



**InfoCAD 23  
User Manual**

**Finite Elements**

**Prestressing**

**Frameworks**

**Dynamics**

**BIM**



# **InfoCAD 23**

**User Manual**

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# Introduction

The **InfoCAD** program system is a civil engineering software package for analyzing 2D and 3D structures. The system includes the following analysis methods:

- Computation of 2D and 3D beam and shell structures, cable structures and solid models.
- Geometrically and physically nonlinear analysis.
- Spring elements with nonlinear characteristic.
- Analysis of contact problems.
- Determination of buckling eigenmodes.
- Element support according to method of bedding or modulus of compressibility with layered subsoil; optional exclusion of tensile bedding.
- Bending design according to DIN 1045, DIN 1045-1, OENORM B 4700, SIA 262, EN 1992-1-1.
- Shear and torsion design, punching shear and crack check according to DIN 1045-1, OENORM B 4700, SIA 262 and EN 1992-1-1.
- Steel structure checks according to DIN 18800 and EN 1993-1-1 (elastic-elastic, elastic-plastic and plastic-plastic).
- Lateral torsional buckling check according to DIN 18800 and EN 1993-1-1.
- Timber checks according to DIN 1052 and EN 1995-1-1.
- Section analysis for polygonal beam sections; profile database.
- Eigenvalue determination, time-step integration, dynamic train crossing, nonlinear cable dynamics.
- Response spectrum method for earthquake check according to DIN 4149, EN 1998-1, OENORM B 4015 and SIA 261.
- Thermal analysis of steel, timber, reinforced concrete and composite sections. Stability check under fire conditions according to EN 1992-1-2, EN 1993-1-2 and EN 1995-1-2.
- Prestressing of beam and shell structures as well as solid models.
- Prestressed concrete checks according to DIN 4227, DIN 1045-1, OENORM B 4750, SIA 262 and EN 1992-1-1.
- Bridge checks according to DIN Technical Report 102 and EN 1992-2.
- Recalculation of road bridges according to the Recalculation Guideline.
- Computation of construction stages.
- Data exchange via DXF, IFC, SAF, steel construction and reinforcement interfaces; BIM Add-In for Autodesk Revit®.

Model editing, analysis control and results output for all structure types are performed in a standard 3D CAD user interface that includes functions familiar from Windows.

InfoCAD manages structures in project files. The information contained in a project file is organized in data sets which you can access from the graphical view, the table view or the *File Service* command in the *File* menu. The structure type is defined for the project file and labeled by a file extension (FEM, etc.) in the *Structure* menu. **One** static system of the following structure types can be stored in each project file:

- Finite Element System (FEM)
- 3D Frame (RSW)
- 2D Frame (ESW)
- Axisymmetric Shell (ROS)

The static system includes the specified structure data such as beams, area elements, tendon groups or section values along with their associated loads. The computation modules identify deformations, internal forces as well as support and soil reactions and store that data together with the check logs in the project file. This means all the results for output and additional processing are available in one central location.

The program also allows you to create and save help diagrams and explanatory notes in the project file.

## Introductory Examples

This documentation is intended to help you get started with editing beam and shell structures and to demonstrate the various options available for displaying the results. You can press the F1 key to access the online help for the function you are currently using in the program. The help system also contains the complete documentation of the program, including explanations of theoretical principles, calculation methods and results.

During installation example files will be saved to the user directory under *InfoGraph/Samples*. They include projects with beam, cable and shell structures as well as applications relating to structural dynamics and prestressed concrete construction.

The examples are provided without results. To view the results, you first need to perform the relevant calculation. Product news, user tips and updated versions of the program are available at [www.infograph.eu](http://www.infograph.eu).

## General Instructions

### Program Start

To start the program, go to the *Programs / InfoGraph* folder in the *Start menu* and select **InfoCAD** or **InfoCAD Study Version**. An empty program window will appear.

Click the  *Open* button to access InfoGraph project files with the following extensions (e.g., example files):

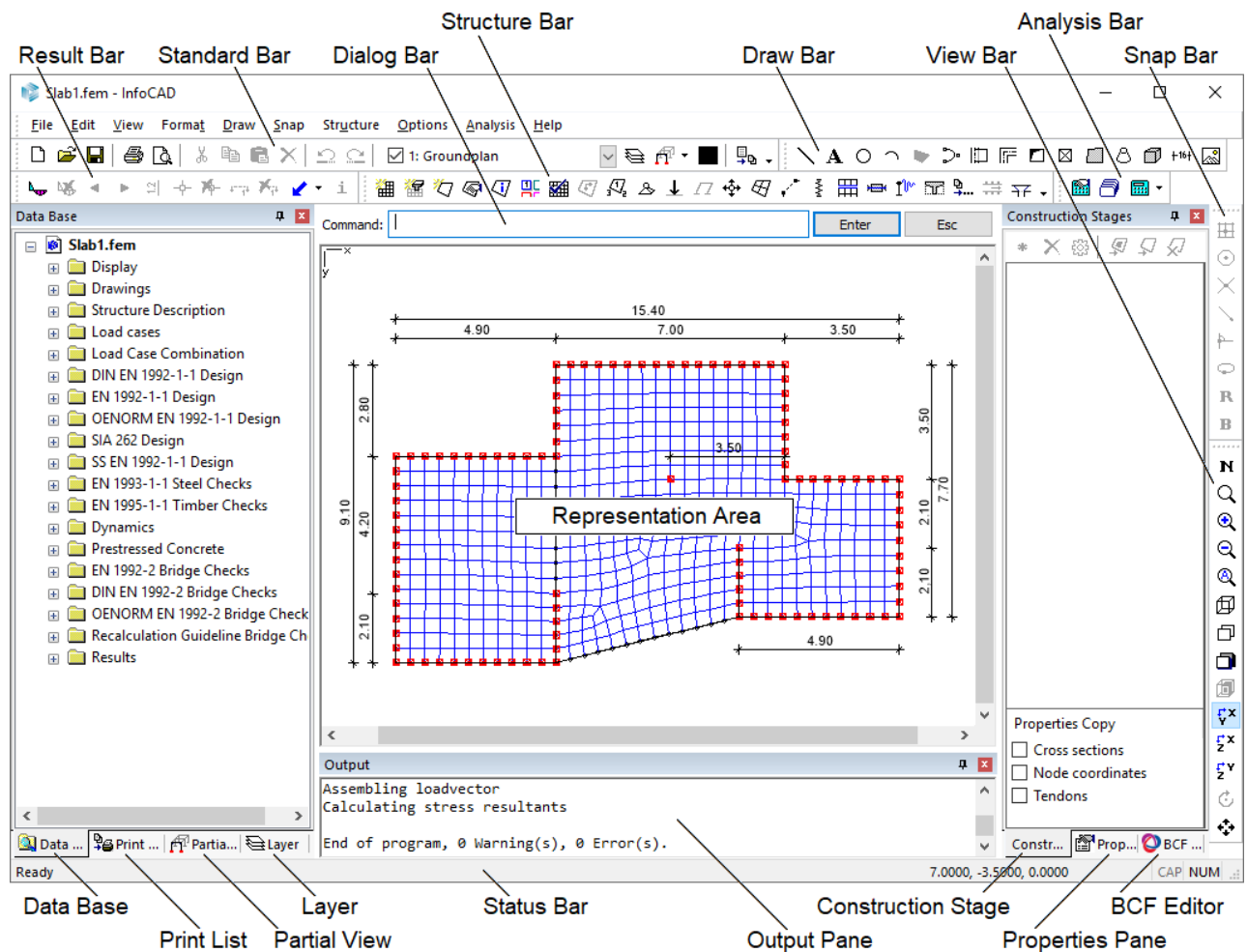
File name	.fem	->	Finite Element Project
	.rsw	->	3D Frame
	.esw	->	2D Frame
	.ros	->	Axisymmetric shell

These file types are registered during installation, which means they can be opened by simply double-clicking the file.

## Operating Concept

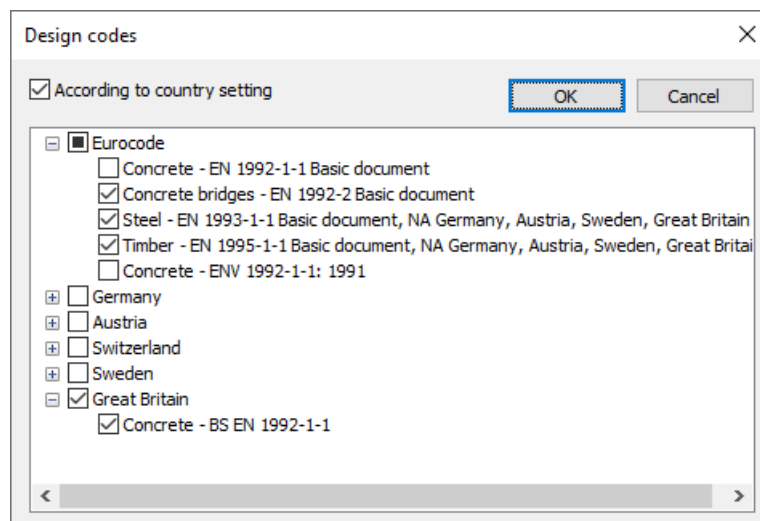
The figure below highlights the most important controls of the **InfoCAD** program interface. For FEM projects, the xy plane appears in the representation area after you select the structure type. Slab and plain stress structures are generally specified in this plane.

Functions can be accessed from a tool bar, a menu item, the dialog bar or the workspace. For graphical functions you normally have to enter additional data, which is then queried in either a separate window or the dialog bar.



## Design Codes

The design codes and associated materials available in the program correspond to the regional settings of your computer. If necessary, you can change these settings in the Options menu.



## Basic Procedures

Structures are normally specified and calculated using the following work steps:

1. Describe the problem with the model objects
2. Specify supports and cross-sections
3. Generate an element mesh
4. Define load cases
5. Define the actions and design situations
6. Perform calculations
7. Display or print out the results

### Model objects are critically important for system generation

Model objects describe the geometry and properties of individual structural components. They form the basis for the program-controlled generation of finite element meshes and are accounted for the automatic mesh generators *form-sensitive* and *grid-shaped*.

The following model objects are used:

<b>Edge</b>	Edges define the borders and the axes of structural components. Depending on their properties, they can also define a line support, a <i>beam series</i> or <i>free beams</i> (e.g. columns). Edges are normally used to describe the whole structure in a wire frame model.
<b>Wall</b>	Like edges, walls define the border of a structural component as well as a line support. The geometry of a component is determined by the wall axes. Walls are specifically suited for creating pure slab systems.
<b>Hole</b>	Holes define openings in the element mesh. They can also define line supports and continuous beams according to their properties.
<b>Column</b>	Columns define a point support or, alternatively, a column head condensation in the element mesh.
<b>Face</b>	Faces describe areas that are automatically meshed by the mesh generator. Edges, walls and holes form the outer border of these meshes. All model objects inside the face are taken into consideration during mesh generation. The face can be assigned various properties for the FEM mesh.
<b>Cone</b>	The <i>Cone</i> model object describes a truncated cone segment for subsequent meshing. You can use the shortcut menu to determine the degree of intersection with other selected cones or model faces. A cone can also be assigned various properties for the FE mesh.
<b>Solid</b>	The model object <i>Solid</i> describes a solid body for the subsequent mesh generation with tetrahedron elements. Properties for the FE mesh can be assigned to the solid.

Model objects have no further significance once the element mesh is generated. However, they should be saved as drawings for reuse at a later time.

For an area to be meshed, a closed polygon must be defined using edges and/or walls. If you want to mesh multiple areas in different planes at the same time, then model faces should be defined as well.

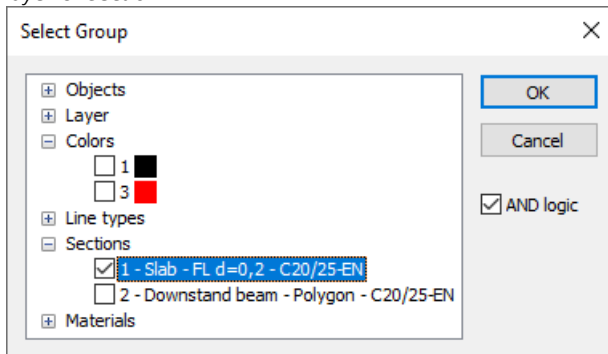
## Selecting

If you want to edit or delete an object, you first need to select it. This can be done in a number of different ways:

1. Select individual objects via mouse click. You can expand or reduce the selection by holding down the *Shift* or *Ctrl* key while clicking the objects.
2. Select an area by holding down the mouse button and dragging the cursor. When you let go of the mouse button, all objects completely inside the drawn rectangle will be selected.

**Selecting with the right mouse button, the shortcut menu with the standard commands for the selected objects will appear.**

3. The *Select* command in the *Edit* menu allows consecutive additions/deletions of the selection using selection boxes for example.
4. The *Select group...* command in the *Edit* menu allows you to select objects based on shared properties such as *color*, *layer* or *section*.



5. The *Select All* command in the *Edit* menu.

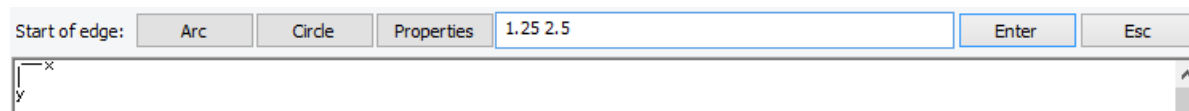
All of these selection methods can be combined with one another. Selections are removed when you click on an empty space in the workspace.

**You can access the shortcut menu for objects that are already selected by holding down the Shift or Ctrl key and pressing the right mouse button.**

## Entering Coordinates

You can enter coordinates with either the keyboard or the crosshair.

If you are using the keyboard, enter the x, y and then z coordinates separated by a space in the dialog bar. The measures are always meter [m]. The positive direction corresponds to the displayed coordinate system.



For 2D editing you only need to enter the first two coordinates. The program will automatically set the third coordinate to zero. The decimal separator (e.g., on the numeric keypad) conforms with the one defined in the regional settings.

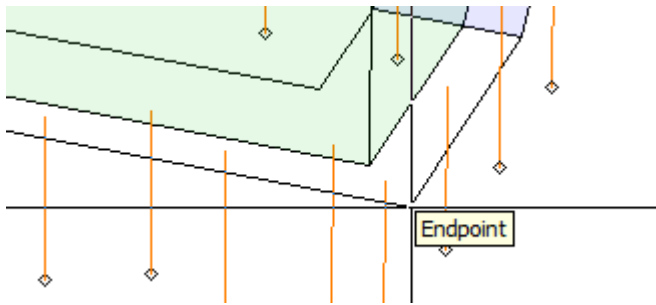
### Global coordinates

These coordinates always refer to the global zero point.

### **R** Relative coordinates

Die These coordinates refer to the last point that was entered (local zero point). A small axis system is always visible at this point. To activate relative coordinate input, click the *Relative* button.

Using the crosshair to enter coordinates, it automatically refers to an existing object.



The current snap option will be displayed.

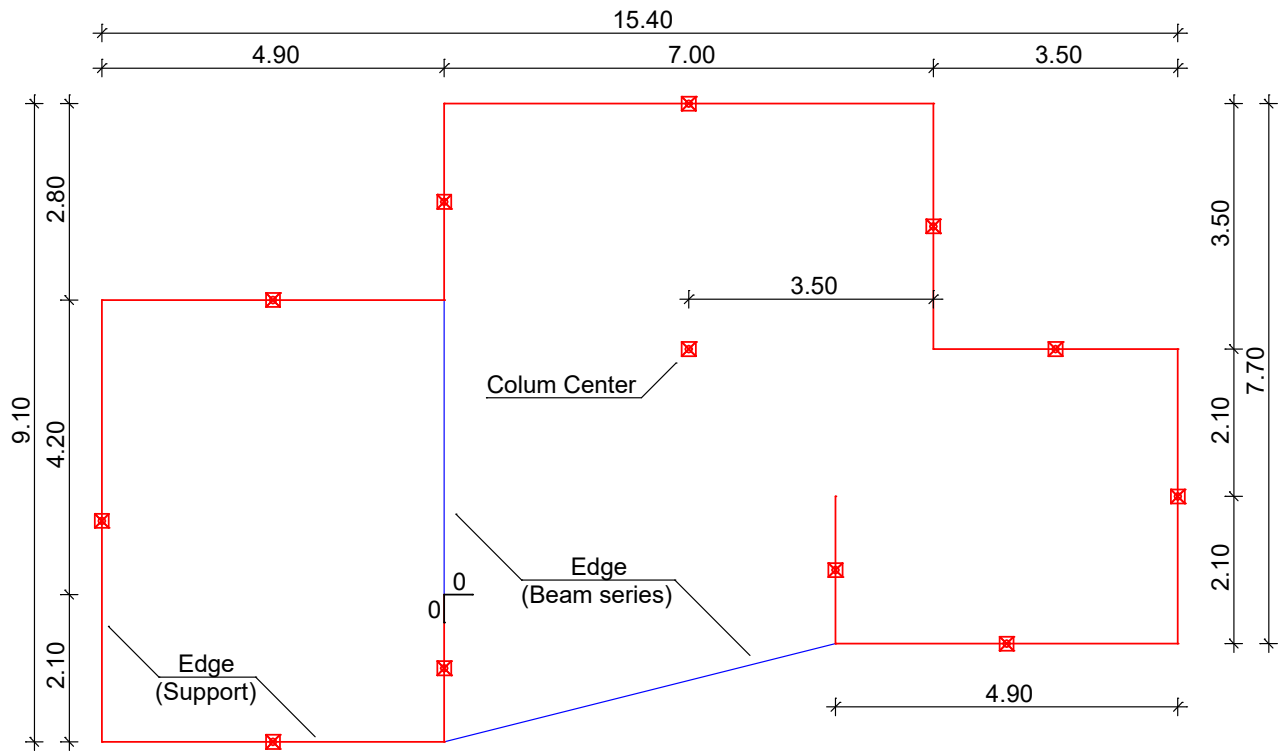
The *Node*, *Midpoint*, *Intersection*, *Endpoint*, *Perpendicular* and *Object* snap functions can be used to define the snap mode and ignore the automatic snap function. Click the appropriate button to activate them.

The snap window will then appear in the crosshair:

The selected snap function will remain active until it is disabled or replaced by another.

### Example 1: Slab with Downstand Beam

This example shows you how to specify and calculate a basic floor plate and then process the results.



Material:	Concrete	C20/25-EN (Concrete EN 1992-1-1)
	Reinforcing steel	BSt 500, axis distance from edge 3.0 cm
Sections:	Slab	Thickness 20 cm
	Downstand beam	
Loads:	Permanent load	Dead load and additional load 1.50 kN/m <sup>2</sup>
	Traffic	Area load 3 kN/m <sup>2</sup>

A new project is started when you launch the program or select *New* from the *File* menu. Choose the **Finite Elements** structure type from the *structure* menu.



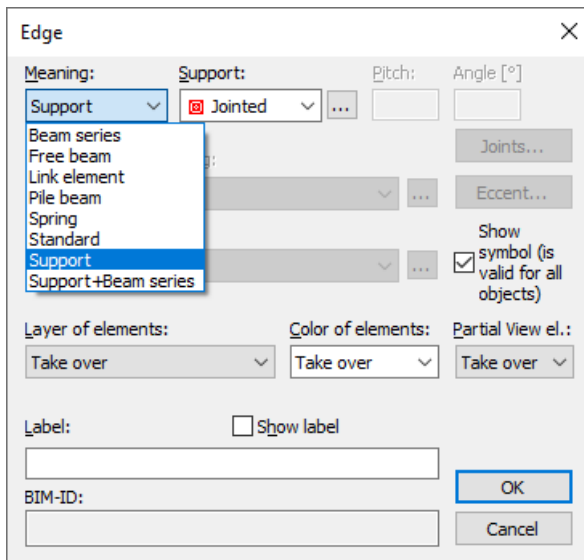
## Drawing Ground Plans with Model Objects

Ground plans and in spatial structures wire frame models are initially described using the Edge, Column and Hole model objects. These objects will later be recognized by the mesh generator and taken into account during mesh generation.

You do not need to make any definitions regarding the area:

**Simply draw your static system. The mesh generator will take care of the rest.**

Click the *Edge* button and select *properties* in the dialog bar.



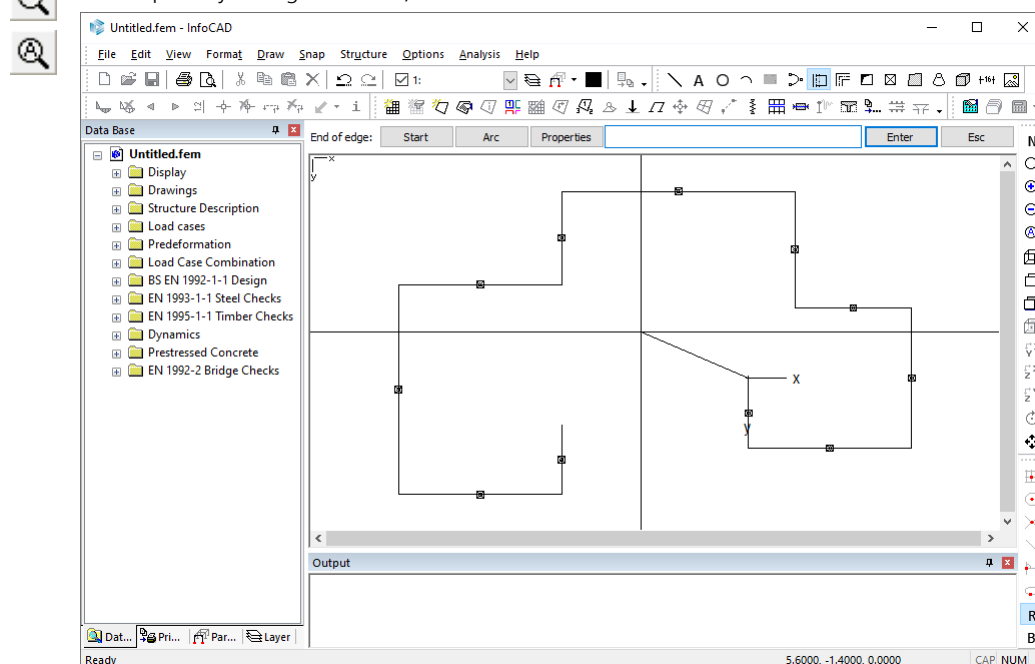
For Meaning set the option *Support* and leave the dialog with *OK*.

A crosshair will appear in the representation area and you will be prompted to enter the coordinates of where the edge starts in the dialog bar. Enter [0 0] to position the start of the edge at the origin. After you press [↵] or Enter, the end of the edge will be queried.

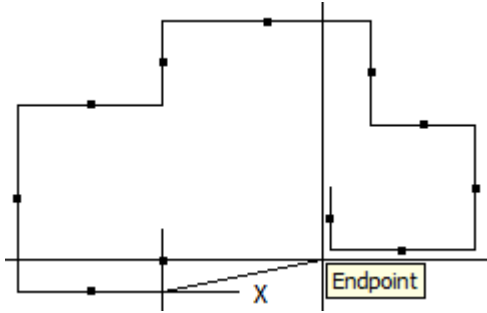
To enter additional coordinates, you should activate relative coordinate input by clicking the *Relative* button. A small axis system will now always appear at the last point inserted. All successive coordinates you enter always refer to this point. Enter [0 2.1] to define the end of the edge in the positive y direction. If you enter consecutive coordinates, the subsequent edge ends will always be queried. The remaining coordinates are therefore:

[ -4.9 0 ] [ 0 -6.3 ] [ 4.9 0 ] [ 0 -2.8 ] [ 7 0 ] [ 0 3.5 ] [ 3.5 0 ] [ 0 4.2 ] [ -4.9 0 ] [ 0 -2.1 ]

To temporarily enlarge the view, click *Zoom -* or *Zoom all*.

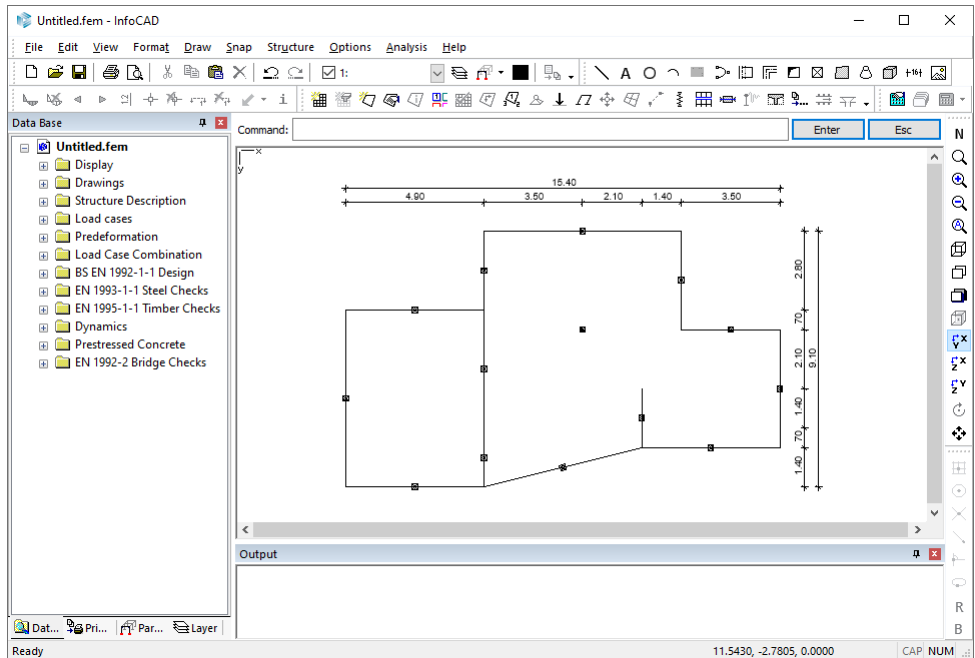


You can begin entering another edge without having to use the command over again by selecting the *Start* option. Now enter the missing edges. In the process you can click on the endpoints of existing edges.



- Click the *Column* button and enter the column midpoint. To move the current reference point for the entry of coordinates, click the *Basepoint* button and select the end of the edges to the right of the column as the new reference point. Confirm the *Old* or *Global* setting (the local reference system is not to be rotated) as the *New direction of x axis*.

The column should be 3.5 m in the negative x direction from the selected edge end point, which means the coordinates for the column midpoint are: [-3.5 0].



- One way to check your input is to dimension the system. To do so, click the *Dimension* button and the *Objects* option. Now select *all* of the specified model objects. Dimension points are the end points of all objects. For example, to position a horizontal dimension chain, select *Horizontal* and click the drawing area below your system.

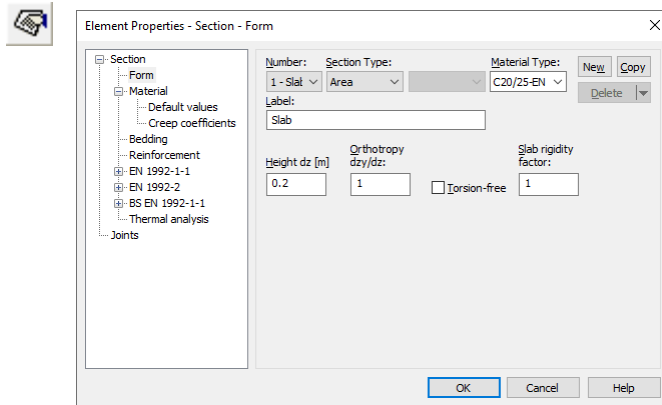
- Click *Save* to store all the data you have entered up to this point. The Windows file dialog will appear and prompt you to enter a file name. The *\*.fem* extension will be assigned automatically. Your drawing will also be given a name (E.g. *Ground\_plan*) and is saved in this file.

### Specifying Supports and Cross-Sections

The properties of the specified Edge and Column model objects are preset as follows:

- Edge: Support jointed
- Column: Support jointed

The correct support conditions are thus already set for this system.



With the selection of the material the possible design standards are determined.

The corresponding properties are now available.

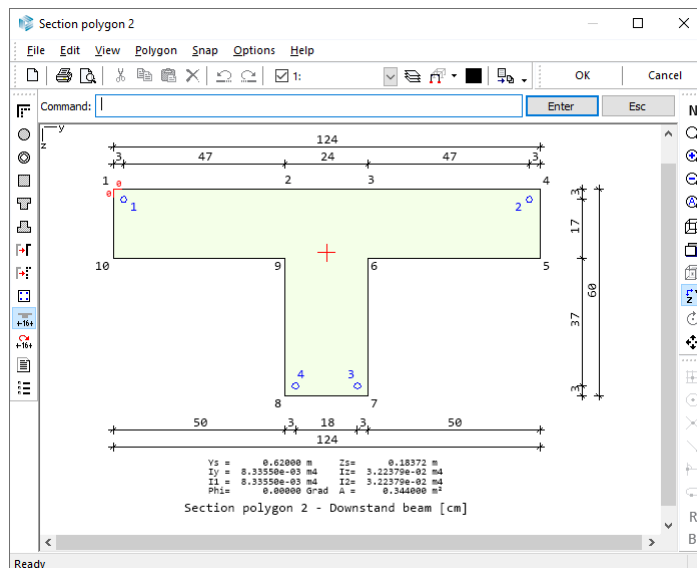
For example the material C ..... -EN-D is to choose when the checks per NA Germany are to be performed.

For the other European standards, the material selection is C..... -EN.

To modify the properties of the area elements, select the *Element Properties* command. Click *New* and then change the section type to *Area* for **Section 1**. The thickness is preset correctly with 0.2 m and the material must be selected to *C20/25-EN*. For the next input this material is preset.

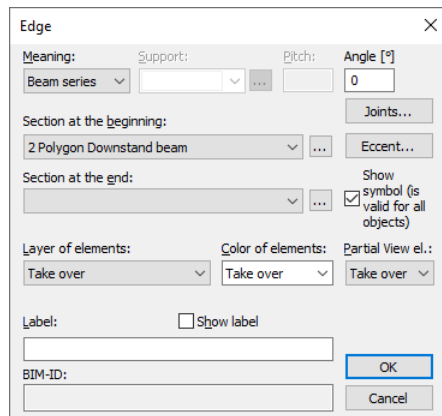
The default settings for the remaining section properties such as bedding, reinforcing steel and EN 1992-1-1 are correct for this example.

Click *New* again and generate **Section 2**. Change the section type to *Polygon*. The input window for graphically describing the polygon is displayed:



You can generate and dimension the polygon shown above using the *Downstand Beam* command. Close the window by clicking *OK*.

Activate the *Reduce dead load* button to decrease the dead load of the T-beam by the portion attributed to the slab.



Beam series (downstand beams) with the specified section should be generated at the two last entered edges to replace the supports. Open the *Edge* dialog by double-clicking the edge you want to modify. Now change the *Meaning* to *Beam series* and select *Section 2*. You do not need to specify the section at the end of the edge.

## Generate the FEM Mesh



The finite element mesh can be generated automatically using the specified model objects. To do so, click the *Mesh Generation* button and select the *Form-sensitive* option. Arrow indicators will now appear marking the outer area limits recognized by the program. Specify the mesh width (0.45 in this example) to generate the FEM mesh, supports and beam series.

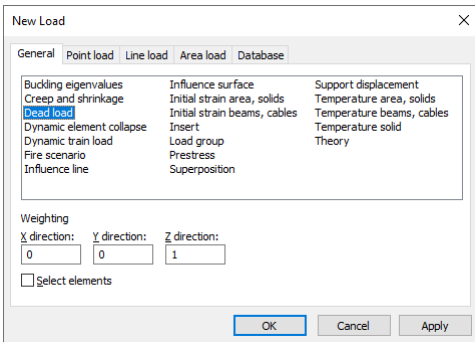
The drawing is not required anymore. Deactivate the drawing in the database  **Ground\_Plan**

## Defining Load Cases

Open the *Load cases* folder in the database and double-click **New load case**. A new load case will be created automatically and the load input and editing buttons will appear in the dialog bar.



Click the *Load creation* button to open the New Load dialog:



Select *Dead load* with a weighting of 1 in the Z direction and then click *Apply*. Using this load type, the dead load will be generated automatically based on the sections and material. This load is indicated by the *Dead Load* label in the upper right corner of the image.

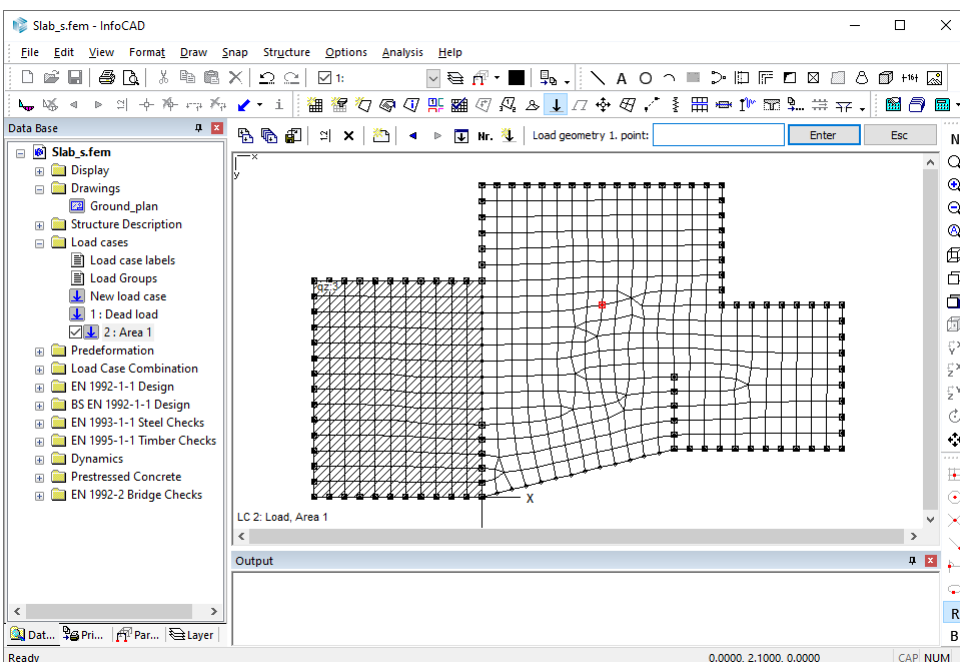
Now select *Uniform region* on the *Area load* tab and select two points to specify a rectangle that surrounds the entire slab. The load ordinate  $q_z$  should correspond to 1.5 kN/m<sup>2</sup>. This load will be applied to all elements located entirely within the area.



Click *Change Number* to modify the load case number or to specify a load case name.



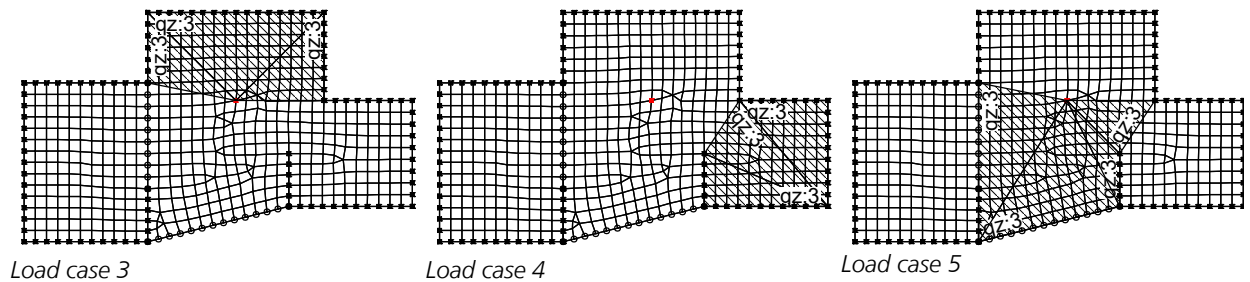
Create the new load case 2 using the function *New load case*. Select the *Uniform rectangle* area load under the *Load creation* option. The load rectangle is now represented by three coordinates. Click on the three corner points of the load field. In this case the load ordinate is  $q_z = 3$  kN/m<sup>2</sup>:



Create the next load case and use the *Uniform polygon* area load for each remaining field.

Trace the next field and then click *Close* and enter the load ordinate  $q_z = 3 \text{ kN/m}^2$ . The program will then apply triangle areas to the defined load area.

Now enter the depicted load cases in consecutive order, each time selecting *New load case*:



**x** Load input is now complete. Exit the load dialog by clicking *Close* (in the dialog bar **x**).

## Specifying Actions and Design Situations

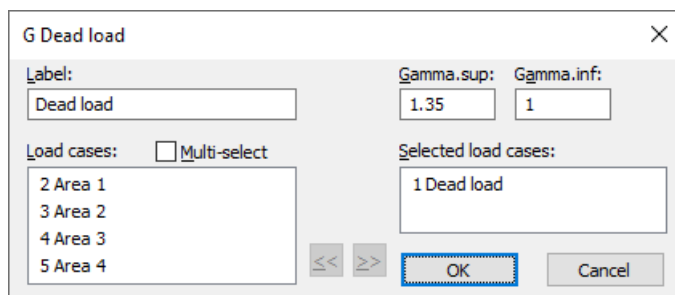
When carrying out the design, first the internal forces of the actions are combined. The program uses cyclic permutation to analyze all variations of the leading and accompanying actions. The safety factors and combination coefficients are automatically taken into account as specified by EN 1990. The extremal internal forces are then used to determine the design values.

This is done by simply assigning the existing load cases to the actions:

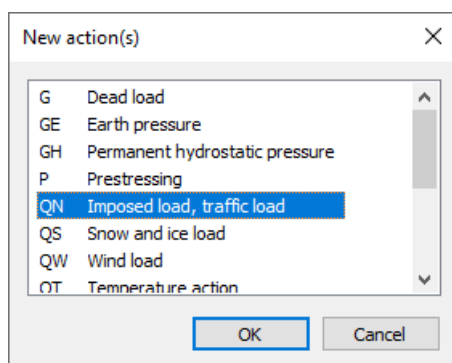
G     Dead load:                     Load case 1  
 QN    Imposed load, traffic load:   Load case 2-5

Open the *EN 1992-1-1 design* folder in the database and select the *Actions* submenu item. A dialog will appear where you can define actions in accordance with EN 1992-1-1. The program suggests a new *G Dead load* action. Confirm by clicking *OK*.

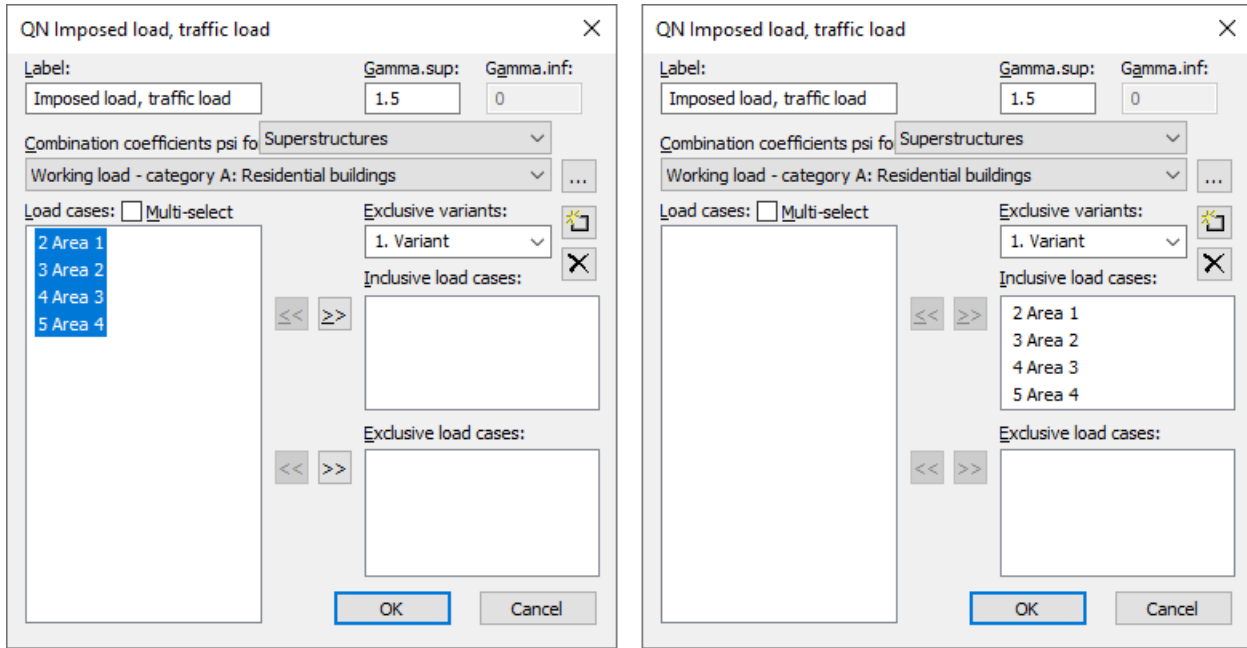
Now assign the relevant load cases to the action by selecting a load case in the left pane and moving it to the *Selected load cases* pane on the right:



Exit the *Dead load* dialog and create a new *QN Imposed load, traffic load* action.



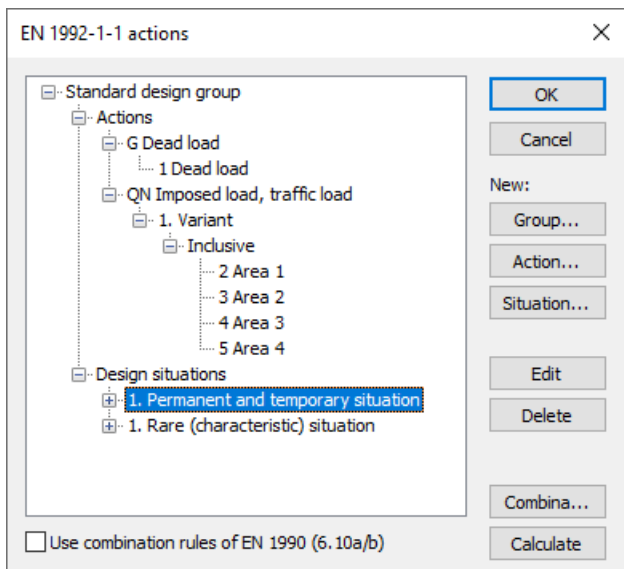
Assign the remaining traffic loads by moving them to the *Inclusive load cases* area.



<i>Inclusive load cases</i>	Selected load cases that can have a simultaneous effect.
<i>Exclusive load cases</i>	Selected load cases that are mutually exclusive (e.g., wind from right / left or individual mobile load positions)

The action *QN* is now defined and you can exit the dialog.

Next click *Situation...*, select *Permanent and temporary situation* and then accept the default settings. The *Rare (characteristic) situation* is required to define the robustness reinforcement. Create this as well.



The actions and situations are now defined. You can edit an action or situation by double-clicking it. The user-friendly tree structure provides an easy way of checking the included load cases.

Click *OK* to exit the *EN 1992-1-1 actions* dialog.

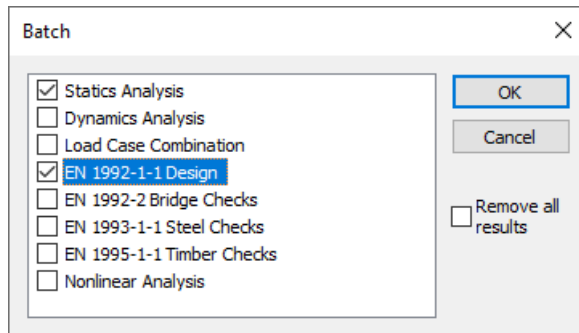
## Performing Calculations

You can now perform the following calculations:

- *Statics analysis* (determines deformations, internal forces, support forces, etc., for each load case)
- *EN 1992-1-1 design* (bending and shear design)



The calculations can be started automatically in consecutive order. To do so, click the *Batch...* button and select the relevant check boxes.



If necessary, you can control the calculation and logging process by clicking the *Calculation settings...* menu item.

The progress of the calculation and any errors or warnings will be displayed on the *output bar*.

## Processing Results

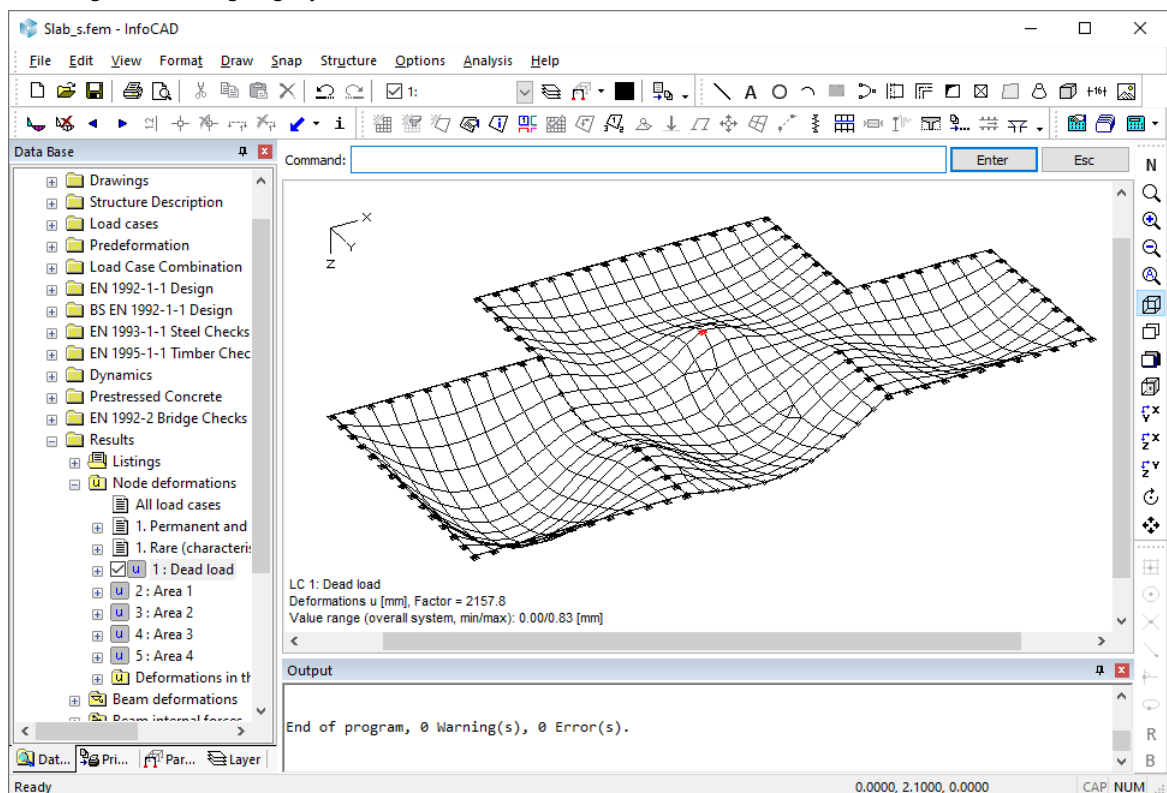
All of the available results are listed in the **Results** folder in the database. Double-click a result to open the available or designated standard view (graphic or table) of that result.



For instance, open the *Node deformations* results folder and then double-click *Load case 1*. The deformed system will be displayed. To see the deformation figure, switch to the *3D view*.

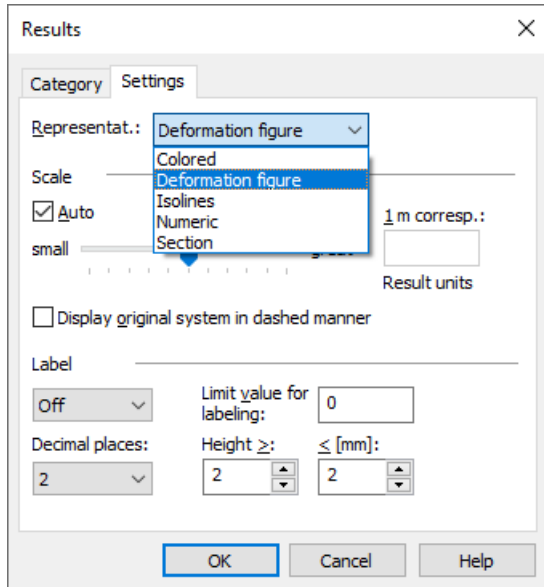


To change the viewing angle you can use the *Rotate with mouse* button.





Click the *Results* button in the results bar to open a dialog that lets you select the display option and settings for all results:



If you select representation *Section*, you can click the *Define section* button in the results bar and define the sections for the results view.

Please note that some view options are not available depending on the category you selected:

- A deformation figure can only be displayed for all components simultaneously and not for an individual displacement component (e.g.,  $u_z$ ).
- The min/max line from an internal forces combination of area elements can only be displayed numerically or in the section.

## EN 1992-1-1 Checks

In addition to the bending and shear design that has already been carried out, you can also perform additional checks in the ultimate limit state and the serviceability limit state:

- Bending with or without normal force or normal force only (EN 1992-1-1, Chapter 6.1).
- Minimum reinforcement against failure without warning (Chapter 5.10.1 (5)P and 9.2.1.1).
- Lateral force (Kapitel 6.2).
- Torsion and combined stressing (Kapitel 6.3).
- Punching shear (Kapitel 6.4).
- Fatigue of longitudinal reinforcement, shear reinforcement and prestressing steel (Chapter 6.8.5, 6.8.6)
- Fatigue of concrete under compressive stress (Chapter 6.8.7)
- Fatigue of the concrete compressive struts under lateral force and torsion (Chapter 6.8.7 (3))
- Limiting the concrete compressive stresses (EN 1992-1-1, Chapter 7.2).
- Limiting the reinforcing steel stresses (Chapter 7.2).
- Limiting the prestressing steel stresses (Chapter 7.2).
- Decompression check (Chapter 7.3.1).
- Minimum reinforcement for crack width limitation (Chapter 7.3.2).
- Crack with calculation (Chapter 7.3.4).
- Limiting deformations (Chapter 7.4).

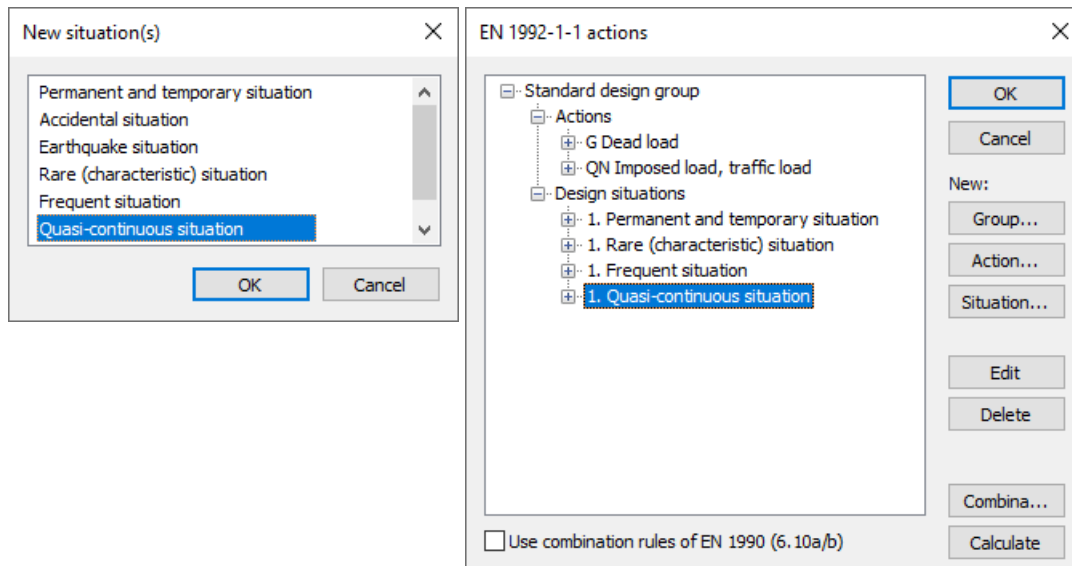
Additional input is required in the  *EN 1992-1-1 design* folder for this purpose:



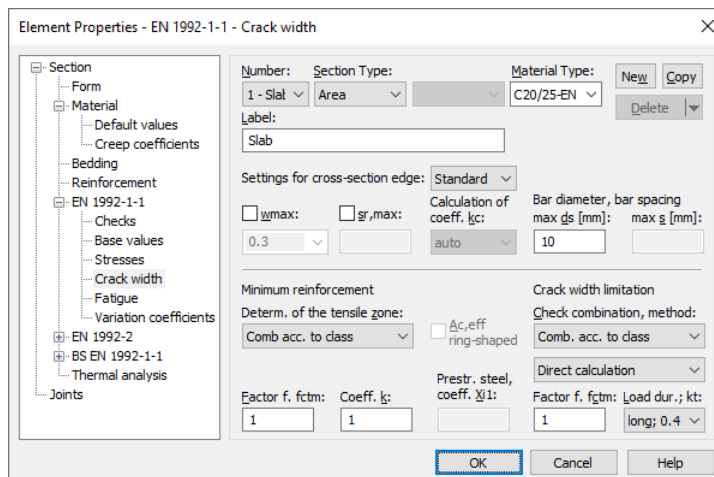
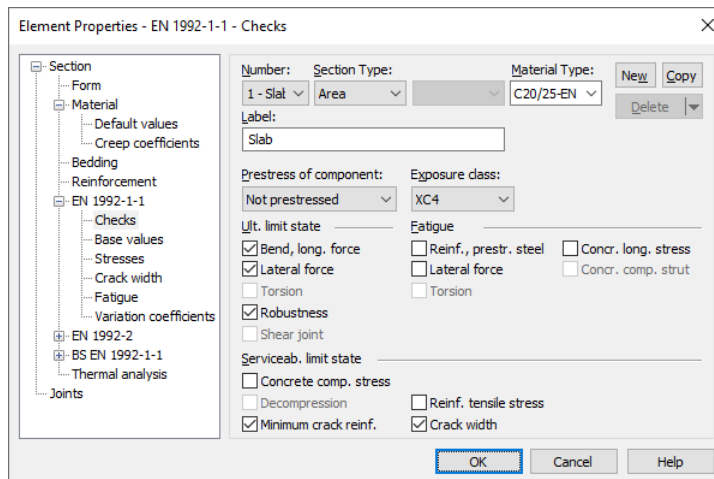
**Actions:** Define the necessary design situation depending on the additional checks and the exposure class.



To do this, click *New: Situation...* in the action dialog and create the required situations.



Settings related to the section are made in the Element properties dialog. Click the *Element properties* button to open this dialog and then enter the check specifications required for each section under *EN 1992-1-1*.



After you are done, start the *EN 1992-1-1 design*. The program will determine the required reinforcements, perform the checks and then generate a log.

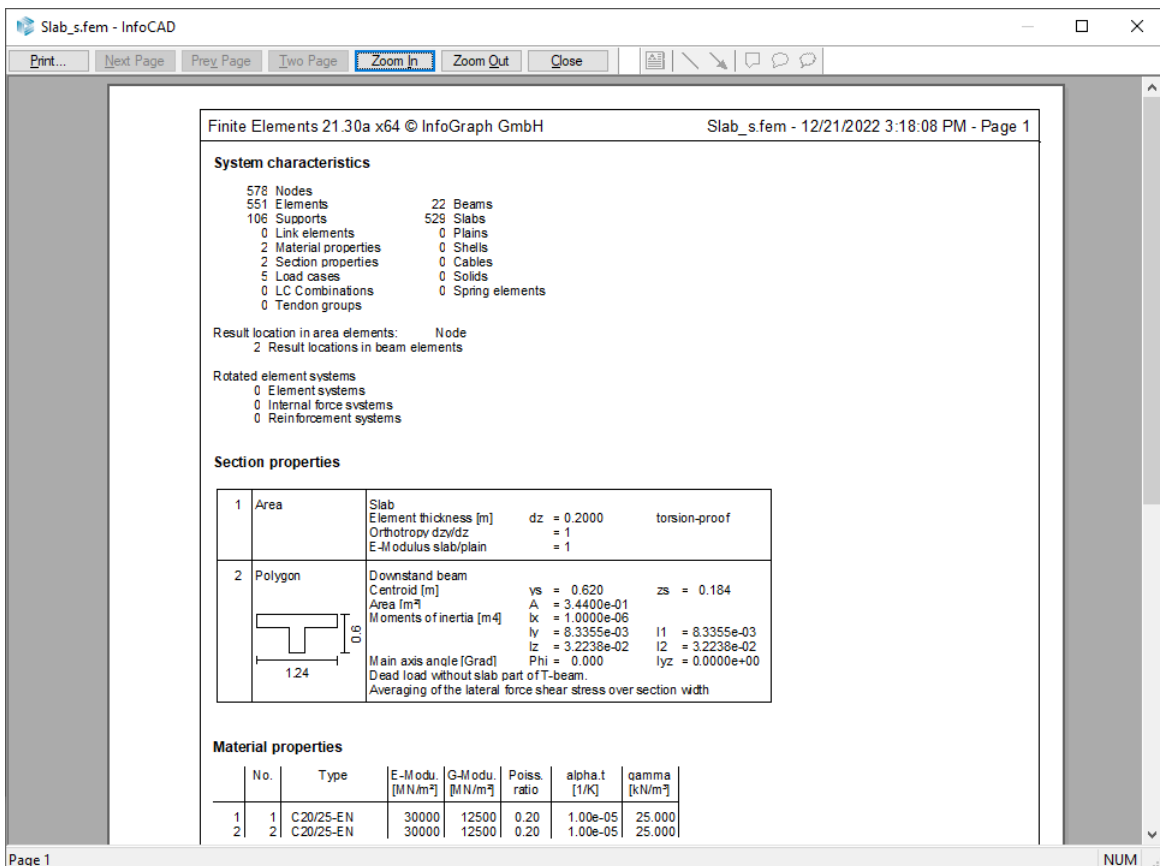
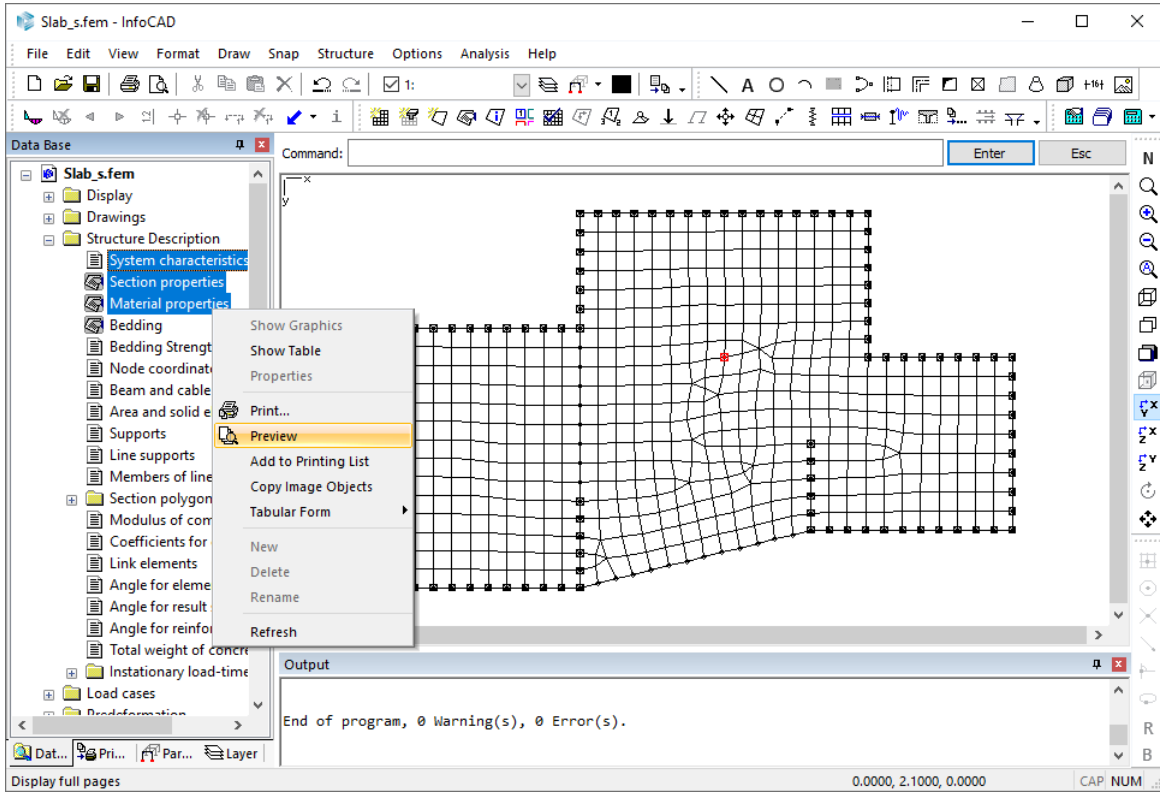
A detailed description of all the checks is available in the chapter *EN 1992-1-1 Design*.

## Printing



You always have the option of printing the current screen view. Click *Print Preview* to see what the actual printout will look like. The settings for the page format, printer selection, etc., can be modified using the *Page Setup...* command in the *File* menu.

Tables or logs need to be selected in the database and then printed out from the shortcut menu (right-click). The *Page Preview* command is also available in the shortcut menu.



## Printing List

Specified views as well as tables and logs can be added to the printing list for printing at a later time. Only display parameters are stored, meaning changes made to the system are automatically reflected in the list.

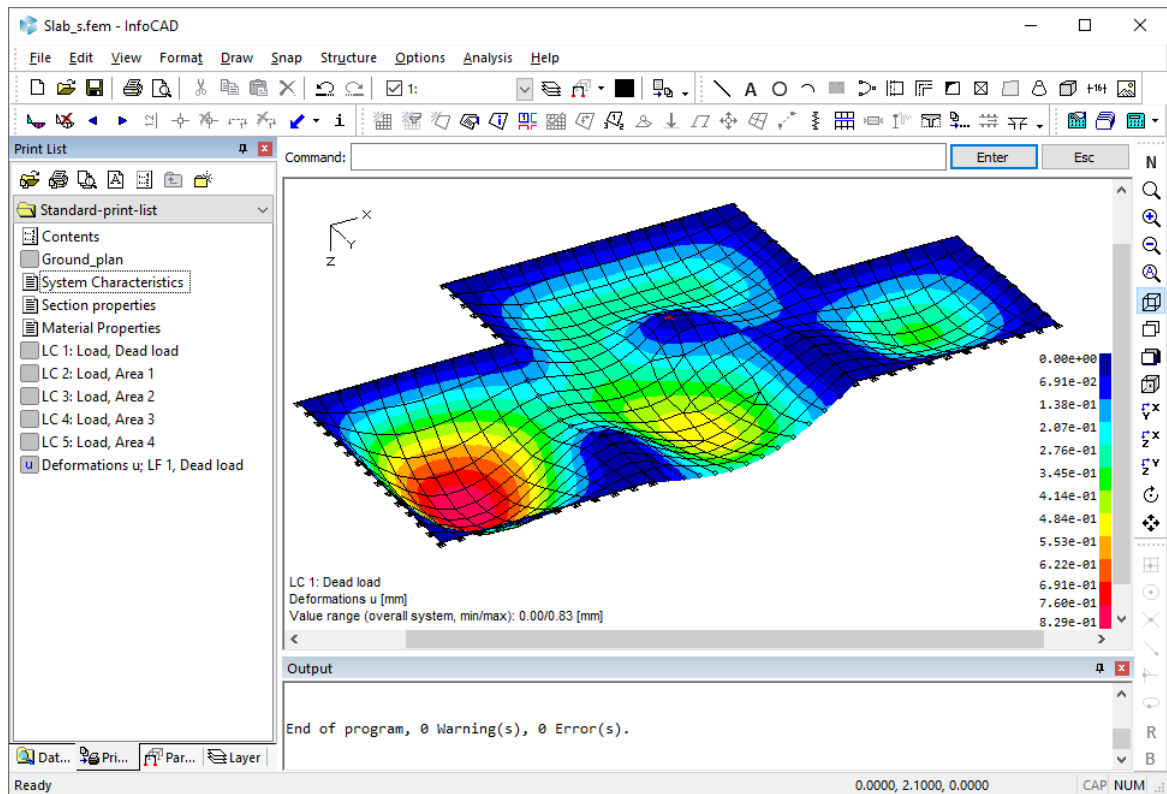
Switch from the *database* to the *printing list*. The *Print* and *Page Setup* commands can also be accessed in the printing list.



Click the *Add to printing list* button to copy the current system view into the printing list. All active settings will be saved with the system view.

Use the shortcut menu to add tables or logs selected in the database to the printing list. You can also add separate text objects, page breaks or a table of contents.

You can sort existing entries in any order you want by simply dragging them with the mouse.



Double-click entries in the printing list to open the corresponding view in the representation area.

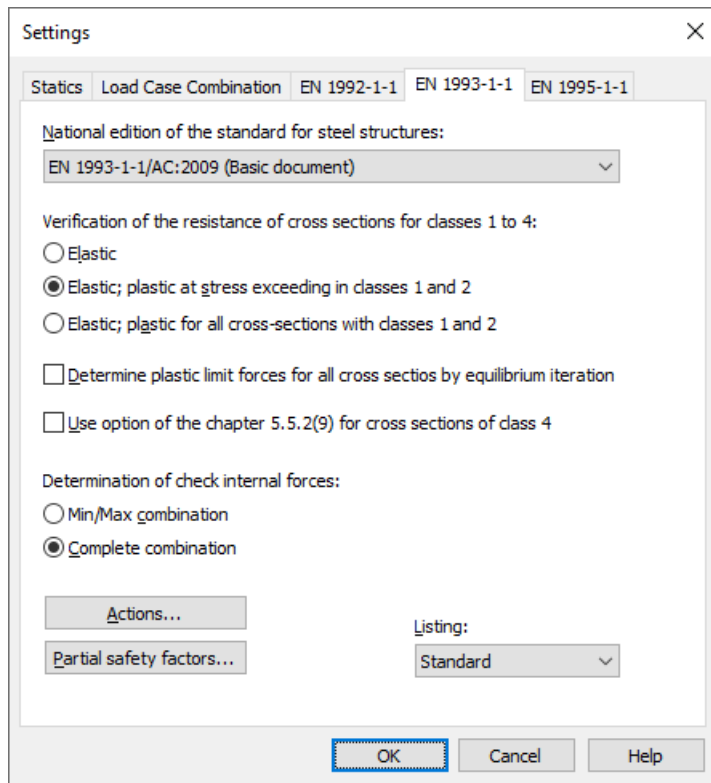
## Example 2: 2D Hall Frame

This example shows you how to specify a 2D hall frame and perform the following calculations:

- Static calculation
- EN 1993-1-1 Steel Checks (elastic; plastic at stress exceeding in classes 1 and 2)

Please select in the menu *Analysis / Settings ... EN 1993-1-1* the required national annex of the standard. This will be used in the subsequent inputs and calculations.

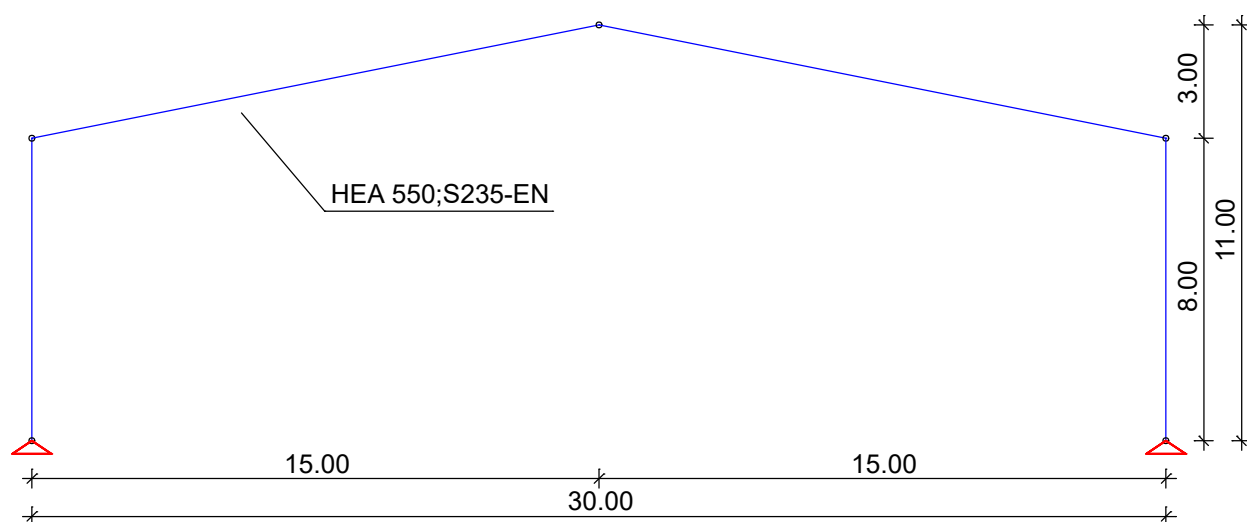
Activate the *Elastic; plastic at stress exceeding in classes 1 and 2* - button.



In accordance with Chapter 6.2.1 of the standard, the elastic cross-section resistance is verified for classes 1 to 4.

If the comparison stress in classes 1 and 2 exceeds the permissible limit, the plastic cross-section resistance will be verified. For every set of internal forces the cross-section class is automatically determined.

## Task



A new project will be started when you launch the program or select *New* from the *File* menu. Choose the **2D Frame** structure type you want to use from the *structure* menu.

The representation area will switch to the x-z view and you can now start specifying the system.

## Specifying the Framework



Click the *Element creation* button and a crosshair will appear in the representation area. You will then be prompted to enter the coordinates of the beam starting point in the dialog bar. Enter [0 0] to position the first beam node at the global origin. Press [↵] or *Enter*. Next, the beam end point will be queried.



To enter additional coordinates, you should activate relative coordinate input by clicking the *Relative* button. A small axis system will now always appear at the last point inserted. All successive coordinates you enter always refer to this point. Enter [0 -8] to define the beam end in the negative z direction. You have now specified the first beam.

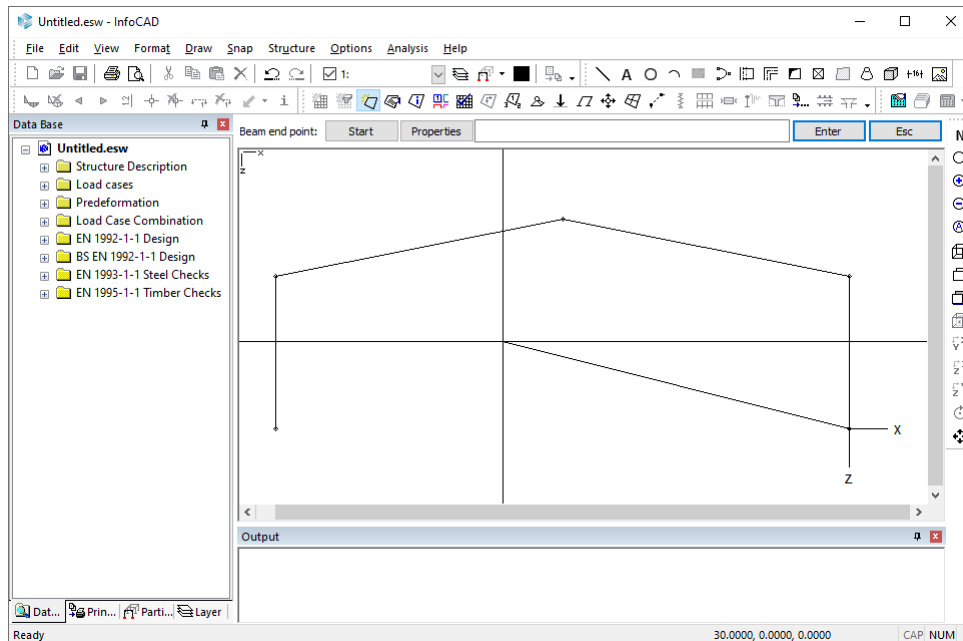


To temporarily enlarge the view, click *Zoom -* or *Zoom all*.



Enter the remaining beam ends in consecutive order and adjust the view where necessary:

[15 -3] [15 3] [0 8]

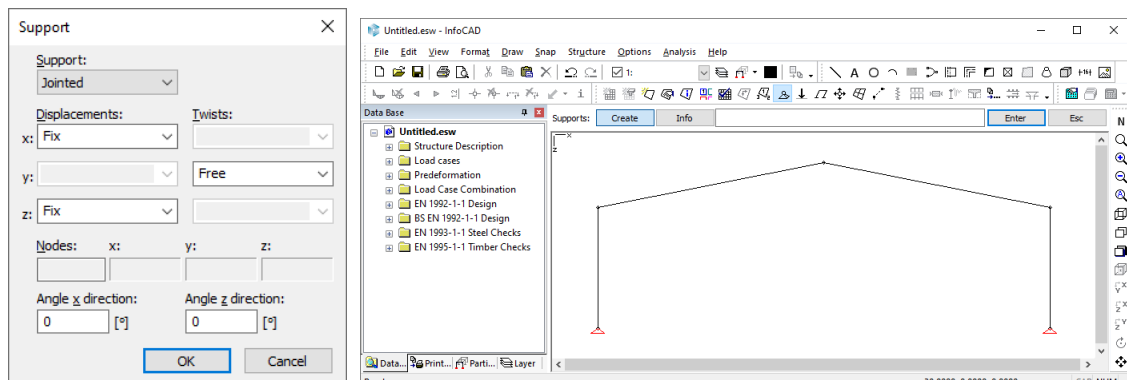


Exit beam input by pressing [↵] or *Enter*.



Now click the *Supports* button and choose the *Create* option. You will be prompted to select the beam nodes in the dialog bar. You can click the nodes directly with the crosshair or select them using the *Window* option (to specify a rectangular snap window). Press *Enter* after you have selected the nodes. Specify *Yes* when the program asks you for the global alignment.

In the *Support* dialog you can individually adjust each degree of freedom if necessary.



Confirm that the support is set to *jointed* and exit the *Support* dialog by pressing *OK*. The supports are indicated by their corresponding icons.



Click *Save* to store all the data you have entered up to this point. The Windows file dialog will appear and prompt you to enter a file name. The \*.esw file extension will be added automatically.

## Defining the Section



Click the *Element properties* button and set the section type to *HEA 550* and the material type to *S235-EN* for section 1.

The 'Beam Properties - Section - Form' dialog box contains the following fields and values:

- Number: 1
- Section Type: HEA
- Material Type: S235-EN
- Label: (empty)
- Dimensions: 0.3 (width), 0.54 (height)
- Properties:
  - A: 0.0212 m<sup>2</sup>
  - I<sub>x</sub>: 3.53e-06 m<sup>4</sup>
  - I<sub>y</sub>: 0.001119 m<sup>4</sup>
  - I<sub>z</sub>: 0.0001082 m<sup>4</sup>
  - I<sub>yz</sub>: 0 m<sup>4</sup>

The 'Section Library' window shows a tree view of steel profiles. The selected profile is '50 x 30 x 2,9' (MSH). Other visible profiles include '50 x 30 x 3,2', '50 x 30 x 3,6', '50 x 30 x 4', and '50 x 30 x 4,5'. The 'RRO 50 x 30 x 2,9 (MSH)' profile is highlighted at the bottom.

You also have access to a detailed section library with international steel profiles.

The complete framework is now defined and the corresponding parts list is available in the workspace in the *Structure description* folder.

### List of steel profiles

	Section	Length [m]	Quantity	Total [m]	Weight [kg/m]	Weight [kg]	Total [t]
1	1 HEA 550 - S235-EN	8.000	2	16.000	166.420	1331.36	2.66
2	1 HEA 550 - S235-EN	15.297	2	30.594	166.420	2545.74	5.09
3	Sum		4	46.594			7.75

## Checking the Structure Properties





You can activate a number of different view options to check the data you have entered. To do so, click the *View options* button:

The 'View Options' dialog box includes the following settings:

- Numbering:  Beams,  Supports,  Node,  Springs
- Text height [mm]: min: 1, max: 4
- Coordinate system: Beams
- Cloring:  Legend
- Shrink mode, factor: 0.8
- Full symbols for supports:

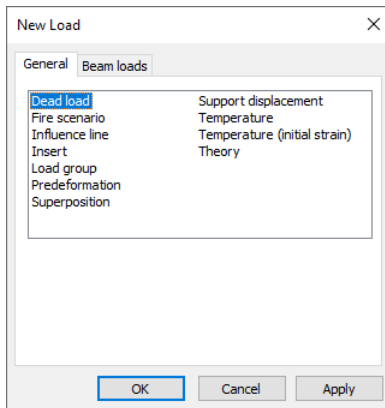
The main window displays a structural diagram of a hall frame with nodes 1, 2, 3, and 4. Beams are labeled '1-HEA 550'. The diagram shows a truss-like structure with vertical supports at nodes 1 and 4, and a horizontal beam at node 3.

## Defining Load Cases

Open the  *Load cases* folder in the database and double-click  **New load case**. A new load case will be created automatically and the load input and editing buttons will appear in the dialog bar.

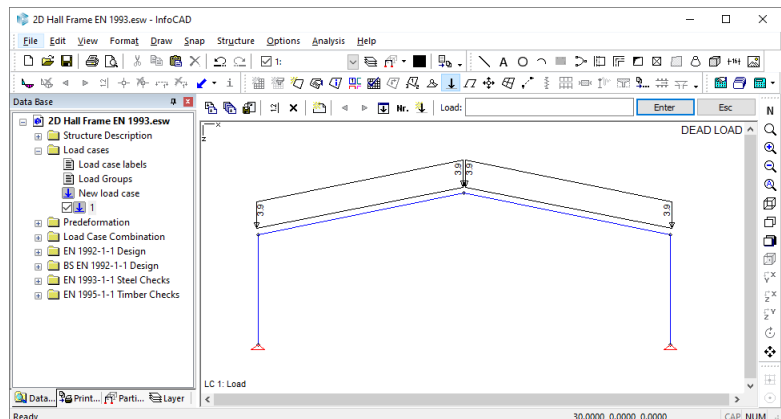
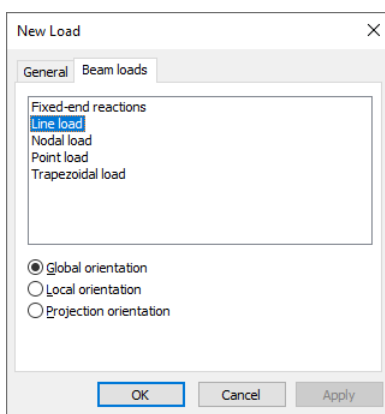


Click the *Load creation* button to open the *New Load* dialog:



Activate the *Dead load* and click *Apply*. Using this load type, the dead load will be generated automatically based on the sections and material. This load is indicated by the *Dead Load* label in the upper right corner of the image.


Now choose *Line load* from the *Beam loads* tab, check the *Global orientation* option, select both beams of the horizontal frame member and enter a load ordinate of  $q_z = 3.9$  kN/m:

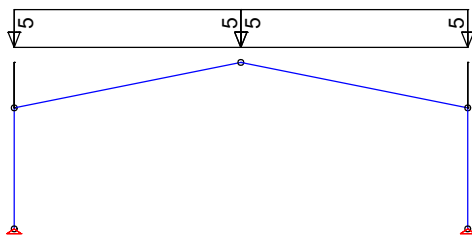


You can adjust the load diagrams in any manner you want using the *Load representation* button.

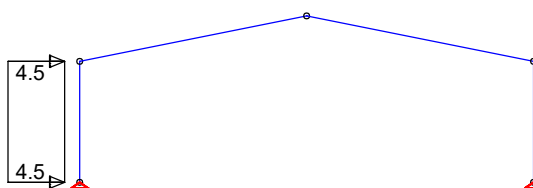


Click *Change Number* to modify the load case number or to specify a load case name.

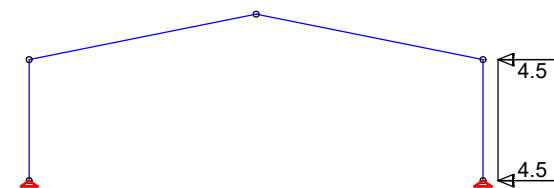
Create the new load case 2 by pressing  **New load case**. Choose *Line load* from the *Load creation* tab, check the *Projection orientation* option and enter a snow load of  $q_z = 5$  kN/m.



Starting with *New load case*, enter the following load cases:

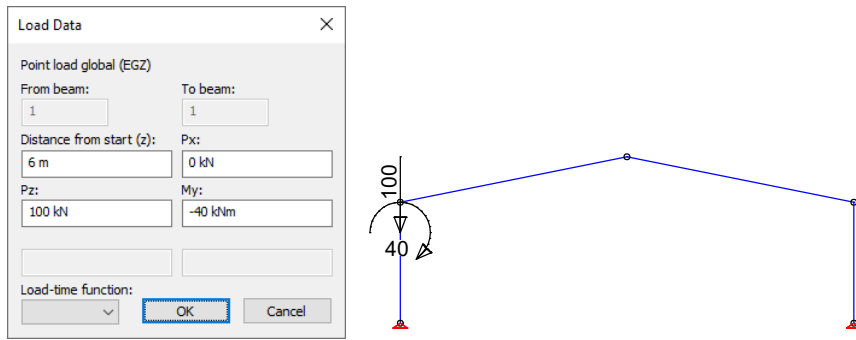


Load case 3: Wind from the left



Load case 4: Wind from the right

The 'Load case 5: Crane load left' requires a single load to be positioned inside the frame column. This load should be applied 2 meters below the frame corner. After selecting the *Point load* load type, simply click the corresponding frame column and enter the ordinates as shown below:



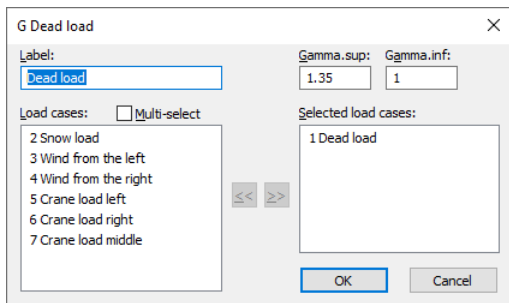
For precise positioning, define a load location with a distance of 6 meters to the beam start.

Now enter 'Load case 6: Crane load right' (distance of 2 meters to beam start) and 'Load case 7: crane load middle' accordingly, with half the load on each frame column.

### EN 1993-1-1 Steel Checks

You first need to describe the actions according to EN 1993-1-1. To do so, assign different actions to the existing load cases:

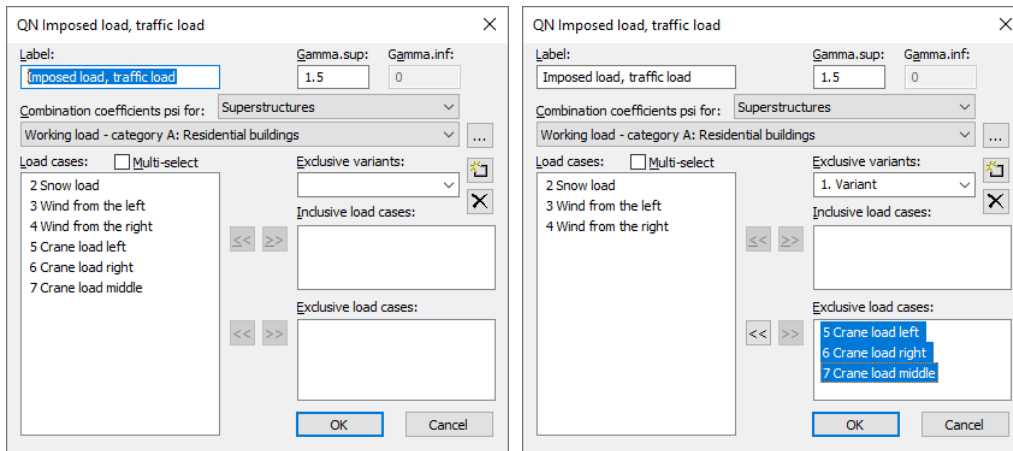
- G Dead load: Load case 1
- QS Snow and ice load: Load case 2
- QW Wind load: Load case 3 and 4
- QN Imposed load, traffic load: Load case 5, 6 and 7



Open the **EN 1993-1-1 Steel Checks** folder in the database and select the **Actions** submenu item. A dialog will appear where you can define actions in accordance with EN 1993-1-1. The program suggests a new *G Dead load* action. Confirm by clicking **OK**.

Now assign the relevant load cases to the action by selecting a load case in the left pane and moving it to the *Selected load cases* pane on the right.

Exit the *Dead load* dialog and create a new *QN Imposed load, traffic load* action. Assign the remaining traffic loads by moving them to the *Exclusive load cases* area.

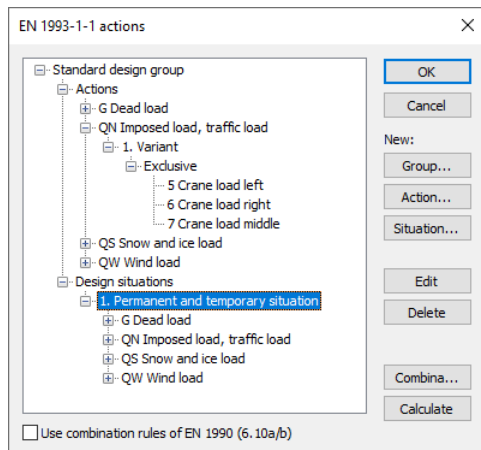


<i>Inclusive load cases</i>	Selected load cases that can have a simultaneous effect.
<i>Exclusive load cases</i>	Selected load cases that are mutually exclusive (e.g., wind from right / left or individual mobile load positions)

Do the same with the *QS Snow and ice load* and the *QW Wind load* action.



Next click *Situation...*, select *Permanent and temporary situation* and then accept the default settings.



The actions and situations are now defined. You can edit an action or situation by double-clicking it. The user-friendly tree structure provides an easy way of checking the included load cases.

Click *OK* to exit the *EN 1993-1-1 actions* dialog.

## Performing Calculations

You can now start the following calculations:

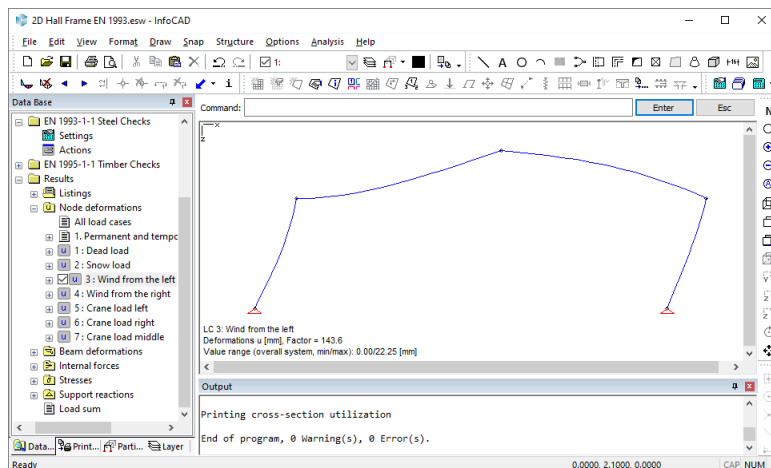
- Static analysis (determines deformations, internal forces, support forces, etc., for each load case)
- EN 1993-1-1 Steel Checks (superposition of actions with calculation of determinant design values, check for elastic or plastic utilizations)

In the standard case, the checks are performed at 5 calculation points per beam. Due to the overlarge element division, it is recommended to select 11 calculation points for beams in the analysis settings for static analysis in this example.

The calculations can be started automatically in consecutive order. To do so, click the *Batch...* command in the calculation menu and activate the relevant check boxes.

The progress of the calculation and program messages, such as errors or warnings, will be displayed in the *output bar*.

## Processing Results



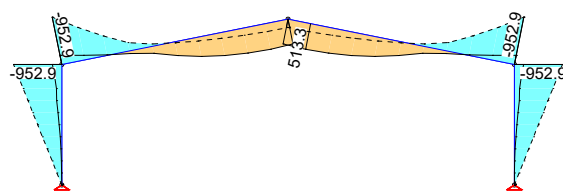
All of the available results are listed in the **Results** folder in the *database*. Double-click a result to open the available or designated standard view (graphic or table) of that result.

For instance, open the **Node deformations** folder and then double-click Load case 3. The deformed system will be displayed.

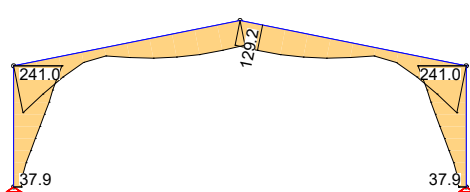


Click the *Results* button to access various display settings.


Internal forces and comparison stresses from the *1. permanent and temporary situation*:

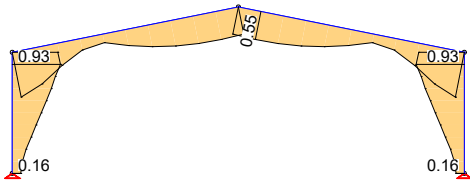



LCC EN1993.SV.1: 1. Permanent and temporary situation, EN 1993-1-1  
Internal forces min,max My. 1043.77 [kNm] =  $\frac{1}{4}$   
Value range (overall system, min/max): -952.9/1043.77 [kNm]

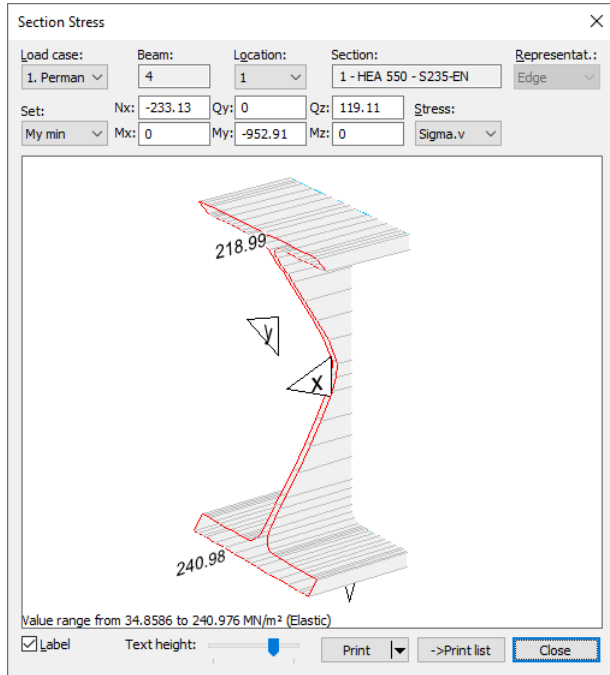


LCC EN1993.SV.1: 1. Permanent and temporary situation, EN 1993-1-1  
Stresses (steel checks) max Sigma.v. 263.95 [MN/m²] =  $\frac{1}{4}$   
Value range (overall system, min/max): 37.9/263.95 [MN/m²]

You can activate the calculated utilizations in the  *Stresses / Steel Checks* folder. Note that the depicted frame can only be checked if the plastic internal force limits are taken into account.




LCC EN1993.SV.1: 1. Permanent and temporary situation, EN 1993-1-1  
 Utilization: 1.02 [-] =   
 Value range (overall system, min/max): 0.16/0.93 [-]

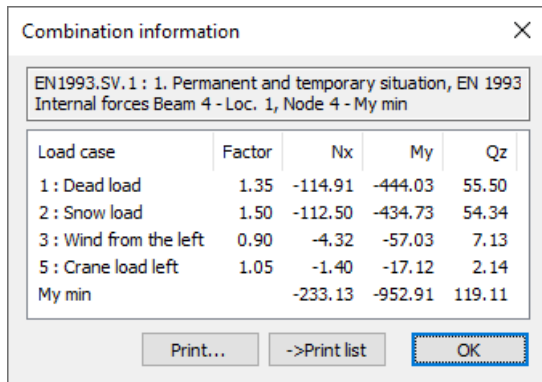


In the stress view right-click any beam and select *Section Stress* from the shortcut menu.

The stresses at the section will now be applied for the selected location.

In the depicted window you can select the results for all load cases and combinations as well as for  $\sigma_x$ ,  $\sigma_y$ ,  $\tau_{xy}$  and  $\tau_{xz}$  stresses.

The checking program records the calculation process in a concise log. You can view the log in the  *Listings* folder.



The combination information shows you the individual factors contributing to an internal force result.

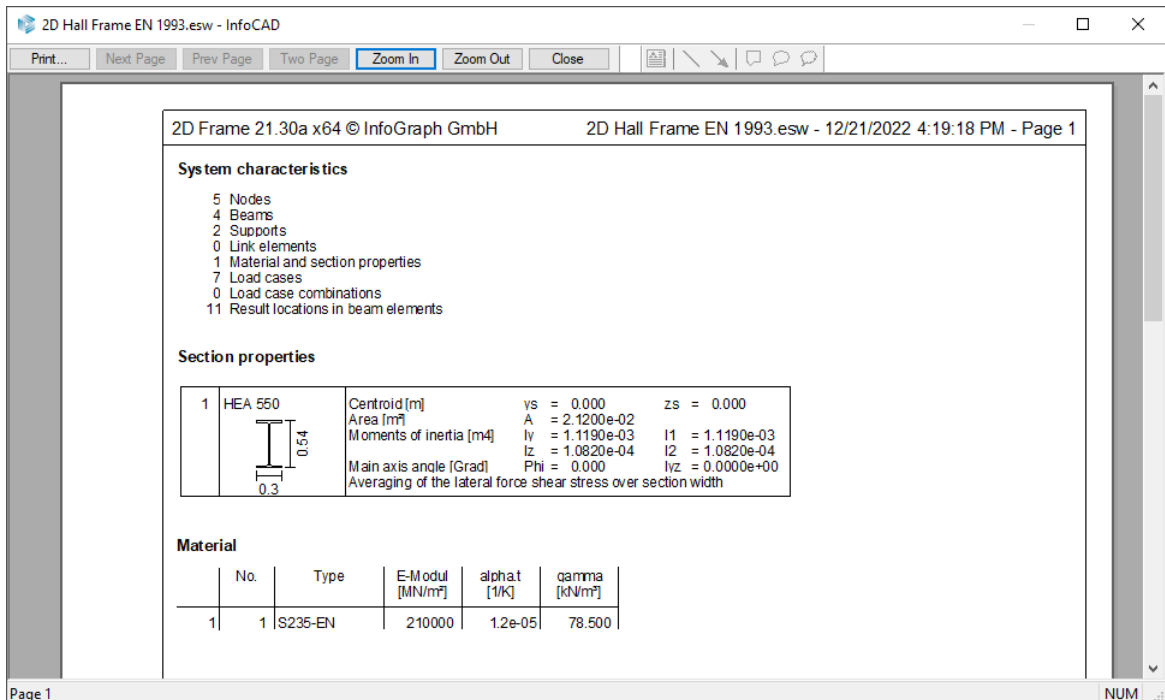
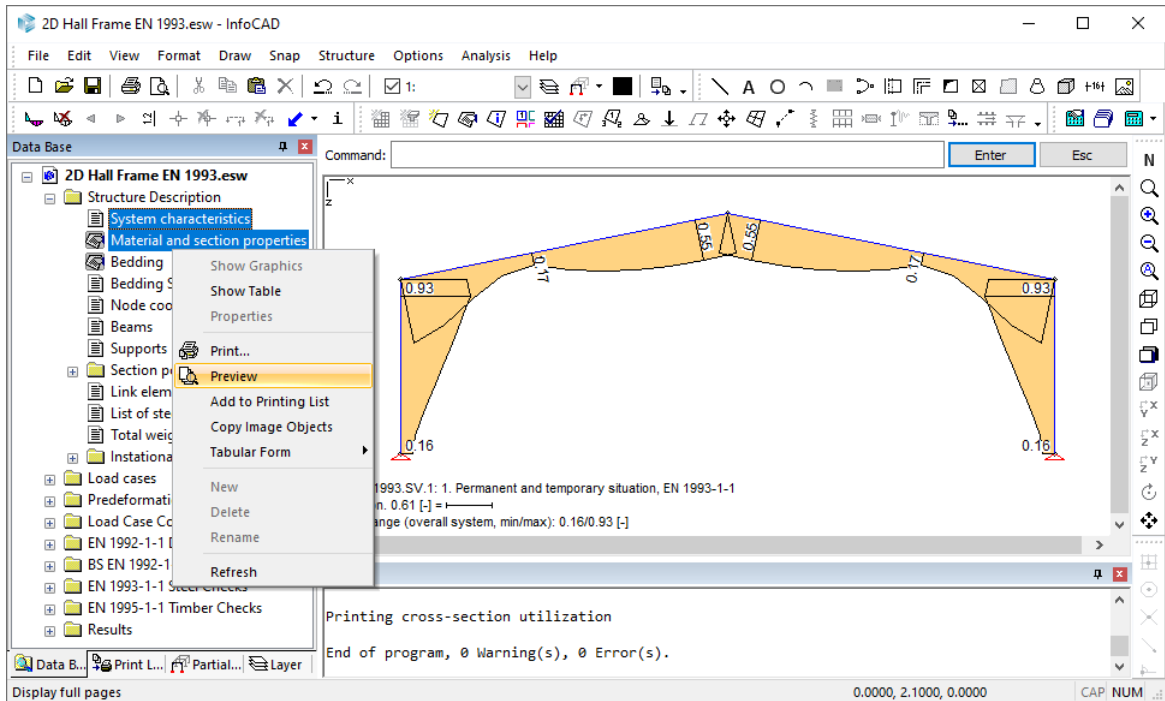
In the results view right-click the desired results location in the beam and select *Combination information* from the shortcut menu.

## Printing



You always have the option of printing the current screen view. Click *Print Preview* to see what the actual printout will look like. The settings for the page format, printer selection, etc., can be modified using the *Page Setup...* command in the *File* menu.

Tables or logs need to be selected in the workspace and then printed out from the shortcut menu (right-click). The *Page Preview* command is also available in the shortcut menu.



## Printing List

Specified views as well as tables and logs can be added to the printing list for printing at a later time. Only references to the specified display parameters are stored, meaning changes made to the system are automatically reflected in the list.

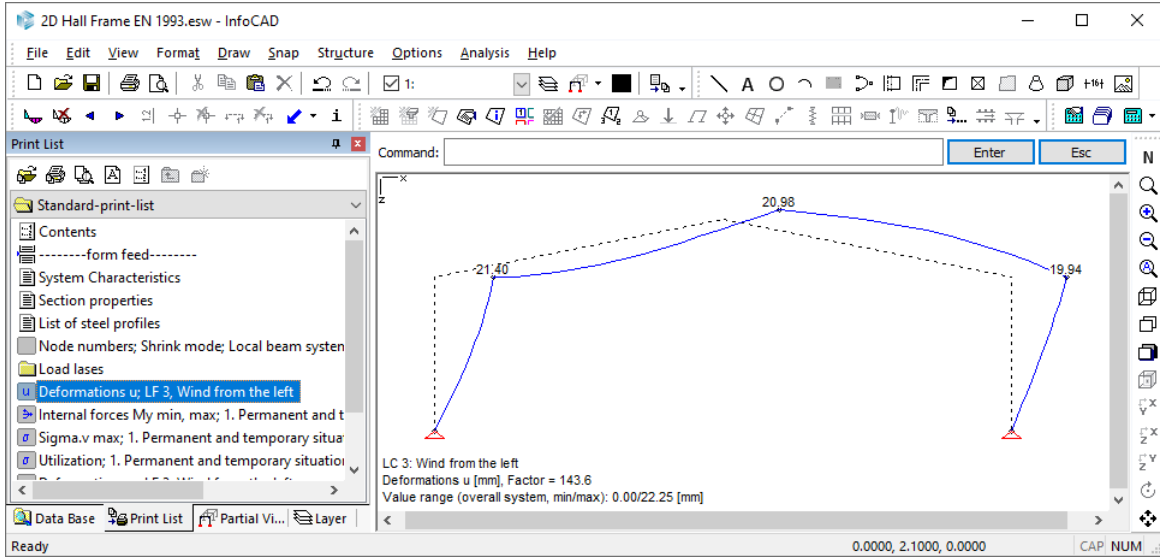
Switch from the *database* to the *printing list*. The printing list has its own *Print* and *Page preview* commands.



Click the *Add to printing list* button to copy the current system view into the printing list. All active settings will be saved with the system view.

Use the shortcut menu to add tables or logs selected in the database to the printing list. You can also add separate text objects, page breaks or a table of contents.

You can sort existing entries in any order you want by simply dragging them with the mouse.

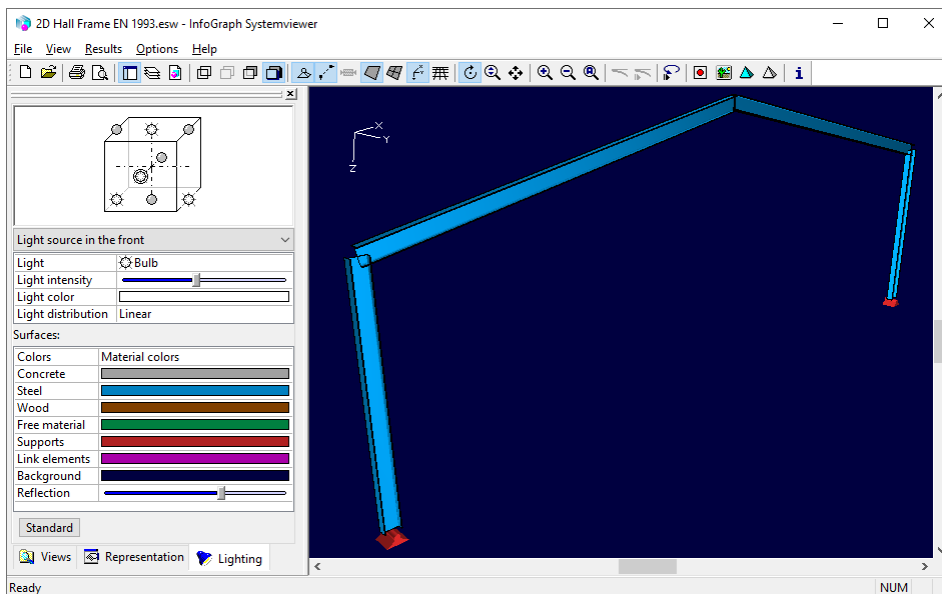


Double-click an entry in the printing list to open the corresponding view in the representation area.

## InfoGraph Systemviewer

The InfoGraph Systemviewer allows you to visualize FEM and framework projects as well as to animate deformations from static or dynamic calculations.

Launch the Systemviewer from the *Programs / InfoGraph* folder in the *Start* menu and open the file you want to view. A variety of settings options are available to help you optimize the graphical output.



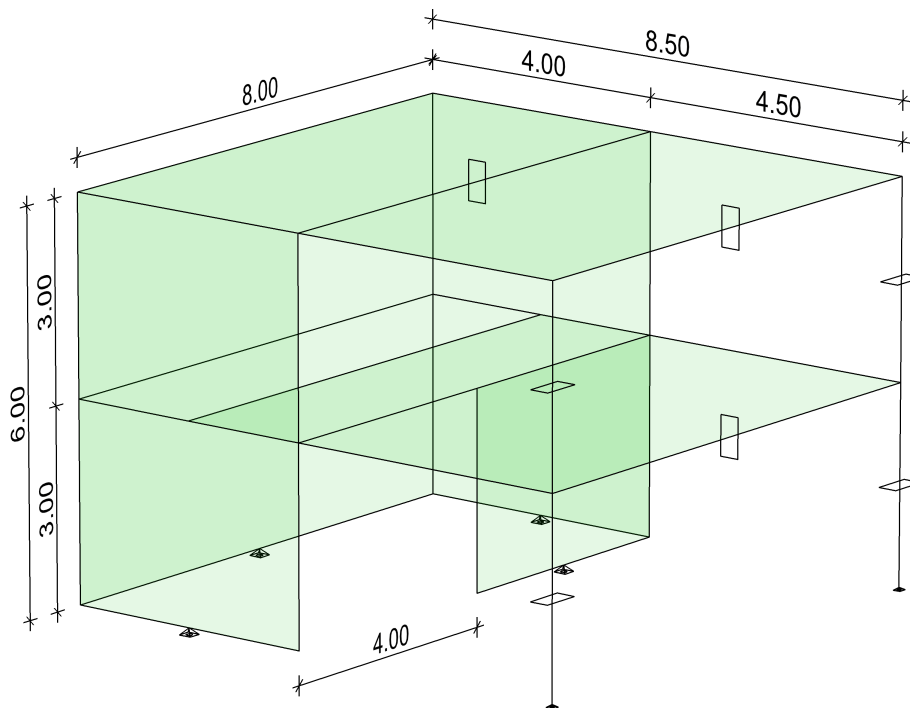
## Example 3: 3D Building

This example shows you how to specify a 3D building and perform the following calculations:

- Static calculation
- Design according to EN 1992-1-1

It is assumed that you have already gone through the previous examples. Hence, some of the processing steps will not be detailed in this example.

### Task



System axes with dimensions [m]

Material: C20/25-EN (Concrete EN 1991-1-1)  
Reinforcing steel BSt 500, axis distance of 3.0 cm from edge


Load: Dead weight, area load, horizontal load


#### Procedure to enter 3D structures:

1. Describe a wire frame model using edges
2. Assign the wanted properties to the edges (support, beam series, free beam)
3. Enter model faces
4. Mesh generation

### Drawing the wire frame model

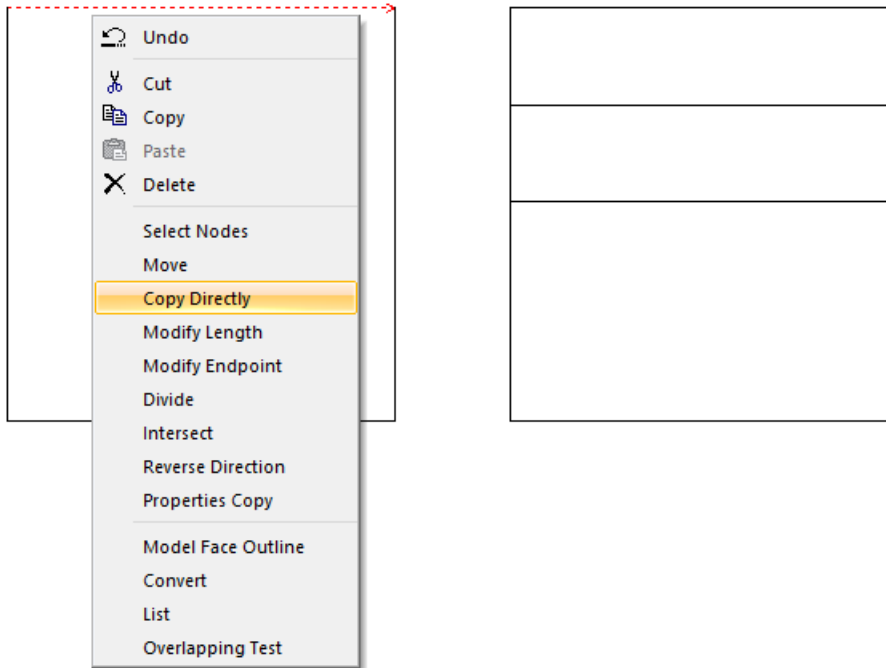
Launch the program or start a new project by clicking *New* and then selecting **Finite Elements** under the *Structure* menu.

 Click the *Edge* button and begin by entering:  
[0 0]

 Activate relative coordinate input and complete the outer edge of the 1<sup>st</sup> level:  
[8 0], [0 8.5], [-8 0]

Point to the start of the 1st edge to connect the lines. End the edge input with *Enter* or *ESC*.

Select the upper edge with the right mouse button and choose *Copy directly* from the shortcut menu.



The program asks for the displacement vector. Select e.g. the upper left end of the edge to define the start point. Now activate the option *Multiple*. For the second point of the displacement vector enter the coordinates [0 2] and [0 4]. The edge will be copied 2 and 4 meters in y direction. Finish the copy process with *Enter* or *ESC*.



Now switch to the 3D view and create the other levels using the copy function.

To do so, select the edges by dragging a box over all the objects. The selected objects will now be displayed with dotted lines.

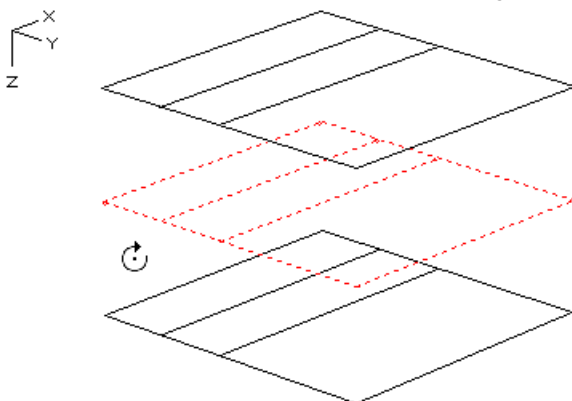
Select *Copy directly* in the *Edit* menu and specify the displacement vectors:

1. Point of displacement vector: [0 0 0]
2. Point of displacement vector: [0 0 3]
2. Point of displacement vector: [0 0 -3]

For 3D editing, always enter 3 coordinates in the order x, y and z. Based on the above coordinates, the displacement vectors point +/- 3 meters in z direction.

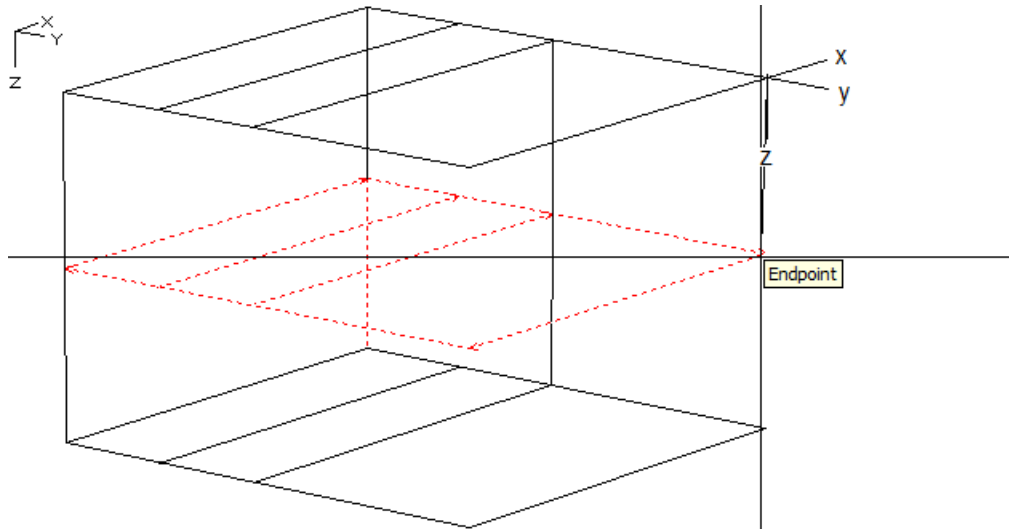


If you want to temporarily change the angle of view, click *Rotate with mouse* and move the cursor while holding down the left mouse button. Click the button again to stop the rotation.



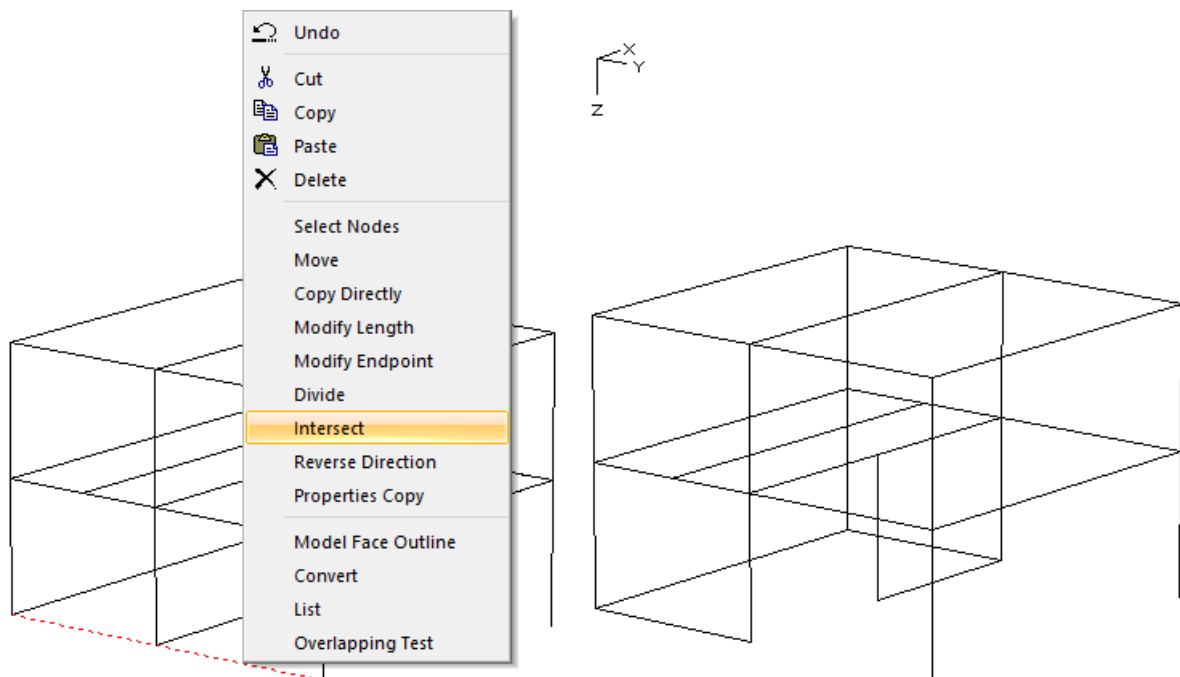
You can adjust the view to fit the new object dimensions using the *Zoom all* button.

- Enter the missing edges between the levels. It can be referred to the existing points with the snap modes *mid-point*, *endpoint* or *intersection*. In this way, further manual coordinate input can be avoided.



- Select the unnecessary edges and *delete* them.

To shorten the lower edges select these edges with the right mouse button and choose the option *Intersect*. For border object select the relevant edge.

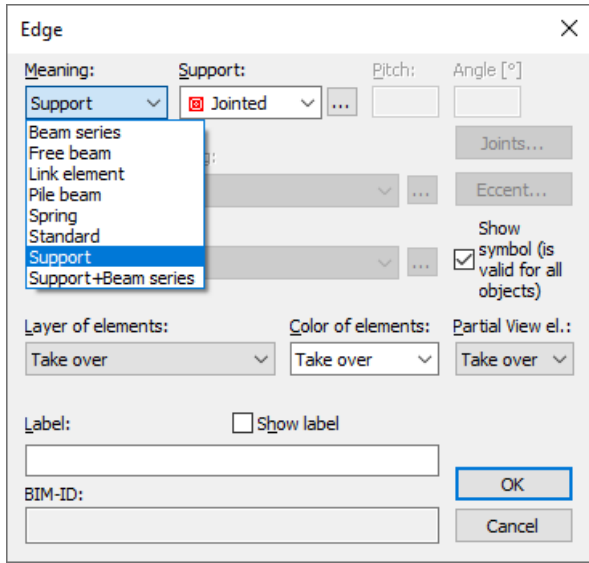


- In order to create supports at the columns later we use the model object *Column* with the option *Point* at the foot points of the columns.

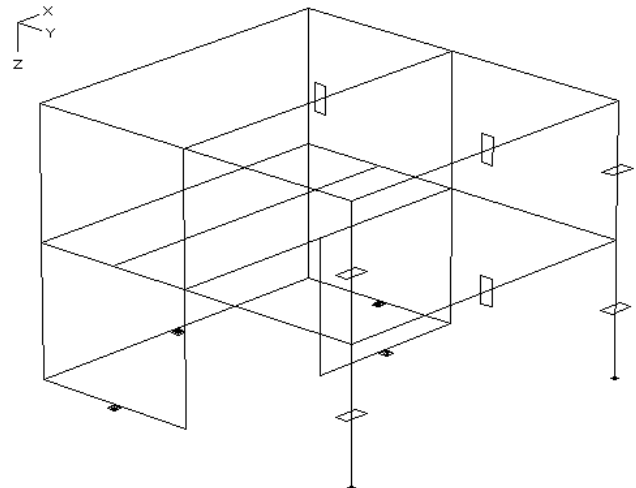
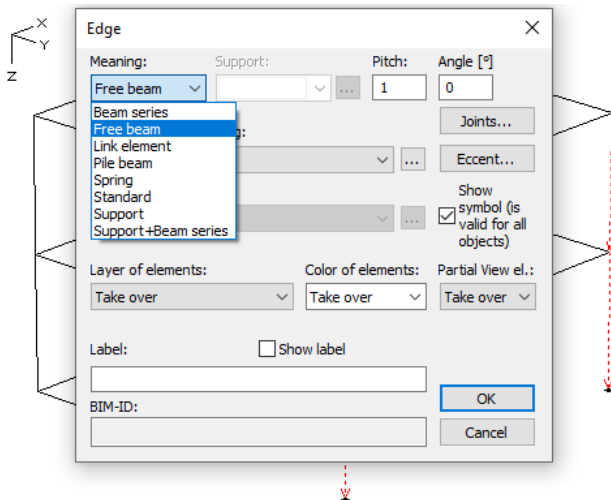
The wire frame model is completed.

## Assigning Properties

Additional properties can be assigned to the edges to be taken into account while generating the mesh. Double-click one or more edges to define the properties and set the wanted *meaning*:



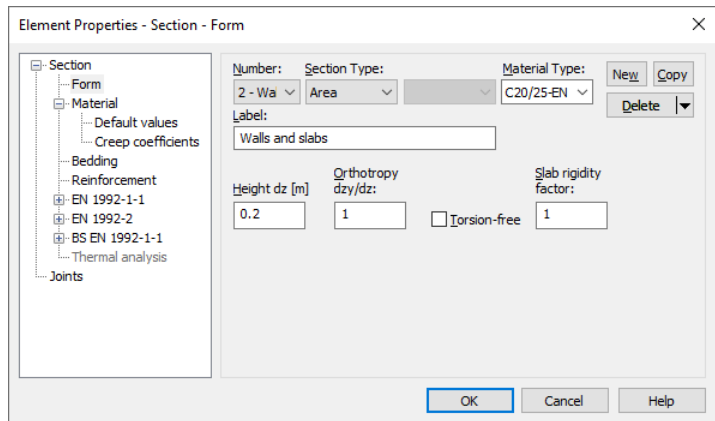
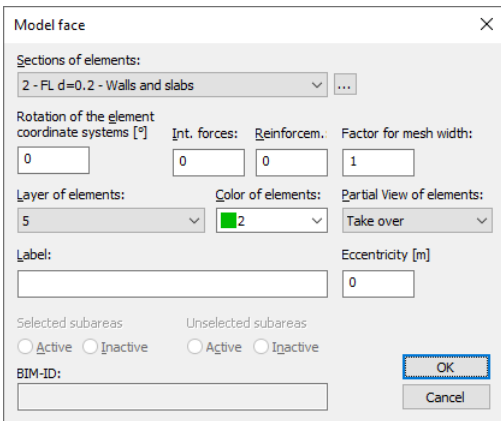
Meaning	Result from the mesh generator
Standard	Area limitation or fixed line inside
Support	Line support on the edge
Beam series	Beam elements connected to the FE mesh, e.g. downstand beams
Free beam	Beam elements with independent intersection, e.g. columns or parts of frames
Support +Beam series	Line support and beam series



For the columns set *Free beam* with *pitch* = 2. Start the section dialog and define the polygon section with the function ... (e.g.: Rectangle 30x60).

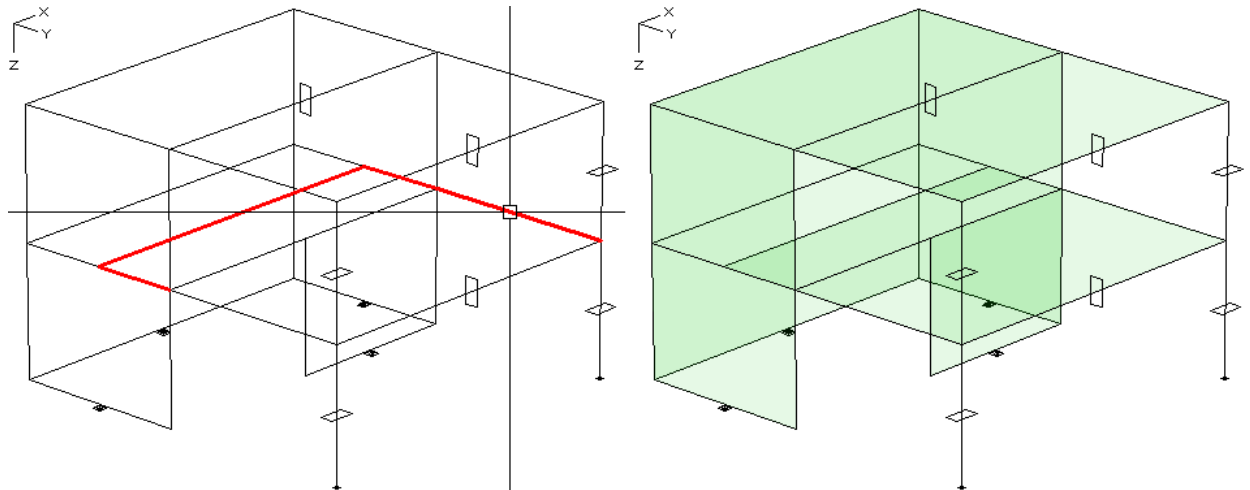
Assign the property *Support* to the supported edges and *Beam series* with the polygon section to the outer downstand beams.


Click the *Face* button and select the *Layer* and *Color* you want to use for the element mesh before defining the area (*Properties* option). Double-click inside the section window to open the section dialog. Set the properties for the area section and leave the dialog.





Now select the limiting objects for the first area. An area will be created as soon as it is fully defined by the selection. Change the *color* now and again if necessary.

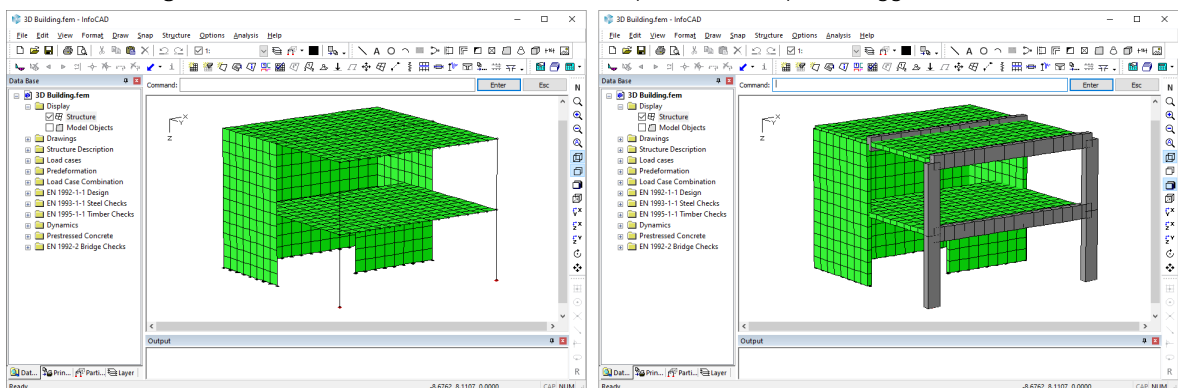



 The geometric description of the system is now complete. Save the file and enter a name for the drawing.


## Generating an Element Mesh

The model structure that is created serves as the basis for FEM mesh generation.




 Click the *Mesh generation* button, select the *Form-sensitive* option and accept the suggested mesh width to 0.5 [m].




 Click the *View options* button to access various settings for how the system is displayed. In the resulting dialog, activate the *Area fill of the elements* option to improve the 3D quality of the structure.

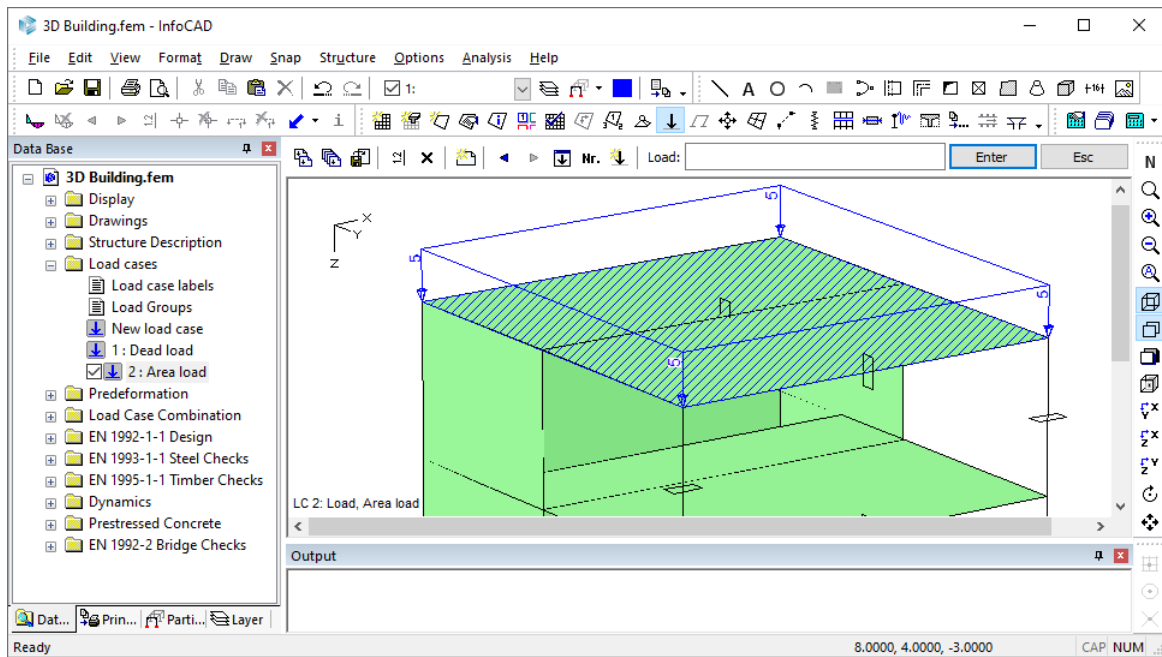
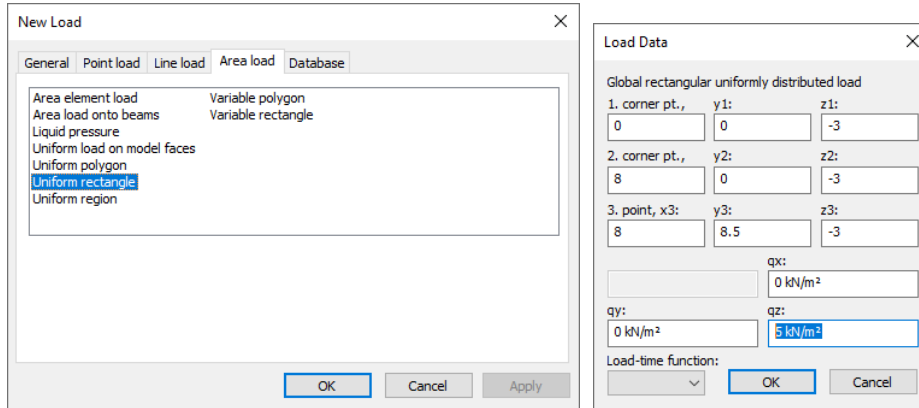
 Check the structure using the *section representation*.

## Defining Load Cases

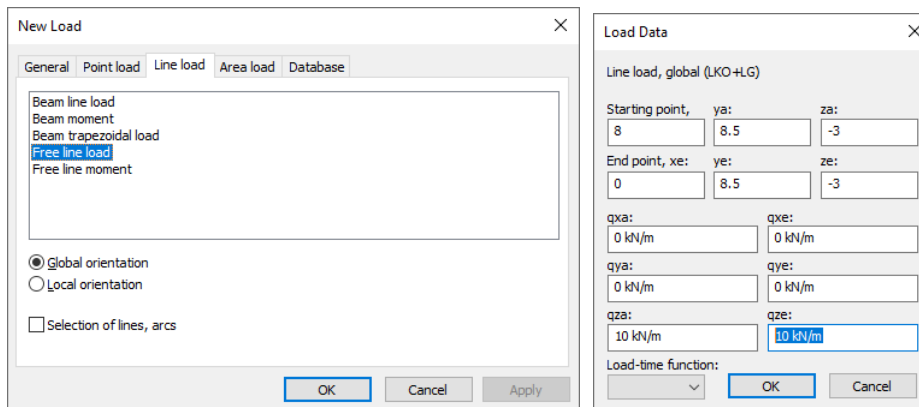
 Open the  *Load cases* folder in the database and double-click  **New load case**. A new load case will be created automatically and the load input and editing buttons will appear in the dialog bar.

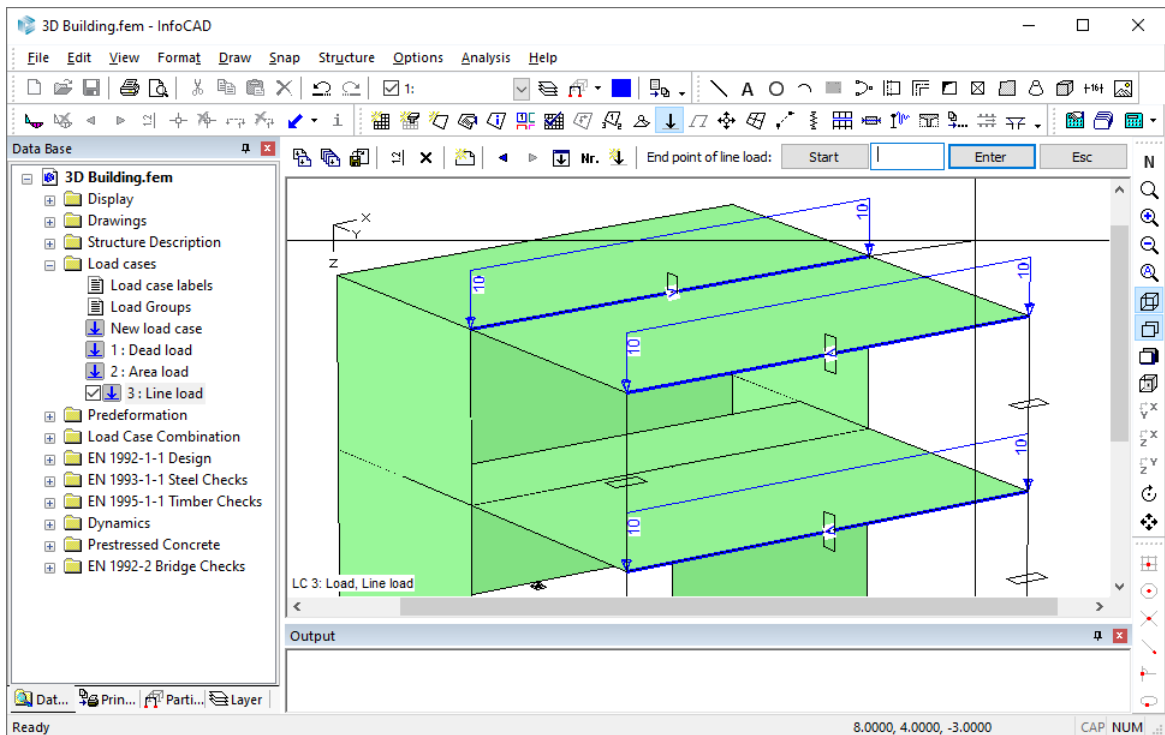
**Nr.** For load case 1, select *dead load* with weighting 1 in the z direction, confirm your selection and then enter a load case label using the *Change Number* button.

Click  **New load case** in the database again to create the new load case 2. Select the area load *Uniform rectangle* and define two rectangle area loads on the floors. When entering coordinates refer to existing nodes and use the functions *Zoom* and *Rotate with mouse*.



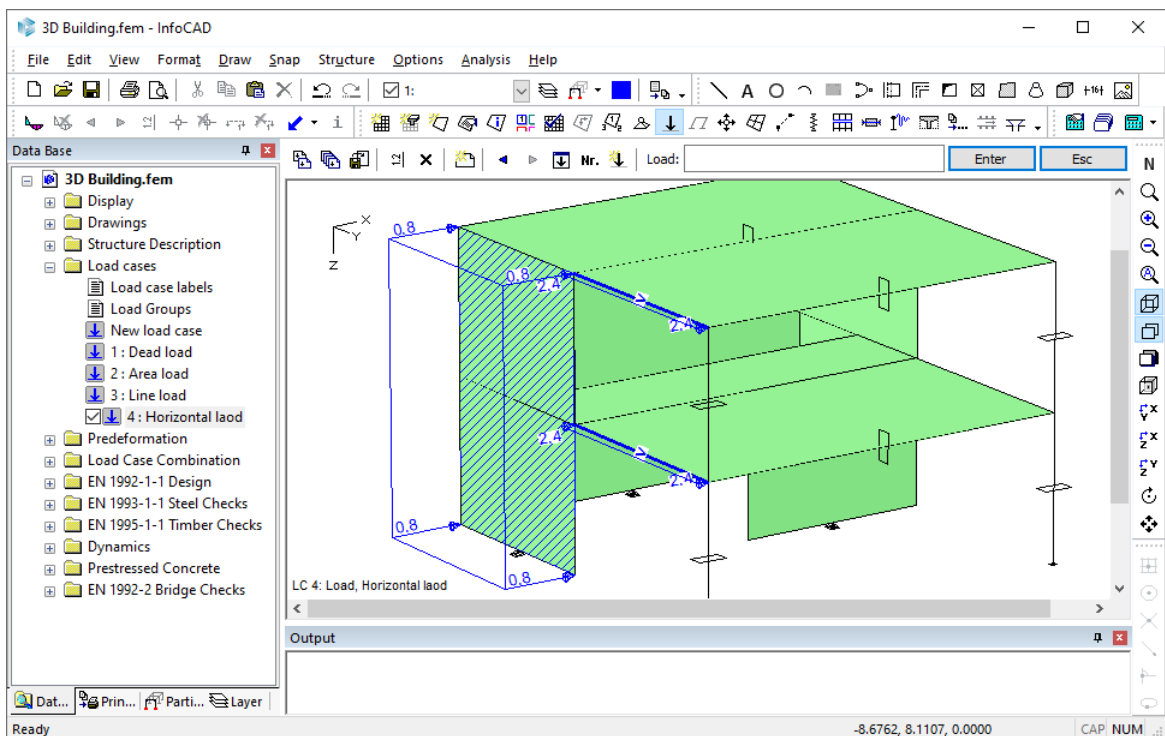
For load case 3, select Line load for area elements with global orientation.





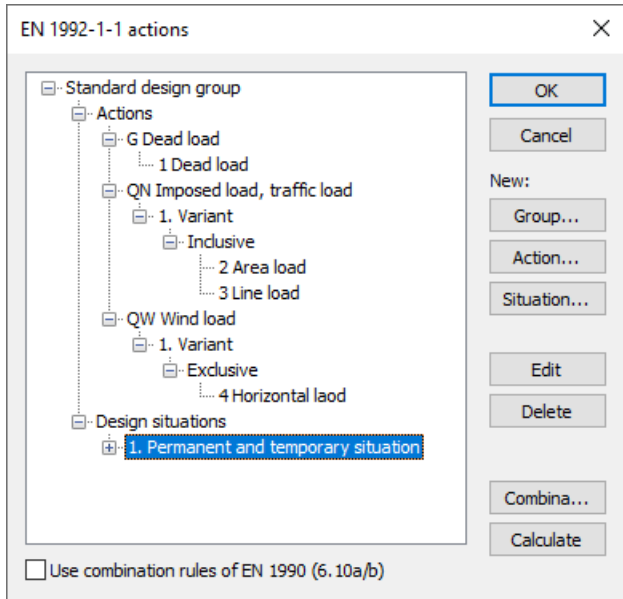
1.2 With the *Load representation* option you can scale the load ordinates.

For the fourth load case create two Line loads with 2.4 kN/m and one area load with 0.8 kN/m<sup>2</sup> in global x direction.



Load input is now complete and you can exit the *Load* dialog.

## Describing Actions

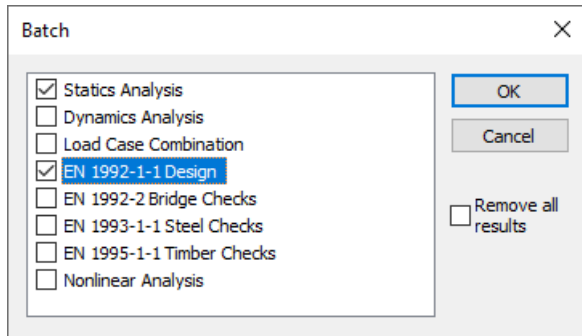


Open *Actions* in the *EN 1992-1-1 Design* folder of the database, assign the following actions to the load cases and create a *Permanent and temporary situation*.

- |    |                                |  |
|----|--------------------------------|--|
| G  | Dead load:                     | Load case 1 (dead load)                  |
| QN | QN Imposed load, traffic load: | Load case 2 and 3 (area load, line load) |
| QW | Wind load:                     | Load case 4 (horizontal load)            |

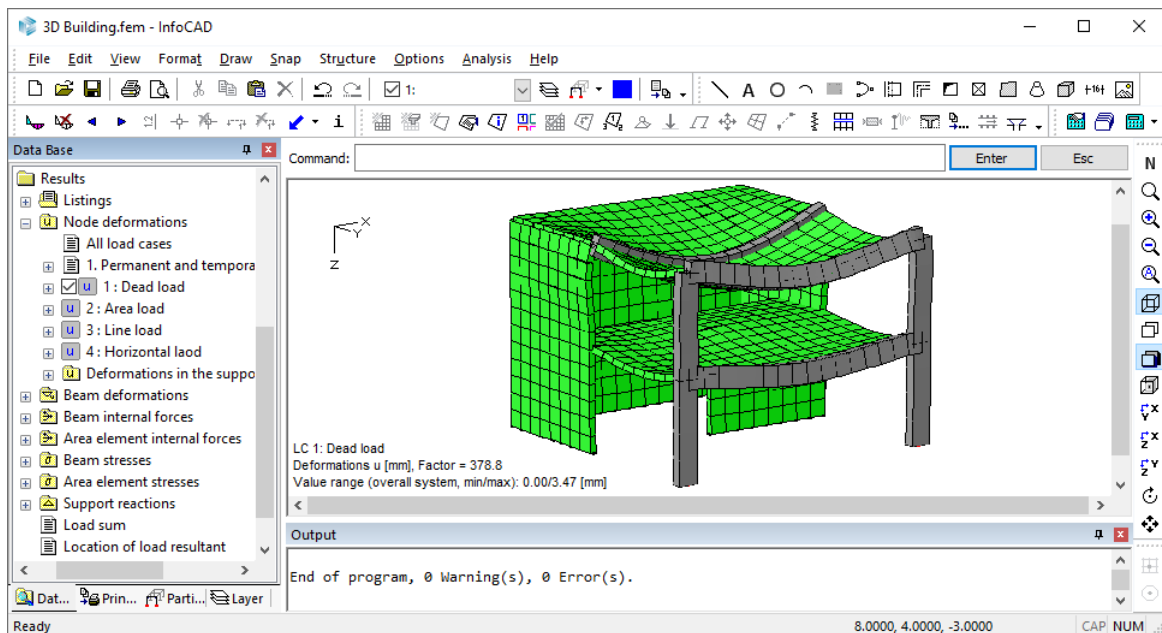
## Calculation and Results

Begin the *batch* calculation with the options selected as shown below.

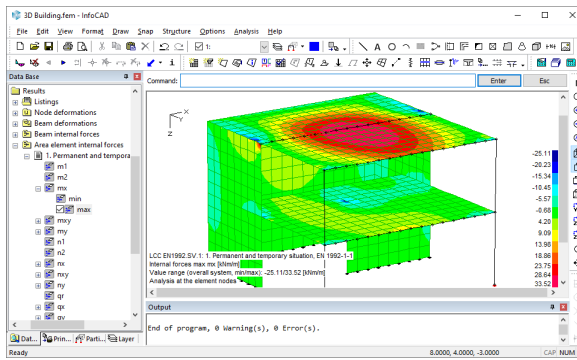


The selected calculations are performed automatically in consecutive order. You can follow the progress of the calculations in the *output bar*.

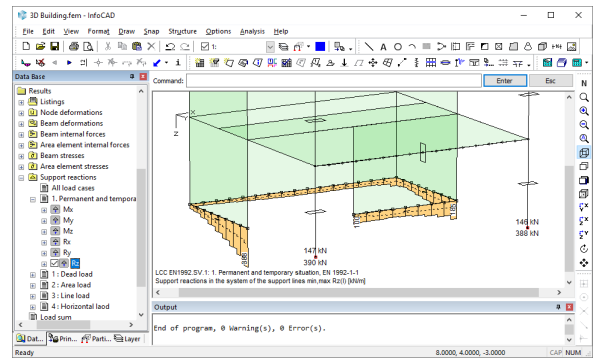
Once the calculation is finished, all the results are available in the database. Open the results category you need and select the appropriate view.



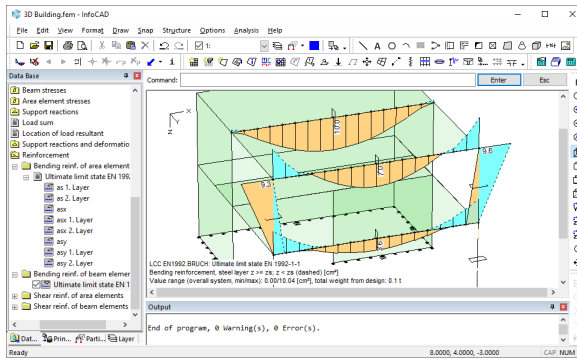
*Deformations load case 1*



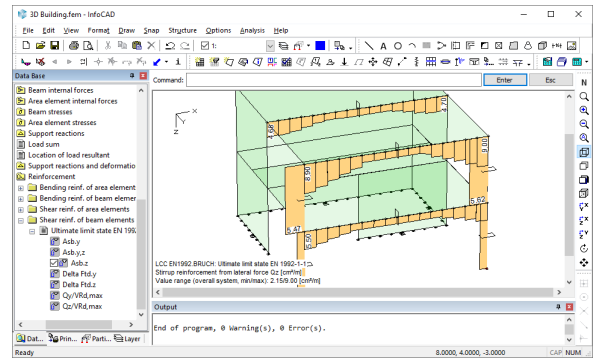
Colored progression of the moments  $max\ mx$



Support reactions  $Rz$



Longitudinal reinforcement of the beams

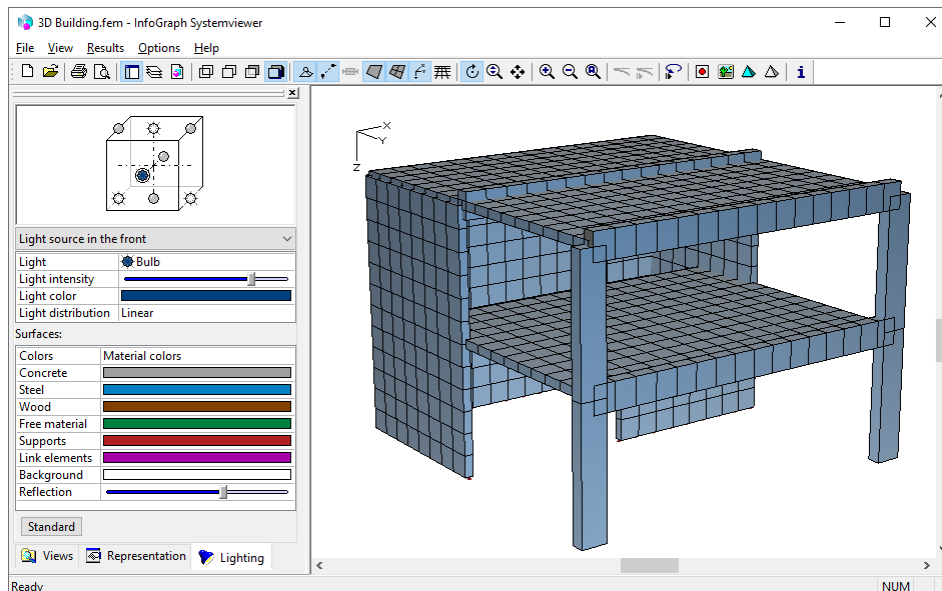


Lateral reinforcement of the beams and columns

## InfoGraph Systemviewer

The InfoGraph Systemviewer allows you to visualize FEM and framework projects as well as to animate deformations from static or dynamic calculations.

Launch the Systemviewer from the *Programs / InfoGraph* folder in the *Start* menu and open the file you want to view. A variety of settings options are available to help you optimize the graphical output.



## Example 4: Prestressed Roof Girder

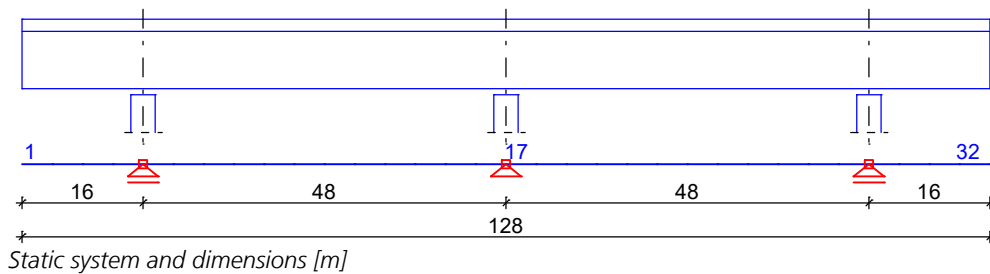
This example involves a wide-spanned roof construction that is represented as a continuous girder over two spans with a double-sided cantilever. To edit this system, perform the following work steps:

- Enter a static system
- Enter a tendon group
- Enter the loads
- EN 1992-1-1 Design
- Display the results

The structure is constructed for exposition class XC1. In accordance with EN 1992-1-1, Table 7.1N no check of decompression is necessary for this exposition class.

It is assumed that you have already gone through the previous examples. Hence, some of the processing steps will not be detailed in this example.

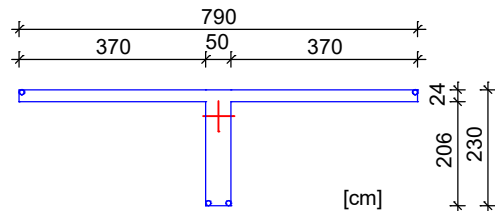
### Task



### Material

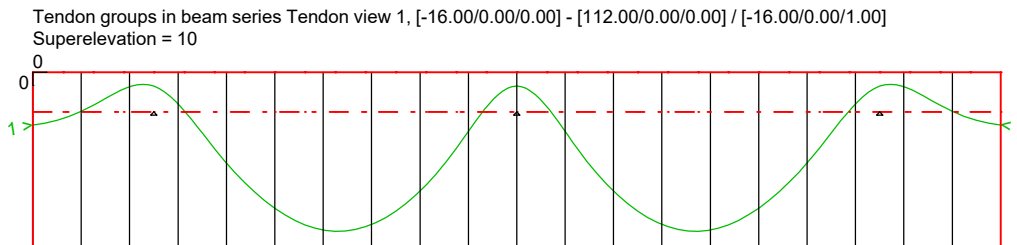
Concrete C45/55-EN  
 Reinforcing steel BSt 500, axis distance from edge 5 cm

### Section



### Tendon groups

Four bundled tendons are arranged in this example. The tendon group guide is shown in the next figure. The depicted tendon group ordinates  $z_v$  of the spline points refer to the upper edge of the section.



Tendon group ordinates  $z_v$  [cm] at the base points

0.00	6.40	12.80	19.20	25.60	32.00	38.40	44.80	51.20	57.60	64.00	70.40	76.80	83.20	89.60	96.00	102.40	108.80	115.20	121.60	128.00	
1	70.0	52.0	19.2	42.0	120.2	180.1	208.8	201.5	157.4	78.3	18.5	78.3	157.4	201.5	208.8	180.1	120.2	42.0	19.2	52.0	70.0

**Prestressing steel and prestressing system**

Prestressing steel quality	St 1500/1770
Certification of the prestressing system	EC2
Number of tendons in the bundle	4
Section surface area $A_z$	1800 mm <sup>2</sup>
E-modulus of the prestressing steel	195000 MN/m <sup>2</sup>
0.1% strain limit (yield strength) of the prestressing steel $f_{p0,1k}$	1500 MN/m <sup>2</sup>
Tensile strength of the prestressing steel $f_{pk}$	1770 MN/m <sup>2</sup>
Permitted prestressing force of the tendon $P_{m0}$	2295 kN
Prestressing loss from relaxation of prestressing steel	4.5 %
Friction coefficients when increasing or releasing strain	0,2
Unintentional angle displacement of a tendon $\beta'$	0.3 °/m
Slippage at prestressed tie bolt	6 mm
Duct diameter	82 mm

**Scattering coefficients of the internal prestressing**

Construction stage ( $r_{sup} / r_{inf}$ )	1.1 / 0,9
Final state ( $r_{sup} / r_{inf}$ )	1.1 / 0,9

**Loads**

Load case 1	Dead load (G1)
Load case 2	Additional loads 11.06 kN/m (G2)
Load case 3	Snow load 7.90 kN/m (Q)
Load case 10	Prestressing (P)
Load case 15	Creep-generating continuous load: G1+P+G2
Load case 20	Creep and shrinkage (CSR) Coefficients: $\varphi_{t\infty} = 2.55$ ; $\rho = 0.8$ ; $\epsilon_{cs,t\infty} = -24.8 \cdot 10^{-5}$ Creep-generating continuous load case: 15 The redistribution of internal forces between concrete and prestressing steel are taken into account.

**Static System**

Launch the program or start a new project by clicking *New* and then selecting **Finite Elements** from the *Structure* menu.

To accurately measure the prestressing and the creep and shrinkage behavior, first divide the structure into beams with a length of 4 meters. Specify the supports based on a fork support of the system.



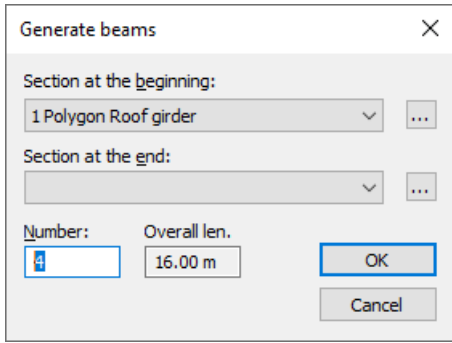
First switch to the xz view.



Click the button *Element creation* and select the element type *RS Beam*. For automatic division activate the option *Generate*. A crosshair will appear in the representation area and you will be prompted to enter the coordinates of the starting point in the dialog bar. Enter [0 0] to position the starting point at the origin. After you press [↵] or *Enter*, the end of generation line will be queried.



To enter additional coordinates, you should activate relative coordinate input by clicking the *Relative* button. A small axis system will now always appear at the last point inserted. All successive coordinates you enter always refer to this point. Enter [16 0] to define the end of the generation line in x direction.



Enter 4 for the number of beams which shall be generated. Prestressed beams must have the cross-section type *Polygon* or *Composite*.

To temporarily enlarge the view, click *Zoom* - or *Zoom all*.

Now enter continuously the remaining section coordinates and choose the number of beams to be generated:

[48 0] - 12 Beams; [48 0] - 12 Beams; [16 0] - 4 Beams

Finish the beam input with [ ↵ ] or *Enter*.

Now click the *Supports* button and choose the *Point support* option. You will be prompted to select the support nodes in the dialog bar. You can click the nodes directly with the crosshair. During the support input take care to support the system free of restraint in the longitudinal direction and to define a fork support.

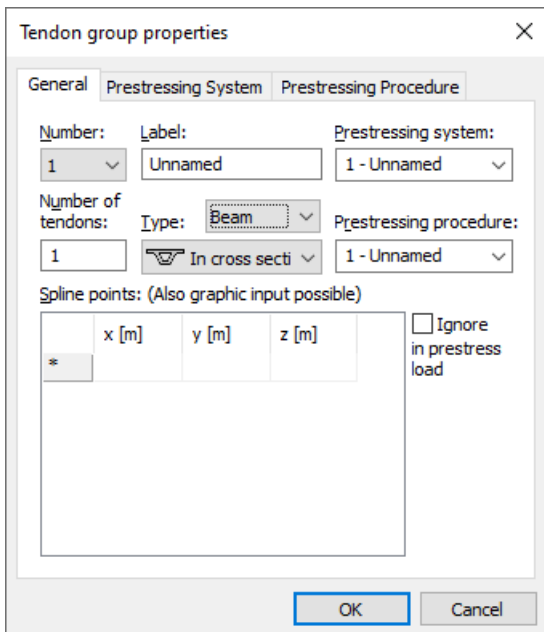


### Entering a Tendon Group

The definition of the tendon groups does not depend on the elements. The assignment and force introduction onto the structure occur while the *Prestressing* load case is being calculated. To avoid any conflicts, a distinction is made between beam and area/solid prestressing.

The tendon curve on which the calculation approach is based is represented by a 3D cubical spline function. The spline function is the curve that runs through all specified spline points with the least amount of curvature. The basis for the prestressing analysis is provided by a tendon group force curve that allows for strain increase, release and slippage. To include prestressing in the FEM analysis, you need to define a load case with the *Prestress* load type.

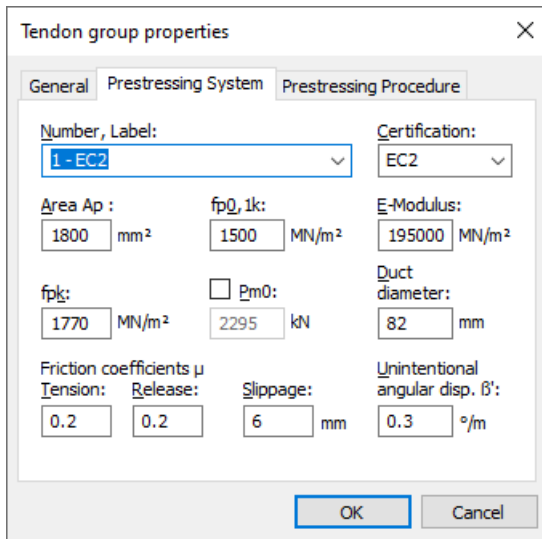
Use the *Prestressing* function to enter and edit tendon groups.



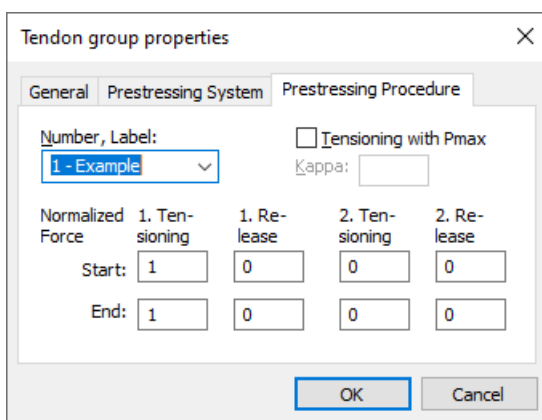
Click *Define* and then select *Beam* for the tendon group type in the resulting dialog.

Normally, the spline points are defined graphically.





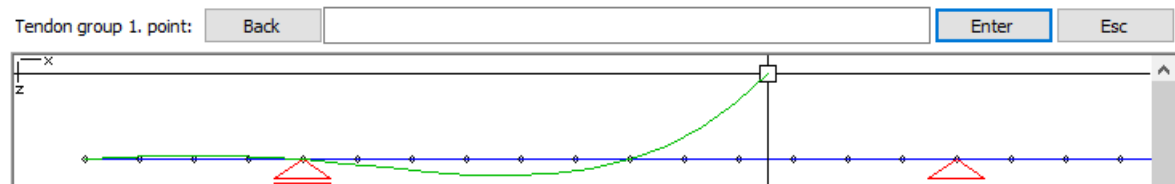
The *Prestressing System* tab contains all the properties that are assigned to the tendon groups. They apply to each individual tendon of the group.



If the allowance value **K** is used, then the factor for over-stressing refers to the maximum permitted force  $P_{max}$ .

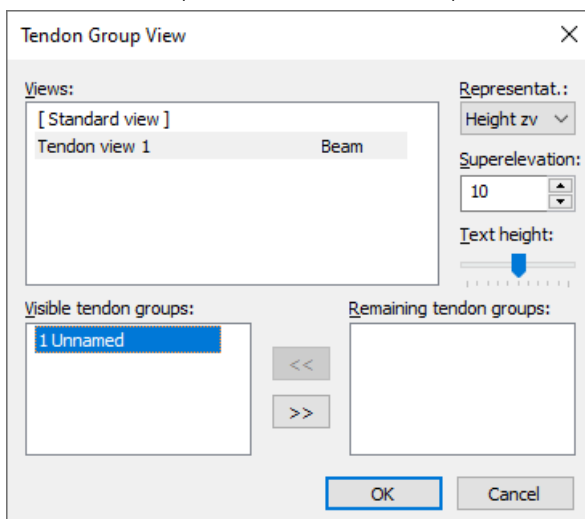
**Using the factor specified for the release, the maximum prestressing force remaining in the tendon group is defined with respect to  $P_{m0}$ . A factor of '0' means no release.**

Exit the dialog with *OK* and enter the spline points of the tendon group (start, supports, midspans) at the nodes of the beam array.

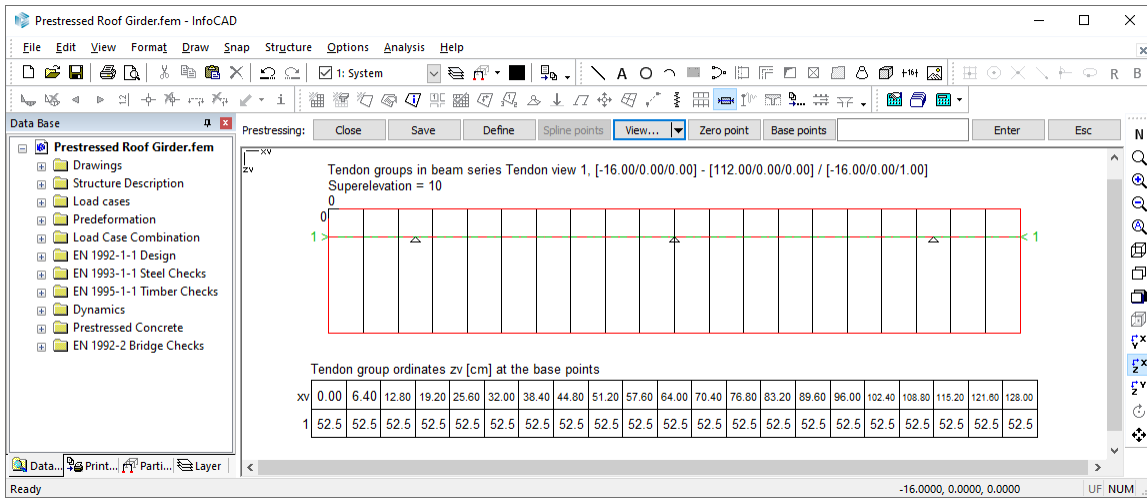


When you are done, click *Enter*. The tendon group no. 1 is now positioned along the centroidal axis of the beam array.

Additional editing should take place in a section view. To access a section view, select the entire beam series with the *View... / Beam* option and define the view plane from the first node to the last node in the global z direction.

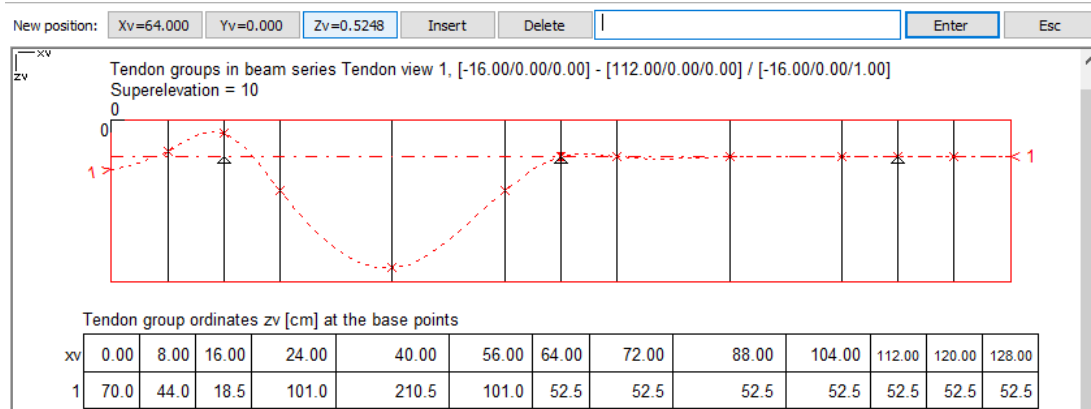


Add tendon group 1 to the *Visible tendon groups* list.

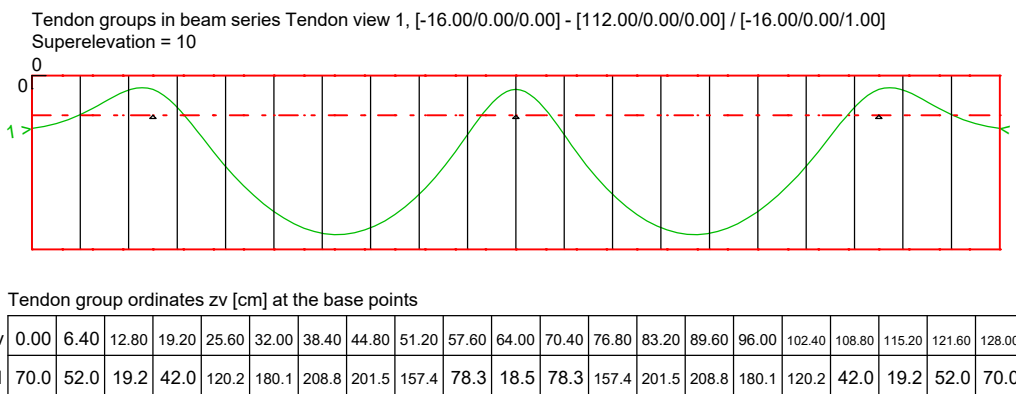


The tendon group is now displayed in the selected section. Next, use the *Zero point* option to move the position of the reference system from the centroid to the upper edge of the section and then in the *Base points* option select *Spline points*.

If the tendon group is selected, you can reposition it by first clicking the existing spline point and then the *Spline points* option. Only change the respective *Zv* position. The *Insert* option allows you to add additional spline points before the active spline point.



Use this method to assign the desired geometry to the tendon group.



In the tendon group view, the coordinates always refer to the reference system.

Click the *Representation* option to activate the resulting prestressing force curve in the *tendon group view*:

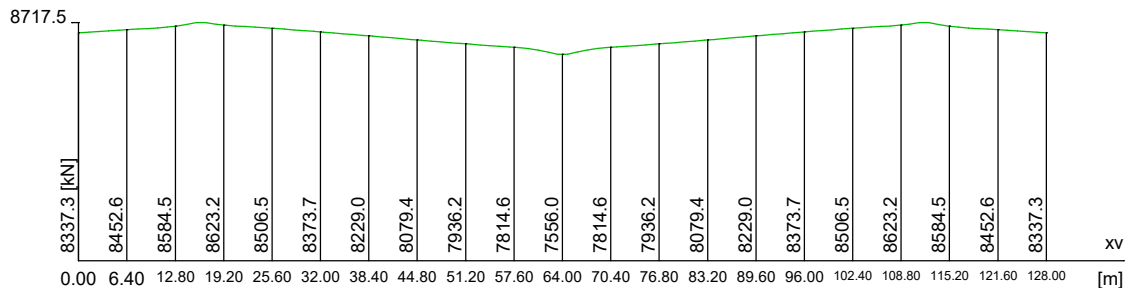
Force function of tendon group 1 (4 tendon(s), l = 128.41 m)

Prestressing system 1 - EC2. Certification according to EC2.

Pm0 = 2295.0 kN, Ap = 1800.0 mm<sup>2</sup>, μa = 0.20, Angle β' = 0.30 °/m  
E-Modulus = 195000 MN/m<sup>2</sup>, Ah = 5281.0 mm<sup>2</sup>, μn = 0.20, Slippage = 6.00 mm

Prestressing procedure 1 - Example

Pre. anchor	:	Start	End
Normal. force	:	1.000	1.000
Pre. force [kN]	:	2295.0	2295.0
Extension [mm]	:	694.5	72.8



## Loads and actions according to EN 1992-1-1

Enter the following loads:

Load case 1 *Dead load (G1)*

Load case 2 *Additional loads 11.06 kN/m (G2)*

Load case 3 *Snow load 7.90 kN/m (Q)*

Load case 10 *Prestressing (P)*

Load case 15 *Creep-generating continuous load: G1+P+G2*

Load cases 1, 2 and 10 are grouped into this load case with the *Insert* load type.

Lastfall 20 *Creep and shrinkage (CSR)*

Coefficients:  $\varphi_{t_{\infty}} = 2.55$ ;  $\rho = 0.8$ ;  $\varepsilon_{cs,t_{\infty}} = -24.8 \cdot 10^{-5}$

The *Creep and Shrinkage* load type lets you calculate the redistribution of internal forces between the concrete and prestressed steel. Specify load case 15 as a creep-generating continuous load case.

The following checks are carried out as a part of this example:

### Checks in the Ultimate Limit States

- Bending with or without normal force or normal force only
- Lateral force
- Minimum reinforcement against failure without warning

### Checks in the Serviceability Limit States

- Limiting the concrete compressive stresses
- Limiting the reinforcing and prestressing steel stresses
- Minimum reinforcement for crack width limitation with  $f_{ctm} = 1$ ;  $k = 1$ ;  $\xi_1 = 0.27$
- Crack with calculation

**Element Properties - EN 1992-1-1 - Checks**

Number: 1 - Roc | Section Type: Polygon | Material Type: C45/55-EN

Label: Roof girder

Prestress of component: Subsequent bond | Exposure class: XC1

Ult. limit state:

- Bend, long. force
- Lateral force
- Torsion
- Robustness
- Shear joint

Fatigue:

- Reinf., prestr. steel
- Lateral force
- Torsion
- Concr. long. stress
- Concr. comp. strut

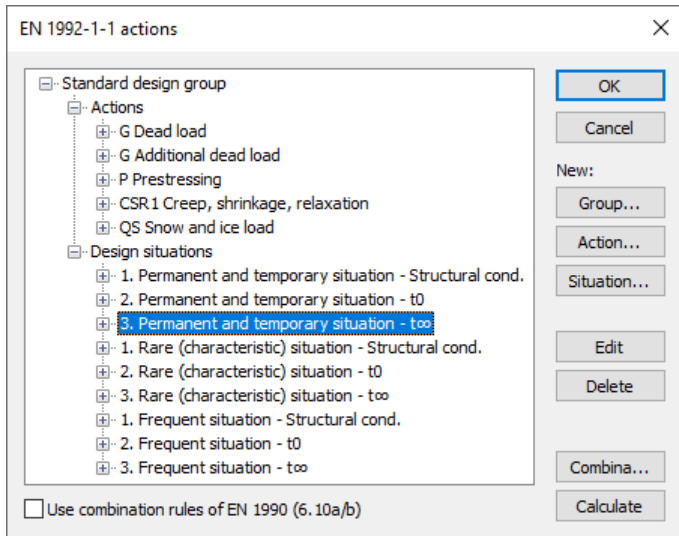
Serviceab. limit state:

- Concrete comp. stress
- Decompression
- Minimum crack reinf.
- Reinf. tensile stress
- Crack width

Buttons: OK, Cancel, Help

The following input is required for these checks:

- Select and adjust the settings for the checks in the element properties
- Define the actions



To carry out the desired checks the following situations are necessary:

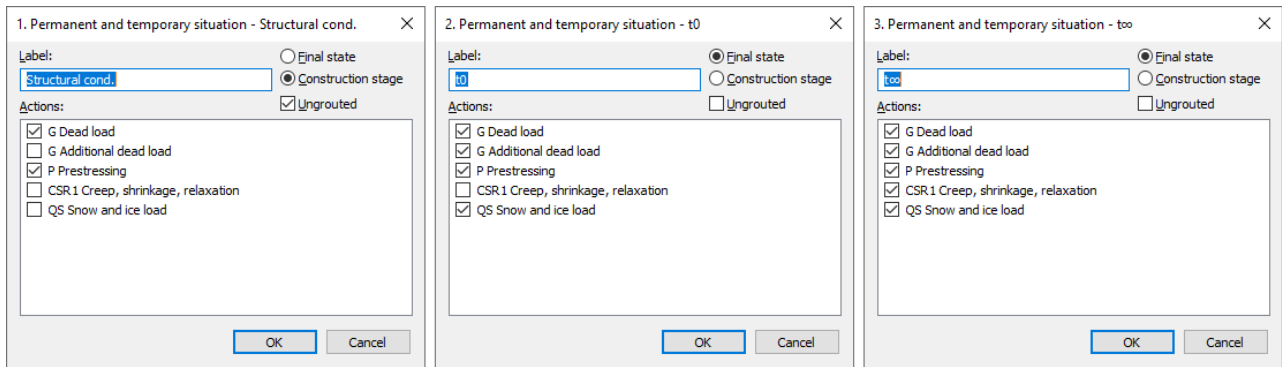
- *Permanent and temporary*
- *Rare (characteristic)*
- *Frequent*

To take different (construction) stages into account, three situations are defined:

Construction stage: Dead load G and prestressing P (tendon not grouted)

$t_0$ : Dead load G, P, additional load and snow load (tendon grouted)

$t_{\infty}$ : Dead load G, P, additional load, snow load and CSR (tendon grouted)



## Calculation and results

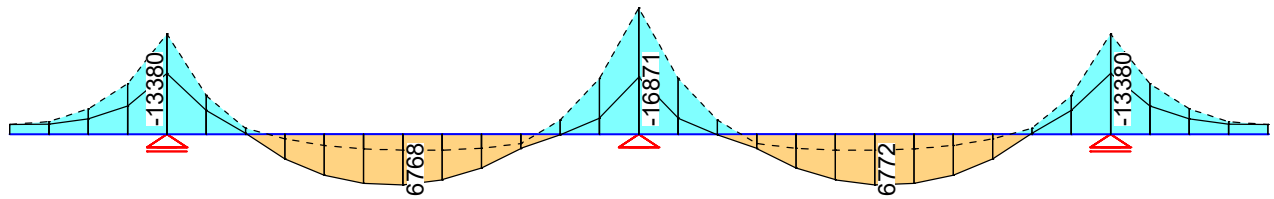
Perform the following calculations after all system specifications have been made:

- Statics
- DIN EN 1992-1-1 Design

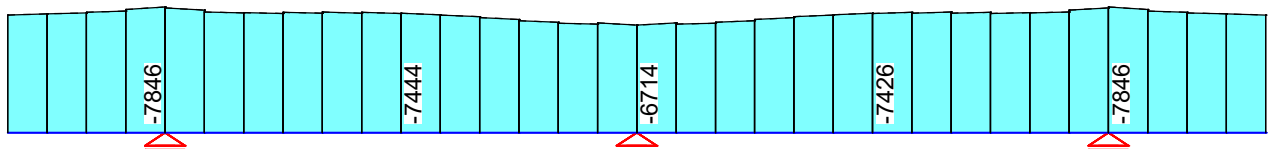
The necessary design situations are used for all checks depending on the requirement class. Each situation is checked independently and the maximum reinforcement for each steel layer is stored.

A number of different calculation results are shown below:

### Internal forces

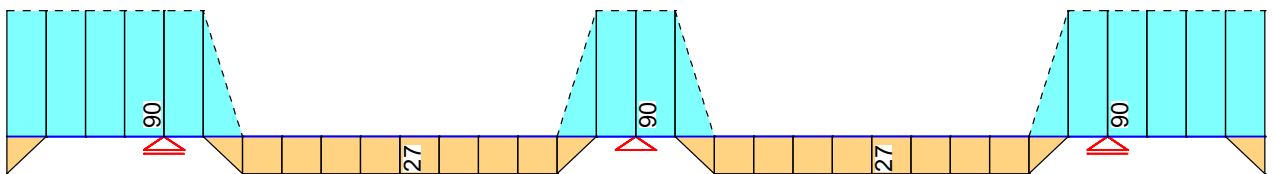


LCC EN1992.SV.3: 3. Permanent and temporary situation, EN 1992-1-1, t8  
 Internal forces min,max My. 9885.63 [kNm] =  $\longleftarrow$   
 Value range (overall system, min/max): -16871.48/6771.54 [kNm]

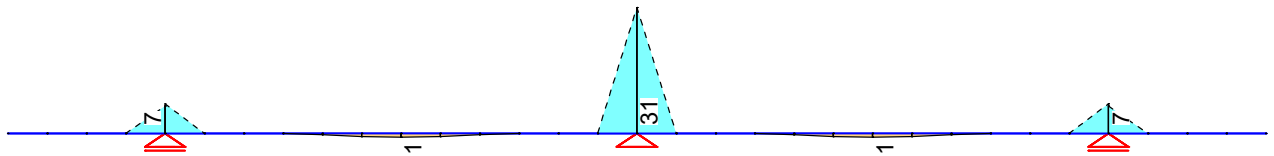


LCC EN1992.SV.3: 3. Permanent and temporary situation, EN 1992-1-1, t8  
 Internal forces min,max Nx. 4597.29 [kN] =  $\longleftarrow$   
 Value range (overall system, min/max): -7846.04/-6714.14 [kN]

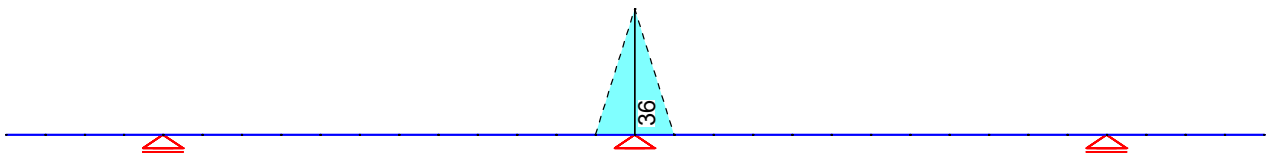
### Longitudinal Reinforcement



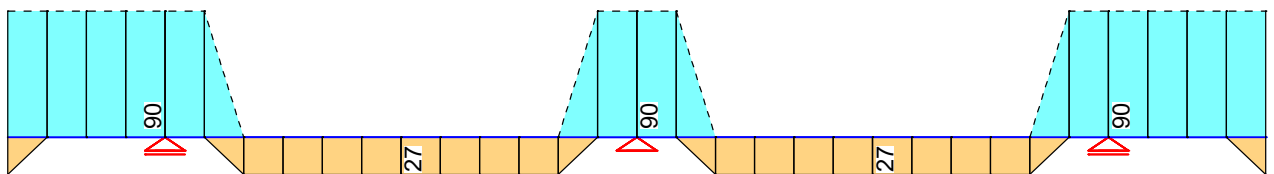
LCC EN1992.MIN: Robustness EN 1992-1-1  
 Bending reinforcement, steel layer  $z \geq z_s$ ;  $z < z_s$  (dashed). 52.63 [cm<sup>2</sup>] =  $\longleftarrow$   
 Value range (overall system, min/max): 0.00/89.82 [cm<sup>2</sup>], total weight from design: 5.5 t



LCC EN1992.RISS: Crack width limitation EN 1992-1-1  
 Bending reinforcement, steel layer  $z \geq z_s$ ;  $z < z_s$  (dashed). 18.02 [cm<sup>2</sup>] =  $\longleftarrow$   
 Value range (overall system, min/max): 0.00/30.75 [cm<sup>2</sup>], total weight from design: 0.2 t

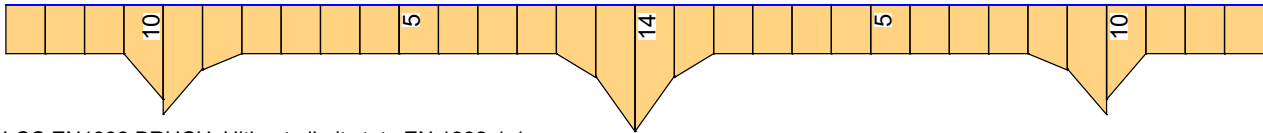


LCC EN1992.BRUCH: Ultimate limit state EN 1992-1-1  
 Bending reinforcement, steel layer  $z \geq z_s$ ;  $z < z_s$  (dashed). 21.22 [cm<sup>2</sup>] =  $\longleftarrow$   
 Value range (overall system, min/max): 0.00/36.22 [cm<sup>2</sup>], total weight from design: 0.1 t



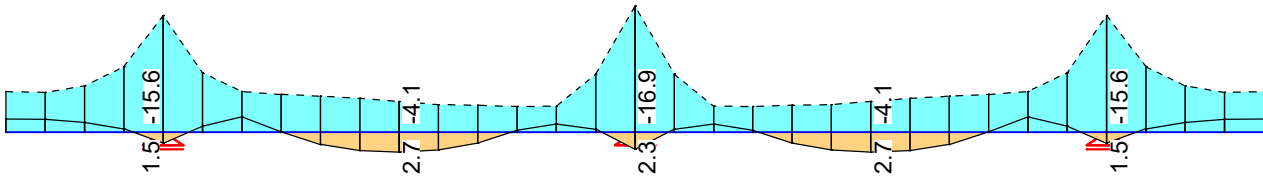
LCC EN1992.MAX: Maximum EN 1992-1-1  
 Bending reinforcement, steel layer  $z \geq z_s$ ;  $z < z_s$  (dashed). 52.63 [cm<sup>2</sup>] =  $\longleftarrow$   
 Value range (overall system, min/max): 0.00/89.82 [cm<sup>2</sup>], total weight from design: 5.5 t

**Shear reinforcement**



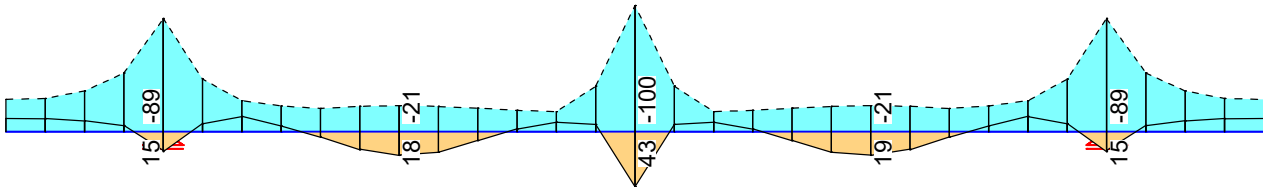
LCC EN1992.BRUCH: Ultimate limit state EN 1992-1-1  
 Stirrup reinforcement from lateral force  $Q_z$ . 8.18 [cm<sup>2</sup>/m] = |-----|  
 Value range (overall system, min/max): 5.37/13.96 [cm<sup>2</sup>/m]

**Concrete compressive stresses**



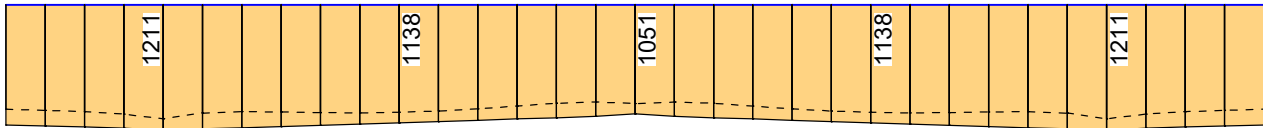
LCC EN1992.C: Maximum, Rare (characteristic) situation, EN 1992-1-1  
 Concrete compressive stresses min,max  $\Sigma_{\sigma.c}$ . 9.89 [MN/m<sup>2</sup>] = |-----|  
 Value range (overall system, min/max): -16.88/2.69 [MN/m<sup>2</sup>]

**Reinforcing steel stresses**

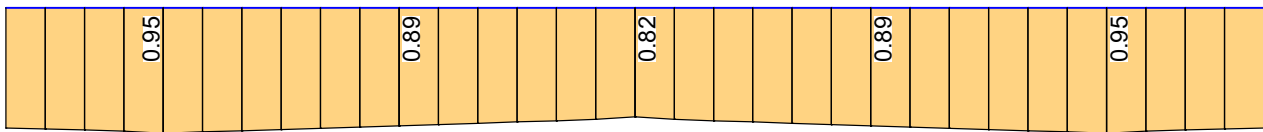


LCC EN1992.C: Maximum, Rare (characteristic) situation, EN 1992-1-1  
 Reinforcing steel stresses min,max  $\Sigma_{\sigma.s}$ . 58.30 [MN/m<sup>2</sup>] = |-----|  
 Value range (overall system, min/max): -99.50/43.29 [MN/m<sup>2</sup>]

**Prestressing steel stresses**



LCC EN1992.C: Maximum, Rare (characteristic) situation, EN 1992-1-1  
 Prestressing steel stresses min,max  $\Sigma_{\sigma.p}$ . 709.29 [MN/m<sup>2</sup>] = |-----|  
 Value range (overall system, min/max): 936.03/1210.52 [MN/m<sup>2</sup>]



LCC EN1992.C: Maximum, Rare (characteristic) situation, EN 1992-1-1  
 Prestressing steel stresses  $\Sigma_{\sigma.p}/\Sigma_{\sigma.p,perm}$ . 0.56 [-] = |-----|  
 Value range (overall system, min/max): 0.82/0.95 [-]

**Excerpt from the 'EN 1992-1-1 Design' log (Standard)**

Beam 17, Location 1



**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. for robustness as per EN 1992-2, 6.1 (109) (Charact. C.)
- (R) Nominal/requ. reinforcement as per 7.3.2/4 for crack width limitation  
 Increase of reinforcement due to crack width check is marked by "!".
- Ap' Part of prestr. steel area  $X_{il} \cdot A_p$  which was used to reduce req.As
- X<sub>il</sub> Bond coefficient for prestressing steel as per Eq. (7.5)

- (B) Design of reinforcement at ultimate limit state  
 Results from a column design using the method with nominal curvatures according to 5.8.8 are marked with "+".  
 In case of dominant bending, compression reinforcement is marked with "\*\*".  
 For section areas acc. to 6.1 (5) the concrete strain is not limited.  
 The minimum reinforcement acc. to 9.2.1.1 and 9.3.1.1 is not determined.  
 For compressive members the minimum reinf. acc. to 9.5.2 is considered.  
 whereby the prestressing steel area of bonded tendons is taken into account.

Beam No.	Se.	Lo.	Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]
17	1	1	1	M	109,56	-11704,67	0,00	.	44,91
				R	-6042,73	-9624,61	0,00	.	15,38!
			2	M	109,56	-11704,67	0,00	.	44,91
				R	-6042,73	-9624,61	0,00	.	15,38!
			3	M	0,06	-6609,23	0,00	.	0,00
				R	0,00	0,00	0,00	.	0,00
	4	1	1	M	0,06	-6609,23	0,00	.	0,00
				R	0,00	0,00	0,00	.	0,00
			2	M	0,06	-6609,23	0,00	.	0,00
				R	0,00	0,00	0,00	.	0,00
			3	M	0,06	-6609,23	0,00	.	0,00
				R	0,00	0,00	0,00	.	0,00

### Design of shear reinforcement

The percentage of nominal reinforcement acc. to Eq. (9.5N) is considered.

VRd, TRd Design value of maximum absorbable lateral force, torsional moment  
 Angle Angle cot Theta between the compressive strut and the beam axis  
 Asb,Asl.T Req. stirrup reinf. from lateral force and torsion, torsional reinf.  
 Exceedings as per Eq. (6.12) are marked by "!"  
 Asl Req. longitudinal reinf. acc. to Fig. 6.3 for req. Asb.

Beam No.	Loc.	Qy/VRd	Asb.y Angle[cm <sup>2</sup> /m]	Qz/VRd	Asb.z Angle[cm <sup>2</sup> /m]	Asl [cm <sup>2</sup> ]	Q/VRd+ Mx/TRd	Asb.T [cm <sup>2</sup> /m]	Asl.T [cm <sup>2</sup> ]
17	1	0,00	2,50	0,00	0,55	2,50	13,96	.	.

### Check of crack widths

The check is led by direct calculation of the crack width.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

wk Calculated value of crack width as per 7.3.4 [mm]  
 wmax Permissible crack width as per specification [mm]  
 Sigma.c Maximal concrete edge stress in state I [MN/m<sup>2</sup>]  
 (CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

Beam No.	Se.	C.	Lo.	Reinf. Layer	Nx [kN]	My [kNm]	Mz [kNm]	Sigma.x [MN/m <sup>2</sup> ]	wk [mm]	wmax [mm]
17	1	TC	1	1	-6042,73	-9624,61	0,00	1,96	0,07	0,20

### Check of concrete compressive stress

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked.

Sigma.x,min Total maximal longitudinal compressive stress [MN/m<sup>2</sup>]  
 Sigma.x,per = 0.60\*fck for Charact. C. (CC) as per 7.2 (2)  
 (t,b) Position of the edge point: above, below of centre

Beam No.	Se.	Loc.	Sigma.x,min [MN/m <sup>2</sup> ]	Sigma.x,per [MN/m <sup>2</sup> ]	Se.- Pnt.	Side t b	Period	Situation
17	1	1	(I) -16,88	-27,00	9	. x	Final	CC.3

### Check of steel stress

For the check, a cracked concrete section is assumed.  
 For tendon groups without bond and/or for situations before grouting, the prestressing steel stress is checked acc. to Eq. (5.43).

Type S Long. reinf. from N and M, layer number, Charact. C. (CC)  
 Type P Prestressing steel, Tendon number, Charact. C. (CC)  
 Sigma.s,per = 0.80 \* fyk resp. 1.0 \* fyk (CK) as per 7.2 (5)  
 Sigma.p,per = 0.75 \* fpk as per 7.2 (5)

Beam No.	Se.	Lo.	Steel Type	As [cm <sup>2</sup> ]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]	Situation
17	1	1	S 1	44,91	43,29	400,00	CC.3
			S 2	44,91	43,28	400,00	CC.3
			S 3	0,00	.	400,00	CC.1
			S 4	0,00	.	400,00	CC.1
			P 1	72,00	1049,44	1275,00	CC.1

# Installation and Configuration

## Licensing

The program system can be used with a single user license or a network license. Each license is protected by a software protection device (USB dongle, computer identification). The current license configuration can be checked in the license information (*Help* menu). Functions that are not licensed are disabled in the interface.

### Single User License

With single user licenses a software protection device is required at each workstation. The usage via remote connections is not supported. If several workstations are set up with different license configurations, then each configuration will require its own protection device.

### Network License

Network licenses do not require a software protection device at the workstations. The licenses are managed by the InfoGraph License Manager, which is installed on a central licensed computer together with the associated software protection device. The program users can access the software from any workstation in the network.

### Internet connection

No Internet connection is required to use perpetual licenses. If activated by the user in the InfoCAD help menu, port 80 is used to check whether new messages or program versions are available on the InfoGraph web server when the program is started.

Temporary licenses always require an internet connection for the application programs (single user license) or for the license manager (network license) to retrieve the internet time via port 80 or port 123.

## Installation

InfoCAD can be installed on the operating systems Windows 8 or Server 2012 R2 or later. All installations require administrator privileges.

The installation is performed either from CD (*Setup.exe*) or with the *InfoCAD\_xxx.exe* file after downloading from [www.infograph.eu](http://www.infograph.eu). A license number is required for download and installation, which is indicated on the back of the CD box and in the license agreement.

### Single User License

The following steps must be performed for installation:

- *Connect the dongle to the USB interface.* When installing without a dongle, the computer whose identifier has been licensed must be used.
- *Install InfoCAD.*  
'C:\Program Files (x86)\InfoGraph' is suggested as the target folder for local installation. The application, help and example files will be stored in the target folder.

### Network License

The network administrator should carry out the installation and configuration of network licenses. Supplementary documents and utilities are available in the Admin subdirectory of the installation folder.



## Network Configuration

- The licensed computer can be a Windows server or workstation.
- On workstations the system limits the number of users to 10.
- The licensed computer must have the '*Client for Microsoft Networks*' and '*Files and Printer Sharing for Microsoft Networks*' network services installed. The firewall should grant access to the file system in order to allow for communication via so called *Named Pipes* using the SMB protocol (SMB = Server Message Block). Port 445 (TCP) must be released for this.
- The program user must have the necessary user privileges on the licensed computer. Alternatively the guest account must be activated (see Windows Function *Control Panel/Administrative Tools/Computer Management/Local Users and Groups/Users/Guest* ) and the '*Password Protected Sharing*' must be switched off.
- The workstation must have the '*Client for Microsoft Networks*' network service installed.

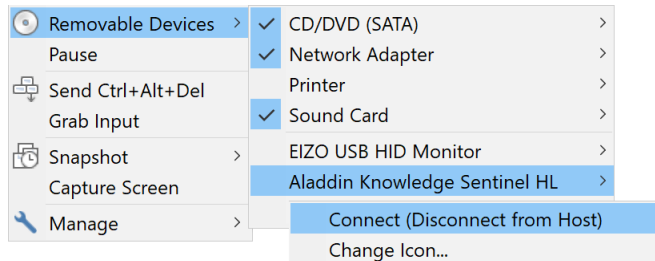
## Tasks on the Licensed Computer

- Connect the network dongle to the USB interface. When installing without a dongle, the computer must be used whose ID has been licensed.
- Install the License Manager. This requires administrator privileges. The installation is done either from CD ( *Admin\Setup.exe* ) or with the file *InfoNetKey\_xxx.exe* after downloading from [www.infograph.eu](http://www.infograph.eu).
- After the installation the license manager starts automatically as a service. The further settings are made with the *InfoMonitor.exe* utility, which is installed in the Admin subdirectory of the selected target folder:

- In the *License Information* dialog you can check the range of functions.

## Virtual Machines Using VMware or Microsoft Hyper-V

If the license manager is installed on a virtual machine using a dongle, the USB dongle must be logically disconnected from the host system and exclusively connected to the virtual machine. On system boot the assignment should be done automatically. Hyper-V requires specific drivers for using the USB interface. When licensing via computer identification, the license is connected directly to the virtual machine.



VMware settings dialog

If no connection to the license manager is possible, the following requirements should be checked:

- Is the server accessible via ping?
- Do server and clients support the SMB protocol?
- Do the firewalls allow communication through SMB?
- Can the clients access shared folders?

## Tasks on the Workstation

- For central installation of InfoCAD on a file server:
  - No further installation is necessary on the workstation.
  - Create shortcuts to the *Infocadw.exe*, *Infocadw64.exe*, *Infocadgl.exe* or *Infocadgl64.exe*, *Bdkn.exe* and *Risse.exe* files and, if necessary, manually register the InfoCAD file types.
  - In order to display the miniature views within the Windows explorer, you have to copy the appropriate DLL file (*infoshex.dll* or *infoshex64.dll*) from the Admin folder of the CD to the workstation. This DLL must be registered using the console command `regsvr32 "drive:\path\name.dll"` which has to be executed as Administrator.
  - Add the file server to the list of trusted sites for the local intranet (Control Panel/Internet Options).
  - Execution of HTML help files needs modifications of the security options in the registry (see file *Admin\_Enu.txt* in the *Admin* subdirectory of the installation folder).
  - If all users are to work with the same settings (e.g. page frame, cross-section data base), the administrator must edit the files *IGRAPH.DAT* and *ANWEND.PRO* - as described in the configuration for single user licenses - and then manually copy these files to the program directory.
- For local installation:
  - Install InfoCAD on the workstation in the same way as for a single-user license. This requires administrator privileges.
  - Make sure that all workstations are using the same program version that matches the License Manager.

## Applications

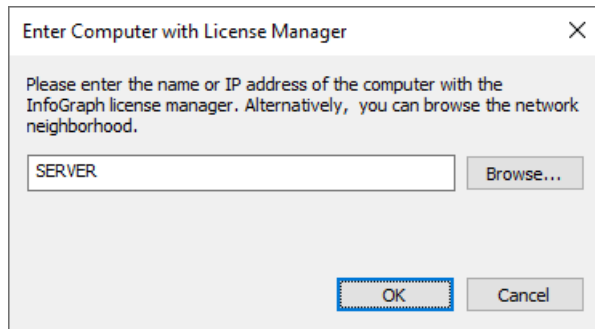
After the installation you can open the applications via shortcuts in the Windows Start menu. The applications include:

- *InfoCAD*  
The program system for editing structures.
- *System Viewer*  
Stand-alone program for realistic system display and results animation.
- *InfoGraph Crack Width Limitation*  
Stand-alone program for checking compliance with SIA and Eurocode.
- *InfoGraph Lateral Torsional Buckling Check*  
Stand-alone program for checking compliance with Eurocode.

Depending on the type of licensing (single user/network), the software protection device required for program execution is searched for in the following order:

- Protection device at the last used location (local/network). This behavior can be deactivated in the InfoCAD license information.
- All local protection devices (USB dongle, computer identification).
- License manager on the computer from which InfoCAD was started.

If none of the above connections can be established automatically, the user is prompted to specify the computer with the license manager. This location is then saved in the Windows registry for the next time the program is started.



The computer in question can be identified by entering its name or IP address or searched for in the network environment. If InfoCAD is always to work with a specific license, this can be specified by parameters in the program shortcut (see *Admin.txt* file in the Admin subdirectory of the installation folder).

## Examples

When the program is started for the first time, the example files stored in the installation folder are copied to the *InfoGraph* subdirectory of the personal document folder.

## User Configuration

The user settings are managed in the following files:

### *IGRAPH.DAT*

Among other things the basic settings of the computer programs, user-defined page formats and other printing settings are stored in this system file. The file is automatically created in the 'C:\Users\name\AppData\Roaming\InfoGraph' folder when InfoCAD is opened for the first time, where 'name' represents the login name of the user. These folders carry the 'hidden' attribute and are displayed depending on the folder options setting. The InfoCAD Options menu provides functions for exporting and importing the system file.

### *ANWEND.PRO*

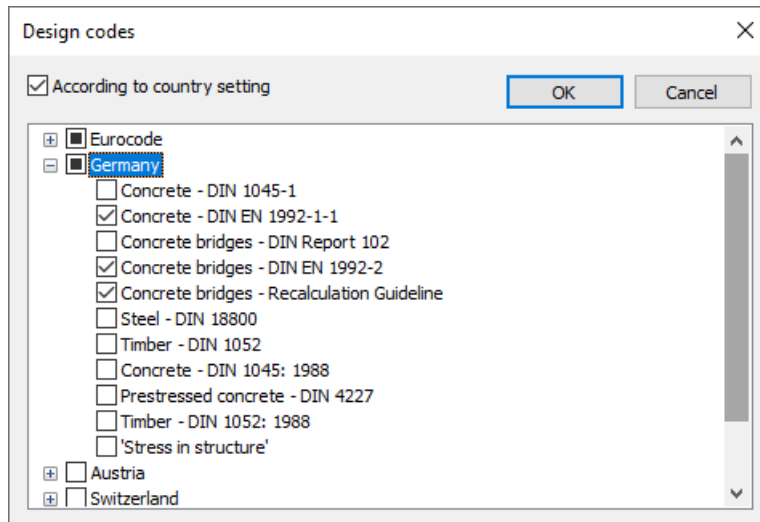
The user database lets you store user-defined beam sections. It is created in the 'Documents\InfoGraph' subfolder of the user.

If all users are to work with the same settings, the administrator must manually copy these files to the program directory. As a result, users will no longer be able to change the settings.

## Language Selection

During installation you will be prompted to select the language of the user interface. You can change the language at any time in the *Options* menu. The analysis logs and results will now be displayed in the language you have selected.

## Standards Selection



The design codes shown in the user interface are preset according to the country setting of the operating system. You can change the selection at any time in the *Options* menu.

## Page Frames

The page frames used for printing can be selected in the Print dialog. Except for the standard page frame, you can design or adjust any of the frames that are available for selection using the *Page Frame* option in the *Options* menu.

## Updates

InfoCAD informs you about new program versions on the InfoGraph web server, unless this has been deselected in the Help menu. The program system is updated in the same way it was initially installed. Updates will not affect the user's configuration files and settings. To update the license data, e.g. after acquiring new modules, the associated function is available in the InfoCAD license information (single-user license) or in the *InfoMonitor* utility program (network license).

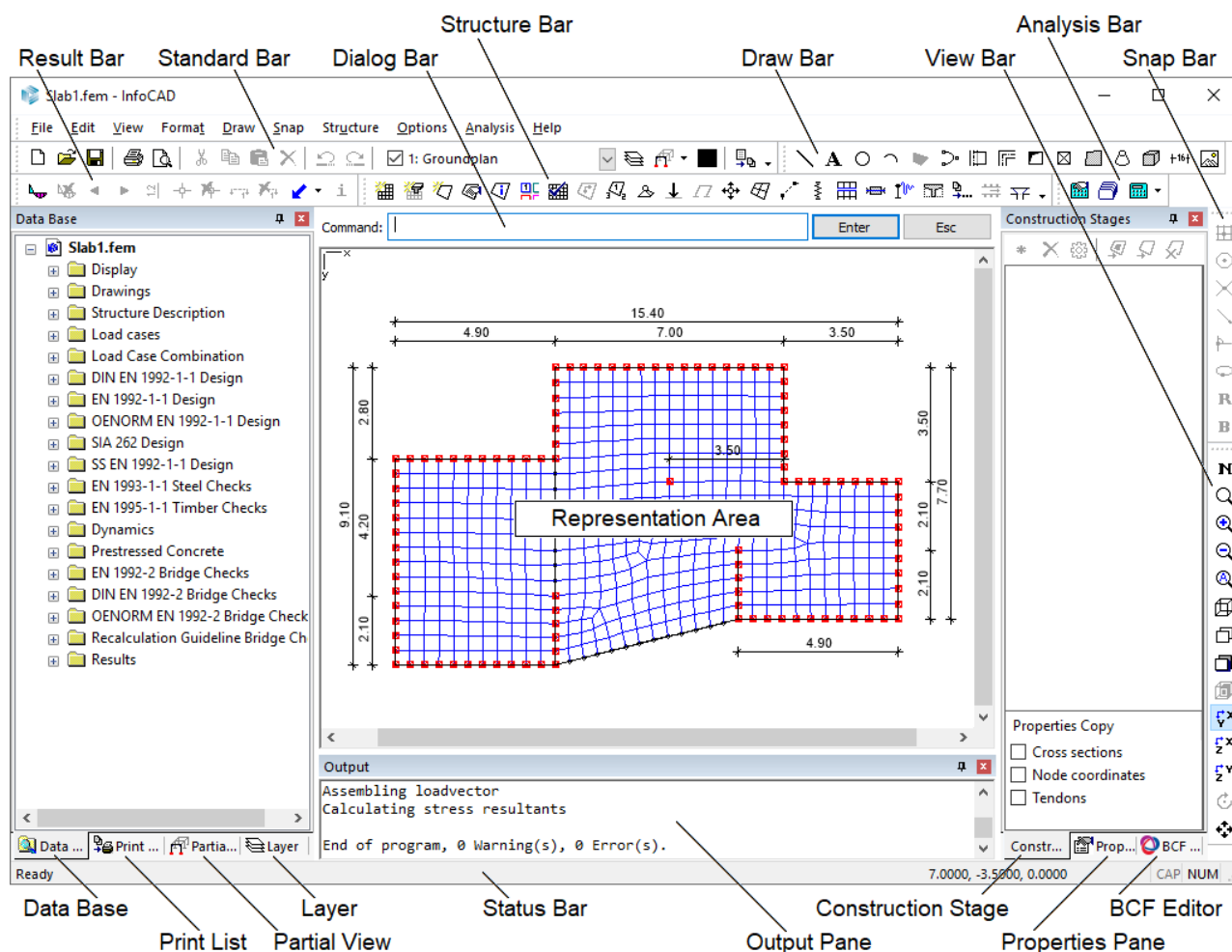
## Uninstallation

To erase the program system from the hard drive, use the option *Software* from the Windows control panel to start the automatic uninstall program for InfoCAD. Uninstallation will not affect any files created by the user.

# User Interface

## Controls

The image below shows how the interface appears after opening a project file.



## Standard Bar

The standard bar contains key functions from the *File* and *Edit* menus. It also allows you to control the layer function, the partial view function, the color selection and the transfer of the current screen view to the print list.

## Draw Bar, Snap Bar, View Bar

These toolbars contain functions for creating drawings, snap functions for selecting object points and other functions for defining the display window.

## Structure Bar, Analysis Bar, Result Bar

The structure bar contains the functions of the *Structure* menu. The functions of the *Analysis* menu are also available in the analysis bar. The results bar offers special functions for showing results.

## Status Bar

Information on the currently selected function is shown in the left part of the status bar. The x, y and z coordinates [m] of the crosshair center or of the last entered point are displayed in the right part of the status bar. Click the status bar to switch to continuous coordinate view.

## Data Base Pane

The database represents all the input data and analysis results of the project in a single tree and allows you to display selected data in the representation area either as a graphic or a table. In addition, you can print out selected sections using the context menu and transfer them to the print list.

## Print List Pane

Graphical and table representations are saved to the print list. These representations can later be accessed using their original settings and then printed individually or in batches. The results contained therein will be automatically updated.

## Layer Pane

In this pane the layer function is displayed and controlled. The layer feature can be used to group graphical objects in a logical manner, e.g., to allow for separate editing.

## Partial View Pane

This pane displays the existing partial views with their state and appropriate controls. Partial views can alternatively be used to group graphical objects in a logical manner.

## Representation Area

The representation area shows either the graphic or table view of the active project.

## Graphical View

The graphical view displays objects in a plan view, front view or 3D perspective. You can freely choose the image plane (viewport) and the display window. The mouse cursor assumes different shapes during editing:



With the left mouse button you can select objects by clicking them or dragging a selection box over them. If the window is dragged from left to right, only the objects within the window are selected. If the window is dragged from right to left, all objects within the window plus any objects crossing the boundaries of the window are selected. When combined with the SHIFT and CTRL keys, you can expand or reduce the selection. To open the context menu, press the right mouse button. To open the Properties dialog, double-click an object.



Indicates a direct transfer operation when holding down the left mouse button.



The crosshair with snap icon is used to select objects in active dialogs. In conjunction with object snap methods, the basepoints of objects are used to enter coordinates.



When the large crosshair is selected (Options menu), a scaled snap window is displayed.



The crosshair with snap window is used to select objects in active dialogs. In conjunction with object snap methods, the basepoints of objects are used to enter coordinates.



With this cursor you can rotate the view in any direction in 3D mode.



This cursor is used to move the display window.



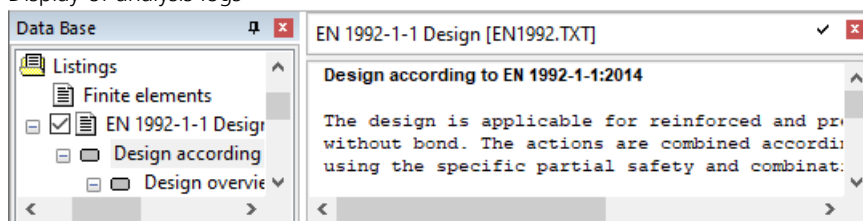
This icon appears if you are not able to execute a function in the representation area.

For the selected objects the color and layer are displayed in the standard bar.

## Table View

The table view enables additional alphanumeric input and output and provides the following functions:

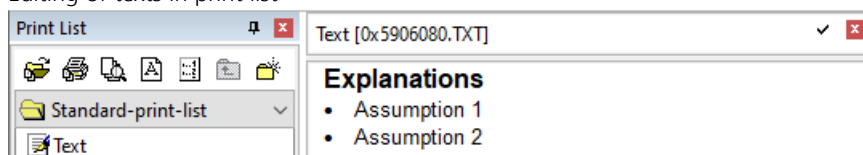
- Input and modification of structural properties in tabular form
- Definition of analysis settings
- Display of analysis logs



- Tabular view of results

	Node	Rx [kN]	Ry [kN]	Rz [kN]
1	131	0.00	0.00	42.38
2	132	0.00	0.00	4.12
3	133	0.00	0.00	10.53

- Editing of texts in print list



The following special functions are available in the menus for the tables, listings and texts in print lists:

<i>Find...</i>	Search for the specified words.
<i>Replace...</i>	Replace specific text with different text.
<i>Goto...</i>	Go to the specified row (number) of the table.
<i>Standard View</i>	Select the preferred view. The reaction pair (e.g., Nx min, corresp. My, corresp. Mz, ...) of a result location will be shown in a table row for combination results.
<i>One Row per Result Location</i>	Select a view of combination results in which all the reactions of a result location will be shown in a table row.
<i>View Selection...</i>	Open the dialog to filter the result representation.
<i>Font ...</i>	Select the font for the current selection.
<i>Bullet Style</i>	Insert a bullet point in the selected rows.
<i>Word Wrap</i>	Word wrap allows you to view the entire text on the screen but does not affect how the text will be printed.
<i>Paragraph...</i>	Open the dialog for specifying the indentation and alignment of the selected paragraph.
<i>Tabs...</i>	Open the dialog for specifying the tab stop in a selected paragraph.
<i>Small Font</i>	Select a small font for the table and listing printing.
<i>Section Stress...</i>	Open the dialog for viewing the section stress of the selected internal forces.

Table contents and analysis logs can be exported to other programs via the clipboard. The analysis logs are recreated after each analysis and can be supplemented with text-based explanations.

Table rows can be marked with the left mouse button and the *Shift* key on the left edge of the table. This selection is taken into account when printing and when transferring to the print list. To e.g. copy only certain columns they can be marked in the header. The name of the data set displayed is shown above the table in square brackets.

## View Selection

With this dialog the table view of results can be limited to specific objects.

### Active objects of the graphic view

Only the table content of objects which are on the active layers or partial views is displayed. Selecting nodes leads to a more limited table view.

### Selected objects of the graphic view

Only the table content of selected objects is displayed. Selecting nodes additionally leads to a more limited table view.

### Selection stored in the print list

The selection is taken from the print list and is independent from the current graphic view.

## Output Pane

The analysis and checking programs display their progress in the output pane. The information in the output pane can be selected and then copied or printed using the context menu. By double-clicking the warnings and error messages of the analysis modules, the affected object (*load case, load line, beam, element, node*) is selected in the graphic or table view and the image section is centered on the object.

## Properties Pane

In this pane BIM properties of selected IFC objects are displayed.

## Construction Stage Pane

In this pane the existing construction stages are listed and can be edited. The current construction stage is marked with '>'. Modified construction stages have a star (\*).

## BCF Editor Pane

The InfoGraph BCF editor is displayed in this pane. This editor allows the exchange of information in the BIM Collaboration Format (BCF), an open standard format recommended by the buildingSMART e.V. It is used by numerous BIM applications to supplement building models with commentaries, image details and 3D views.



## Dialog Bar

The functions in the menu extend the range of input options when using the dialog bar. They are as follows:



The provided buttons can be accessed with the left mouse button.



You may enter data into the input field using the keyboard. If the text cursor is not in the input field, simply click the field to move it there. Conclude all data inputs by pressing the Enter button or key. Right-click to access a context menu.

You can cancel dialogs by pressing the *Esc* button or key.

Functions and their options can also be accessed by their name. The first letters of the name are sufficient as long as they are unique (e.g. *ed* for *Edge*). Upper and lower case are not distinguished. The input of a function name is to be terminated with the *Enter* key.

## Coordinate Input

Object coordinates are saved in InfoCAD using three components. You can enter the coordinates using either the crosshair or the keyboard.

Crosshair input is carried out in the representation area. In 2D view the third component is specified based on the constants that have been defined for the viewport (e.g., the constant *z* value for the *xy* viewport). Object points near the crosshair are snapped automatically. The effective snap radius can be defined via the *Snap* menu.

If you enter coordinates using the unit meter in the dialog bar with the keyboard, separate the individual components with a space. The decimal separator must be entered in accordance with the regional settings of the operating system. You can define a local reference system for coordinate input using the *Snap* menu functions.

The following input variants are available:

### Cartesian coordinates

Assign the first two values to the axes of the viewport in 2D view (e.g., *x* and *y*). The constant specifically defined for that viewport will be used as the third component. In 3D view, coordinates must be entered in the order *x*, *y* and *z*. The *&* character can be used to define relative coordinates which refer to the previous point.

Example: In the *xy* viewport, entering '& 1.55 3.05' will yield a point with  $\Delta x=1.55$  and  $\Delta y=3.05$  relative to the last point.

### Cylinder coordinates

Specify radius [m] and rotation angle [°] against the horizontal axis in 2D view (polar coordinates). In 3D view, the radius, the rotation angle against the *x* axis and the *z* component are required. The angles are defined in a positive, clockwise direction.

Example: In 3D view, entering '1.4142 <45 1' will yield the Cartesian coordinate (1 1 1).

The *x*, *y* and *z* coordinates of the crosshair center or the last specified point are displayed in the right part of the status bar. Click the status bar to switch to continuous coordinate view.

## Arithmetic

Numerical entries in the dialog area can be specified as arithmetic expressions. The expressions can be composed of constants and variables. They can be grouped by parentheses, but spaces are not permitted. The arguments of the functions must be in parentheses.

### Operators

+	Addition
-	Subtraction
*	Multiplication
/	Division
**	Exponentiation








### Functions

SIN	Sine of an angle	ABS	Absolute value
COS	Cosine of an angle	SQRT	Square root
TAN	Tangent of an angle	EXP	E-function
ASIN	Arcsine	LOG	Natural logarithm
ACOS	Arccosine	LOG10	Base-10 logarithm
ATAN	Arctangent		

Example:  $1.50 * \text{SIN}(30)$  results in 0.75.


## File

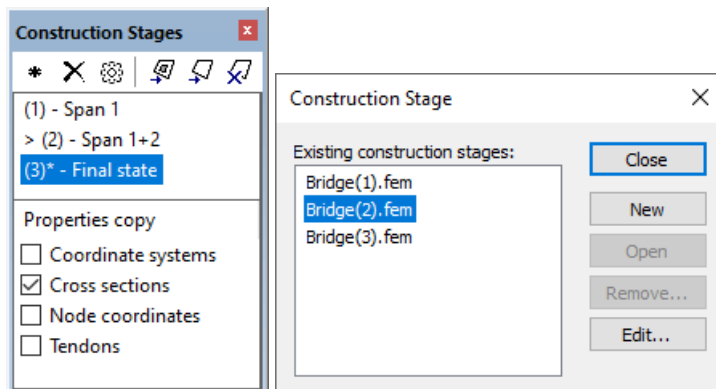
The *File* menu contains functions for project file management. In addition to the structure, the project file can contain any number of drawings. These drawings are only composed of drawing objects and can be opened and saved using separate functions. To edit drawings without the structure, you first need to open the project file with the *Open Drawing* function.

	<i>New</i>	Start a new structure. The current project file will close.
	<i>Open...</i>	Open an existing project file. The structure will be automatically loaded and displayed.
	<i>Save</i>	Save the current project file.
	<i>Save As...</i>	Save all the input data of the current project with a new name.
	<i>Save Copy As...</i>	Save all the input data of the current project with a new name. The current project will remain open.
	<i>Construction Stage...</i>	Generate or select a construction stage.
	<i>Drawing</i>	<ul style="list-style-type: none"> <li><i>New</i> Create a new drawing. The current drawing will be closed.</li> <li><i>Open</i> Open an existing drawing.</li> <li><i>Save</i> Save the current drawing.</li> <li><i>Save As</i> Save the current drawing with a new name.</li> <li><i>Picture creation</i> Generate a 2D drawing of the current screen view.</li> </ul>
	<i>Print...</i>	Printout of the current screen view.
	<i>Print Preview</i>	Preview of the pages to be printed.
	<i>Page Setup...</i>	Page layout for printing.
	<i>Add to Printing List</i>	Add the active document to the printing list.
	<i>Import</i>	Import a project file or other data formats.
	<i>Export</i>	Export a project file to other data formats.
	<i>File Service</i>	Display and edit the data sets of the project file.
	<i>1 - 8</i>	Open one of the eight most recently used project files.
	<i>Send...</i>	Send the current project file via e-mail. This function is only available if a Windows e-mail client such as MS Outlook is installed.
	<i>Only Input Data</i>	Button for sending project files via e-mail. When limiting to input data, the file size is normally reduced significantly.
	<i>Exit</i>	Close the current project file and exit the program.

## Construction Stage

The *Construction Stage* function supports the calculation of structures in which different construction phases are to be analyzed. The function is based on the following principles:

1. The calculation model of each construction stage is stored in a separate project file.
2. The names of associated project files are identical except for a consecutive number. The file name for the first construction stage must contain the number (1) (e.g., 'System(1).fem'). This is required for the function to work. Gaps in the numbering are not permitted.
3. When creating a new construction stage , the program simply uses the current system without the load cases. The new construction stage also includes the number of 'inherited' elements, nodes, supports and tendon groups as well as the load cases of the previous system. In the case of 'inherited' tendon groups, the *Ignore in prestress load* button is selected.
4. The structural components in the new construction stage can be added or removed in any manner you choose. Doing so will have no effect on the object numbers included from the previous stages.
5. The section and material properties of transferred elements may be modified.
6. You can define any number of new load cases (recorded load case numbers are protected).
7. The settings regarding the number of result locations in the beams and the result locations in the area elements must be the same in all construction stages.
8. Before each calculation (Statics), the results from the load cases of the previous system are applied to the current system. This process only takes into account the included objects (see 3.). More specifically, this involves:
  - Node deformations (DEFORM.\*)
  - Beam deformations (ELDEF.\*)
  - Support reactions (AUFLR.\*)
  - Deformations in the support system (AUFLDEF.\*)
  - Internal forces of the beams (SREAK.\* / LERG.\*)
  - Internal forces of the areas and stresses of the solid elements (REAK.\*)
  - Soil pressures (SIGB.\*)
  - Resulting soil pressures (SIGRB.\*)
  - Tendon group forces from creep and shrinkage (VSPZV.\*)
9. The transferred results are available in the standard form for additional processing. This includes the superposition or the combination with other load cases, for example.



The existing construction stages are listed in the construction stage pane. The current construction stage is marked with '>'. Modified construction stages have a star (\*).

### New

Insert a new construction stage behind the open construction stage.

### Delete

Delete the selected construction stages that follow the currently open construction stage. These deletions are permanent. You will be prompted to confirm the deletions.

### Copy Object Properties

Copy the object properties of selected objects into the selected construction stages. The copied properties also apply to all other objects with the corresponding number (cross-section, prestressing system, etc.).

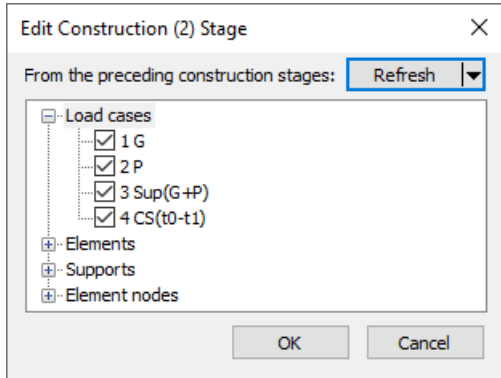


**Copy Objects**

Copy the selected objects into the selected construction stages.



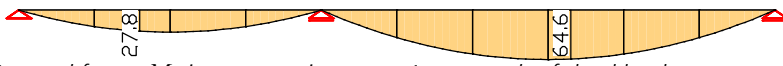
**Edit**



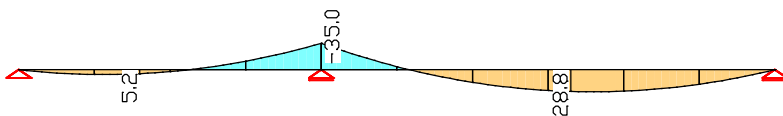
Edit the current construction stage. The left dialog shows the load cases and structure objects whose results are taken over from the previous construction stage for each calculation process (Statics). During the creation of a new construction stage with the function *New \**, the marks for all load cases and structure objects existing at this moment are set automatically. If in previous construction stages load cases are added subsequently, a take over of the results to the current construction stage only occurs if the new load cases are marked here. Results of a new structure object can only be transferred if it exists in the current construction stage with the same number and it is marked for transfer.

**Example 1: Girder**

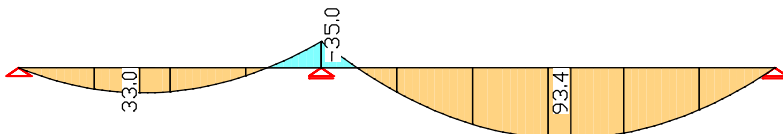
In this example, two single-span beams are connected retroactively. A detailed explanation of this example can be found in the section 'Finite elements > Examples > Construction stages, creep redistribution'.



Internal forces  $M_y$  in construction stage 1 as a result of dead load



Internal forces  $M_y$  in construction stage 2 as a result of additional load

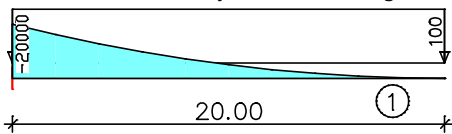


Sum of internal forces  $M_y$  from construction stages 1 and 2 (Superposition load type)

**Example 2: Bridge With Auxiliary Support**

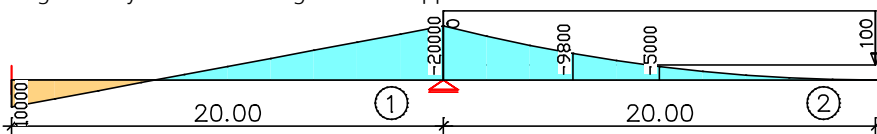
This example shows a bridge with an auxiliary support. For the purpose of simplification, only the following three construction stages will be considered:

1. Fixed cantilever shortly before reaching the auxiliary support



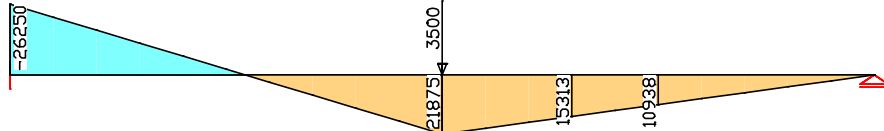
Internal forces  $M_y$  in construction stage 1 as a result of G1

2. Bridge shortly before reaching the end support

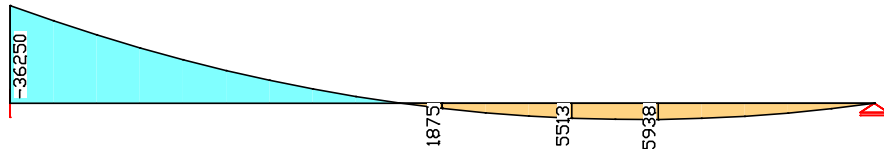


Internal forces  $M_y$  in construction stage 2 as a result of G2

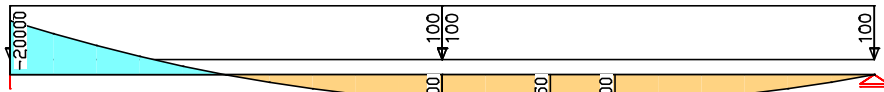
3. Bridge after the auxiliary support is removed



Internal forces  $M_y$  in construction stage 3 as a result of removing the auxiliary support



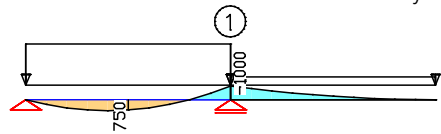
Sum of internal forces  $M_y$  from construction stages 1, 2 and 3 (SUP load type)



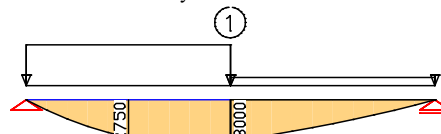
For comparison purposes, the calculated internal forces  $M_y$  in the final state as a result of G

### Example 3: Incrementally Launched Bridge

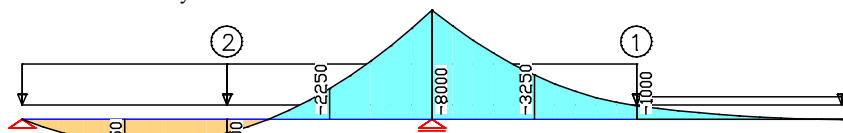
In this example, a bridge is moved to the end support using a launching nose. Unlike example 2, the beam nodes are moved in this case. The field lengths measure 20 m and the dead load of the bridge is 100 kN/m. The 10 m long launching nose has a dead load of 20 kN/m. An auxiliary support is used temporarily in the first stage.



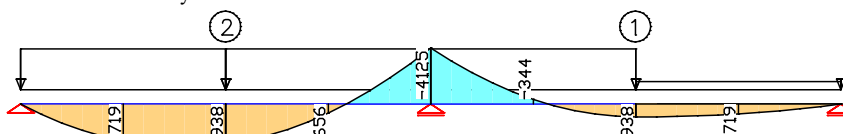
Internal forces  $M_y$  [kNm] in construction stage 1 with auxiliary support as a result of G



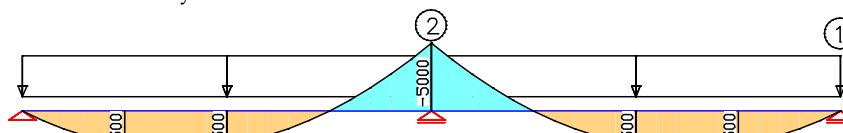
Internal forces  $M_y$  [kNm] in construction stage 2 as a result of G



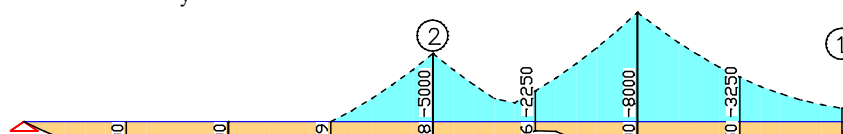
Internal forces  $M_y$  [kNm] in construction stage 3 as a result of G



Internal forces  $M_y$  [kNm] in construction stage 4 as a result of G



Internal forces  $M_y$  [kNm] in construction stage 5 as a result of G (these internal forces correspond to the monolithic model).



Internal forces min, max  $M_y$  [kNm] from construction stage 1 through 5 as a result of G

## Print, Page Setup

This function allows you to adjust the print settings.

### Page frame

Selected page frame for printing. User-defined page frames can be modified in the *Options* menu.

### Page no.

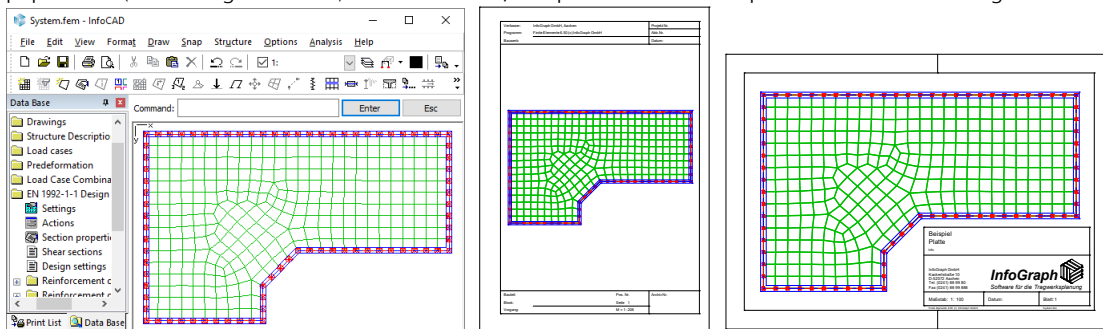
Page number used for the next page to be printed. This number is increased by one after each printed page. You can enter '0' to suppress numbering.

### Part numbers

The part number is the consecutive number of pages in a print job. If the part number deviates from the page number, you can have it printed in parentheses in front of the page number.

### Scale

If the *auto* button is selected, the representation area will be printed out in the largest size that fits within the printable paper area (see the figure below). If not selected, the printout will correspond to the scale setting.



### B/W-Print

Print the lines in black. Colored areas will be converted to gray values. You can define the gray values for the graphical representation of results on the *Gray Values* dialog page.

### Axes

Print the axes together with a dimension line in the upper left corner of the drawing area.

### Date, Time

This information appears automatically in the standard page frame. In the other page frames the information is positioned based on the *@Datum* or *@Zeit* placeholders.

### Project, building number, Title

This text appears automatically in the standard page frame. In other page frames the information is positioned based on the *@Projekt* or *@Titel* placeholders.

### Use print list text

If the switch is set, the label texts of graphical views of the print list are output instead of the title. Printing graphical views by using the context menu of the database leads to the same result.

### Texta, b, c

Text that can be added to the page frame using the *@Texta*, *@Textb* and *@Textc* placeholders. The fields are released for editing if the placeholders are used in the selected frame. Any text entered in these fields can be saved in the current project file.

### User name, structural engineer

The user indicated here will be used in the conjunction with the standard page frame and with the *@Benutzer* placeholder in other page frames. It is saved in the system file *lgraph.dat*.

### Printer...

Open the Windows printer dialog. In this dialog you can select a different printer or adjust printer-specific settings.

**Page...**

Page orientation, paper size and margin settings.

**Line Widths**

Assigns line widths to the line colors for printing. This setting can be disabled using the line width indicated in the line type. A minimum number of pixels can be set for the line width for display on the screen.

**Gray Values**


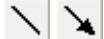

Coloring of result diagrams and slab reinforcement is carried out with the colors defined here. You can assign grayscale values to the colors for black and white printing. When selecting the Windows background color, coloring of result diagrams is suppressed.

**Text Options**

Text that can be added within the page frame using the @T1-@T6 placeholders.

**Print Preview**

This function opens the print preview. The buttons and symbols contained therein have the following meaning:


<i>Print...</i>	Printout of the current screen view.
<i>Next Page</i>	Preview of the next page.
<i>Prev Page</i>	Preview of the previous page.
<i>One Page</i>	Preview of one page.
<i>Two Page</i>	Preview of two consecutive pages.
<i>Zoom In</i>	Enlarges the view.
<i>Zoom Out</i>	Reduces the view.
<i>Close</i>	Closes the print preview.
	If the icon is activated, a text field is inserted by clicking and dragging.
	If the icon is activated, a line or an arrow is inserted by clicking and dragging.
	If the icon is activated, a corresponding legend is inserted by clicking and dragging.

If it is a print preview of a print list, text boxes, lines, arrows or legends are saved in the relevant print list entry. Otherwise, these objects are used for the subsequent normal printouts and are retained as long as the program remains open.

The video "[https://download.infograph.de/video\\_de/Kommentarfeld%20in%20Druckausgabe%20platzieren.mp4](https://download.infograph.de/video_de/Kommentarfeld%20in%20Druckausgabe%20platzieren.mp4)" (German language) shows the handling of text boxes and arrows in the print preview.

## Print List








Save graphical and tabular views to the print list. They can be accessed with their original settings and printed out individually or in groups. Since the print list only contains references, the data printed out will always be current.

To add views to the print list, click the  button in the standard toolbar. The following information will be saved for each entry:

- Current drawing
- Current load case
- Analysis result with settings
- Visibility of layers and partial views
- Display window and its configuration
- Scale if layout view is enabled
- Table selections
- Paper size and page orientation
- Page frame

If you only want to see specific details rather than the entire graphical view, you can drag a selection box by holding down the ALT key. The display window inside the selection box will then be added to the print list.

The content of the print list can be edited using the toolbar of the same name. The icons in the toolbar have the following functions:

-  Open the dialog to select or define print lists.
-  Print all or selected content.
-  Show the page view for all or selected content.
-  Open the Page Setup dialog. If entries are selected, the icon will show the current page orientation and allows you to modify the paper size, page orientation and page frame.
-  Generate a table of contents at the beginning of the print list or update the existing table of contents.
-  Switch to the previous folder.
-  Create a new folder.

Double-click any entry in the list to restore the view.

The context menu contains additional functions:

- *Copy image objects*: Copy image objects directly to the clipboard. This allows you to insert graphics, tables and similar items into other Windows programs.
- *Metafile export*: Generate a metafile in the WMF or EMF format for all or selected content.
- *Undo*: Changes of the print list can be undone.
- *Cut, copy, paste*: Cut, copy and paste entries via the clipboard. Entries can also be moved using drag and drop.
- *Refresh*: Transfers the current graphical view to the selected entries.
- *Refresh with selection*: Adopts selected properties of the current graphical view to the selected entries.
- *Next to predecessor position*: Arrange the display window next to the predecessor if there is enough space.
- *Insert form feed*: Force a page break when printing.
- *Insert text*: Insert user-defined text.

## Refresh with Selection

The following options are available for updating selected graphical views from the print list:

- *Label*: Updates the label of the selected entries depending on the updated graphical view.
- *Setting of the view*: The current graphical view with the associated display options is adopted.
- *Layer, partial views, 3D section box*: The active layers, partial views and the 3D section box of the current graphical view are taken over.
- *Drawing*: The drawing name stored in the entry is replaced by the name of the currently selected drawing or, if no drawing is selected, removed.
- *Load cases*: The currently displayed load case is adopted.
- *Result case*: The results are changed to the load case, or combination, of the currently displayed result.
- *Result component*: The currently displayed result component is adopted.
- *Result representation*: The current settings for the result representation are adopted.



## Import and Export

These functions can be used to exchange data in the following formats:

- AutoCAD DXF format
- Data exchange interface for steel construction of DSTV 4/2000 as of 10/2002, file extension \*.stp
- IFC (Industry Foundation Classes), file extension \*.ifc
- InfoGraph project files with extensions \*.fem, \*.rsw, \*.esw, and \*.ros
- InfoGraph interface files with extension \*.icx
- SAF (Structural Analysis Format), file extension \*.xlsx

## Data Exchange Between Project Files

Element data and drawings are exchanged between project files.

### Import

<i>Drawing</i>	Insert the selected drawing of another project file.
<i>Element data</i>	The elements of another project file can be inserted into the current file. The numbers of the inserted nodes and elements may change in some cases.
<i>Insertion point</i>	Location where the zero point of the imported data is inserted.
<i>Angle of rotation</i>	Angle by which the imported data is rotated.
<i>Scaling</i>	Scaling factors in the x, y and z direction for the inserted objects.

### Export

<i>Select objects</i>	Select the objects to be exported.
<i>Drawing</i>	The selected drawing objects can be exported with a new name. Element data is exported without further user interaction.

## Data Exchange With InfoGraph Interface Files (ICX)

The interface bases on the STEP (**ST**andard for the **E**xchange of **P**roduct model data) technology and uses the ASCII format to exchange the following data:

- Drawing objects, model objects
- Beams, cables, area and solid elements
- Design objects
- Link elements
- Spring elements, supports
- Tendons
- Section, material, joints
- Loads, load case combination, actions for checks

The EXPRESS file *icx-03.exp* installed in the program directory describes all objects supported by the interface with their properties and data types. The properties are explained in the file *icx-03.html*. The video "[https://download.infograph.de/video\\_delProgrammierschnittstelle%20ICX.mp4](https://download.infograph.de/video_delProgrammierschnittstelle%20ICX.mp4)" (German language) shows this handling of the interface.

## Data Exchange via DXF Format

The data exchange is based on AutoCAD Version 12. The following DXF objects are included in the import:

<i>3DFACE</i>	<i>ARC</i>	<i>BLOCK</i>	<i>CIRCLE</i>	<i>DIMENSION</i>
<i>INSERT</i>	<i>LINE</i>	<i>LWPOLYLINE</i>	<i>MTEXT</i>	<i>POLYLINE</i>
<i>SOLID</i>	<i>TEXT</i>	<i>TRACE</i>		

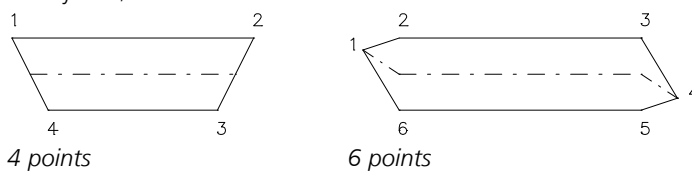
The *LWPOLYLINE* and *POLYLINE* objects are converted into lines and the *DIMENSION* object is converted into lines and text. During export the current drawing and element data is stored as *3DFACE*, *ARC*, *CIRCLE*, *LINE*, *POLYLINE*, *SOLID* and *TEXT*. Layer and color information will be retained.

### Input

*Unit* Measurement units in the DXF file (mm, cm or m).  
*Ground plan* You can choose whether the model information is to be kept for automatic mesh generation (column, hole, edge, wall).

With respect to exchanging model information, the following conventions apply:

*Walls* On layer *G\$WAND* as closed *POLYLINE* in two forms:



*Edges* On layer *G\$RAND* as *LINE* or *ARC*.  
*Holes* On layer *G\$LOCH* as *LINE* or *ARC*.  
*Columns* On layer *G\$STUETZE* as *POLYLINE* with 4 points (rectangular column),  
*CIRCLE* with  $r > 0.001$  (round column),  
*CIRCLE* with  $r = 0.001$  (point column)

### Note

Area and solid elements are exported as *3DFACE* objects, beam elements as *LINE* objects. The context menu allows to convert imported *3DFACE* objects into area elements and imported *LINE* objects into model edges or beam elements.

## IFC Data Transfer

The data exchange format IFC (Industry Foundation Classes) is a manufacturer-independent transnational interface, which supports the exchange of all geometric and alphanumeric BIM data.

buildingSMART International develops and establishes IFC as an open standard for the building industry. IFC is registered as an international standard in ISO 16739.

The data can be exchanged using the import or export function. The IFC version IFC 2x3 or IFC 4 can be used.

Additional project information can be transferred via the BIM Collaboration Format (BCF) using the BCF Editor.

### Coordinate System

In the IFC product model the z axis of the global coordinate system typically runs from below to above. This is counter to the convention used in InfoCAD. To avoid all objects standing on their heads after an export or import, they are turned 180° around the global x axis.

### Import

The data import consists of two functional areas:

- Importing the Analysis models (frames or model objects)
- Importing the Geometry of Building Elements

## 1. Importing Analysis models (frames or model objects)

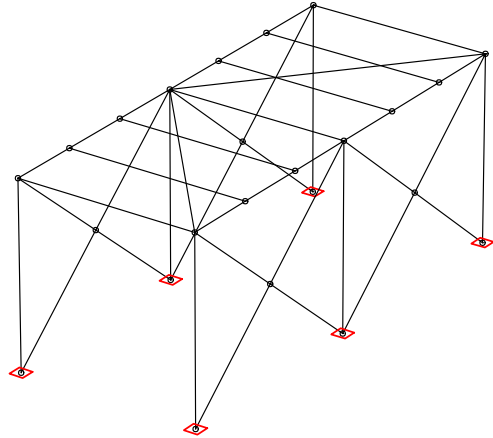
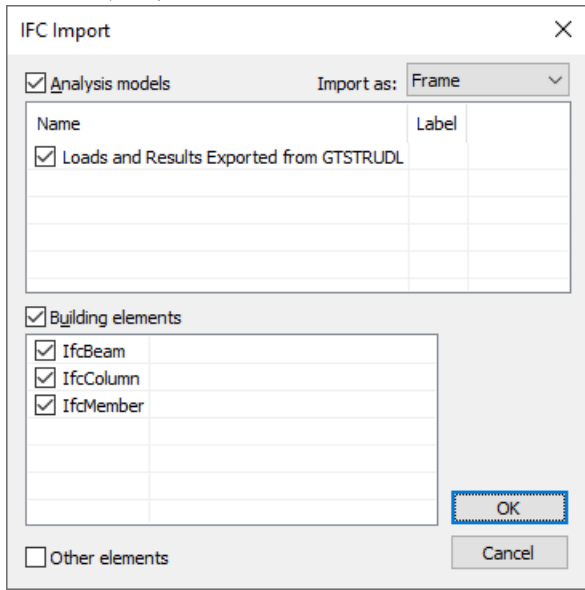
The import is carried out for the static analysis models (IfcStructuralAnalysisModel) saved in the transfer file and selected by the user. The following table displays the relevant objects.

<b>IFC Object</b>	<b>Imports as</b>	<b>Remarks</b>
<b>IfcArbitraryClosedProfileDef</b> <b>IfcArbitraryProfileDefWithVoids</b>	Polygon section	The geometry must be described with IfcPolyline
<b>IfcAsymmetricIShapeProfileDef</b>	Polygon section	
<b>IfcBoundaryNodeCondition</b>	Support, beam joint	Beam joints must be defined in the beam system
<b>IfcCircleHollowProfileDef</b>	Tube profile or polygon section	
<b>IfcCircleProfileDef</b>	Polygon section	32 polygon points
<b>IfcIShapeProfileDef</b>	HEA, HEB, HEM or IPE profile or polygon section	
<b>IfcLShapeProfileDef</b>	L profile or polygon section	
<b>IfcMaterial</b>	Material	
<b>IfcRectangleHollowProfileDef</b>	Hollow profile or polygon section	
<b>IfcRectangleProfileDef</b>	Rectangle section	
<b>IfcRelAssociatesMaterial</b>	Beam material	
<b>IfcRelAssociatesProfileProperties</b>	Beam section	
<b>IfcRelConnectsStructuralMember</b>	Beam node	
<b>IfcStructuralCurveMember</b>	Beam or edge	Constant section
<b>IfcStructuralLinearAction</b>	Linear load	
<b>IfcStructuralLinearActionVarying</b>	Trapezoidal load	
<b>IfcStructuralLoadGroup</b>	Load case	
<b>IfcStructuralLoadLinearForce</b>	Line load	
<b>IfcStructuralLoadPlanarForce</b>	Area load	
<b>IfcStructuralLoadSingleDisplacement</b>	Support displacement	
<b>IfcStructuralLoadSingleForce</b>	Point load	
<b>IfcStructuralLoadTemperature</b>	Temperature load	
<b>IfcStructuralPlanarAction</b>	Area load	
<b>IfcStructuralPointAction</b>	Punctiform load	
<b>IfcStructuralPointConnection</b>	Node	
<b>IfcStructuralProfileProperties</b>	General beam section	Will be ignored if a different section is imported
<b>IfcStructuralSurfaceMember</b>	Model face	
<b>IfcTShapeProfileDef</b>	T profile or polygon section	
<b>IfcUShapeProfileDef</b>	U profile or polygon section	

To import analysis models, either the FEM or the RSW mode must be active.

### Example

The following illustrations display the import of a framework from file *gtstrudl\_physical.ifc*. Source: <http://cic.nist.gov/vrml/cis2.html> (NIST, National Institute of Standards and Technology)



Dialog for selecting the import data

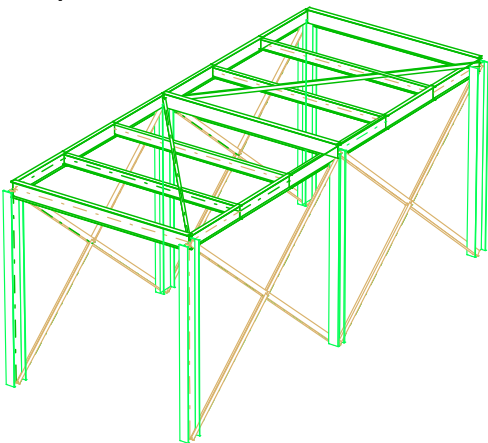
Imported framework

The file contains a analysis model with the name 'Loads and Results Exported from GTSTRUDL' and also building elements of various types that have been deselected for import.

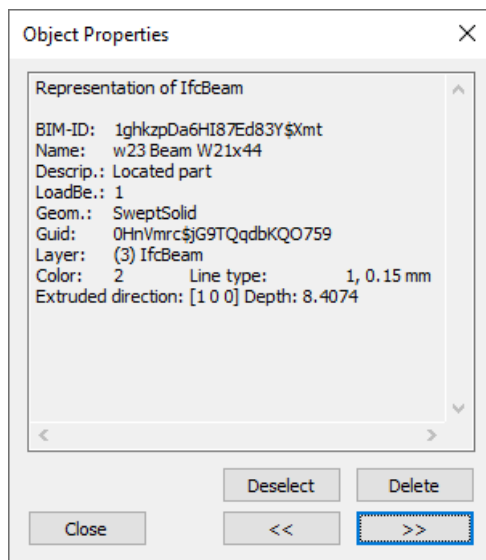
## 2. Importing the Geometry of the Building Elements

This functional area calls for construction model objects saved in the transfer file whose type is derived from *IfcBuildingElement*. They include, for example, *IfcBeam*, *IfcColumn*, *IfcSlab* and *IfcWall(StandardCase)*. Only objects with the geometric display *SweptSolid*, *Clipping*, *Brep*, and *MappedRepresentation* are considered. In InfoCAD special drawing objects are generated from these that make it possible to continue to use the geometry of the building elements. The snap modes *End*, *Middle*, and *Normal* allow you to use, for instance, characteristic points for measurements or for the design of model objects.

### Example



Building elements from the *gtstrudl\_physical.ifc* file. Source: <http://cic.nist.gov/vrml/cis2.html> (NIST, National Institute of Standards and Technology)



The figure above shows several further characteristics that are imported in addition to the geometry of the building elements. These include

- *GlobalId*: identification of the building element
- *Name*, *Descr.*: description
- *Material*: associated material description
- *LoadBe(aring)*: displays whether the element bears load (1) or not (0)

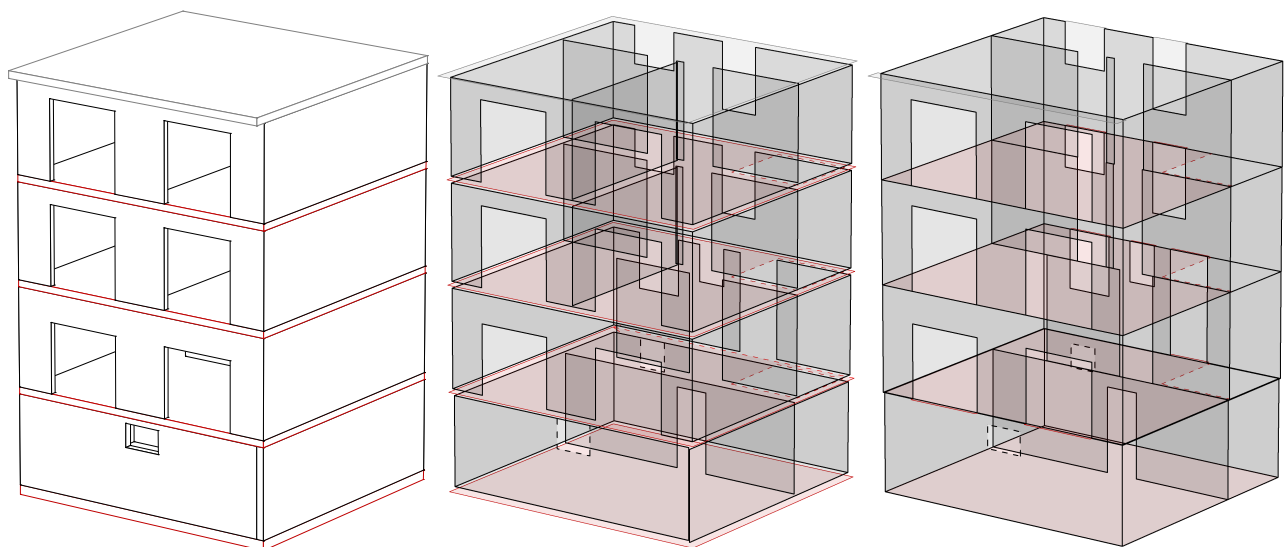
### Transformation into Model Objects

Imported building elements whose geometric display is based on the extrusion of a base area (*IfcExtrudedAreaSolid*) and the building elements

- *IfcBeam*
- *IfcColumn*
- *IfcMember*
- *IfcSlab*
- *IfcWall(StandardCase)*

can be transformed, with the help of the program, into model objects (select *Convert* from the context menu, FEM mode). As part of this process, linear elements are turned into edges with the meaning *Free beam*. The generated beam section corresponds to the profile definition used for the base area. Objects originating from *IfcSlab* and *IfcWall* whose base area consists of a traverse with exactly four points are converted into model surfaces.

The following illustration displays some of the building elements from the file *AC-90Smiley-West-14-07-2005.ifc*, as well as the resulting model surfaces. Source <http://www.iai.fzk.de/www-extern>.



Geometry of the building elements

Model objects, not joined

Model objects, joined

Because the model objects are created based on the wall or slab axis surfaces, gaps usually exist between neighboring

objects. As a remedy, all objects are joined with one another during the transformation and, if necessary, marginally reduced or enlarged. In addition, openings that are close to the edge are adjusted. The described operations help to improve the continuity between the model objects. This can eliminate the need for manual corrections in many cases.

### Export

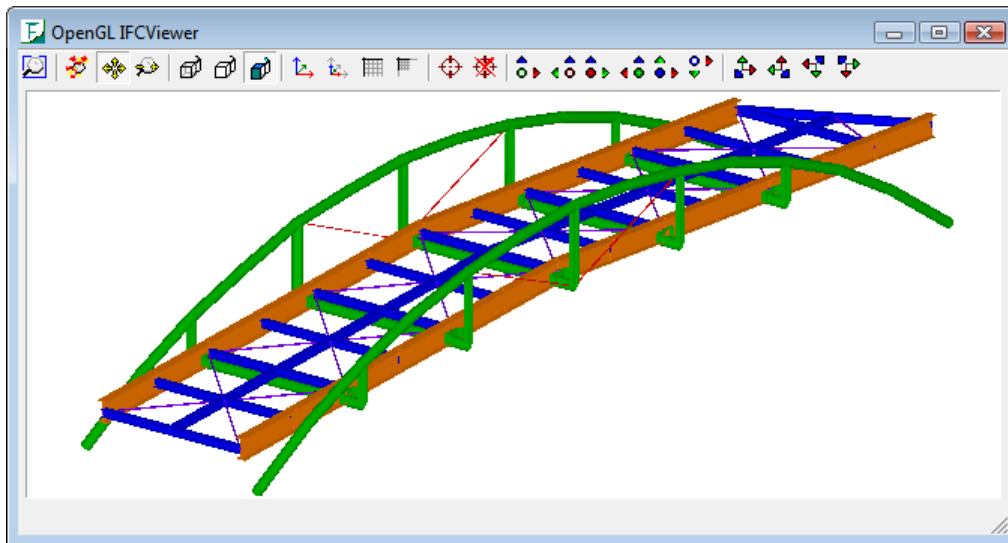
The function applies to model objects and frameworks. The following data will be exported:

#### Model objects

- Model edges with meaning 'free beam'
- Model faces

#### Framework, if no model objects exist

- Static analysis model without results
- Construction model of the 3D framework with the objects *lfcBeam*, *lfcColumn*



Framework exported with InfoCAD displayed in *lfcStoreyView*  
(Forschungszentrum Karlsruhe, Institut für angewandte Informatik)

## SAF Data Exchange

The SAF (Structural Analysis Format) exchange format is a file format based on MS Excel for the exchange of analysis models. Data exchange is performed within the import and export functions. During export the model objects of the active drawing are taken into account.

### Import / Export

The following model objects are handled:

- *Edge* with attribute *standard*, *beam series*, *support* and *free beam*
- *Hole* with attribute *standard*, *beam series* and *support*
- *Face*
- *Column*

## File Service

All input and results data are stored and managed in the project file as data sets. The file service represents the table of contents of the data sets in the project file. You can also use it to delete, copy or rename data sets. These actions are permanent and cannot be undone. Improper use may make the project file unusable.

Name	Size	Type	Modified	Rows	Columns	Identification
DEFORM.5	16156	R*4	29.01.24 14:40	577	7	-1
ELDEF.1	1672	R*4	29.01.24 14:40	22	19	-1
ELDEF.2	1672	R*4	29.01.24 14:40	22	19	-1
ELDEF.3	1672	R*4	29.01.24 14:40	22	19	-1
ELDEF.4	1672	R*4	29.01.24 14:40	22	19	-1
ELDEF.5	1672	R*4	29.01.24 14:40	22	19	-1
ELEMENTE	28340	I*4	29.01.24 14:39	545	13	1
EXTREMA	300	R*4	29.01.24 14:40	5	15	-1
FEM.TXT	9824	C*1	29.01.24 14:40	9824	1	-1
FESTH	3816	R*4	29.01.24 14:39	106	9	1

The data sets can be sorted according to the terms in their headers. The menu includes the following functions:

### File, Delete Results

Delete all analysis results.

### File, Compress

During compression unused parts are removed from the file. This can reduce the amount of space the files use on the hard drive.

### Records, Copy, Delete, Rename

With these functions you can use the \* wildcard to select groups of data sets (e.g. *LOAD.\**).

### View, Selection

You can select the data sets based on their names and properties for display in the table of contents. Input data sets have a positive identifier and result data sets have a negative identifier.

## Data Export in Text Format

Table contents can be copied directly from the table view to other programs via the clipboard.

Data sets in table formats (type I\*4 or R\*4) can also be exported via a command line.

```
[path]prog.exe [path]file.ext /ExportTxt:[path]cmd.txt /Out:[path]out.txt
```

These are:

[path]	Complete path specification;
prog.exe	InfoCAD program file, either <i>infocads.exe</i> for the study version or <i>Infocadw64.exe</i> for the full version;
file.ext	InfoCAD project file;
cmd.txt	Input file with specification of the data sets to be exported;
out.txt	Output file with the exported data sets. The table columns are separated by a tabulator (0x09). The first line of each column contains a short text describing the column content. Undefined column descriptions appear as question marks "?".

Return values

0	The export was successful.
-1	The export failed. If the output file could be created, it contains an error message.

If a path or file name contains spaces, the name must be enclosed in quotation marks,

e.g. `/Out:"c:\users\administrator\own exports\platte export.txt"`

## Example

```
"c:\program files\infograph\infocadw64.exe" platte_s.fem /ExportTxt:cmd.txt /Out:out.txt
```

### Cmd.txt

```
?AUFLR // Table of contents of the support reactions
KNOTEN // Node coordinates
FESTH // Supports (node-related)
AUFLR.* // Support reactions for all load cases (node-related)
DEFORM.1 // Node deformations for load case 1
```









### Out.txt

```
BEGIN ?AUFLR N=6
AUFLR.1 TIME=25.01.24 11:55
AUFLR.2 TIME=25.01.24 11:55
AUFLR.3 TIME=25.01.24 11:55
AUFLR.4 TIME=25.01.24 11:55
AUFLR.5 TIME=25.01.24 11:55
AUFLR.6 TIME=25.01.24 11:55
END ?AUFLR
BEGIN KNOTEN N=615 TIME=25.01.24 11:55
? ? ?
1 -2.23475 -0.559114 0
2 -2.22496 -0.110335 0
3 -2.66703 -0.126483 0
...
131 -0.445455 2.1 0
132 -0.890909 2.1 0
133 -1.33636 2.1 0
...
END KNOTEN
BEGIN FESTH N=106 TIME=25.01.24 11:55
? ? ? ? ? ? ?
131 614 615 -2 -2 -2 -1 -1 -1
132 614 615 -2 -2 -2 -1 -1 -1
133 614 615 -2 -2 -2 -1 -1 -1
...
END FESTH
BEGIN AUFLR.1 N=106 TIME=25.01.24 11:55
NR RX RY RZ MX MY MZ
131 0 0 -18.4156 0 0 0
132 0 0 2.7276 0 0 0
133 0 0 1.84187 0 0 0
...
END AUFLR.1
...
...
BEGIN AUFLR.6 N=106 TIME=25.01.24 11:55
NR RX RY RZ MX MY MZ
131 0 0 -19.5786 0 0 0
132 0 0 2.01733 0 0 0
133 0 0 -1.38568 0 0 0
...
END AUFLR.6
BEGIN DEFORM.1 N=577 TIME=25.01.24 11:55
NR UX UY UZ PX PY PZ
1 0 0 0.000759544 -0.000129361 0.000146784 0
2 0 0 0.000686177 -0.000189877 0.000154264 0
3 0 0 0.000726218 -0.000183836 1.0907e-005 0
...
END DEFORM.1
```



## Edit

The *Edit* menu contains the following functions:

	<i>Undo</i>	Cancel the previous action.
	<i>Redo</i>	Restore the previously canceled action.
	<i>Cut</i>	Transfer selected objects to the clipboard.
	<i>Copy</i>	Copy selected objects to the clipboard.
	<i>Insert</i>	Insert objects from the clipboard.
	<i>Delete</i>	Delete selected objects.
	<i>Select</i>	Select objects.
	<i>Select Group...</i>	Select objects according to group classification.
	<i>Select All</i>	Select all active objects.
	<i>Move</i>	Move, mirror, rotate and scale objects.
	<i>Copy Directly</i>	Copy objects without using the clipboard.
	<i>Generate</i>	Duplicate objects.
	<i>Properties Copy</i>	Assign the specified properties of a reference object to the selected objects.
	<i>Layer</i>	Manage a layer or assign it to selected objects.
	<i>Color</i>	Preset the drawing color or assign it to selected objects.
	<i>Line Type</i>	Preset the line type or assign it to selected objects.
	<i>Modify</i>	Modify the properties of selected objects (text, solid, circle, arc).
	<i>Modify Length</i>	Modify the length of the selected object.
	<i>Modify Endpoint</i>	Change an endpoint of the selected objects.
	<i>Intersect</i>	Intersect a selected object with another one.
	<i>Wall Aperture</i>	Insert a wall aperture into the selected wall.

## Undo / Redo

You have the option of canceling the previous changes you made to project data. Any action that is canceled in this manner can also be restored.

## Cut

The cut function allows you to transfer and save selected objects to the clipboard for insertion at a later time. This function is disabled if non-transferable objects are selected.

## Copy

There are two ways of using the copy function:

### Copy selected objects

The selected objects are copied to the clipboard and stored for insertion at a later time. This function is disabled if non-transferable objects are selected.

### Copy a rectangular image section

While holding down the ALT key, you can use the left mouse button to create a selection box in the drawing area. When

you use the copy function, the image section will be saved to the clipboard as a WMF file and can then be transferred to the analysis logs. You can also use this method to export graphics to other programs that support the WMF format.

## Insert

Objects saved in the clipboard are inserted with their 3D coordinates. This will replace any objects you have selected. You can move the inserted objects to place them at any specific position you want.

## Delete

All selected objects are deleted.

## Select

This function is used to select objects. Selected objects are displayed with dotted lines.

### Selection

<i>Objects</i>	Selection of individual objects using the mouse.
<i>Number</i>	Selection based on a number (beam, element, tendon group, load line).
<i>Window</i>	Selection based on a window. All objects that are completely within the window will be selected. If the window is dragged from right to left, all objects within the window plus any objects crossing the boundaries of the window are selected.
<i>4P</i>	Search region with four points. All objects that are completely within the area will be selected.
<i>last</i>	Selects the object specified last.
<i>&lt;-</i>	Switches to deletion of selected objects.
<i>-&gt;</i>	Switches from deletion mode back to selection mode.

### Special features for object selection

- Areas and solid elements can also be identified inside an element.
- Text must be identified within the area that is limited by the text.
- The option '*Object selection with crossing window (from right to left)*' in the snap settings can be deselected.

## Select Group

With this function you can select the objects based on logical aspects. This also enables direct access to individual object groups, even for complicated structure models. Using *AND logic*, all objects are selected that meet every specified condition.

## Select All

Select all visible objects.

## Deselect All

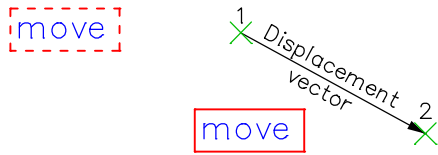
Deselect all visible objects. This command can be assigned to the edit menu or to a shortcut key using the function *Customize* of the view menu.

## Move

You can move selected objects using the following methods.

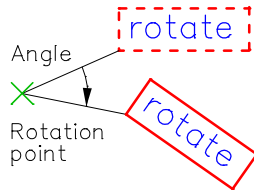
### Displacement vector

Movement through displacement. The displacement vector is defined by two points.



### Rotate

Movement through rotation. A rotation point is defined in 2D mode while a rotation axis is defined in 3D mode.



### Mirror

Movement through mirroring. A mirror axis is defined in 2D mode while a mirror plane is defined in 3D mode.



### Scaling

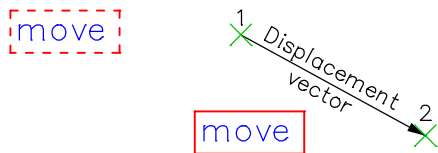
Movement through scaling in the global or local reference system. You can align objects on an axis or plane by specifying a scaling factor of zero.

## Copy Directly

Selected objects can be copied using the following methods.

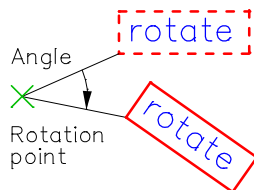
### Displacement vector

Copying through displacement. The displacement vector is defined by two points. The *Multiple* option allows you to make multiple copies of the selected objects.



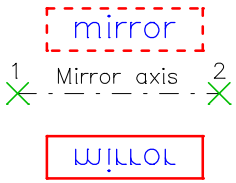
### Rotate

Copying through rotation. A rotation point is defined in 2D mode while a rotation axis is defined in 3D mode.



**Mirror**

Copying through mirroring. A mirror axis is defined in 2D mode while a mirror plane is defined in 3D mode.



**Parallel**

Lines and model edges can be copied through parallel displacement.

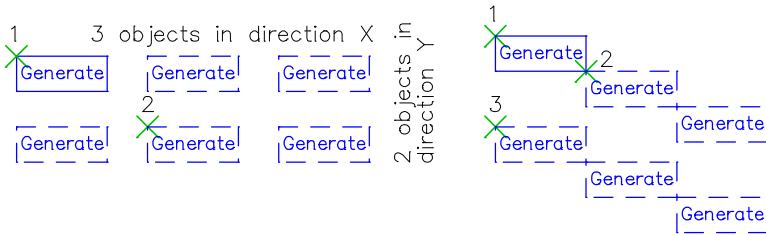
**Generate**

Selected objects are generated through displacement, rotation or a mixture of both. The original objects are always included in the number of objects to be generated.

**Displacement vector**

Generation through displacement. After you specify a displacement vector, the generation will take place in orthogonal form through the incremental addition of the displacement vector components (see image to the left). The specified number of copies will also be generated if the component equals zero in a direction. This may result in an overlapping of the copied objects.

After you specify a second displacement vector (and a third one in 3D mode), an *oblique generation* will be carried out through the incremental addition of each displacement vector (see image to the right).

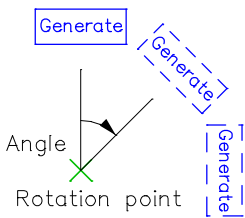


Global generation

Oblique generation

**Rotate**

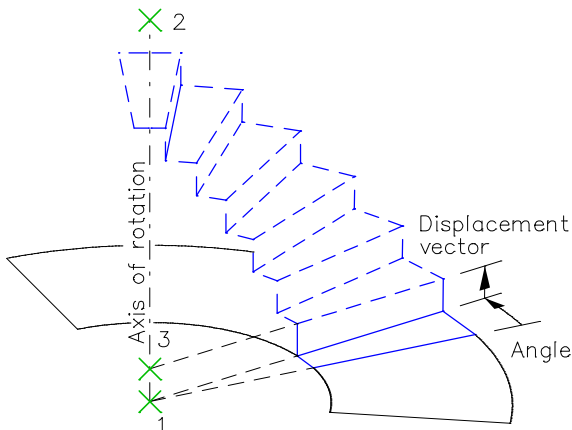
Generation through rotation. A rotation point is defined in 2D mode while a rotation axis is defined in 3D mode.



**Polar**

The generation is carried out by means of rotation around the specified rotation angle [°] and subsequent translation with the displacement vector. This allows you to create a helix, for example, in a 3D layout.

A stair step was initially specified as the prototype in the illustrated example. This step is incrementally rotated around the P1-P2 rotation axis through polar generation and then moved to the level of the next step with the P1-P3 vector.



## Color

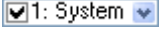

The color can be predefined for objects to be created or modified for selected objects. The current color is shown in the toolbar. If objects are selected, their color will be displayed.

## Layer

All graphical objects have a layer property to put them into logical groups. A Layer is identified by its number (from 1 to 255). For a description label can optionally be specified. For the graphical representation specific layers can be switched invisible, i.e. all objects with this property will be hidden. The layer property is also applicable for locking layers. Afterwards the affected objects are still visible but cannot be modified.

New objects get a layer preset by the user when they are created. It can be changed anytime later.

There are different controls for the management of layer properties of objects:

- Combination list box  1: System in the standard bar
- Dialog which is accessible using the layer button  in the standard bar or the edit menu
- Layer pane

## Line Type

The line type can be preset for new objects or modified for selected objects.

### Style

You can choose from the following line styles:

- 1 Solid
- 2 Short dashed
- 3 Long dashed
- 4 Dot dashed
- 5-15 Solid

### Width

Specify the line width for the objects. Alternatively, you can use the line width that has been defined for the object color (see *Print*).

## Modify

This function can be used to modify the properties of selected objects (text, solid, circle, arc).

### Text

<i>Position</i>	Define new text position.
<i>Alignment</i>	SThe possible text alignments are <i>Left</i> , <i>Middle</i> or <i>Right</i> .
<i>Direction</i>	Specify point in the direction of the text or align the text horizontally.
<i>Height</i>	Change the text height [m].
<i>Font</i>	Change the text font. You can use <i>Standard</i> , <i>Standard mono</i> , <i>Text</i> or <i>Text mono</i> vector fonts as well as true type fonts. Vector fonts of type <i>mono</i> have a fixed character width.
<i>Text</i>	Change the content of the text.

### 2D solid

Change the corner point of a solid.

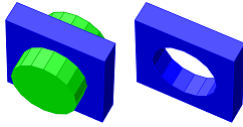
### Circle, arc

Change the radius.

### Solid

<i>Simplify</i>	Add surfaces of a solid together, if they are in the same plane and touch each other.
<i>Remove ports</i>	Remove holes from solids.

- Divide** Solids are cut into pieces by a defined plane.
- Add** Solids which touch or penetrate each other are added together to one solid.
- Subtract** Subtract a solid from another solid.



Example of the subtraction of two solids.

## Modify Length

You can use this function to modify the length [m] of a linear object that has been selected. The endpoint of the object that is closest to the selection point will be changed.

## Modify Endpoint

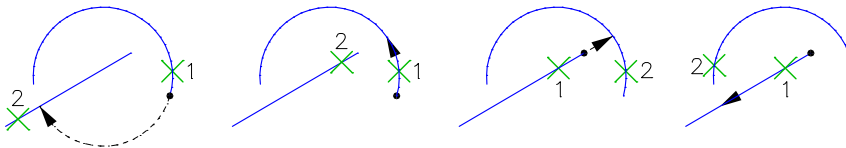
With this function you can change the endpoints of selected objects (e.g. lines, arcs, edges, holes, solid). The endpoint of the object that is closest to the last selection point will be changed. The radius of arcs will remain unchanged.

## Intersect

You can use this function to intersect selected objects (lines, arcs, edges, holes, walls).

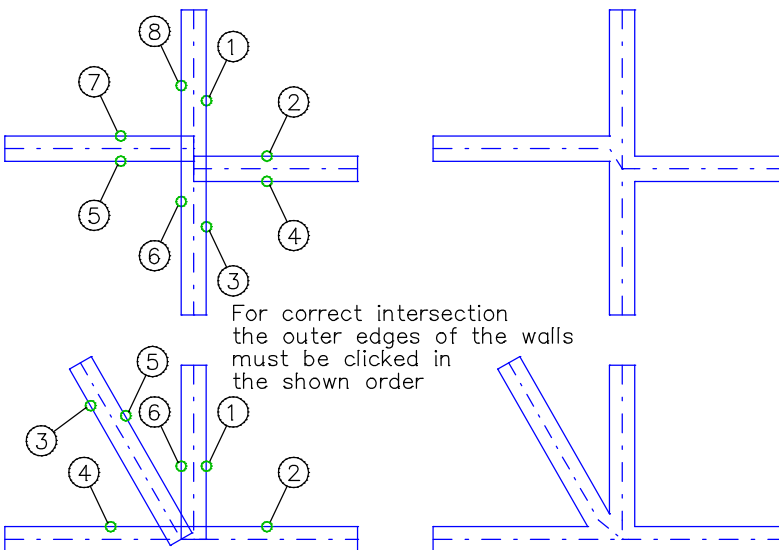
The figures below provide four examples of how lines and arcs are intersected. Proceed in the following order:

- Select the object to be modified at the end that you want to intersect (point 1).
- Activate the function.
- Select the limiting object (point 2).



If several lines are selected before the function is called, the ends of the lines closest to the selected limiting object are intersected with the function.

The figures below show you the steps involved in intersecting walls.



## Wall Aperture

This function allows you to insert an aperture in the selected wall. The resulting ends of the wall axes are connected by an edge object. The length of both of the resulting walls must measure at least half of the wall width since short walls may lead to very unfavorable or even improper aspect ratios of the elements during mesh generation.

## Divide

You can use the context menu to divide selected lines, arcs, edges, holes, model faces or solids.

## Join

With this function from the context menu selected lines, arcs, edges, holes or line supports can be joined to one object. The properties of the first selected object remains.

### Lines (also edges, holes)

The objects must be collinear. There can be gaps between them.

### Arcs (also edges, holes)

The objects must be on a imaginary circle. There can be gaps between them. The new arc starts at the starting point of the first selected arc.

### Line supports

Line supports are treated like lines or arcs, but no gaps are allowed.

## Convert

Using the context menu you can convert the following selected objects:

- line, circle and arc drawing objects into edge and hole model objects,
- edge and hole model objects into the corresponding drawing objects,
- line and 3D face drawing objects into structure elements (beams or area elements),
- line, circle and arc objects into NURBS curves,
- specific IFC building elements into model objects,
- free point and line loads into beam loads and vice versa.










Conversion is useful if a drawing comes from a DXF import, for example. In general, only drawing objects are transferred. After conversion into model objects is complete, you can use any of the automatic mesh generators to generate the FE mesh.

## Find GUID

This function in the context menu opens a dialog for searching objects by their GUID number. In the dialog, a list of GUID numbers can be entered line by line, which are then selected. The GUID number of objects can also be displayed using the *List* function. The search is especially helpful when searching for specific IFC or SAF objects.

## View

The *View* menu contains the following functions:

	<i>Redraw</i>	Redraw the representation area.
	<i>Zoom</i>	Zoom in on an image section.
	<i>View...</i>	Specify the view options for the structure.
	<i>3D</i>	Switch between 2D and 3D view.
	<i>Rotate</i>	Rotate in the 3D perspective.
	<i>3D-View</i>	Define any 3D view direction.
	<i>Projection</i>	Define central or parallel perspective.
	<i>Hidden</i>	Enable or disable hidden view.
	<i>Section Representation</i>	Enable or disable section representation.
	<i>3D Section Box</i>	Crop the visible area of the 3D view.
	<i>XY-Viewport</i>	Switch to the XY viewport.
	<i>XZ-Viewport</i>	Switch to the XZ viewport.
	<i>YZ-Viewport</i>	Switch to the YZ viewport.
	<i>1 Viewport</i>	Switch to 1-viewport display.
	<i>2 Viewports</i>	Switch to 2-viewport display.
	<i>Viewport...</i>	Specify the screen viewport.
	<i>Load...</i>	Load view with viewport setting.
	<i>Save...</i>	Save view with viewport setting.
	<i>Normal</i>	Activate normal view.
	<i>Layout</i>	Activate layout view.
	<i>Grid Points</i>	Enable or disable the auxiliary grid.
	<i>Scroll Bars</i>	Activate scrollbars to move the image section.
	<i>Toolbars</i>	Show or hide the toolbars.
	<i>Status Bar</i>	Show or hide the status bar.
	<i>Reset Window Layout</i>	Restores the default toolbar and docking window layout.
	<i>Standard View</i>	Show the results of load case combinations in the standard view. This view shows a reaction pair (e.g., Nx min, corresp. My, corresp. Mz, ...) of a reference point in a table row.
	<i>One Row per Result Location</i>	Show the results of combinations in a row depending on the result location.

## Redraw





This function is used to redraw the representation area.




## Zoom

The Zoom function allows you to specify which portion of the image is to be displayed.

### Input

	<i>1st point of zoom window</i>	Define the image section by entering a rectangle.
	All	The image detail will be displayed such that all objects are visible.
	Pan	Move the image section by entering a vector.
	+	Enlarge image.
	-	Reduce image.
	<< >>	The previous or next zoom level is activated.
	Sc.=1:100	Adjust the scale of the image section.

If your mouse has a scroll wheel, you can also use it to zoom while holding down the CTRL key. In addition, the  icon and the middle mouse button allow you to move the image section by dragging the mouse.

## 3D Representation

This function is used to switch between 2D and 3D view.

### 3D-View

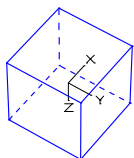
With this function you can define a specific view direction.

#### Input

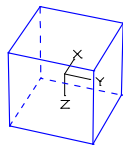
	<i>1st point of view plane</i>	Describe the view plane with three points. The view direction is defined perpendicular to the plane.
	Axis	View in the direction of a global axis.
	Direction	Specify a point that defines the view direction to the centroid of the object.
	Vector	Enter the components of the direction vector from the object's centroid to the new view location.
	Standard	Return to the standard view.

## Rotate

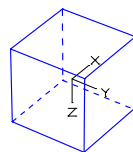
With this function you can change the view direction of the observer in 3D view. The rotation direction refers to the view location, meaning the movement occurs around the stationary object. The specified angle of tilt [°] is relative and refers to the current direction. A rotation cannot exceed the zenith or nadir point.



Up: 15°



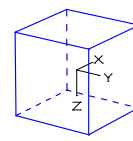
Left: 15°



Default setting



Right: 15°



Down: 15°



Alternatively, you can use this cursor to rotate the view in any direction in 3D mode.

If your mouse has a scroll wheel, you can use it to rotate around the vertical axis while holding down the Shift key or around the horizontal axis while holding down the Shift and CTRL keys.

## Projection

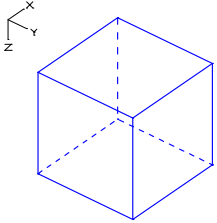
This function allows you to choose the projection type for the 3D perspective.

### Input

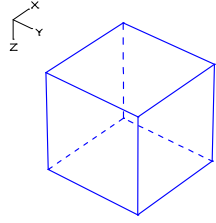
*Central 0...9* Display in central perspective with no change to the angle of view. The setting '0' produces the effect of viewing the object from a large distance using a long focal length (telephoto lens), while the setting '9' produces the effect of viewing the object from up close using a short focal length (wide-angle lens).

*Parallel* Display in parallel perspective (isometric) with no change to the angle of view.

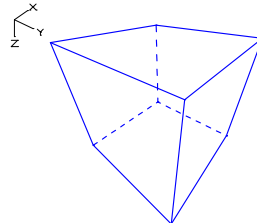
The following images show both the central and parallel perspectives.



*Parallel*



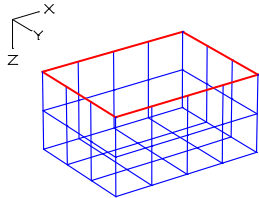
*Central 0*



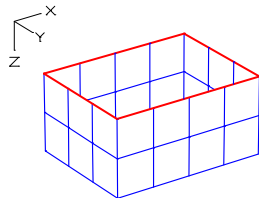
*Central 9*

## Hidden

Use this function to prevent hidden objects parts from being drawn in 3D view.



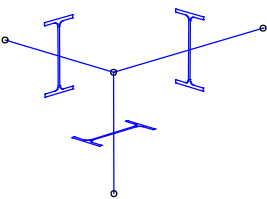
*Hidden: Off*



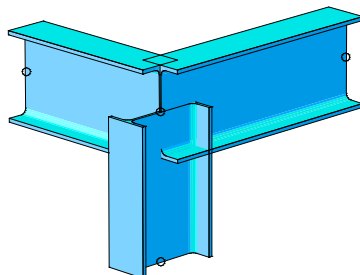
*Hidden: On*

## Section Representation

Click this function to display all elements and their section dimensions in 3D view.



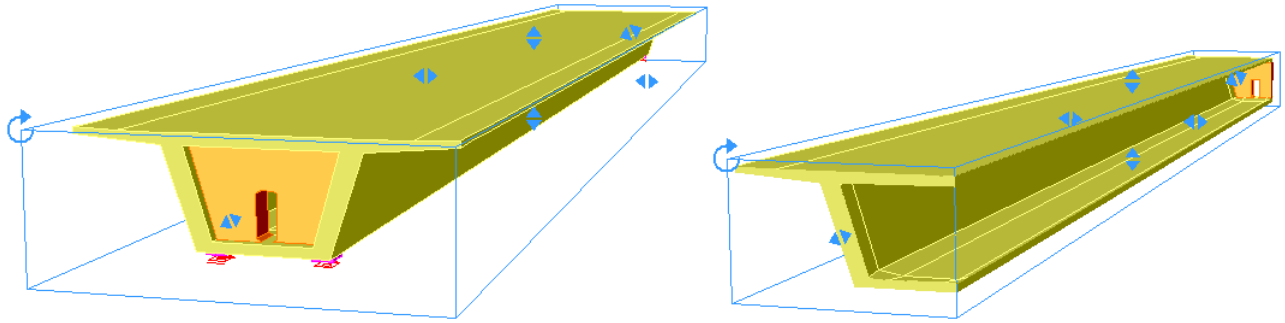
*Section Representation: Off with beam section*



*Section Representation: On*

## 3D Section Box

The 3D section box curtails the visibility of the 3D view. The boundaries of the 3D section box can be modified using the controls (blau arrows).



*3D section box after activation*

*Modified 3D section box*

The handling is also shown in the video "[https://download.infograph.de/video\\_en/3D-Sectionbox.mp4](https://download.infograph.de/video_en/3D-Sectionbox.mp4)" (German).

## Viewport

This function is used to adjust the viewport setting.

### Input

<i>Number of viewports</i>	Select 1 or 2 viewports.
<i>1st viewport</i>	Specify the desired coordinate system for the left viewport.
<i>2nd viewport</i>	Specify the desired coordinate system for the right viewport.
<i>Constant coordinate</i>	If only one viewport is selected, the constant third coordinate will be added to the 2D coordinates. This allows you to set the xy viewport to a specific z level, for example.
<i>View Direction</i>	Selection of the direction of view.

## Layout

Use the Layout function to switch to the layout view. Along with the defined page formats, the system is displayed in the same manner as it is printed. You can work in the layout view exactly as you would in the normal view.

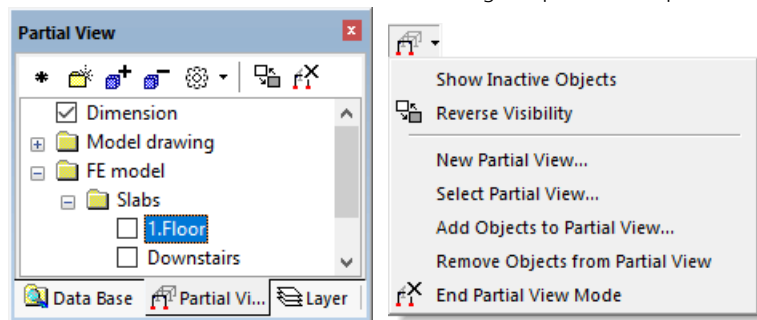
## Partial View

For a structured representation of larger constructions, it is possible to create *partial views*. Every object can be part of multiple partial views. The assignment to a partial view does not change the object properties. Representations can be realized, which can't be realized by exclusively using layers. If, for example, in a high-rise construction the layers have been structured separately for each floor, it is impossible to display only the objects in one axis of the building. With the corresponding partial view this can be accomplished without modifying the objects.

The user can create any number of partial views, which can be named freely. They can subsequently be renamed without any problems, because the name is not used for identification. Groups of partial views can be combined in folders.

If one or more partial views are activated for representation, the so-called 'Partial view mode' is automatically activated. In the combination list box of the standard bar the existing partial views are displayed. Even in this mode the layers of the objects keep their relevance. In other words, objects in disabled layers remain invisible. Objects created in the partial view mode are added to the active partial view.








Partial views can be created and modified using the partial view pane or the partial view button menu.



Partial view pane

Partial view button menu

The following functions are available in the partial view pane:

- Selected partial view.
-  Create a new partial view including the selected objects.
-  Create a new folder below the current partial view.
-  Add the selected objects to the selected partial view (e.g. **1.Floor**).
-  Remove the selected objects from the activated as well as the selected partial views.
-  Reverse the visibility of the objects.
-  Close the partial view mode and return to the view according to the layer feature.
-  Extras
  - Sort
  - Remove empty partial views
  - Show inactive objects
  - Divide model structure into planes
  - Divide model structure into storeys

## Draw

This menu contains all relevant functions for entering drawing objects.



*Line*



*Text*



*Circle*



*Arc*



*2D Solid*



*Nurbs*

*Model*



*Edge*



*Wall*



*Hole*



*Column*



*Face*



*Cone*



*Solid*



*Dimension*



*Image* Adds images of formats *bmp, ico, jpg, png, gif, tif, emf, wmf* in the drawing.

For more detailed information on how to use drawing objects, refer to the *Introductory\_Examples.pdf* document which is installed in the program directory.

## Line

This function specifies a line.

### Input

*1st point* Specify the start of the line. Additional lines can be entered consecutively once the endpoint has been specified.

*Start* Specify a new start point.

*Close* Connect the last point with the first start point.

## Text

This function is used to enter text.

### Input

*Text starting point*  
Specify the text starting point.

*Left, Middle, Right*  
Select left-aligned, centered or right-aligned text input. The text direction and text height [m] must also be specified.

*Font*  
Preselect the text font for further text input. You can use *Standard, Standard mono, Text* or *Text mono* vector fonts as well as true type fonts. Mono type vector fonts have a fixed character width.

Text height 

## Circle

Use this function to specify a circle.

### Input

- Center point*      Input using center point, circle plane (only in 3D) and radius.
- 3Points*            Input using 3 circle points.

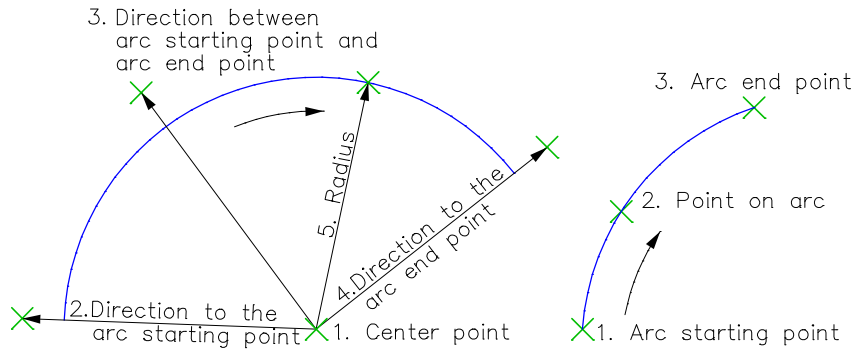
## Arc

This function is used for entering arcs.

### Input

- Center point*      Specify an arc using the following determinants:
  1. Center point,
  2. Direction to beginning of arc,
  3. Direction between arc beginning and end,
  4. Direction to arc end,
  5. Radius.

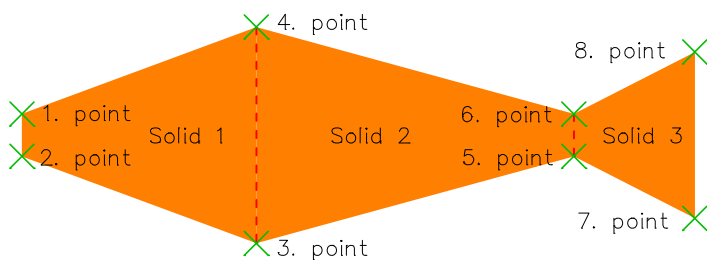
- 3Points*            Input using 3 arc points.



Variants for specifying arcs.

## 2D Solid

A 2D solid is used to describe a square color surface. The following figure shows the order of data input.



It can be specified that 2D solids are always drawn behind other objects in the 2D view (selecting 'draw order' during the input of the first point). In the standard case the objects are displayed in the order of their input.

## NURBS

**Non-Uniform Rational B-Splines** are mathematically defined curves, surfaces and solids that can be used to model any kind of (free-form) shape. In principle you can use NURBS to render any shape that is technically constructible or that exists in nature. Here they are used to generate an FE mesh.

NURBS can normally be edited interactively in an intuitive manner using control points. These points are either directly on the NURBS or act as a rubber band that connects to a NURBS.

### Definition of a NURBS Curve

A NURBS curve with a degree of  $p$  is defined by:

$$C(u) = \frac{\sum_{i=0}^n N_{i,p}(u) w_i P_i}{\sum_{i=0}^n N_{i,p}(u) w_i} = \sum_{i=0}^n R_{i,p}(u) P_i ; \quad a \leq u \leq b$$

with

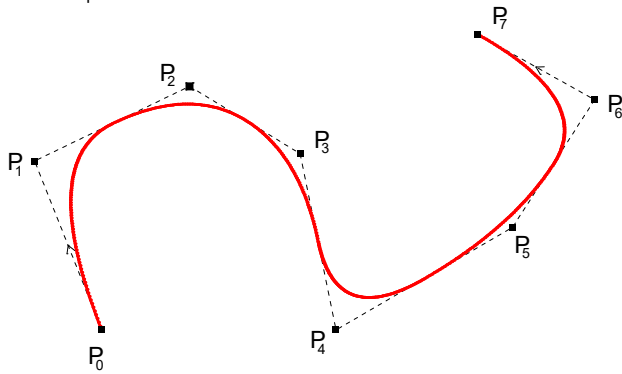
- $u$  Curve parameter.
- $n$  Index of the control point at the curve end.
- $p$  Polynomial degree of the curve.
- $P_i$  Control points; these generate the control polygon.
- $w_i$  Weights of the control points.
- $N_{i,p}(u)$  B-spline base function of degree  $p$ .
- $R_{i,p}(u)$  Rational base function of degree  $p$ .

The curve parameter  $u$  is defined on the node vector

$$U = \{a, \dots, a, u_{p+1}, \dots, u_{m-p-1}, b, \dots, b\}$$

This parameter has  $m+1$  elements. The first and last nodes are repeated  $p+1$  times. The elements of the node vector are arranged in ascending, non-cyclic order, yet they can be unevenly distributed. Normally  $a=0$  and  $b=1$  are set. You can generally choose any degree for the base functions. For construction and computer-related reasons,  $p$  should be between 1 and 5.

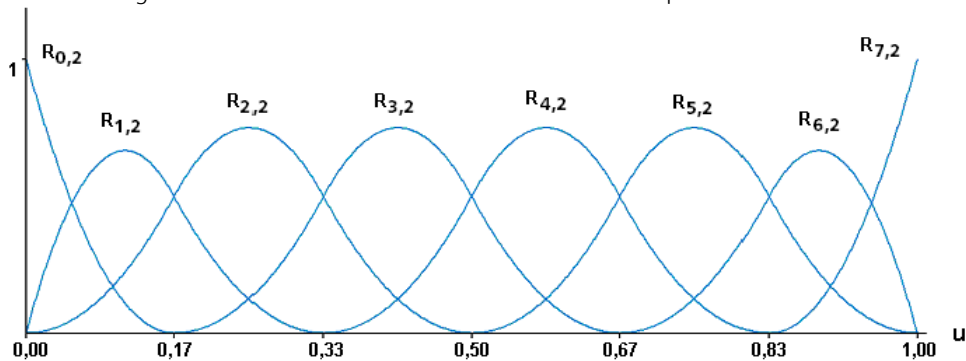
The following illustration shows a NURBS curve of degree 2 with the associated control polygon. The weighting of all control points is  $w = 1$ .



The base node vector is:

$$U = \{0, 0, 0, 1/6, 1/3, 1/2, 2/3, 5/6, 1, 1, 1\}$$

The following illustration shows the course of the associated B-spline base function.



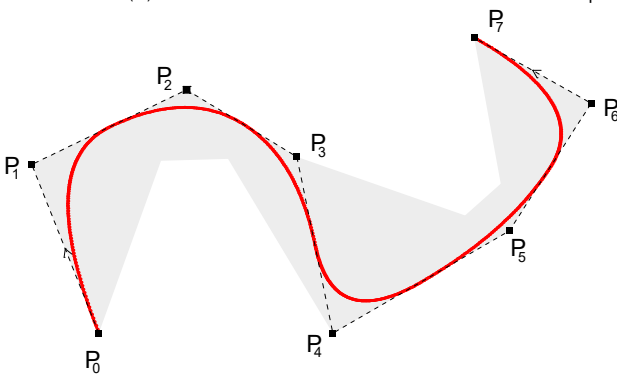
**Important properties of the rational base functions  $R_{i,p}$**

- $R_{i,p}(u) \geq 0$  for all  $u \in [a, b]$ .
- The sum of all base functions at each location  $u \in [a, b]$  is equal to 1.
- Each base function has exactly one maximum in the interval  $u \in [a, b]$ .
- $R_{i,p}(u) = 0$  outside the interval  $[u_i, u_{i+p+1}]$ .
- For  $w_i = \text{const.}$ ,  $R_{i,p}(u) = N_{i,p}(u)$  applies.
- All derivatives of  $R_{i,p}(u)$  exist within a node span, at a node  $R_{i,p}(u)$  is always differentiable  $(p-k)$  times.  $k$  is the multiplicity of the respective node.

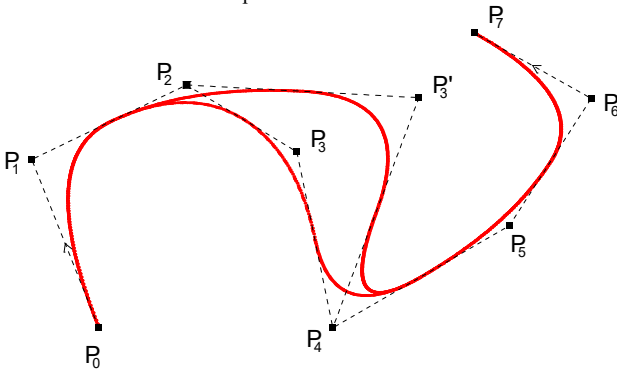
**Important properties of NURBS curves**

The above properties of rational base functions result in the following properties of NURBS curves:

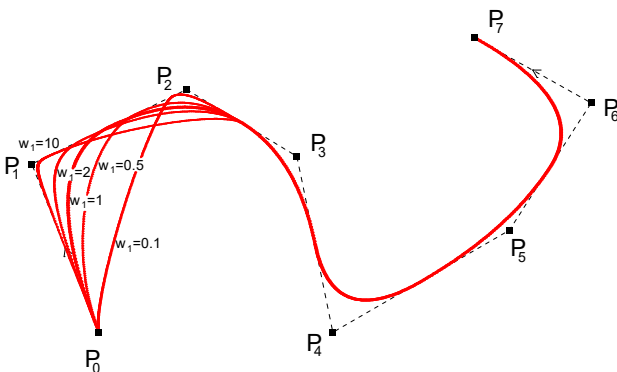
- $C(a) = P_0$  and  $C(b) = P_n$  for  $u \in [a, b]$ .
- An affine transformation of NURBS curves is performed by transforming the control points.
- The curve  $C(u)$  lies within the convex shell of the control polygon.



- A NURBS curve without inner nodes is a rational Bezier curve. Consequently, NURBS curves contain both nonrational b-spline curves as well as nonrational Bezier curves.
- Changing the coordinates or the weight of a control point  $P_i$  will only affect the part of the curve that is assigned to the interval  $u \in [u_i, u_{i+p+1}]$ .



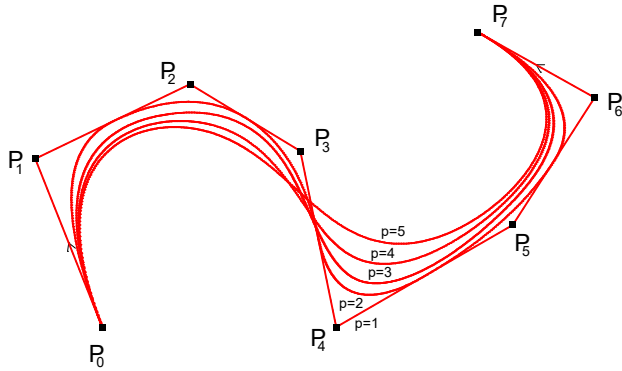
Moving the control point  $P_3$  to  $P_3'$



Changing the course of the curve by varying the weight of the control point  $P_1$

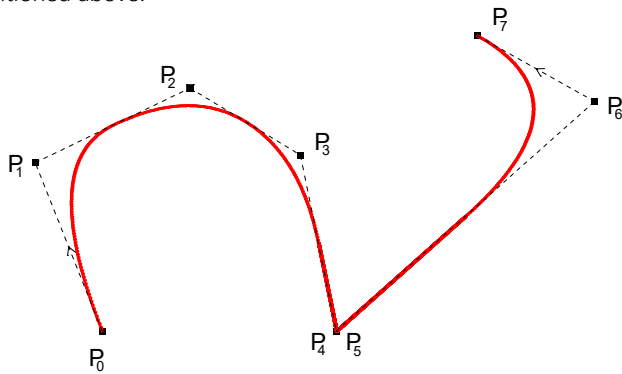


- The smaller the degree of a NURBS curve, the smaller the deviation from the associated control polygon. The following curves all have the same control polygon and the same node vector, but their polynomial degree  $p$  differs.



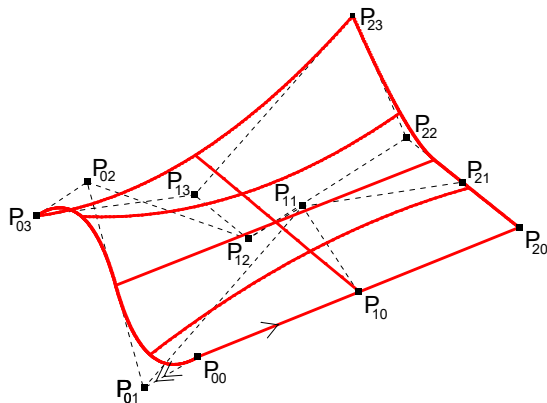
- The use of multiple control points results in buckling in the curve. This is directly due to the above condition of the convex shell.

The next illustration shows the course after the control point  $P_5$  has been moved to  $P_4$ . The NURBS curve is linear in the vicinity of the buckling. A comparable effect can be achieved by using  $p$ -times nodes since this results in a discontinuity as mentioned above.



**NURBS surfaces and solids**

The properties of NURBS curves can formally be transferred to surfaces and solids. All principles are still valid in this case. The numeric processing, however, is significantly more complex due to the two- or three-dimensional parameter range. For more information on the mathematical principles involved, refer to the relevant technical literature.



*NURBS surface with associated control polygon*

## Input and Editing

A NURBS curve is entered using either its control points or the interpolation points on the NURBS curve. The following context functions are available to edit NURBS:

- Reverse curve direction
- Move control point
- Change control point weight
- Connect curves
- Invert surface directions  $u, v$
- Insert nodes

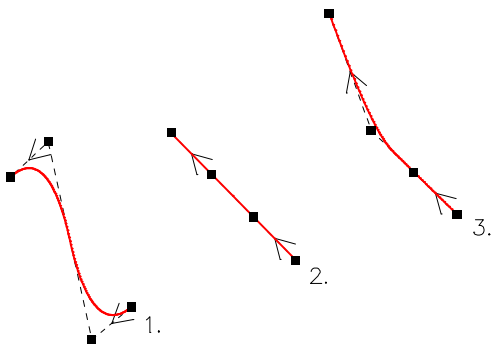
NURBS surfaces are created based on NURBS curves. The following context functions are available for this purpose:

- Create surface from profile curves
- Create surface from 4 edge curves
- Create surface of revolution

NURBS solids are created based on NURBS surfaces. The following context functions are available for this purpose:

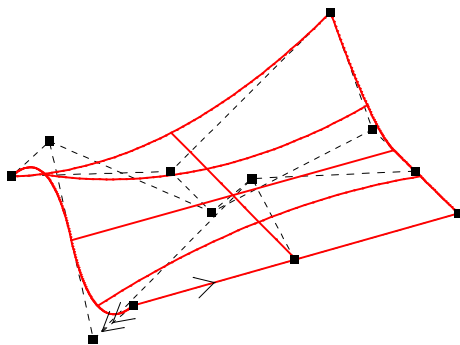
- Create solid of revolution
- Create solid from surfaces

### Example of a surface created from profile curves



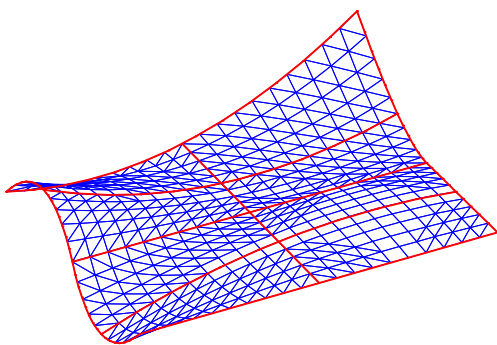
Three NURBS curves

The function *NURBS curve* is used to draw three NURBS curves. It is important that the curves are aligned uniformly.



NURBS surface

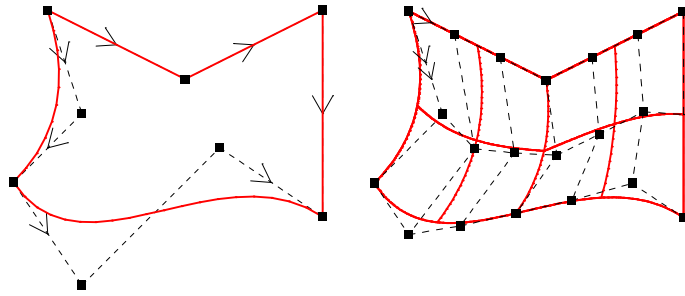
The three NURBS curves are selected in order (1/2/3). Once the NURBS curves have been selected, the *Create surface from profile curves* context function is chosen. The degree of the surface in the 2nd direction is 2 in this example.



NURBS surface with FE mesh

The *Generate FE mesh* context function of the NURBS surface is used to create an FE mesh.

**Example of a surface created from four boundary curves**

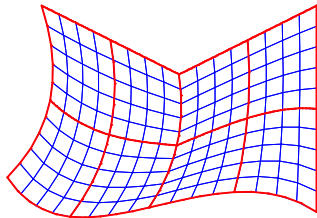


Two NURBS curves

NURBS surface

The *NURBS curve* function is used to draw four different and connected NURBS curves.

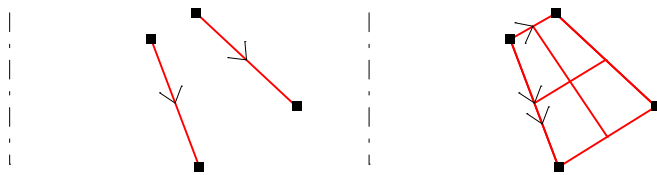
Once the NURBS curves have been selected, the *Create surface from 4 boundary curves* context function is chosen.



NURBS surface with FE mesh

The *Generate FE mesh* context function of the NURBS surface is used to generate an FE mesh.

**Example of a rotational solid**

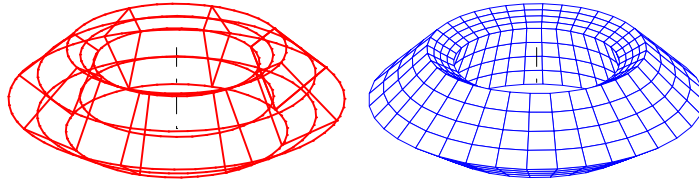


Two NURBS curves

NURBS surface

The *NURBS curve* function is used to draw NURBS curves of the first degree with two control points each.

Once the NURBS curves have been selected, the *Create surface from profile curves* context function is chosen.



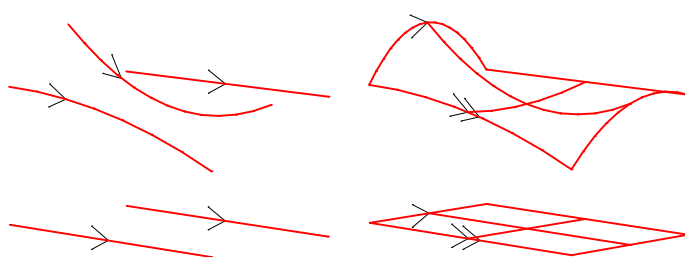
NURBS solids

FE solid model

The *Create solid of revolution* context function of the NURBS surface is used to generate a NURBS solid.

The *Generate FE mesh* context function of the NURBS solid is used to generate solid elements (VQ83).

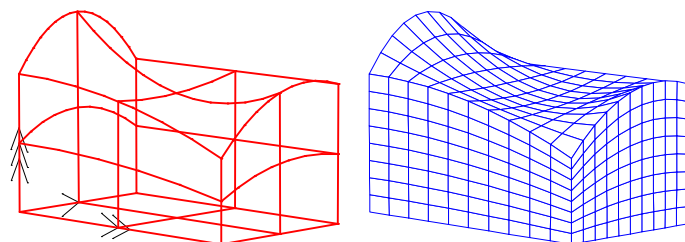
**Example of a solid created from surfaces**



NURBS curves

NURBS surfaces

The *NURBS curve* function and the *Create surface from profile curves* context function are used to create two 3D NURBS surfaces. It is important that the NURBS are aligned uniformly.



NURBS solids

FE solid model

Once the NURBS surfaces have been selected, the *Create solid from surfaces* context function is chosen. Finally, the *Generate FE mesh* context function of the NURBS solid is used to create solid elements (VQ83).

## Model

Model objects describe the geometry and the properties of single structural components. They provide the basis for the program-controlled generation of the finite element mesh by a mesh generator (see also *Mesh generation > Form-sensitive or grid-shaped from model*).

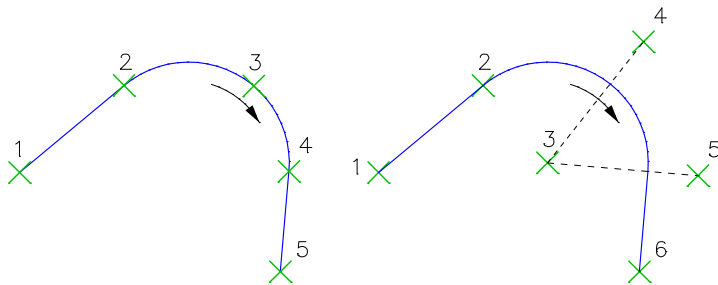
A framework can also be created with the function *Generate beams* of the context menu.

## Edge

Edges define the border and the axes of structural components. Depending on their properties, they can also define a line support, a beam series, free beams (e.g. columns), springs or link elements (see *Property dialog for edge and hole objects*). Together with the model object *Wall*, edges are used to limit a plane area for the mesh generation.

### Input

- Start of edge* Specify the start of the edge.
- End of edge* Specify the end of the edge.
- Start* Specify the start of a new edge.
- Arc* Specify an arc-shaped edge section.
- Circle* Specify a circular edge.
- Properties* Preset the edge properties.



Variants for constructing edges

- |                       |                                   |
|-----------------------|-----------------------------------|
| Input via arc points: | Input via center point:           |
| 1 edge start point    | 1 edge start point                |
| 2 edge endpoint       | 2 edge endpoint                   |
| 3 point on arc        | 3 center point                    |
| 4 arc endpoint        | 4 direction between start and end |
| 5 edge endpoint       | 5 direction of arc endpoint       |
|                       | 6 edge endpoint                   |

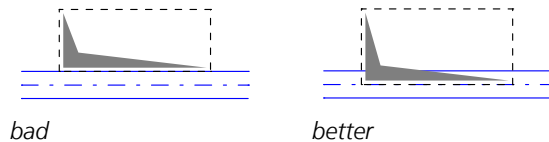
## Hole

Hole objects define openings for mesh generation. The hole objects have to be lines or arches and must form a closed polygon. They are displayed short dashed (line style 2) and can, for example, define a beam series (see *Property dialog for edge and hole objects*).

### Input

- Start of hole* Specify the beginning of the hole object.
- End of hole* Specify the endpoint of the hole object.
- Close* Close the polygon.
- Start* Specify the beginning of a new hole object.
- Arc* Specify an arc-shaped hole object.
- Circle* Specify a circular hole.

With respect to mesh generation, note that holes along the walls should be extended to the wall axes since otherwise elements between the hole and the wall axis will be generated.



## Property Dialog for Edge and Hole Objects

Basically the meaning for the mesh generation is defined here.

The following options can be selected:

- *Standard*  
Boundaries or fixed lines for generating area elements.
- *Support*  
Line support in the area element mesh. The support properties are specified in the dialog fields under *Support*.
- *Beam series*  
Additional beam series in the area element mesh (e.g., for downstand beams). The section is selected using the *Section at the beginning* or *Section at the end* dialog fields. In conjunction with the *Polygon*, *Rectangle* and *Beam* section types, you can specify different start and end sections. The program will then interpolate a section for each beam element. This enables the modeling of a haunch. The position of the polygon points are interpolated for polygon sections. For this to work, the number and arrangement of the polygon points must match one another. In addition, beam eccentricities can be defined.
- *Support+Beam series*  
Line support and beam series in the area element mesh.
- *Free beam*  
Beam elements with the same length are created based on the specified pitch (for sections, see *Beam series*). The coordinate system of the beams is rotated around the beam axis using the selected angle. Using this property at the edge or inside area element meshes is not effective!
- *Pile beam*  
For each ground layer, beam elements and bedding properties are created based on the specified pitch. The layers of the bedding profiles (see example *Pile foundation*) and the ordinate  $z_0$  apply in global z direction. Beams above  $z_0$  and below the bedding profiles get the bedding properties of the *Section at the beginning*. The coordinate system of the beams is rotated around the beam axis using the selected angle. Using this property at the edge or inside area element meshes is not effective!
- *Spring*  
Springs are created between the begin and the end of each edge or hole object.
- *Link element*  
Link elements are created between the nodes at the begin and end of each edge or hole object.

The layers and colors of the objects to be created are determined by the properties defined in the interface (*Standard*), the properties of the edge (*Convert*) or through direct selection.

## Wall

Walls define the edge of structural components for mesh generation and a line support as well. The geometry of a component is determined by the wall axes. Walls are specifically suited for creating slab systems.

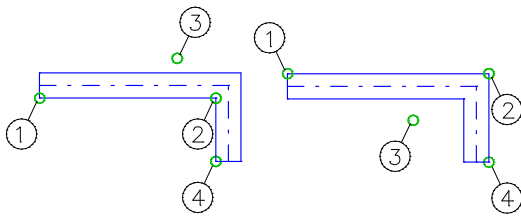
### Input

*Start of wall* Specify the start point of the wall.

*End of wall* Specify the endpoint of the wall. The length of wall must measure at least half of the wall width since short walls may lead to very unfavorable or even improper aspect ratios of the elements during mesh generation.

*Side on that the wall is*

Input another point to specify on which side the wall is placed in relation to the defined start and end points. The following figures show two methods for entering the third point and continuing wall input with a fourth point.



Instead of specifying the wall side, you can also select the wall axis.

- Start* Specify the start of a new wall.
- W =* Specify the wall width.
- Axis: On* Enable or disable the wall axes view.
- Properties* Preset the wall properties. These properties are the support conditions and the wall name.

With respect to mesh generation, note the following:

Problem	bad	better
Avoid wall protrusions. These will result in unfavorable aspect ratios during mesh generation and might yield unwanted singularities in the results.		
Buttressing walls should be avoided since the supports created by mesh generation will often result in unwanted restraints.		

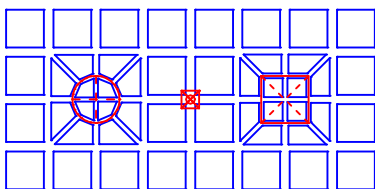
## Column

Columns define a point support and, optionally, a column head condensation in the element mesh.

### Input

- Column center* Specify a point column without measurements.
- Circle* Specify a circular column based on the center point and diameter.
- Rectangle* Specify a rectangle column based on the center point, the measurements in the x and y directions and the angle of rotation.
- Corners* Specify a rectangle column based on three corner points.
- Properties* Preset the column properties. These properties include the support conditions, a name and the column head condensation for which the following symbols are used:
  - Condensation enabled:
  - Condensation disabled:

The figure below shows the *Circular column*, *Point column* and *Rectangle column* with the corresponding FEM mesh after automatic mesh generation (representation in shrink mode).

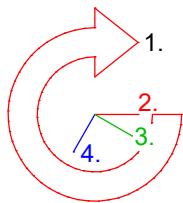


With respect to mesh generation, note the following:

Problem	bad	better
Columns should be placed centrally under the downstand beams (edges) since otherwise there would be no direct support for the downstand beams.		
Buttressing walls should be avoided since the wall and column supports created by mesh generation often result in unwanted restraints.		
For column measurements $\gg 2/1$ the column head condensation should be disabled or a wall should be drawn in the place of the column since otherwise the aspect ratio of the elements will be unfavorable.		 or 

### Face

A model face describes a 2D area for later meshing using a mesh generator. Edges, walls and holes form the outer boundary of the area and must therefore be specified before the model face. All model objects inside the face are included in the mesh generation.

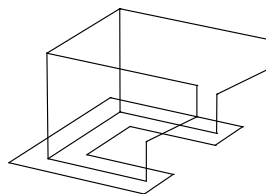


The sign of a selected model face:

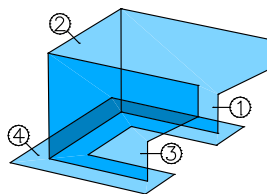
1. Rotational direction of the local z-axis. The direction can be reversed using the context menu.
2. Local x-axis of the element system (red).
3. Local x-axis of the internal force system (green, if different from the element system).
4. Local y-axis of the reinforcement system (blue).

You can assign line hinges to a face using the context menu or additional properties for the FE mesh using a separate dialog.

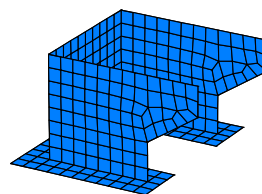
The intersection curves for the selected model faces and cones can be calculated using the context menu. The subareas which result are made visible with indented contours (see example).



Edges



Model faces



FE mesh

### Input

Select border objects

Selection of edges, walls or hole objects that form the surrounding edge of the model face. Depending on the selection, the program will search for other limiting objects until the face is unambiguously described. It cannot be limited exclusively by hole objects. Click selected objects again to deselect them.

Search for boundary Properties

Search for limiting objects that describe the greatest possible model face. Preset the properties.

### Properties Dialog

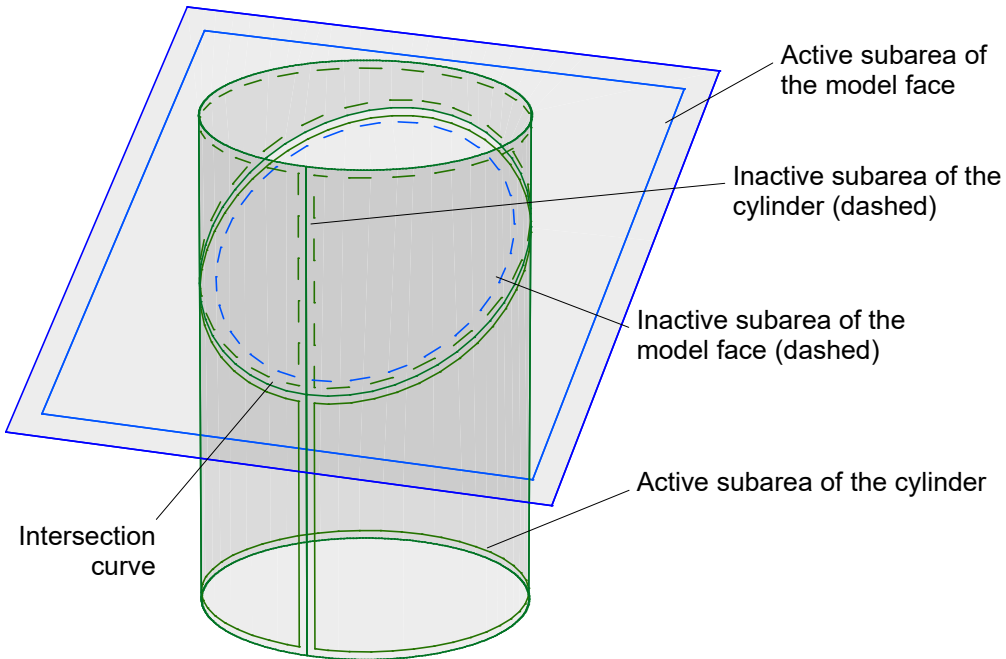
The dialog contains the following properties:

- Section of elements
- Layer of elements
- Color of elements
- Partial View of elements
- Name
- Factor for mesh width
- Consideration of subareas during mesh generation
- Rotation of the element, internal force and reinforcement systems [°]
- Eccentricity of elements in their local z direction [m]

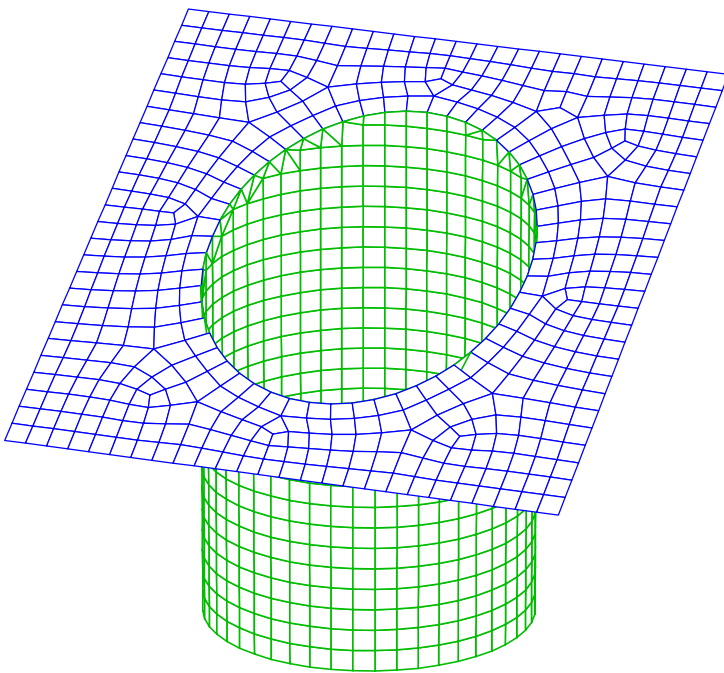
If *Standard* is selected, the property defined in the surface will apply; if *Take over* is selected, the property of the model face will apply. The significance of subareas is illustrated in the example below.

### Example: Connection of a cylinder to a model face

The example shows the significance of the active and inactive subareas of a cylinder and a model face.



*Model face and cylinder with intersection curve and subareas.*

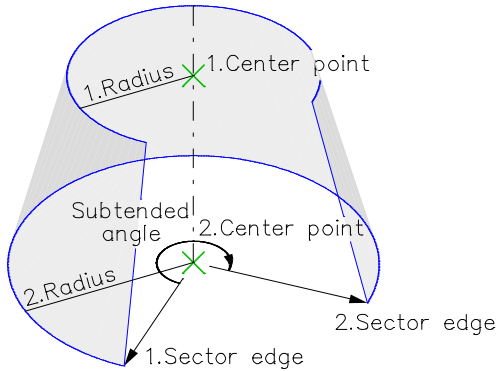


*Generated element mesh*



## Cone

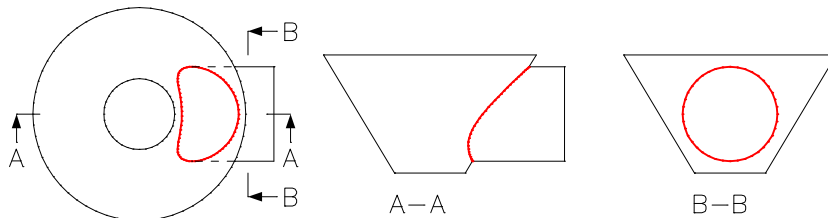
The *Cone* model object describes a truncated cone segment for later meshing. You can use the context menu to determine the intersection with other selected cones or model faces. A cone can also be assigned various properties for the FE mesh.



A truncated cone is defined using the first and second center points, first and second radiuses and the sector. Alternatively it is possible to select boundary objects (edges) to define the cone.

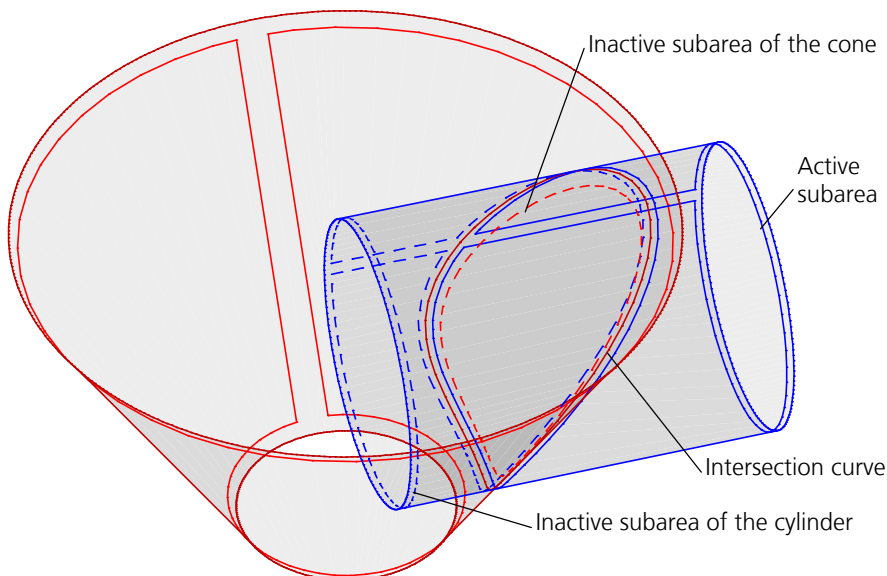
### Example: Drain of a container

The example shows how a cone and cylinder are meshed while taking their intersection into account.

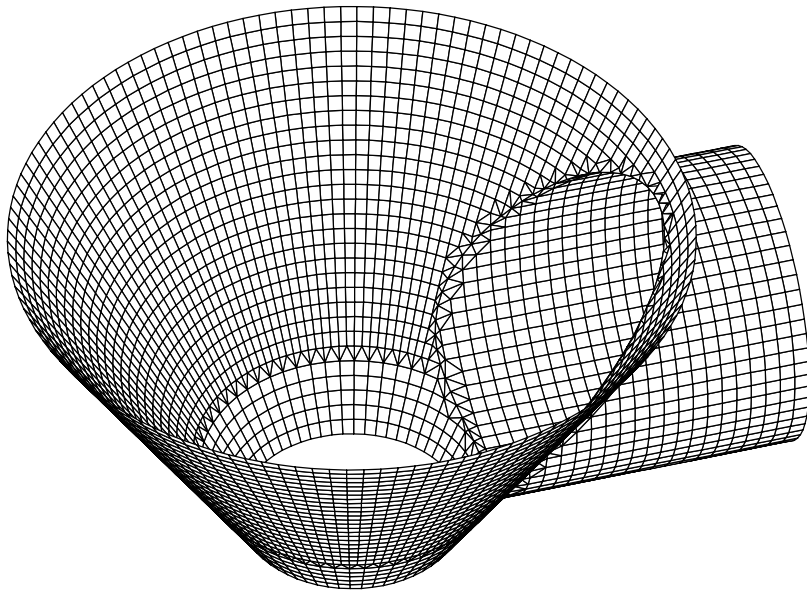


Proceed as follows:

1. Draw the cone and cylinder.
2. Select both objects and determine the intersection (context menu).  
A indented continuous line is inserted to make the resulting subareas visible and selectable.
3. Select and deactivate the interior subareas of the cone and the cylinder (*Model Face, Cone* from the context menu).  
Inactive subareas are shown with dashed lines.
4. Generate the element mesh with the form-sensitive mesh generator.



Cone and cylinder with intersection curve and subareas.



Generated element mesh

### Solid

The model object *Solid* describes a solid body for the subsequent mesh generation with tetrahedron elements *VT10*. Properties for the FE mesh can be assigned to the solid. The solid can be created in the following ways:

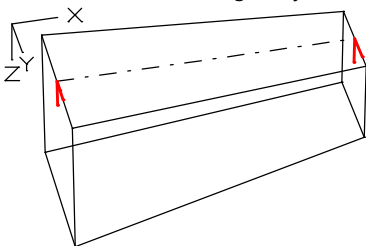
- Extrusion
- Definition of a cuboid by 8 discrete points

### Extrusion

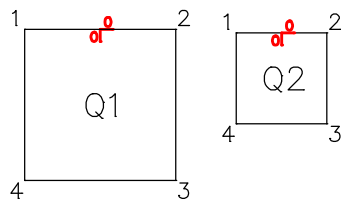
In the case of extrusion, the volume is described by the cross-section course along an axis. The cross-section course can be constant or linearly variable. The linearly variable cross-section course is defined via a start and end cross-section. Both cross-section polygons must have the same number of polygon points. If no end cross-section is defined, the cross-section course is constant. The extrusion axis runs between the origins of the polygons. The sections are perpendicular to the axis.

The axis required for extrusion can be defined in the following ways:

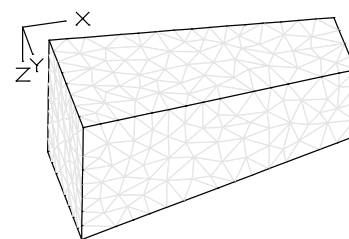
- Entry of the start, end and any number of intermediate points on the extrusion axis and the degree of the curve (see *NURBS*)
- Selection of a previously drawn arc
- Selection of a previously drawn *NURBS* curve
- Selection of a design object



Solid with extrusion axis



Section at the start and the end



Tetrahedron elements

A curved course of the extrusion axis is approximated by a selected number of segments. A linear course of the axis is assumed within a segment. Under certain circumstances, a point in the local *xz* plane is required to define the cross-section rotation while entering the axis (only in 3D). When creating a volume using a design object, the cross-section course and the number of segments are defined by the design object.

### Solid properties

The dialog contains the following properties:

- Section of elements
- Layer of elements
- Color of elements
- Partial view of elements

- Label
- Criteria for regulation of the element quality during the mesh generation
  - *Total volume / Element volume*: This ratio (V) limits the size of the elements.
  - *Circumsphere radius / min. Edge length*: This ratio (B) describes the quality of the element shapes. Small values increase the quality of the elements but also increase the number of elements.
- Coordinate system of the stresses. This can be modified with the function *Element System*.

### Surface properties

The following properties can be assigned to each surface of a solid figure using the context menu:

- Color
- Label
- Bedding with the bedding moduli defined for the section
- Description of contact problems (see *Contact*)

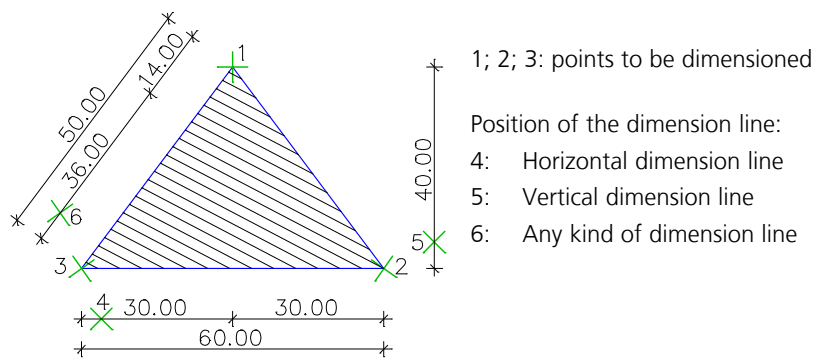
To select surfaces the 3D view and the *hidden* or *section representation* have to be activated. The surface properties are numbered.

### Modify, divide and modify endpoint

Solid bodies can be edited with the context functions *Modify*, *Divide* and *Modify Endpoint*.

## Dimension

This function allows you to carry out semi-automatic 2D and 3D dimensioning of objects.











### Input

<i>Point</i>	Select individual points that are to be dimensioned.
<i>Section</i>	Define lines whose intersection points with objects are to be dimensioned.
<i>Objects</i>	Select objects whose basepoints (e.g., beginning, end) are to be dimensioned.
<i>Horizontal</i>	Horizontal orientation of the dimension line in 2D mode.
<i>Vertical</i>	Vertical orientation of the dimension line in 2D mode.
<i>Arbitrary</i>	Orientation of the dimension line through definition of a free direction.
<i>Re-input</i>	Reselect the points to be dimensioned.
<i>Position</i>	Modify the position of the dimension line.
<i>Font</i>	The font for the dimension line.
<i>Text height</i>	Specify the height of the measurement figures [m].
<i>Unit</i>	Set the output accuracy to [cm] or [mm].
<i>Type</i>	Define whether partial dimensioning, full dimensioning or both (default) is to be generated.

Selected points can be dimensioned consecutively in multiple directions. With respect to walls, the axes or the edges are dimensioned using the *Objects* and *Section* dimensioning options depending on whether the wall axes are visible or hidden. The saved dimensioning consists of lines and texts and is not associative.

## Snap

The *Snap* menu contains auxiliary functions for coordinate input. The snap modes are used to specify the exact coordinates of object points with the crosshair. The snap mode can be enabled while the dialog is running and will remain active until it is disabled or another mode is selected. The selected mode overrides the automatic snap mode.

	<i>Node</i>	Enable the snap function for beam and element nodes.
	<i>Endpoint</i>	Enable the snap function for the endpoints of lines, arcs, solids, walls and columns.
	<i>Mid-point</i>	Enable the snap function for the mid-point of circles, arcs and columns as well as the middle of lines.
	<i>Intersection</i>	Enable the snap function for the intersection points of lines, circles, arcs and beams.
	<i>Perpendicular</i>	Enable the snap function for perpendicular points. With this function you can calculate the normal from the last point on a line, wall, circle, arc, beam or area element.
	<i>Objectpoint</i>	Enable the snap function for the nearest point of lines, walls, circles or arcs.
	<i>Relative Coordinates</i>	Enable or disable permanent relative coordinate input.
	<i>Basepoint</i>	Define the local reference system.
	<i>Orthogonal</i>	Enable or disable orthogonal mode.
	<i>Snap mode</i>	Enable or disable the snap grid.
	<i>Settings...</i>	Adjust the settings for the snap grid, grid points, orthogonal mode and snap radius.

## Relative Coordinates

Use this function to switch to the permanent relative coordinate input. You now no longer need to enter the & character in front of the coordinate values. If this mode is active, the reference system will be displayed at the current basepoint.

## Basepoint

This function is used to specify a local reference system for entering relative coordinates with the keyboard.

### Input

<i>New base point</i>	Specify a new basepoint.
<i>Old</i>	Retain old basepoint.
<i>Standard</i>	Reset the reference system to standard setting.
<i>Display</i>	Display the current reference system.
<i>New direction of local x-axis</i>	Specify the point for defining the local x axis.
<i>Old</i>	Retain the old local x axis.
<i>Global</i>	Select the direction of the global x axis.
<i>Angle</i>	Specify the angle [°] against the global x axis.

## Snap Settings

This dialog allows you to define the snap functions.

### Snap mode

The snap grid is used for coordinate input with the crosshair. If the snap grid is enabled, the crosshair will jump to the next snap point. The snap points are defined by their distance [m].

### Grid points

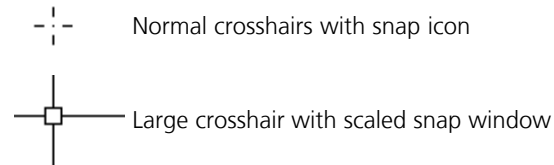
Enable or disable the auxiliary grid. The distance of the grid points [m] must be specified.

### Orthogonal mode

Enable or disable orthogonal mode.

### Snap radius for objects

A snap icon appears in the center of the crosshair when it is used to select objects. The objects will be selected if they are located inside the snap area or touched by it. The size of the snap area is defined by the snap radius in pixels. In the Options menu, the preferred crosshair with the associated display can be selected.



### Object selection with crossing window (from right to left)

If dragging a window, all objects within this window are selected. If this option is activated and a window is dragged from right to left, all objects within the window plus any objects crossing the boundaries of the window are selected.

## Format

The *Format* menu contains the following functions for formatting tables as well as text and analysis logs.

#### *Font ...*

Select the font for the current selection.

#### *Bullet Style*

Insert bullet points in the selected lines.

#### *Word Wrap*

Word wrap is an option that makes the entire text on the screen visible yet does not affect the appearance of the text when it is printed.

#### *Paragraph...*

Open a dialog in which you can control the formatting of selected paragraphs in the following ways:

- Text indent from left margin [cm].
- Text indent from right margin [cm].
- Text indent of first line from left margin [cm].  
You may enter a negative value to create a hanging indent.
- The selected paragraph can be aligned to the left, center or right. This function is only available if *Word wrap* is switched on.

#### *Tabs...*

The function opens the dialog for specifying tab stops in a selected paragraph.

- To set a tab stop position, enter the position of the new tab stop (e.g., 0.5) in the *Tab Stop* field and accept by clicking the *Set* button [cm].
- To remove a tab stop position from the selected paragraph, select the relevant entry from the list and click the *Delete* button.
- To remove all tab stop positions from the selected paragraph, click the *Clear All* button.

#### *Superscript*

Select superscript characters for the current selection.

#### *Subscript*

Select subscript characters for the current selection.

#### *Small Font*

Select a small print font for printing tables.

## Options

The *Options* menu contains the following functions:

<i>List</i>	Display information about the objects.
<i>Overlapping test</i>	Test drawing objects for overlap.
<i>Coordinates</i>	Determine coordinates.
<i>Distance</i>	Determine distance.
<i>Area</i>	Determine area and perimeter.
<i>Angle</i>	Determine angle.
<i>Variable</i>	Define a numeric variable.
<i>Page Frame</i>	Edit page frame.
<i>System File</i>	Import or export the system file <i>lgraph.dat</i> with user specific settings.
<i>Archive...</i>	Archive the selected project files.
<i>Auto Save...</i>	Set the time interval for automatic save.
<i>Drag Selected Objects</i>	Enable transfer operation of selected objects while holding down the left mouse button.
<i>Window Background Black</i>	The background of the graphical view is displayed black.
<i>Large crosshair</i>	During the coordinate input, a crosshair is displayed in the size of the graphic view.
<i>Design Codes...</i>	Open the dialog for setting the design codes included in the user interface.
<i>Import Material Conversion...</i>	Starts the dialog for defining a conversion table for the material.
<i>Language</i>	Change the interface language.

## List

This function shows the properties of the selected objects.

## Overlapping test

This function in the context menu can be used to test drawing objects for overlap. The test checks all visible objects of a drawing (except *NURBS curves* and *images*) for overlapping. The overlapping objects are summarized in the newly created partial view *Overlapping objects*.

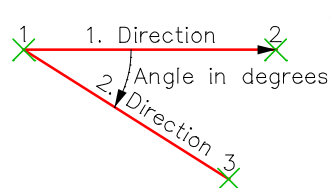
## Distance

The Distance function calculates the distance between two points or the length of a traverse. The result is shown in the status bar [m].

## Area

This function determines the area and perimeter of a 2D traverse. The results are shown in the status bar.

## Angle



The angle between two directions can be determined by entering three basepoints. The result is shown in the status bar.

## Page frame

Page frames are used for printing. The following special functions can be accessed from the *File* menu in the page frame window:

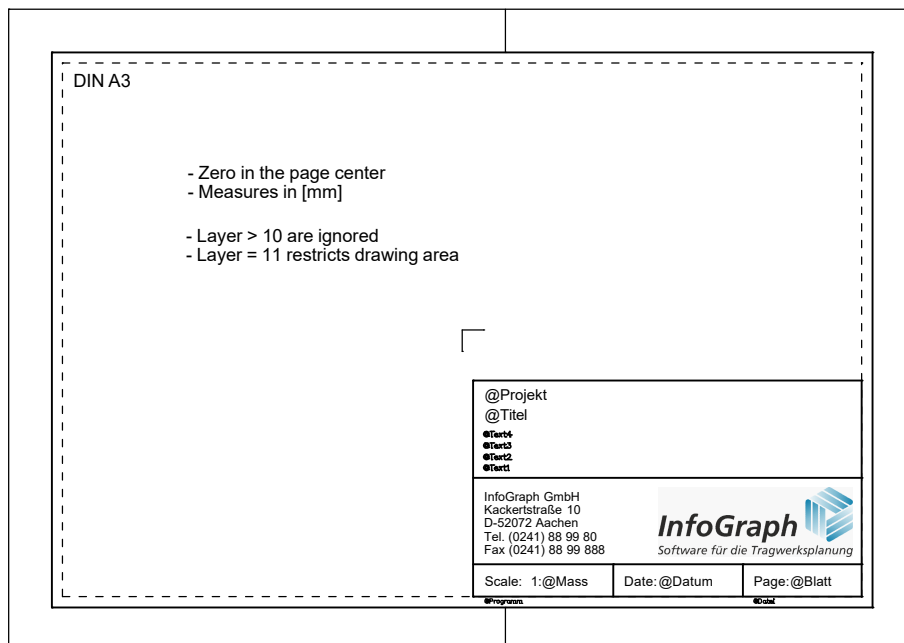
<i>New</i>	Create a new page frame. The current page frame will close.
<i>Open</i>	Open an existing page frame.
<i>Save</i>	Save the current page frame.
<i>Save As</i>	Save the current page frame with a new name. The name can be saved in the project file for a specific project or in the <i>lgraph.dat</i> system file for all projects.
<i>Exit</i>	Quit the page frame window. You are prompted whether you want to save the changes in the current page frame.

All drawing objects can be used to design the page frame. In addition, the following text variables can be used:

@Benutzer	User name, structural engineer. *)
@Blatt	Page number. *)
@Datei	Name of the project file.
@Datum	Current date. *)
@Kapitel	Complete folder path for entries of the print list.
@Mass	Plot scale. *)
@Objekt	Label text of graphical views of the print list.
@Ordner	Current folder name for entries of the print list.
@Ordner1, 2, 3	Folder name of the corresponding level for entries of the print list.
@Pfad	Complete path to the project file.
@Programm	Version name of InfoCAD.
@Projekt	Project name. *)
@Texta, b, c	Free variables with additional text. *)
@Text1...9	Explanations of the respective views. If these text variables are not available, the explanations will be displayed in the lower left corner of the drawing area.
@Titel	Description of the figure. *)
@T1...6	Additional title lines. *)
@Zeit	Time. *)

\*) When printing, these variables will be replaced by the text specified in the *Page Setup* dialog in the *File* menu.

If a variable is within a text line, there must be another variable, a blank space or the end of the text after this variable. Other special features are shown in the illustration below:



When printing, the zero point of the drawing frame is placed in the middle of the page. The longer side of the drawing area is aligned based on the longer paper edge regardless of whether you are printing in portrait or landscape mode.

The drawing area is bounded by the rectangle specified on layer 11. If no clear boundary is found on layer 11, the lines on layers 1-10 are used to determine the borders. Lines on layers 1-9 in the lower or upper right corner of the drawing area

result in additional cutouts in these corners.

## Variable

Variables are used to save scalars so that they can be used again during later keyboard input. The beginning of the variable names may not correspond to a function expression. Variables can be used together with constants in arithmetic expressions.

## Archive

This function is used to archive the input data of project files. Analysis results are generally not archived because they can be restored from the input data at any time.

## Auto Save

You can specify a time interval for the auto save feature. Any input data for the current project file that has not yet been saved will be saved as *Backup of ....* If a project file has not been specified, the backup file will be saved in the *My Documents\InfoGraph* folder. The backup file will be automatically deleted after the project file is properly closed.

## Import Material Conversion

In this dialog, material assignments are defined. These are used in the following cases:

- Import an analysis model via the Autodesk® Revit® Add-In "InfoCAD BIM RV".
- Import an analysis model via an IFC file.
- Transformation of IFC building elements into model objects.

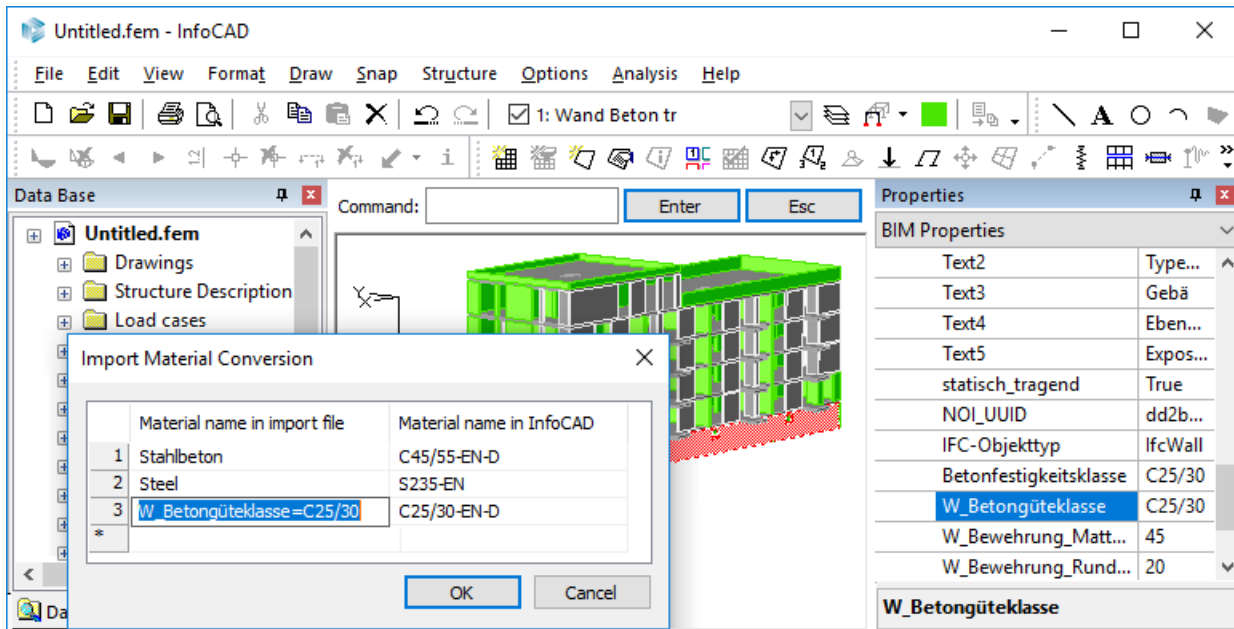


Table with entry for converting IFC building elements.



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# Help

The Help menu contains the following functions.

## Help Topics

This function presents the help topics of the *InfoCAD* documentation. Explanations for current program functions can be accessed pressing the F1 key.

## User manual

In addition to the help system, the user manual (PDF) can be used. In order to view, the Adobe Acrobat Reader® or a compatible application is required.

## InfoGraph Homepage

Open the homepage on the InfoGraph web server.

## News

Open the webpage with the news on the InfoGraph web server.

## Release notes

Open the webpage with the release notes on the InfoGraph web server.

## Downloads

Open the service webpage on the InfoGraph web server.

## Check for updates

This function checks whether a newer version of *InfoCAD* is available on the InfoGraph web server.

## About InfoCAD

Software information, version number and copyright.

## License Information

This dialog contains the customer number and the available license configurations at the workstation. The license file *lgraph.def* in the *InfoCAD program* folder is decisive when using a single user license. The *Update* button can be used to transfer the current license data from the InfoGraph web server.

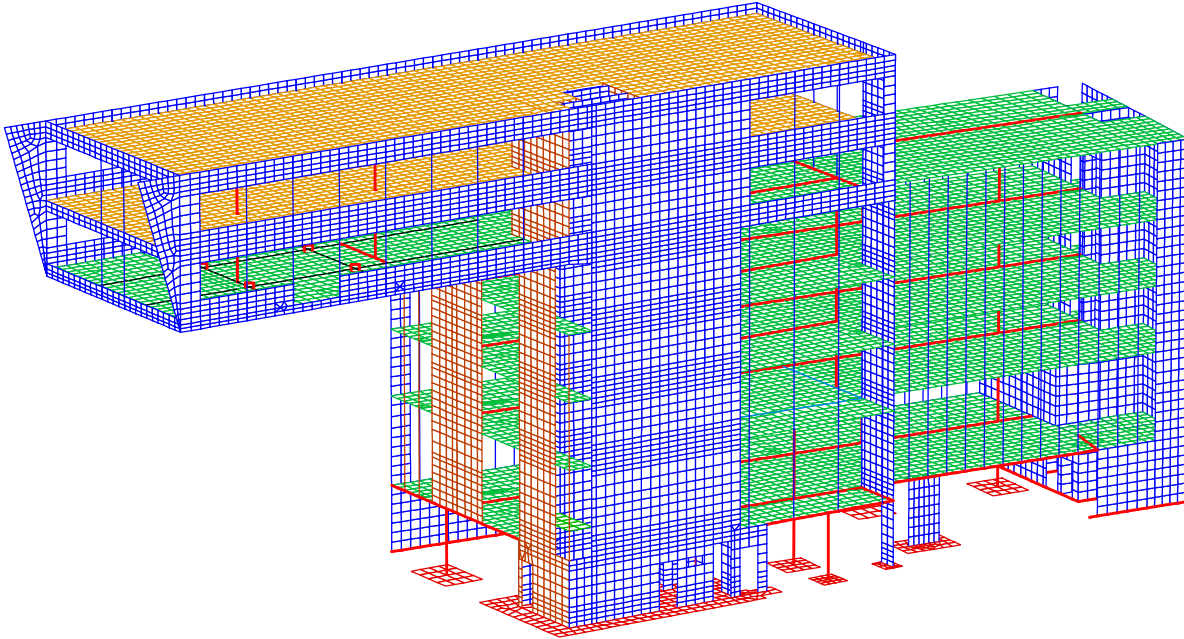
Network licenses use the license file from the *license manager* program folder. This license data can be updated using the *InfoMonitor* program.

A table with the license information can be copied to the Windows clipboard via the context menu.

# Finite Elements

## Basics

The Finite Element Method (FEM) is used in structural design for numerically solving structural problems involving slab, shell, frame and continuum analysis. It has become an accepted tool whose handling has been greatly simplified as a result of the powerful graphical user interface. Especially with complex structures, however, correct modeling and the appropriate interpretation of the results still require an experienced engineer with a good knowledge of statics.



*Static system of a building*

## Analysis Method

The basic principle for solving problems of structural mechanics using the Method of Finite Elements is the division of an entire structure into discrete elements that are connected to one another in the element nodes.

The unknowns of each node are the global deformations  $u_x$ ,  $u_y$ ,  $u_z$ ,  $\varphi_x$ ,  $\varphi_y$  and  $\varphi_z$ , which can be determined by applying the six equilibrium conditions to the node. This is done by constructing element stiffness matrices in the respective local system, transformation to the global coordinates and mapping into the global stiffness matrix of the system. After transformation of the loads to the global coordinates and insertion of the support conditions of the structure, a linear system of equations ( $K \cdot u = p$ ) is derived that can be used to determine the unknown node deformations with

$K$  global stiffness matrix,

$u$  deformation vector of the global node deformations,

$p$  global load vector.

## Element Library

The FEM module provides an element library. All elements that are used fulfill the Patch-Test, meaning that regardless of the type of arrangement, they converge to the exact solution of the boundary value problem as the element mesh is refined.

The beams are analyzed according to the general deformation method and deliver exact results according to first-order theory.

The plain stress elements with 3 or 4 nodes are hybrid element types with rotational degrees of freedom  $\varphi_z$ . The plain stress elements with 6 nodes are pure deformation elements with two unknown displacements per node and a quadratic displacement function. All plain stress elements are fully compatible and are used for calculation of 2D stress states.

The slab elements are either '*discrete Kirchhoff theory*' (DKT) elements or '*Mindlin-Reissner*' (MR) elements. For the first element type (DKT) the distortion-displacement relationship is formulated for thick slabs in order to then introduce the Kirchhoff conditions for thin slabs at discrete points of the element boundaries. This allows the unknowns of the deformation formulation to be expressed by the corner node deformation for these elements as well. The advantage over a purely displacement formulation for slab elements becomes evident in the significantly improved convergence behavior.

Elements of the second element type (MR) are shear-flexible slab elements with independent approaches for the rotations/curvatures and the shear distortion. As a result, the moments as well as the lateral forces are taken into account in the equilibrium conditions of the system of equations. With adequate small elements the lateral forces are in equilibrium with the external forces. Whereas the lateral forces of Kirchhoff elements result from the derivation of the moments, they result directly from the computed shear distortion according to the Reissner plate theory. To get results comparable to the Kirchhoff plate theory for jointed support of the edges, Navier boundary conditions are to be specified for the supported nodes (see example *Slab with Mindlin-Reissner elements*).

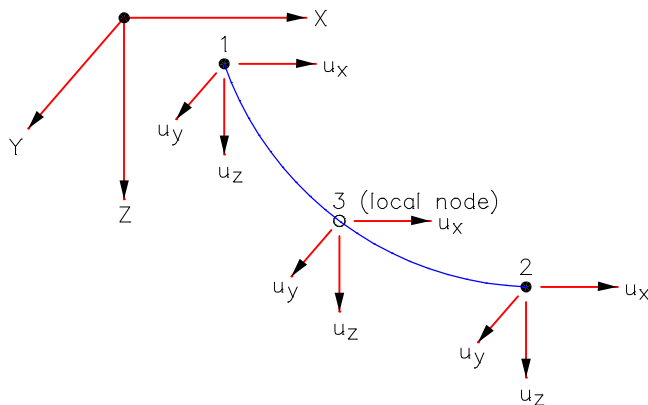
The shell elements are a combination of the element stiffness matrices of the plain stress elements and slab elements, meaning a curved shell area is approximated using facets. This does not mean, however, that there are any relevant limitations with respect to the results that can be achieved.

The solid element VQ83 is an element with 8 nodes, up to 4 of which can be identical. This allows the modeling of any 3D geometry. Use of identical nodes results in reduced accuracy.

The solid element VT10 is a tetrahedron element with 4 corner nodes and 6 side middle nodes. Because of the complete quadratic displacement formulation herewith linear stress curves can be calculated for all stress components.

The cable elements are calculated according to the theory of large displacements and are based on a quadratic isoparametric approach for geometry and displacement. The geometry is described by two external nodes specified by the user and an inner node generated by the program. Each node has the global displacement degrees of freedom  $u_x$ ,  $u_y$ , and  $u_z$ .

These make it possible to map a linearly variable curve of the cable force in the element. A linear-elastic material behavior is assumed. The used strain formula (Green strain) consider great deformations but only small strains.

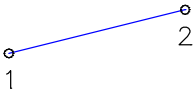
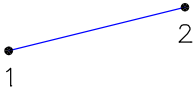
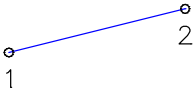

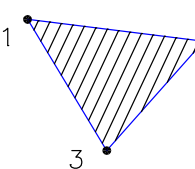
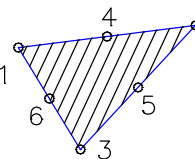
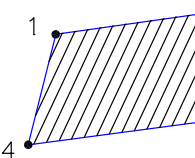
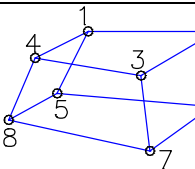
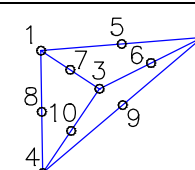


### Cable geometry and global node displacements

The following element approach allows for handling larger deformations. This leads to a nonlinear system of equations that can be solved by means of a Newton-Raphson iteration. Usually, good convergence behavior is exhibited, but this can vary from case to case. Here, in addition to the original cable geometry the level of prestressing is of critical importance. The final cable geometry is load dependent and can only be determined by performing the equilibrium iteration. A superposition of individual load cases is not permissible due to the nonlinear properties as described above.

For ropes, the equilibrium is always calculated on the deformed system. This also applies to the iterative error forces. For combined load-bearing models e.g. consisting of rope, beam or area elements, the forces refer to the undeformed system (first-order theory) regarding the other elements. In order to ensure the overall equilibrium in conjunction with large displacements, the structure should then be calculated according to the second-order theory.

The spring elements are also analyzed according to the general deformation method. The force-deformation-curves are defined separately for each degree of freedom by the user. The curves can have nonlinear characteristics (e.g. contact, slippage, elastic-plastic).

Geometry	Name	Element type	Degrees of freedom
	RF	3D truss element	$u_x, u_y, u_z$
	RS	3D beam element	$u_x, u_y, u_z, \varphi_x, \varphi_y, \varphi_z$
	SEIL	3D cable element	$u_x, u_y, u_z$
	Spring	3D spring element	$u_x, u_y, u_z, \varphi_x, \varphi_y, \varphi_z$
	SD33 PD33 SH36	Plain stress element Slab element Shell element	$u_x, u_y, \varphi_z$ $u_z, \varphi_x, \varphi_y$ $u_x, u_y, u_z, \varphi_x, \varphi_y, \varphi_z$
	SD62	Plain stress element	$u_x, u_y$
	SV43 PV43 SH46	Plain stress element Slab element Shell element	$u_x, u_y, \varphi_z$ $u_z, \varphi_x, \varphi_y$ $u_x, u_y, u_z, \varphi_x, \varphi_y, \varphi_z$
	VQ83	Solid element	$u_x, u_y, u_z$
	VT10	Solid element (tetrahedron)	$u_x, u_y, u_z$

In order to avoid numerical problems during mesh generation, graphical processing and static calculations, *InfoCAD* requires a minimum node distance of 1 mm.

## Coordinate Systems

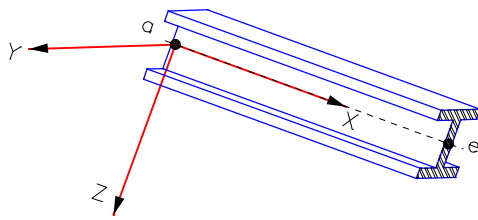
An FEM model consists of the following structure objects with their own (local) coordinate systems:

- Beams
- Cable elements
- Area elements
- Solid elements
- Supports
- Spring elements

All coordinate systems are right-handed systems.

**Beams**

The following illustration depicts the local coordinate system of a beam.



The local beam coordinates have their origin in the beam's start point (a). The local x axis runs from (a) to (e), the beam's end point. The local z direction is determined by a direction vector. When in standard orientation, the local y axis runs parallel to the global xy plane. If this does not establish a clear direction, as for example for columns, then the local y axis runs parallel to the global y axis. The start and end point of a beam element can differ from the positions of the nodes, because of the eccentricities.

**Cable elements**

The local coordinate system of a cable element has its origin in the cable element's start node. The local x axis points to the cable element's end node.

**Area elements**

*Element System*

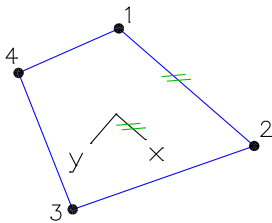
By default, the local x axis points from the first to the second element node. The local y axis is aligned perpendicular to the local x axis and runs in the direction of the third element node. The element properties such as orthotropy and bedding correspond to this coordinate system. It can be rotated about the local z axis. The z axis points to the underside of the element and must coincide with the global z axis for slabs and plain stress elements.

*Internal forces system*

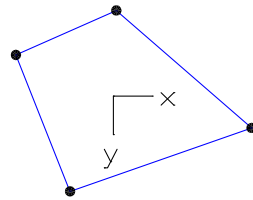
It describes the orientation of the internal forces. By default its x axis runs horizontal and if possible parallel to the global x axis. This coordinate system can also be rotated about the local z axis.

*Reinforcement system*

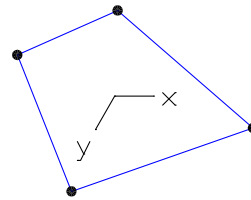
This coordinate system describes the orientation of the area reinforcement and is by default oriented like the internal force system. The local y axis may be defined to deviate from a 90° angle.



*Element system (default)*



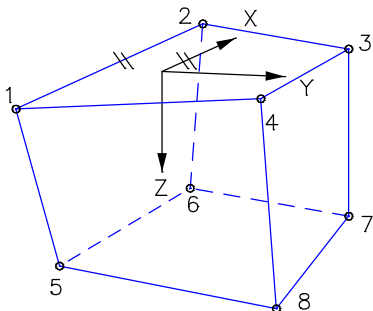
*Internal force system (default)*



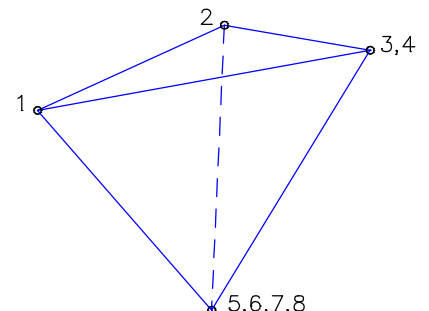
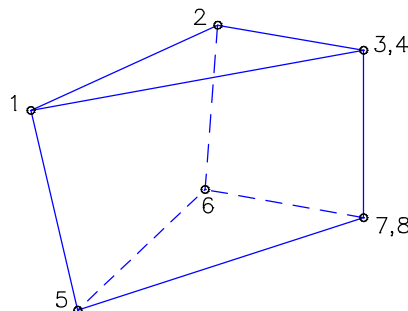
*Reinforcement system (y direction changed)*

**Solid elements**

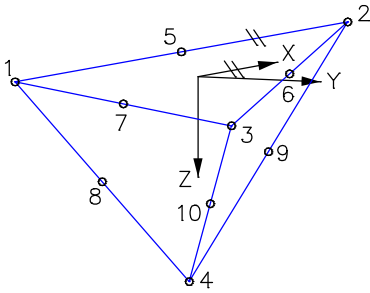
The directions of the element stresses are determined by an internal force system that can be freely oriented. For the orientation of the element system VQ83 the same conditions are valid as for the area elements, however, the z axis must point into the element.



*Solid element VQ83 default orientation Variants of solid elements VQ83 of the element system*



The element system of the tetrahedron element *VT10* is unchangeable.



Solid element *VT10*

**Supports**

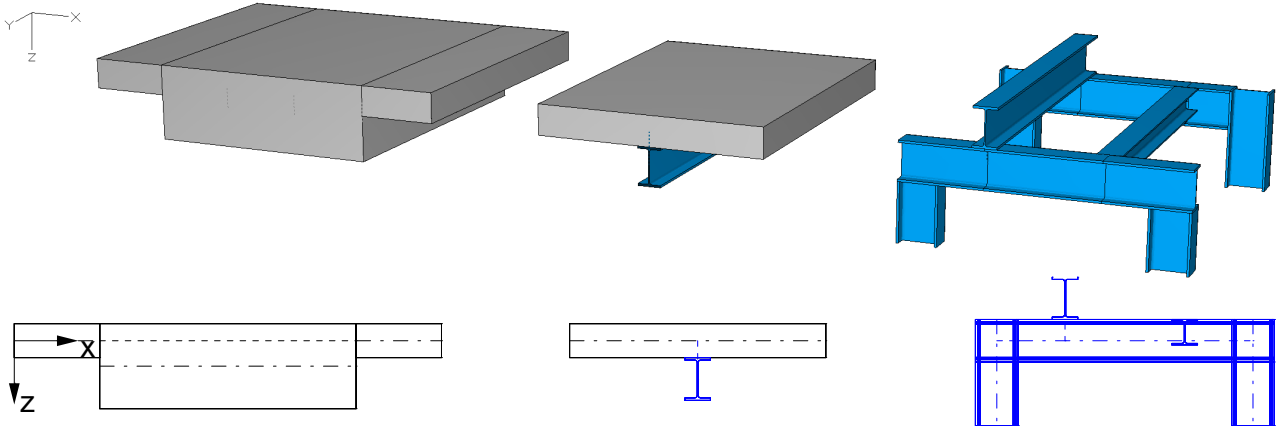
The cartesian support system is defined by a vector for the local x direction and one for the xy plane. The default orientation corresponds to the global coordinate system. The user can define a differing orientation.

**Spring elements**

The local x axis runs from the start node to the end node. If both nodes are at the same location, the x direction have to be defined. The local z direction is defined by the user.

**Eccentricities**

If the element axis of beams or the center plane of area elements should be offset, eccentricities can be adequate. They can be entered via the dialog page *Eccentricity* in the *Element properties* dialog or via model objects. The cross-sections, coordinate systems, beam joints and element results refer to the offset axis. The nodes are still in their original location.

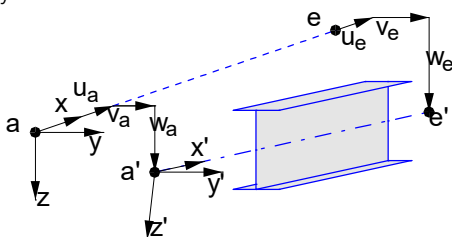


**Eccentricity of area elements**

Area elements can be moved by the dimension *w* in the direction of the local z axis. This can be useful if, for example, a change of thickness occurs within a surface and the surface should remain flat. Because such an offset generally leads to normal forces, the eccentricity of surfaces can only be used for shell elements.

**Eccentricity of beam elements**

Beam elements can have different eccentricities (*u*, *v*, *w*) at their start and end in their local coordinate system. The x axis points from the starting node (*a*) to the end node (*e*) of the beam element. The beam axis points along the eccentric axis from the beam starting point (*a'*) to the beam end point (*e'*), on which possible joints also are located. The local z direction is determined by a direction vector. When in the standard orientation (angle = 0), the local y axis runs parallel to the global xy plane. If this does not establish a clear direction, as for example for columns, then the local y axis runs parallel to the global y axis.



## Nonlinearities in the Calculation

Nonlinear properties in the structure or a load case mean that there can be a nonlinear relationship between load and result. Or in other words, the nonlinear influences do not necessarily produce twice the result when the load is doubled. Therefore, such calculations should only be performed for "realistic" load compositions.

The solution of all nonlinear problems requires an iterative calculation. The measure for controlling the iteration is the length of the error vector, the residuum. It is displayed in the output bar for each iteration step. The iteration is terminated when the remaining residuum in relation to the initial residuum has reached the specified error threshold. The initial residuum is the length of the load vector. If the load case definition also includes terms such as temperature, support displacement or shrinkage, the initial residuum can become very large. To avoid a premature end of iteration, the error threshold preset to 1% should be reduced in this case. It can be set to a smaller value with the load type *Theory* or *Load group*. If the error threshold cannot be reached, the calculation program issues a message.

In addition to the residual, the sum of soil pressures and support reactions must never deviate from the load sum by more than 1%. This can occur, for example, with nonlinear bedding (*failure of tension bedding*), nonlinear supports (*failure of tension supports*) or also with solid structures with contact. Therefore, a maximum of 10 additional steps may be iterated even after the required residual has been reached. If the required equilibrium is still not achieved, the calculation is aborted with a corresponding message.

The following properties or settings lead to a nonlinear calculation:

1. Statics **without** nonlinear structure analysis
  - Failure of tension supports by corresponding support properties or by the corresponding selection in the load type *Theory* or *Load group*
  - Foundation without tensile stresses by the selection "*No soil pressure > 0*" in the load type *Theory* or *Load group*
  - Cross-section type tension member
  - Cross-section type beam with the longitudinal bearing "*Only pressure*" or "*Only tension*"
  - Spring elements can have nonlinear spring characteristics
  - Line hinges can have contact properties and nonlinear spring characteristics
  - Calculation according to second- or third-order theory in accordance with the specification in the load type *Theory* or *Load group*
  - Structures with cable elements
  - Volume structures with contact
2. Statics **with** selected nonlinear structure analysis (in addition to the properties listed under 1.)
  - Nonlinear material behavior of standard concrete, steel or free material with bilinear stress-strain curve or yield condition for calculations in ULS or SLS
  - Bedding with bilinear curve (the load option "*No soil pressure > 0*" is not used)
  - Tendons in concrete bond with bilinear stress-strain curve
  - Nonlinear thermal material behavior of standard concrete, steel and timber in calculations in case of fire
3. Dynamics
  - Eigenvalues, stationary response, response spectrum or modal time-step integration with the *stiffness matrix according to second-order theory* of a selected load case
  - Direct time step integration with the properties listed under 1.
  - Pushover analysis according to EN 1998-1:2013 with the properties listed under 2.

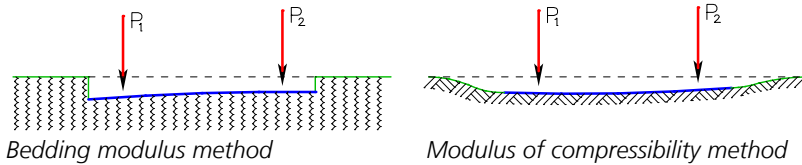
As described above, the superposition principle no longer applies to nonlinear calculations, i.e. they must not be additively superimposed with other results. Further usage in load combinations or design actions is only permitted if these do not contain any other than mutually exclusive actions.

Further information can be found in the section "*Load > General > Load group*".

## Foundation Models

The subsoil can be described by the following models:

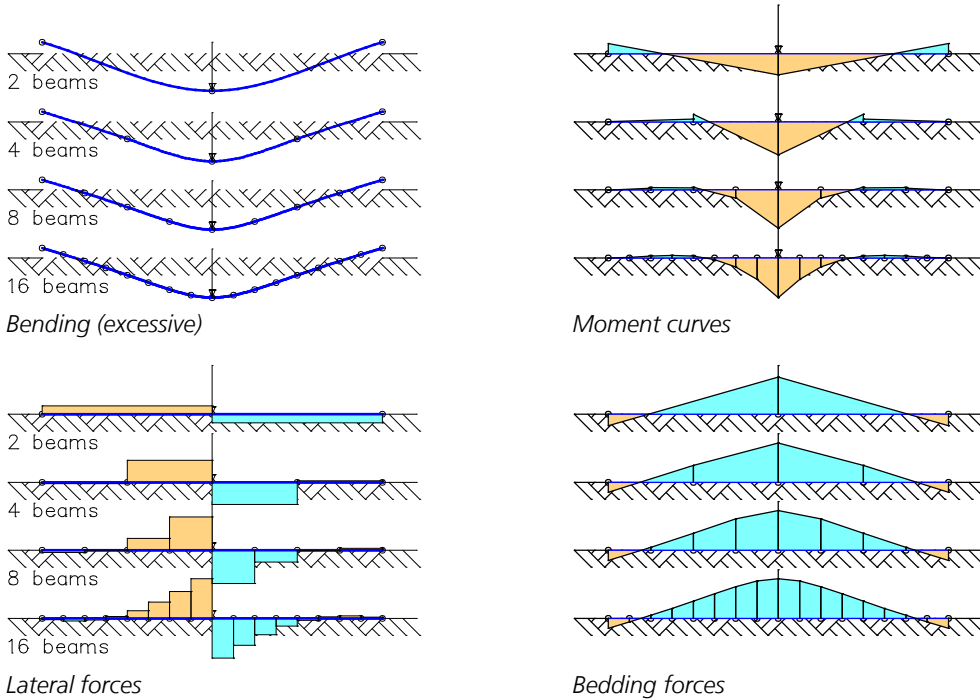
- The *Bedding modulus method* is based on the assumption that the subsidence ( $s$ ) is proportional to the soil stress  $\sigma_0$ , meaning  $\sigma_0/s = k_b = \text{constant}$  ( $k_b = \text{bedding modulus}$ ). This means that a vertical load only causes subsidences under its direct influence area.
- The *Modulus of compressibility method* is based on the assumption that the subsidence hollow and the deformation of the foundation body coincide. The subsoil is represented using the theory of elastic, layered half space. The analysis according to the modulus of compressibility method is performed by an iterative improvement of the bedding moduli according to Ahrens/Winselmann. Additional convergence criteria are the global equilibrium and minimization of the residual. Systems in which the subsidence hollow and the deformation of the foundation body have different signs due to the load can not be calculated exactly with the iteration method used.



### Beam bedding

The modeling of elastically bedded beams is carried out with the help of a cubic element approach under consideration of the bedding terms within the stiffness matrix. Beam loads are transformed into equivalent node forces. Due to this approach, bending moments can be represented by linear curves and lateral forces by constant curves. An improvement of the results can thus only be achieved by setting up a finer beam subdivision. The following example demonstrates the influence of the beam subdivision.

#### Example: Bedded beam with point load



### Foundation without tensile stress

Both of the above methods can be calculated with the exclusion of tensile stresses in the foundation area. If an area or solid element rises above the ground, all ground stresses will be lost. For beams, loss of tensile stresses can occur in the  $y$  or  $z$  directions of the beam system. If the loss occurs in both directions, then jacket friction ( $x$  direction) will be ignored.

### Nonlinear bedding

As part of a nonlinear system analysis, a bilinear bedding characteristic is used. The tensile and compressive strength of the bedding are specified separately for the local directions  $x$ ,  $y$  and  $z$ . If the tensile strength of the bedding of area and solid elements in local  $z$  direction and of beams in local  $y$  and  $z$  direction is set to zero, the program behaves as described for *Foundations without tensile stress*.



## Second- and Third-Order Theory

The second- or third-order theory program module is an extension of the finite element program and makes it possible to solve the following elastic problems:

- Beam buckling.  
In this case the beam forces can relate to either the deformed or undeformed beam chord.
- Slab and shell buckling.
- Calculation of combined structures according to the second-order theory (equilibrium of the deformed system with the assumption of small deformations ( $\sin \varphi = \varphi$  and  $\cos \varphi = 1$ )).
- Calculation of combined structures according to the third-order theory (equilibrium of the deformed system with the assumption of large deformations).

The equilibrium iteration is carried out according to the Lagrangian method based on the total tangential stiffness matrix according to the second- or third-order theory. The iteration method normally converges after a few steps. Stability failures are displayed by the singularity of the global stiffness matrix.

The following requirements must be met:

- Beams are assumed to be straight.
- Area elements are assumed to be plane.
- The area and beam cross section is constant for each element.
- Section dimensions are small compared with the system dimensions.
- The section form of an element remains unchanged during structure deformation, meaning that section warping is also not taken into account.
- Deformations are small compared with the other dimensions (second-order theory).
- The mathematical curvature is linearized (second-order theory).
- The load is slowly increased to its final value and no deviation of direction occurs as a result of the system deformation.

In order to take into account the influences according to second- or third-order theory, the tangential stiffness matrix  $K_T$  is required. This is obtained by formulating the linearized virtual work and variation according to the nodal degrees of freedom.

In general form:

$$K_T = \underbrace{K_G}_{\text{geometric matrix}} + \underbrace{K_0}_{\text{matrix for small deformations}} + \underbrace{K_L}_{\text{matrix for large deformations}}$$

In other notation:

$$K_T = \int_V B_G^T S B_G dV + \int_V B_0^T D B_0 dV + \int_V (B_0^T D B_L + B_L^T D B_L + B_L^T D B_0) dV$$

with

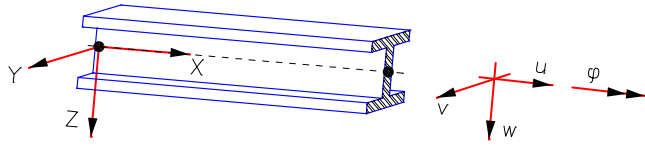
- $B_0$  Differential operator (first-order theory)
- $B_G$  Differential operator (second-order theory)
- $B_L$  Differential operator (third-order theory)
- $D$  elasticity matrix
- $S$  stress matrix

When performing an analysis according to the second-order theory, the matrix for large deformations (third-order theory) is not taken into account. In third-order theory, all terms are considered.

The matrices  $K_G$  and  $K_L$  are in contrast to the linear matrix  $K_0$  functions of the unknown node displacements as these are indirectly contained in the stress matrix  $S$  and the differential operator  $B_L$ . Therefore, as mentioned above, an iterative solution is necessary here as well.

### Formulation for beam elements

The stiffness matrix of the 3D beam element is generated through the formulation of the virtual work of the inner forces. For the deformations in the beam system ( $u, v, w, \varphi$ ) the following signs apply:



The following applies:

$$\delta A_i = \int_V \delta \varepsilon \sigma_x dV + \int_L \delta \varphi' M_x dx$$

*(Normal stresses) (St. Venant torsion)*

with

$$\varepsilon = u' - yv'' - zw'' - y\varphi w'' + z\varphi v'' + \frac{1}{2} [u'^2 + v'^2 + w'^2 + (y^2 + z^2)\varphi'^2]$$

$$\sigma_x = E(u' - yv'' - zw'') \text{ (linearized)}$$

$$M_x = G I_x \varphi'$$

After implementing the above relationships, neglecting higher order effects and carrying out the variation and integration of the section-dependent values, we arrive at:

$$\begin{aligned} \delta A_i = & \int_L [\delta u' EAu' + \delta v'' EI_z v'' + \delta w'' EI_y w'' \\ & \text{(Linear shares from normal force and bending)} \\ & + \delta \varphi' GI_x \varphi' \\ & \text{(Linear share from St. Venant torsion)} \\ & + \delta u' N_x u' + \delta v' N_x v' + \delta w' N_x w' + \delta \varphi' M_{rr} \varphi' \\ & + \delta v'' M_y \varphi + \delta w'' M_z \varphi \\ & + \delta \varphi M_y v'' + \delta \varphi M_z w''] dx \\ & \text{(Geometric nonlinear shares)} \end{aligned}$$

The section integral  $M_{rr}$  (Kindmann) is described as

$$M_{rr} = M_{rr}(N_x, M_y, M_z) = \int_A \sigma_x (y^2 + z^2) dA$$

and can be formally regarded as an internal force.

Because the consideration of  $M_{rr}$  in the element stiffness matrix requires additional section internal forces that can, in general, only be determined by complex integration, it is neglected at this point. The mode of action of the other internal forces follows the common conventions of structural analysis.

The first term of the geometrically nonlinear component is only taken into account in the calculation according to the third-order theory.

As long as nothing to the contrary has been defined, the fixed-end forces according to second-order theory relate to the undeformed beam chord. Internal forces according to third-order theory as well as according to second-order theory with nonlinear system analysis are always related to the deformed beam chord.

In order to be able to perform the integration, the four independent deformation functions are replaced with interpolation polynomials of the node deformations. For  $u(x)$  and  $\varphi(x)$  linear interpolation is performed, for  $v(x)$  and  $w(x)$  a cubic approach is chosen.

### Nonlinear iteration

The geometric nonlinear terms of the work equation are linearized by introducing the internal forces  $N_x, M_y, M_z, M_{tr}$ . To take into consideration the nonlinearities, an iteration of the internal forces is carried out in the program by repeated formation of the element stiffness matrices. In this case the convergence criterion is the length of the error vector, the residuum. The mentioned shape functions ensure exact results for a calculation according to the first-order theory.

When performing a system analysis according to the second- or third-order theory, the interpolation polynomials only yield an approximation of the deformation curve. For components subjected to high load, a finer discretization, meaning at least one intermediate node per compression beam, is recommended.

## Buckling Eigenmodes

Stability problems of the second-order theory are described by the following eigenvalue problem:

$$K_0 \Phi = \lambda K_G \Phi$$

with

$K_0$  linear stiffness matrix according to the first-order theory

$K_G$  nonlinear geometric stiffness matrix acc. to the second-order theory

$\lambda$  buckling load factors (eigenvalues)

$\Phi$  buckling eigenmodes

The basis for the calculation are the internal forces of the load case according to the first-order theory. These are used for the construction of the geometric stiffness matrix  $K_G$ . The determined buckling load factors  $\lambda$  correspond to the load case.

While  $K_0$  is always positively definite, this cannot be assumed for  $K_G$ . This means that in general positive as well as negative eigenvalues  $\lambda$  exist. In order to ensure an unambiguous assignment, only the positive eigenvalues with the corresponding eigenmodes are made available under the number of the load case. This allows for multiple stability load cases to be examined side by side. If not enough positive eigenvalues exist in the defined eigenvector space, also the smallest negative eigenvalues are saved.

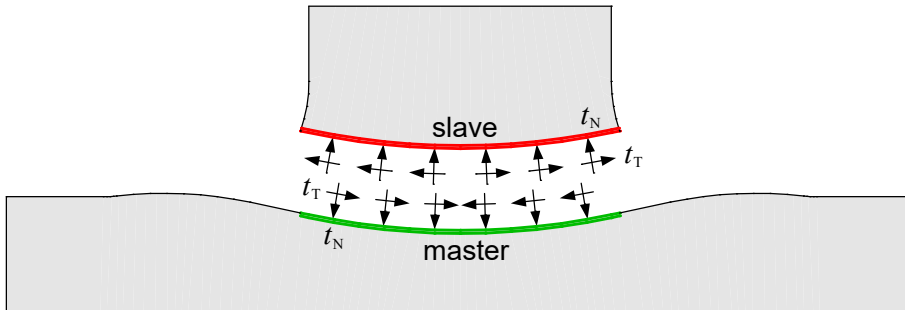
The analysis of buckling eigenmodes does not replace a check according to the second-order theory with equilibrium of the deformed system.

## Contact

There are numerous possible applications for the numeric simulation of contact problems. When bodies without a monolithic connection interact, normally actions occur which are transferred by contact.

Construction examples are head plate connections, supports, anchor plates, connection of prefabricated components, reservoir dams etc. Also the case that structural components, which initially do not touch each other but come in contact because of their deformation, has to be taken into account.

The following figure shows schematically two bodies, which come in contact.



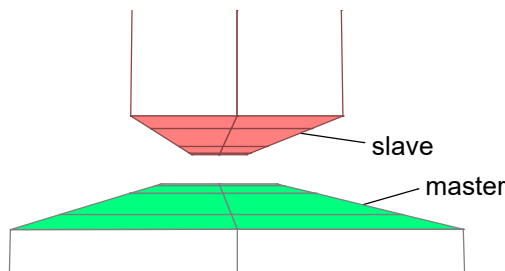
Master and slave surface with contact stresses

In this process normal stresses and possibly shear stresses (friction) occur. To define the interacting surfaces a distinction between slave and master surfaces is made. The contact analysis must ensure that points on the slave surface do not penetrate the master surface. An essential part of the analysis is the detection of the surface parts that come in contact considering the structural deformations. Except for a few special cases these are not known.

Because of the reasons given above, contact problems are highly nonlinear.

Contact problems can be modeled with the finite element method by point/point, point/face or face/face contacts. In the following the handling of contact problems between tetrahedron elements is described. Thereby face/face contact elements are used. For the derivation we reference to the given literature.

To make the modeling for the program user as simple as possible, the contact elements do not need to be entered directly. For the description of the contact problem only master and slave surfaces, which are arranged in pairs, have to be defined.



Model solids with master and slave surfaces

The figure above shows model solids with master and slave surfaces. Because the exact contact surfaces are not known they can be selected generously. With these information the program automatically determines the matching contact areas and internally creates corresponding contact elements. It is not necessary that the meshing of the master and slave surfaces are identical. But it has to be ensured that the nodes of the master and slave surfaces are not connected.

The contact stiffness is also determined internally by analyzing the involved element stiffnesses.

Beside the input data described above additional contact properties are provided:

### Factor for contact stiffness

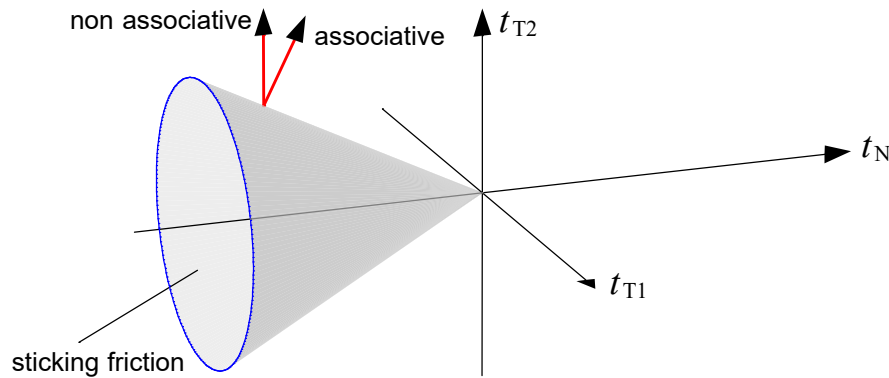
The user can modify the contact stiffness entering an adjustment factor to reach a more precise compliance with the contact conditions or a better convergence.

An overvalued contact stiffness leads to a bad condition of the system of equations and therewith to convergence problems.

**Coefficient of friction**

A coefficient of friction must be defined to take into account the Coulomb friction. The friction law is described by the following yield criterion.

$$f = \|t_T\| + \mu \cdot t_N = 0$$



*Yield surface for Coulomb friction*

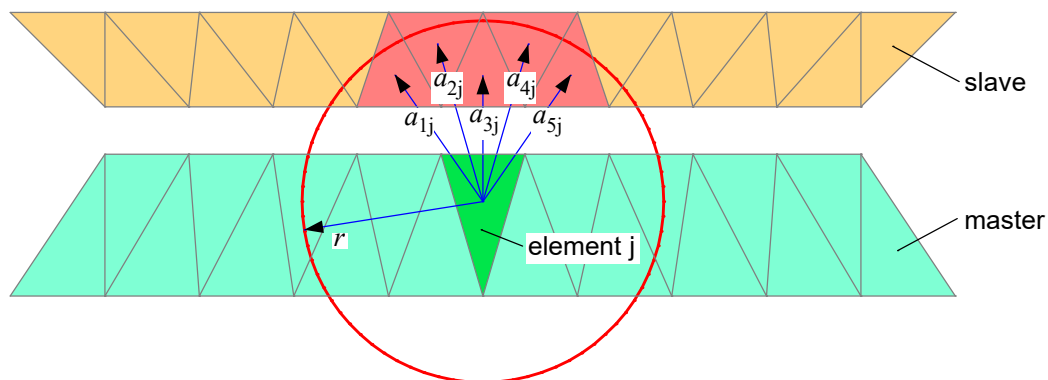
A transmission of shear forces is only possible for negative normal stresses. As long as the resulting shear stress remain within the cone, sticking friction occurs otherwise sliding friction. Sliding friction cannot occur in case of static loads. The consideration of sliding friction requires time step integration and should only be used for dynamic problems.

To fulfill the contact conditions a penalty formulation is implemented.

This method does not increase the number of unknown variables compared with the method of the Lagrange multipliers. The friction law described above requires the consideration of a non-associative yield law. As a consequence the stiffness matrix becomes asymmetric. Because currently the program always assumes a symmetric stiffness matrix the friction law for sliding friction can only be approximated.

**Influence radius**

The following figure shows schematically the influence radius with the corresponding enclosing sphere. With this value the user can limit the area where the program searches for tetrahedron elements which are in contact with the current element. Herewith the installation of internal contact elements is limited to the relevant area. This accelerates the convergence and reduces the time of calculation.



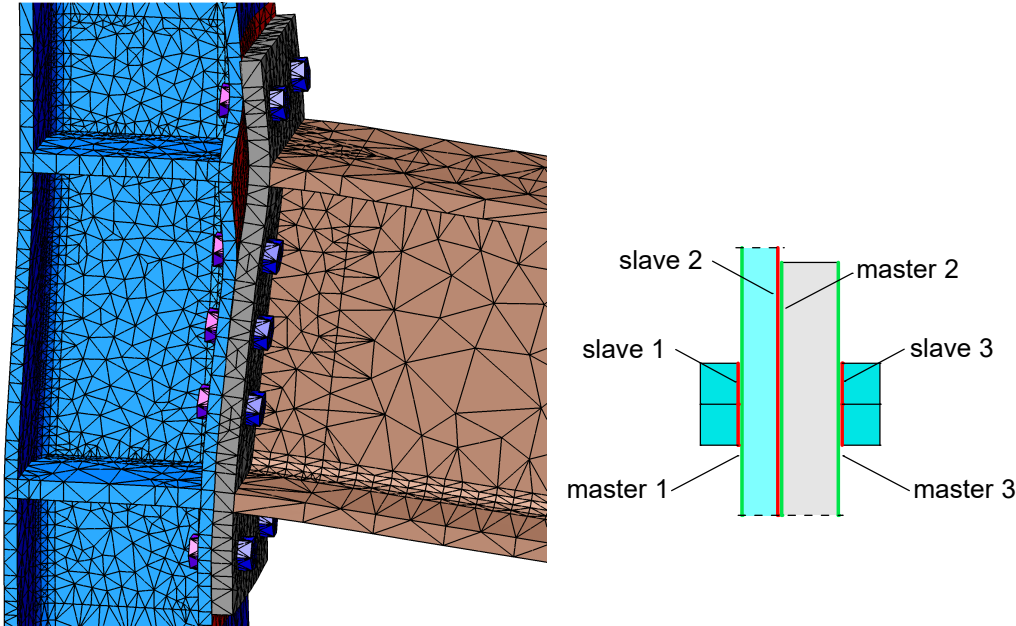
$r$  = influence radius of the enclosing sphere

$a_{ij}$  = spatial distance between element 'i' of the slave surface and element 'j' of the master surface with  $a_{ij} < r$ .

Elements of the slave surface, which distance from the master element 'j' is larger than the influence radius, do not get an internal contact element (i-j) and so cannot get into contact during the calculation.

**Special cases**

The following figure shows the deformation figure of a head plate connection.



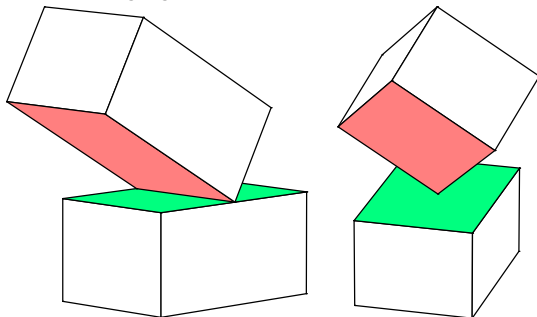
*Head plate connection*

Three different master/slave areas are used to describe this contact problem:

1. Column flange (inside) / screw nut
2. Column flange (outside) / head plate (inside)
3. Head plate (outside) / screw head

Here a separation of contact areas is absolutely necessary because otherwise they would overlap and unwanted interactions occur. The prestressing of the screws has been taken into account here. The whole connection is only beard by the contact.

The following figure shows two special cases which cannot be modeled by the face/face contact elements used.



*Special cases which cannot be modeled*

Even if the contact conditions are strictly satisfied the points of the slave surface which are intended for contact penetrates the master surface. To prevent this, point/face or line/face contact elements must be used.

**Like mentioned above contact problems are highly nonlinear. Therefore always complete weighted load situations have to be used for actions. Consequently the superposition principle is no longer valid, meaning the results may not be additively superposed with other results.**

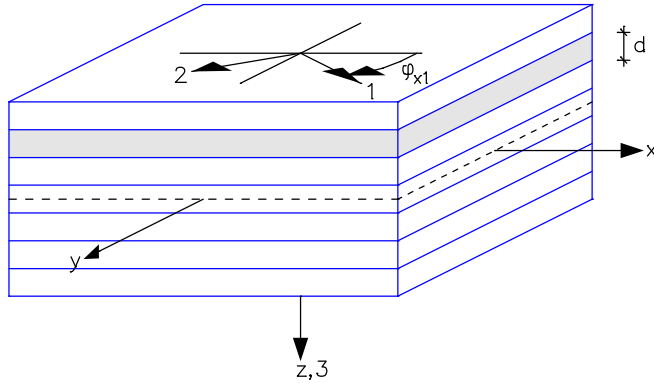
**Because of the nonlinearities the convergence of the iteration cannot be ensured for all cases.**

**After their implementation the deactivation of contact conditions is no longer possible because of the node separation of master and slave surfaces. Otherwise the bodies could penetrate each other or instability of the structure could appear. To create a structure with monolithic connections a new mesh generation is necessary.**

## Laminate

Laminates are used for the numerical simulation of shell structures with a layered material. For example, that can be glass fiber reinforced plastic or plywood.

A laminate cross-section has multiple orthotropic layers, which can be orientated differently. The orthotropic directions '1' and '2' of a layer, in which the flexibility matrix is defined, are specified by the angle  $\varphi_{x1}$ . The reference plane for the elasticity matrices is the center plane of the shell element. In approach, it is assumed that a constant curvature exists over the section thickness. In the standard case a perfect bond between the layers is provided for the analysis. Optionally the calculation can be performed without shear bond between the layers. In sections without shear bond the average strains of the layers are equal to the average strain of the section.



Laminate with layers, element coordinate system  $(x, y, z)$  and coordinate system of one laminate layer

The flexibility matrix of a laminate layer in the orthotropic directions is defined by:

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_1} & -\frac{\mu_{21}}{E_2} & 0 \\ -\frac{\mu_{12}}{E_1} & \frac{1}{E_2} & 0 \\ 0 & 0 & \frac{1}{G_{12}} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{Bmatrix} \quad \text{with} \quad \frac{\mu_{12}}{E_1} = \frac{\mu_{21}}{E_2}$$

The values  $d$ ,  $\varphi_{x1}$ ,  $E_1$ ,  $E_2$ ,  $\mu_{21}$ ,  $G_{12}$ ,  $G_{13}$ ,  $G_{23}$  and the thermal expansion coefficient  $\alpha_t$  are specified separately for each layer. The shear moduli  $G_{13}$  and  $G_{23}$  are only used in conjunction with shear flexible 'Mindlin-Reissner' elements. Therefore the shear correction coefficients  $\kappa_{13}$  and  $\kappa_{23}$  for the  $z$  direction (3) are assumed to be  $5/6 = 0.8333$ .

For isotropic material  $E_1 = E_2 = E$ ,  $\mu_{21} = \mu_{12} = \mu$ ,  $G_{12} = E / [2 \cdot (1 + \mu)]$  is applied.

If the laminate layers are not arranged symmetrically in the cross-section and are joined together resistant, coupling terms, which are not zero, exist in the integral elasticity matrix. These coupling terms cause normal forces due to curvature or bending moments due to strains. The several terms of the elasticity matrix are listed in the log.

$$\begin{Bmatrix} n_x \\ n_y \\ n_{xy} \\ m_x \\ m_y \\ m_{xy} \end{Bmatrix} = \begin{bmatrix} D_S & D_K \\ D_K & D_P \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \\ \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix}$$

Integral elasticity matrix with  $D_P = \text{plain}$ ,  $D_S = \text{slab}$ ,  $D_C = \text{coupling part}$ .

## Equation Solver

### Parallel Sparse Solver

The *parallel sparse solver* is a direct equation solver based on the Intel® Math Kernel Library that is optimized for sparse systems of equations and can be used for all applications relating to statics and dynamics as well as the analysis of stability problems. Thanks to efficient data organization, the required amount of memory space is much lower compared to the standard equation solver and the number of necessary computing operations is minimized. The parallel sparse solver can be selected from the statics analysis settings.

Parallel processing based on the OpenMP standard results in an additional speed increase on multi-processor systems. All available processor cores are activated by default. You can override this behavior using the `OMP_NUM_THREADS` environment variable. For example, the `OMP_NUM_THREADS=1` setting causes the equation solver to use a single processor core only. The environment variables can be defined in the Control Panel. You may need to restart the system for changes to take effect.

For very large or densely populated systems of equations, the memory requirement may exceed the address space of the 32-bit version of the program. In such cases the calculation will abort with the following error message:

```
Triangulating system of equations
** Errors **
FEM-F147: Sparse Solver: Not enough addressable memory.
Abort of program, 0 Warning(s), 1 Error(s).
```

This problem can be avoided by using the 64-bit edition.

For relocatable systems, the calculation will abort either with an error message due to singularity of the system of equations or with a warning that the deformations are too large. Stability failures are generally displayed by a singularity of the system of equations.

### Iterative Equation Solver

The iterative solution of the system of equations is carried out according to the 'preconditioned conjugated gradient algorithm' (Schwarz, 1984). The fundamental advantage of this method is the compact storage of the coefficient matrix. Only the non-zero coefficients of the original system of equations are stored. In addition, a shorter solution time can result for certain structures. The disadvantage of an iterative solution is that for each load case the iteration must be started anew. The number of required iteration steps depends on the problem. For larger systems of equations the number of steps often is between 5 and 15% of the number of unknowns. In contrast to solving by triangulation, a singularity of the system of equations, for instance due to missing support or stability failure according to the second-order theory, cannot be determined directly.

The iteration method determines the minimum of the quadratic functional

$$f(x) = 1/2 \cdot x^T \cdot A \cdot x + b^T \cdot x$$

The necessary condition is given by the system of equations

$$A \cdot x + b = 0$$

which is to be solved. The minimum of  $f(x)$  is determined iteratively by using the direction of the gradient  $f(x)$  in every individual step.

The convergence behavior of an iterative solution method is directly dependent on the condition number of the coefficient matrix  $A$  of the system of equations. The smaller the condition number, the fewer iteration steps are necessary for solving. The algorithm at hand uses a preconditioning per iteration step with the conditioning matrix

$$H = I + \omega \cdot E$$

with

$I$  Identity matrix

$E$  Lower triangle of the scaled coefficient matrix

$\omega$  Parameter

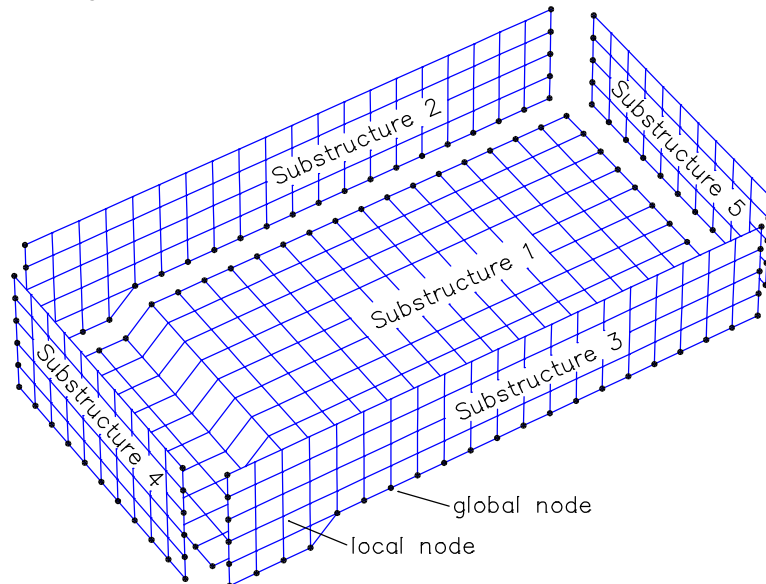
Given an optimal selection of the parameter  $\omega$ , the condition number of the preconditioned matrix is equal to the square root of the original condition number. In the program the parameter  $\omega$  has been set to 1.10.



## Substructure Technique

Substructure engineering represents an alternative direct solution approach. For special structure models such as, for example, a high-rise whose floor slabs are only weakly linked by the rising parts (walls and columns), this method requires less memory than the standard equation solver.

The basic concept of this method is to subdivide the structure model into a number of substructures that are coupled via common (global) nodes. Supports and linking nodes are always global nodes. The degree of freedom of all other (local) nodes of a substructure can, after condensation, be expressed as a function of the global nodal degrees of freedom. The following illustration depicts a simple structure with a possible substructure division.



*Substructure division of a rain storage reservoir, exploded view*

As a result of the condensation process, subelement matrices result that only contain global nodal degrees of freedom. These are then fed into the global stiffness matrix or global mass matrix of the global nodal degrees of freedom. A similar approach is used for the load vector. After calculating the global nodal degrees of freedom, the local degrees of freedom can be determined from the subelement matrices.

### Condensation process for static problems

In formal terms, the condensation process can be broken down as follows:

1. Construct a system of equations for a substructure  
 $K \cdot u = b$
2. Separation of local (index  $l$ ) and global (index  $g$ ) degrees of freedom

$$\begin{bmatrix} K_{ll} & K_{lg} \\ K_{gl} & K_{gg} \end{bmatrix} \cdot \begin{bmatrix} u_l \\ u_g \end{bmatrix} = \begin{bmatrix} b_l \\ b_g \end{bmatrix}$$

3. Elimination of the local degrees of freedom of a substructure

$$K_{\text{cond.}} = K_{gg} - K_{gl} \cdot K_{ll}^{-1} \cdot K_{lg} \quad b_{\text{cond.}} = b_g - K_{gl} \cdot K_{ll}^{-1} \cdot b_l$$

*condensed stiffness matrix*

*condensed load vector*

4. Construction and solution of the global stiffness matrix of the global nodal degrees of freedom
5. Calculation of the local degrees of freedom of the substructures

$$u_l = K_{ll}^{-1} \cdot (b_l - K_{lg} \cdot u_g)$$

### Condensation process for dynamic eigenvalue problems

In formal terms, the condensation process can be broken down as follows:

1. Formulate an eigenvalue problem for a substructure  
 $K \cdot u = \lambda \cdot M \cdot u$
2. Separation of local (index  $l$ ) and global (index  $g$ ) degrees of freedom

$$\begin{bmatrix} K_{ll} & K_{lg} \\ K_{gl} & K_{gg} \end{bmatrix} \cdot \begin{bmatrix} u_l \\ u_g \end{bmatrix} = \lambda \cdot \begin{bmatrix} M_{ll} & M_{lg} \\ M_{gl} & M_{gg} \end{bmatrix} \cdot \begin{bmatrix} u_l \\ u_g \end{bmatrix}$$

3. Elimination of the local degrees of freedom

$$u_l = -(K_{ll} - \lambda \cdot M_{ll})^{-1} \cdot (K_{lg} - \lambda \cdot M_{lg}) \cdot u_g = T_{lg} \cdot u_g$$

As the eigenvalue  $\lambda$  is unknown, it is set to zero in the sense of a static condensation. The following condensed subelement matrices result:

$$K_{cond.} = K_{gg} - K_{gl} \cdot K_{ll}^{-1} \cdot K_{lg}$$

*condensed stiffness matrix*

$$M_{cond.} = M_{gg} + T_{lg}^T \cdot M_{lg} + M_{gl} \cdot T_{lg} + T_{lg}^T \cdot M_{lg} \cdot T_{lg}$$

*condensed mass matrix*

4. Construction of the global stiffness and mass matrices of the global degrees of freedom. Solution of the eigenvalue problem for the global degrees of freedom.
5. Calculation of the local degrees of freedom of the eigenvectors of the substructures.

As the condensation of the eigenvalue  $\lambda$  is set to zero, the condensed eigenvalue problem does not deliver precise eigenvalues. For modes of vibration whose predominant share is determined by the global degrees of freedom, however, the derived values are of sufficient accuracy.

The main advantage of the process described is that the global stiffness and mass matrices of all nodal degrees of freedom are not needed at any time, having the effect that the required memory space and calculation time are in general considerably reduced. It should be noted, however, that given an unfavorable relationship between global and local nodes longer calculation times and larger memory space requirements may result. The use of the substructure technique typically makes sense for large structure models as the process described is significantly more complex and time-consuming than the conventional approach. In order to gain an overview of the substructure topology beforehand, you can use the calculation settings to request an information protocol which provides information regarding the size and number of the local and global nodes of a substructure. The following example is the list given for the above rain reservoir.

**System characteristics overall system**

Nodes	488
Elements	458
Unknown variables	2928
Bandwidth	144
Stiffness matrix	2.7 MB

**Assembly of the substructures**

Sub-structure	global nodes	local nodes	Elements	Bandwidth	Stiffness matrix
1	171	60	200	66	2.5 MB
2	79	30	84	48	622.9 KB
3	79	30	84	48	622.6 KB
4	45	21	50	60	308.1 KB
5	36	19	40	48	221.1 KB

As the values indicate, in this example the use of a substructure system offers no advantages when compared with a calculation as an entire system. In general the substructures should be chosen so that the required memory space of the stiffness matrix of a substructure is smaller than the available core memory as the condensation process would otherwise take very long.

## Singular System of Equations

Global stiffness matrices are solved through triangulation in the standard and sparse equation solver and when using substructure technique. If the system of equations is not explicitly solvable, meaning it is singular, the calculation is aborted with an error message.

```
Load case 1
Assembling total stiffness matrix (100 %)
Installing supports
Triangulating system of equations
Line 1270
** Errors **

FEM-F001: Set of equations is singular, line 1273, node 138 (Ux).
Abort of program, 0 Warning(s), 1 Error(s).
```

The error indicates the nodal degree of freedom at which the singularity was determined. The following causes are possible:

- The system is relocatable.
- After a failure of the tensile bedding or tensile support, the degrees of freedom are no longer supported.
- When calculating according to the second-order theory, the system fails due to beam or shell buckling.
- In structures with tension members or cable elements, the system becomes unstable due to the failure of a beam or cable element.
- When performing a nonlinear system analysis, the creation of plastic hinges leads to a system failure.

In most cases the relocatability of the system is due to erroneous input data:

- Beam series in slabs or plain stress structures are not supported. Beam elements always have 6 nodal degrees of freedom, but are supported by slabs or plain stress elements with only 3 degrees of freedom. The extra degrees of freedom must be supported.
- Slabs or plain stress structures erroneously contain individual shell elements. Of the shell elements' 6 degrees of freedom, in a slab or plain stress element only 3 degrees of freedom are supported.
- Shell structures are only supported in the z direction and thus have open degrees of freedom.
- A framework has joint chains that make the system kinematic. When troubleshooting it can be helpful to replace all joints with stiff connections and then reactivate them one by one until the error occurs.
- A linear beam series has a ball joint at the beginning and end and can therefore rotate about its longitudinal axis.
- Calculations excluding tensile bedding or tensile support are carried out under unrealistic load assumptions, for instance without dead load, and lead to lift-off of the structure.
- A nonlinear system analysis for structures made of reinforced concrete is carried out without sufficient initial reinforcement.

## Capacity Limit Under Windows

For the calculation of larger structures, especially shell and solid structures, the amount of data that is dealt with during the calculation (global stiffness matrix, mass matrix, load vector, etc.) can reach an order of magnitude of several hundred megabytes (MB) to several gigabytes (GB). If the available memory space is not sufficient, the calculation is terminated with an error message. This may be due to one the following causes:

- The swap file of the operating system is too small. The size of this file is specified in the control panel.
- The maximum capacity of the 32-bit version of the program of 2 GB was reached. This limit is a result of the 32-bit address space, which is independent of the amount of RAM or hard drive space of the computer.

If the calculation is aborted, you should check the messages listed in the output bar. They can be copied or printed out using the context menu. The messages for a very large shell structure (high-rise) are given below:

### System characteristics

```
Nodes           86359
Elements        80968
Supports         44
Unknown variables 517980
Bandwidth        2106
Stiffness matrix 5567.4 MB
Mass matrix      104.5 MB
```

```
** Errors **
```

```
Out of memory.
```

```
Abort of program,  0 Warning(s), 1 Error(s).
```

The above system of equations with 517,980 unknowns has, despite bandwidth optimization and shell-oriented memory technology, a memory requirement of 5.5 GB, exceeding the capacity of the operating system. The following options are available to solve the problem:

- Use of the parallel sparse solver,
- Use of the iterative equation solver,
- Use of the 64-bit edition,
- Use of the substructure technique,
- Reduction of the number of nodes by simplifying the system (e.g., by taking advantage of symmetries).

Using the iterative equation solver usually results in the smallest memory requirement. It should be noted that in contrast to the direct equation solvers, a singularity of the system of equations, for instance due to missing support or stability failure according to the second-order theory, cannot be determined directly. This fact is usually only recognized once after a large number of iterations - around 15% of the number of unknowns - still no convergence is in sight.

## Program Capacities

```
Nodes           9999999
Elements        999999
Load cases      9999
Load case combinations 9999
Tendon groups  999
Dynamic eigenmodes 999
```

### Restrictions for the study version

```
Nodes           1000
Elements (FEM)  1000
Beams (ESW, RSW) 10
Area sections   1
Beam sections   1
Tendon groups  1
Dynamic eigenmodes 1
```

## Analysis Results

The sections below describe the following results:

- Deformations
- Internal forces for beams , spring elements and area elements
- Stresses and strains for beams and area elements
- Stresses, strains and damage for solid elements
- Bedding forces for beams
- Soil pressure
- Support reactions
- Velocities and accelerations from time step integration
- Stationary response
- Eigenmodes and eigenvalues
- Buckling eigenmodes
- Influence lines and influence surfaces
- Stresses of tendon groups

### Deformations

The node deformations refer to the global coordinate system and are designated as follows:

$u_x, u_y, u_z$  Node displacements [m] without predeformation in the direction of the global axes.

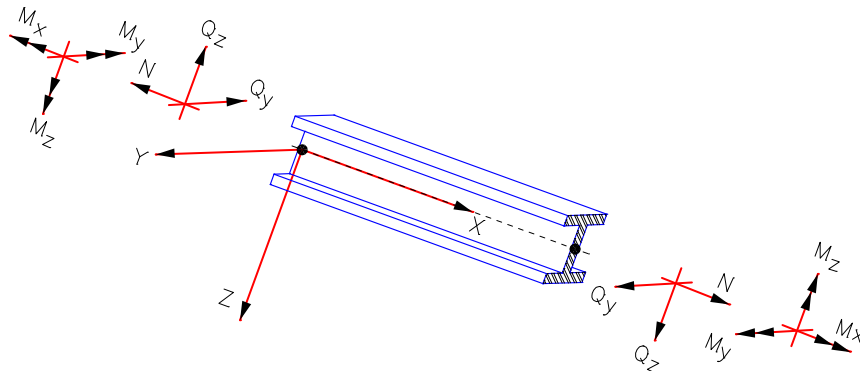
$\varphi_x, \varphi_y, \varphi_z$  Node rotations [rad] around the global axes.

For supported nodes additionally the deformations in the coordinate system of the supports are displayed. For beams the bending curve is also calculated.

### Internal forces for beams

The bending moments  $M_y$  and  $M_z$  as well as the normal force  $N_x$  refer to the section's centroid. The torsional moment  $M_x$  and the lateral forces  $Q_y$  and  $Q_z$  relate to the shear center. The program assumes that the shear center and centroid coincide. For asymmetrical sections the effect of the eccentricity of the shear center on the fixed-end reactions is therefore not accounted for. The internal beam forces are calculated at the beginning and end of the beam as well as at equidistant points along the beam. The number of result locations can be set. Moments are given in [kNm] and forces in [kN].

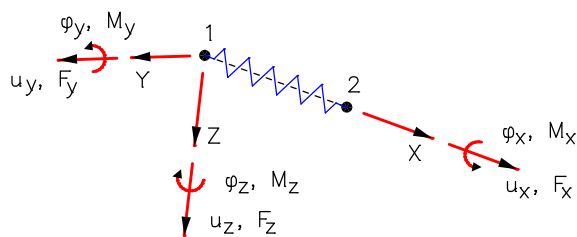
For the calculation of the internal forces according to the second-order theory, the beam forces can correspond to either the deformed or undeformed beam chord.



Beam system with positive internal forces

### Internal forces for spring elements

Depending on the spring characteristics the forces and moments arise from the following local relations:



Deformations and internal forces in the local coordinate system of the spring elements

$$F_x = F(\Delta u_x = u_{x,2} - u_{x,1})$$

$$F_y = F(\Delta u_y = u_{y,2} - u_{y,1})$$

$$F_z = F(\Delta u_z = u_{z,2} - u_{z,1})$$

$$M_x = F(\Delta \varphi_x = \varphi_{x,2} - \varphi_{x,1})$$

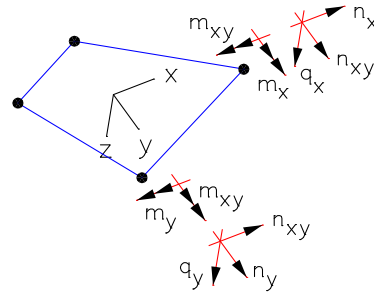
$$M_y = F(\Delta \varphi_y = \varphi_{y,2} - \varphi_{y,1})$$

$$M_z = F(\Delta \varphi_z = \varphi_{z,2} - \varphi_{z,1})$$

**Internal forces for area elements**

Depending on the selection, the internal forces are calculated in the *Nodes*, in the *Centroid* or in the *Side middle*. The following internal forces are determined:

- $n_x, n_y$  Normal forces [kN/m]
- $n_{xy}$  Shear force [kN/m]
- $m_x, m_y$  Bending moments [kNm/m]
- $m_{xy}$  Torsional moment [kNm/m]
- $m_1, m_2$  Principal moments [kNm/m]
- $n_1, n_2$  Principal normal forces [kN/m]
- $q_x, q_y$  Lateral forces [kN/m]



Internal force system with positive internal forces

$$q_r = \sqrt{q_x^2 + q_y^2} \quad \text{Resultant lateral force [kN/m]}$$

$$q_{xr} = q_x + \frac{\partial m_{xy}}{\partial y} \quad \text{Edge lateral force [kN/m] (DKT element)}$$

$$q_{yr} = q_y + \frac{\partial m_{xy}}{\partial x} \quad \text{Edge lateral force [kN/m] (DKT element)}$$

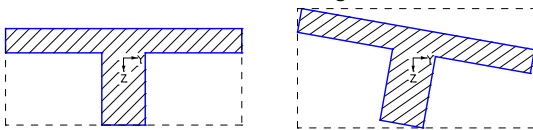
**Stresses and strains for beams**

The following calculation results are determined at the result locations of the internal forces.

- $\sigma_x, \sigma_{x \text{ top}}, \sigma_{x \text{ bottom}}, \sigma_{x \text{ min}}, \sigma_{x \text{ max}}$  Normal stresses [MN/m<sup>2</sup>]
- $\tau_{xy}, \tau_{xz}$  Shear stresses from lateral forces and torsion [MN/m<sup>2</sup>]
- $\sigma_v = \sqrt{\sigma_x^2 + 3 \cdot \tau_{xy}^2 + 3 \cdot \tau_{xz}^2}$  Comparison stress according to the Huber-von Mises yield criterion [MN/m<sup>2</sup>]
- $\epsilon_x$  Longitudinal strain of the beam axis [‰]
- $\epsilon_{x \text{ top}}, \epsilon_{x \text{ bottom}}$  Edge strain [‰]
- $\epsilon_{x \text{ min}}, \epsilon_{x \text{ max}}$  Minimum and maximum edge strain [‰]
- $\kappa_y, \kappa_z$  Curvature about the y or z axis [1/km]
- $\sigma_s \text{ min}, \sigma_s \text{ max}$  Minimum and maximum reinforcing steel stresses of all steel layers [MN/m<sup>2</sup>]

The shear and comparison stresses are determined as part of the stress checks or a nonlinear system analysis. The strain, curvature and the reinforcing steel stresses are calculated as part of a nonlinear system analysis.

The edge strain  $\epsilon_{x \text{ min}}$  or  $\epsilon_{x \text{ max}}$  and normal stresses  $\sigma_{x \text{ top}}$  or  $\sigma_{x \text{ bottom}}$  relate to the first polygon point that lies on the upper or lower edge of the enclosing rectangle. The z-axis of the beam points to the lower edge. The other stresses are extreme values on the section edge.



Sections with enclosing rectangles

**Stresses and strains for area elements**

The stresses [MN/m<sup>2</sup>] and strains [‰] are defined in the internal force system. They are determined at the result locations of the internal forces on the upper and lower section edges. For laminate sections, the stresses  $\sigma_x, \sigma_y$  and  $\tau_{xy}$  at the upper and lower edges of each layer are stored.

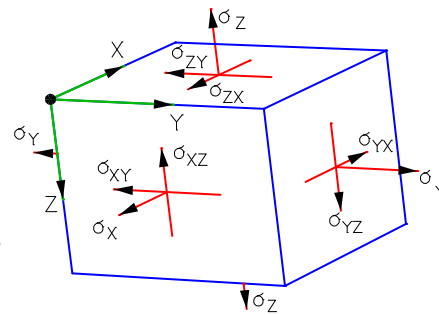
- $\sigma_{x \text{ top, bottom}}$  Normal stresses (elastic  $\sigma_{x \text{ top, bottom}} = \frac{n_x}{d} \mp \frac{m_x}{d^2/6}$ )
- $\sigma_{y \text{ top, bottom}}$  Normal stresses (elastic  $\sigma_{y \text{ top, bottom}} = \frac{n_y}{d} \mp \frac{m_y}{d^2/6}$ )
- $\tau_{xy \text{ top, bottom}}$  Shear stresses (elastic  $\tau_{xy \text{ top, bottom}} = \frac{n_{xy}}{d} \mp \frac{m_{xy}}{d^2/6}$ )
- $\sigma_1 \text{ top, bottom}$  Maximum principal stresses

$\sigma_{2 \text{ top,bottom}}$	Minimum principal stresses
$\sigma_v \text{ top,bottom}$	Comparison stress according to the Huber-von Mises yield criterion $\sigma_v = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \cdot \sigma_y + 3 \cdot \sigma_{xy}^2}$
$\varepsilon_{1 \text{ top,bottom}}$	Maximum principal strains
$\varepsilon_{2 \text{ top,bottom}}$	Minimum principal strains
$\varphi_{\text{top,bottom}}$	Principal axis angle of the strains [rad]
$\varepsilon_{\text{crack top,bottom}}$	Strains perpendicular to the crack direction when the tensile strength of the concrete is exceeded
$\sigma_{sx/y \text{ i.Layer}}$	Reinforcing steel stresses of layer i in x or y direction
$\varepsilon_{sx/y \text{ i.Layer}}$	Reinforcing steel strains of layer i in x or y direction

**Stresses for solid elements**

The stresses [MN/m<sup>2</sup>] can be calculated either in the nodes or at the centroid. The directions of the stresses are determined by the internal force system.

$\sigma_x', \sigma_y', \sigma_z$	Normal stresses
$\sigma_{xy}', \sigma_{yz}', \sigma_{zx}$	Shear stresses
$\sigma_1$	Maximum principal stresses
$\sigma_2$	Intermediate principal stresses
$\sigma_3$	Minimum principal stresses
$\sigma_v$	Comparison stresses according to the Huber-von Mises yield criterion



Internal force system with positive stresses

$$\sigma_v = \sqrt{\sigma_x^2 + \sigma_y^2 + \sigma_z^2 - (\sigma_x \cdot \sigma_y + \sigma_x \cdot \sigma_z + \sigma_y \cdot \sigma_z) + 3 \cdot (\sigma_{xy}^2 + \sigma_{xz}^2 + \sigma_{yz}^2)}$$

**Strains for solid elements**

In the nonlinear system analysis (material type *Frei*), both the total strains and the plastic or inelastic strains are determined. In analogy to the stresses, the principal and comparison strains are determined.

Total, plastic strains (see Eq. V12) [-]:	Inelastic strains (see Eq. V32 or. Eq. V33) [-]:
$\varepsilon_x', \varepsilon_y', \varepsilon_z$ Normal strains	$\varepsilon_x^{\text{in}}, \varepsilon_y^{\text{in}}, \varepsilon_z^{\text{in}}$ Inelastic normal strains
$\varepsilon_{xy}', \varepsilon_{yz}', \varepsilon_{zx}$ Shear strains	$\varepsilon_{xy}^{\text{in}}, \varepsilon_{yz}^{\text{in}}, \varepsilon_{zx}^{\text{in}}$ Inelastic shear strains
$\varepsilon_1$ Maximum principal strains	
$\varepsilon_2$ Average principal strains	Equivalent plastic strains [-]:
$\varepsilon_3$ Minimum principal strains	$\varepsilon_t^{\text{pl}}, \varepsilon_c^{\text{pl}}$ (tension and compression, see Eq. V26)
$\varepsilon_v$ Comparison strains	

**Damage for solid elements**

In the nonlinear system analysis (material type *Frei*) with a damage or combined material model, the *damage* [-] is also determined.

**Normal forces and stresses for cable elements**

The normal forces and stresses relate to the axes of the deformed cable elements and are calculated in the nodes and in the center of the cable.

$N_x$	Normal force [kN/m]
$\sigma_x$	Normal stress [MN/m <sup>2</sup> ]

**Bedding forces for beams**

The bedding forces  $B_x, B_y$  and  $B_z$  [kN/m] are defined in the beam system and represent the soil pressures corresponding to the beam width (for instance,  $B_z = \sigma_z \cdot d_y$ ). Tensile forces in the soil have a positive sign. For bedded beams, the reactions are only calculated in the nodes, and the reaction curve is interpolated linearly between them.

**Soil pressures**

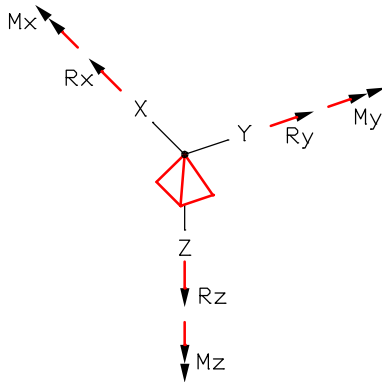
The soil pressures  $\sigma_x$ ,  $\sigma_y$  and  $\sigma_z$  [kN/m<sup>2</sup>] are defined in the element system except for solid elements *VT10*. The z axis of the solid element *VT10* is perpendicular to the bedded surface and the x axis runs horizontal in the bedded surface and if possible parallel to the global x axis. Tensile stresses in the soil have a positive sign. Additionally, the soil reactions are integrated for each element and saved in the following values:

- $R_x, R_y, R_z$  Resultant soil reaction [kN]
- $d_x, d_y, d_z$  Coordinates of the resultant soil reaction [m]

**Support reactions**

The support reactions are calculated in the coordinate system of the supports. They act on the corresponding support. The following reactions are provided:

- $R_x, R_y, R_z$  Support forces [kN]
- $M_x, M_y, M_z$  Support moments [kNm]



*Support system with positive support reactions.*

**Velocities and accelerations**

The node velocities and accelerations are calculated for dynamic time step integrations. They refer to the global coordinate system and are designated as follows:

- $v_x, v_y, v_z$  Node velocities [m/s] in the direction of the global axes
- $a_x, a_y, a_z$  Node accelerations [m/s<sup>2</sup>] in the direction of the global axes

**Stationary response**

A stationary response can be determined as part of a dynamic analysis. The results refer to the global coordinate system and are designated as follows:

- $U$  Absolute node deflection
- $U.real$  Real part of the node deflection
- $U.imaginary$  Imaginary part of the node deflection
- $Phase$  Phase shift
- $DU$  Absolute node velocity (first derivative)
- $DDU$  Absolute node acceleration (second derivative)

**Eigenmodes and eigenvalues**

Dynamic eigenmodes and their corresponding eigenvalues can be determined as part of a dynamic system analysis. The eigenmodes are standardized.

- $u_x, u_y, u_z$  Node displacements in the direction of the global axes
- $\varphi_x, \varphi_y, \varphi_z$  Node rotations around the global axes
- $\omega$  Circular eigenfrequency [rad/s]

**Buckling eigenmodes**

Buckling eigenmodes and their corresponding load factors are determined as part of an analysis of load cases with load type *Pki*. The buckling eigenmodes are standardized.

- $u_x, u_y, u_z$  Node displacements in the direction of the global axes



$\varphi_x, \varphi_y, \varphi_z$  Node rotations around the global axes.

In the listing *Finite elements*:

*Load factors* Buckling load factors of the buckling eigenmodes

$sk.y, sk.z$  Buckling lengths [m] of the beams of the first buckling eigenmode

*Pki* Buckling load [m] of the beams of the first buckling eigenmode

$EI_y, EI_z$  Stiffness of the beams [kNm<sup>2</sup>]

### Influence lines and influence surfaces

Influence lines or influence surfaces are determined as part of a static analysis of load cases with load type *Efl* or *Eff*. The ordinates of the resulting influence surface are expressed as deformation in the direction of the load to be analyzed.

$u_x, u_y, u_z$  Ordinate value [m] in the direction of the global axes.

$\varphi_x, \varphi_y, \varphi_z$  Ordinate value [rad] around the global axes

### Stresses of tendon groups

























The stresses in the tendon groups are determined within a non-linear system analysis, if tendon groups have been considered. The results are related to the tendon group axis are designated as follows:

$\sigma_p$  Normal stress [MN/m<sup>2</sup>]

$\sigma_p/f_{p0,1d}$  Utilization of the tendon group

## Structure Description

The following *Structure* menu functions can be used for system input and the graphical representation of the results:

-  *Mesh Generation*
-  *Beam series*
-  *Element Input*
-  *Element Properties...*
-  *Element Info*
-  *View...*
-  *Mesh Check*
-  *Element System*
-  *Number Sort*
-  *Supports*
-  *Load*
-  *Predeformation*
-  *Moving Nodes*
-  *Condense*
-  *Link Elements*
-  *Spring Elements*
-  *Line Hinges*
-  *Prestressing*
-  *Point Masses*
-  *Design Objects*
-  *Print Input Data...*
-  *Results...*
-  *Reinforcement Export*
-  *Punching Shear Check*

The general procedure for data input, analysis and data representation of a structure is described in the document *Introductory\_Examples.pdf* installed in the program directory.

## Mesh Generation

The *Mesh Generation* menu item is used to create a finite element mesh. The available methods can generally be divided into two groups:

### Mesh generators using model or NURBS objects

These methods allow you to mesh a two- or three-dimensional structure constructed from model objects (*Edge, Wall, Hole, Column, Face, Cone, Solid*) or *NURBS* objects.

*Form-Sensitive from Model (Lp Norm or Paving)*

Mesh generation of area structures affine to the constraints with consideration of all visible model objects.

*Grid-Shaped from Model*

Mesh generation of area structures through an orthogonal grid with consideration of all visible model objects.

*Beams from Selected Edges*

This option generates beams from edges with the meaning *Free beam* or *Pile beam*.

*Selected Model Faces and Cones*

Mesh generation of the selected model faces and cones analogous to the form-sensitive mesh generation with consideration of all visible model objects within the model faces.

*Selected NURBS Objects*

Mesh generation of the selected *NURBS* objects.

*Tetrahedrons from Solid*

Element generation of structures described by the model object *Solid*. In this process tetrahedron elements *VT10* are created.

The video "[https://download.infograph.de/video\\_en/Input%20of%20a%20small%20building%20construction.mp4](https://download.infograph.de/video_en/Input%20of%20a%20small%20building%20construction.mp4)" (German language) shows the usual handling of model objects.

### Mesh generators with direct description of the generation area

When using these mesh generators, the generation area is interactively specified in a dialog. The element properties and the support conditions are assigned manually.

*Polygonal Area*

Mesh generation of a polygonal bordered area with the *Form-Sensitive from Model (Paving)* method.

*Circular or Conic Shape*

Creates a circular, cylindrical or cone-shaped element mesh.

*Parabolic Bordered Square*

Creates an element mesh specified through four corner points and four side points.

*Triangular Area*

Mesh generation of a triangular area where all edges can be freely divided.

*Square - Grid on Four Edges*

Mesh generation of a square area where all edges can be freely divided.

*Square - Variable Grid on Two Edges*

Mesh generation of a square area with freely assignable grid spacing in both principal directions.

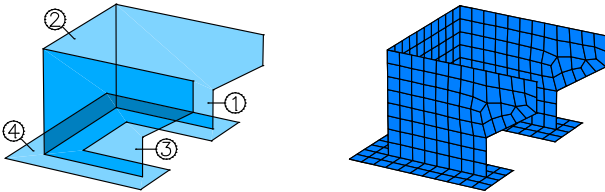
*Square or Cuboid*

Mesh generation of a square area or octagonal body with an identical number of elements on the opposite edges.

## Form-Sensitive or Grid-Shaped From Model

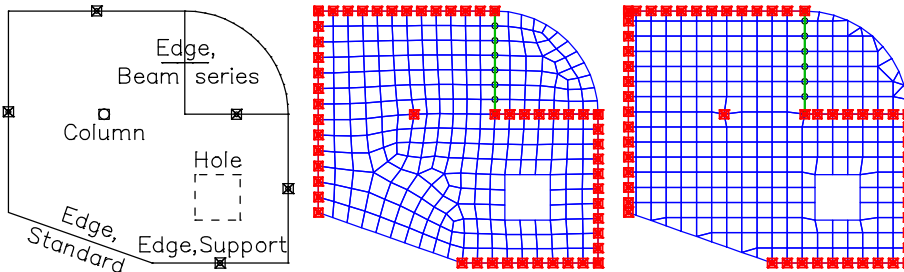
The *Form-Sensitive* and *Grid-Shaped* generators automatically mesh a structure defined by model objects (*Edge, Wall, Hole, Column, Face, Cone*). In this process, either an orthogonal grid or a mesh affine to the constraints is created. Optionally, beam, element, and support properties are adopted from the model objects. Before a new mesh generation all elements within the area, which is to be meshed, and on the outer edges are deleted together with the corresponding nodal, beam and element loads. The program distinguishes between the following cases during mesh generation:

1. Model faces or cones are present.  
In this case only these are meshed. However, all visible model objects inside the areas are accounted for.



3D mesh generation of a structure defined with 4 model faces.

2. No model faces or cones are present.
  - A. The model objects are in the xy plane and are displayed in 2D mode. The boundary of the area to be meshed is automatically recognized. All visible model objects inside the located area are accounted for during mesh generation.



Form-sensitive and grid-shaped 2D mesh generation

- B. The model objects are not in the xy plane or are not displayed in 2D mode. A mesh generation plane has to be defined by selecting model objects. The model objects contained in the mesh generation area must also be selected here. This process must be performed separately for each plane for which a mesh is to be generated.

### Input

<i>Mesh width</i>	Grid spacing or average node distance for mesh generation.
<i>Type</i>	Selection of the element type ( <i>Slab, Plain stress</i> or <i>Shell</i> ).
<i>Column head</i>	Enable/disable FEM meshing of the column geometry.
<i>Direction</i>	Set the zero point and direction of the grid in case 2. This allows you to center the mesh on a fixed point and align it along a fixed line. This option is available for the mesh generator <i>Grid-shaped from model</i> .
<i>Form</i>	Choose the preferred element form (triangle or square) for the grid. This option is available for the mesh generators <i>Grid-shaped from model</i> and <i>Form-sensitive from model (Lp Norm)</i> .

## Polygonal Area

Automatic mesh generation of an area that is delineated by the constraints listed below. The meshing corresponds to the result of the *Form-Sensitive from Model (Paving)* mesh generation. No model objects are necessary for this mesh generation.

### Input

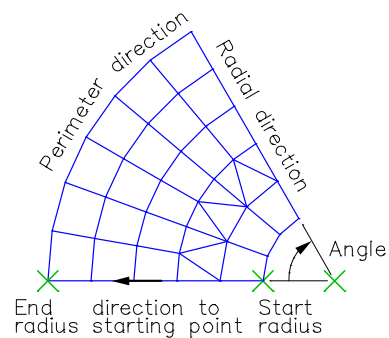
<i>Boundary</i>	The outer edge of the area to be meshed must always be specified.
<i>Hole</i>	Holes in the mesh generation area.
<i>Fixed line</i>	Lines within the described area that are considered as boundary conditions during mesh generation.
<i>Fixed point</i>	Points within the described area that are to be considered during mesh generation.

### Circular or Conic Shape

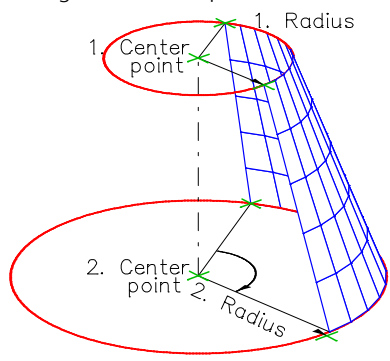
Generates a circular, cylindrical or cone-shaped mesh.

#### Input

- 2D** Generation of an annulus with the following determinants: midpoint, arc beginning, opening angle, start radius (inside), end radius (outside), number of elements in radial and perimeter direction.
- 3D** Generation of a truncated cone using the following determinants: first and second center point, first and second radius, sector, number of elements in the meridian and perimeter direction. The cylinder or cone shapes can be realized using corresponding determinants.
- Variable/constant** The element arrangement in the perimeter direction is variable (flexible) or constant (fixed).



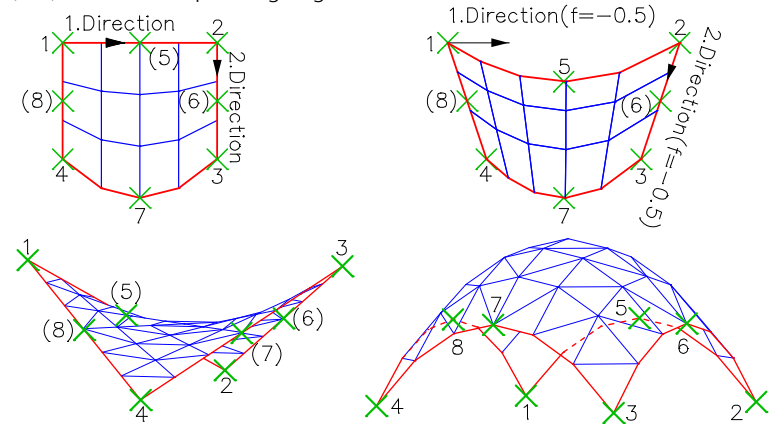
Circular, 2D (variable)



Circular, 3D (constant)

### Parabolic Bordered Square

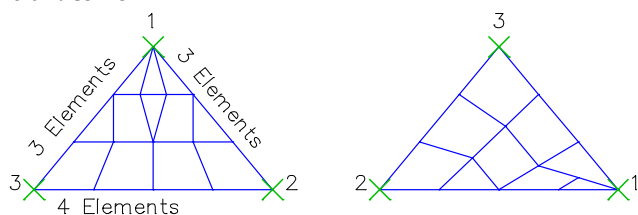
Mesh generation of a parabolic bordered area by specifying four corner points as well as one side point each. The side points define the parabolic fillet of the edges. Opposite edges are assigned the same number of nodes. Condensation factors with values from -1 to +1 allow a concentration of the element mesh towards the start point (< 0) or the end point (> 0) of the corresponding edge. Area or cable elements can be selected for mesh generation.



Points in parentheses where suggested on the side middles and accepted.

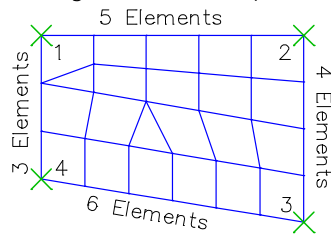
### Triangular Area

Mesh generation of a triangular area with a variable number of elements on all three edges. The element mesh is focused towards the first corner. Thus, for an acute-angled triangle, best results are achieved by first specifying the point at the blunt corner.



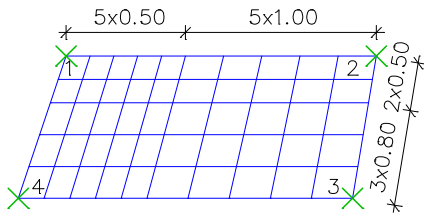
### Square - Grid on Four Edges

Mesh generation of a square area with variable numbers of elements on all four edges.



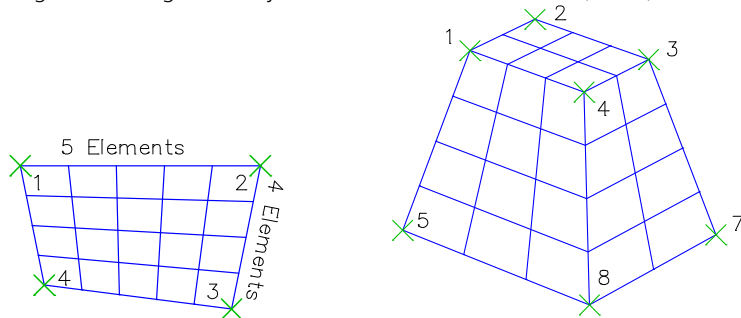
### Square - Variable Grid on Two Edges

Mesh generation of a square area with preset grid spacing in both principal directions. Opposite edges have the same number of nodes.



### Square or Cuboid

For area and cable elements a square area is meshed which features an identical number of elements on the opposite edges. An octagonal body is meshed for solid elements (VQ83).

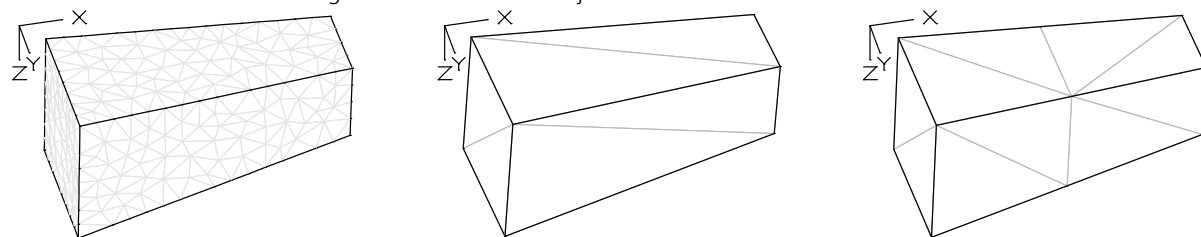


### Tetrahedron From Solid

The generator automatically meshes structures described by the model object *Solid* using the Delaunay triangulation method. Thereby all visible edges within the solid are taken into account. The mesh generator creates an element mesh of tetrahedron elements VT10. The properties (*Section*, *Layer* and *Color of elements*) of the solids are adopted. To regulate the mesh generation the following criteria can be adjusted:

- *Total volume / Element volume*: This ratio (V) limits the size of the elements.
- *Circumsphere radius / min. Edge length*: This ratio (B) describes the quality of the element shapes. Small values increase the quality of the elements but also increase the number of elements.

For these criteria also the settings of the solid model objects can be used.



Element mesh:  $V = 1000; B = 3$

$V = 1; B = 3$

$V = 1; B = 1,5$

The video "[https://download.infograph.de/video\\_de/Volumenmodell%20eingeben.mp4](https://download.infograph.de/video_de/Volumenmodell%20eingeben.mp4)" (German language) shows the usual handling of solid models from tetrahedra.

## Beam Series

This function is used to automatically create beam elements that connect to the existing mesh nodes (modeling of downstand beams, for example).

### Input

<i>Start</i>	Specify the start point of the beam series.
<i>Endpoint</i>	Specify the end point of the beam series.
<i>Properties</i>	The element properties dialog is called up with this option.

## Element Input

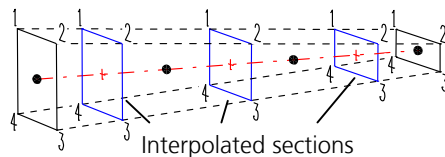
Individual finite elements can be defined with this menu item. The element types are described in detail in the *Element library* section (*Basics* chapter).

### Elements

<i>RS</i>	3D beam element
<i>RF</i>	3D truss element
<i>SEIL</i>	3D cable element
<i>PV43</i>	Square slab element
<i>PD33</i>	Triangular slab element
<i>SV43</i>	Square plain stress element
<i>SD33</i>	Triangular plain stress element
<i>SD62</i>	Triangular plain stress element with side middle nodes
<i>SH46</i>	Square shell element
<i>SH36</i>	Triangular shell element
<i>VQ83</i>	Solid element with eight corner nodes
<i>VT10</i>	Tetrahedron element with side middle nodes

### Input

<i>Starting point</i>	Input a beam from starting to end point.
<i>Generate</i>	The entered line is split into n beams of the same length. In conjunction with the polygon, square and beam section types, you can specify different start and end sections. The program then interpolates the selected number of sections. This enables the modeling of a haunch. The position of the polygon points are interpolated for polygon sections. For this to work, the number and arrangement of the polygon points must match one another.



<i>Beam series</i>	Automatic creation of beams that connect to existing net nodes, for example, for modeling downstand beams.
<i>Edge points</i>	Specify the individual node coordinates of the elements.
<i>Properties</i>	The element properties dialog is called up with this option.

When entering beams in 3D mode, a point in the local xz plane is queried. For 2D input, the program uses the default orientation.

Slab and plain stress elements must be entered in a clockwise manner and parallel to the xy plane. The form of the area elements can be varied up to an aspect ratio of 1/10 and an internal angle of 180°.

## Element Properties

This dialog is used to edit the properties of the *selected elements* or preset the properties for new elements yet to be created. The dialog consists of the following pages:

- *General or Joints*
- *Eccentricity*
- *Equivalent beam length*
- *Section*
- *Deformation Check*

## General

This dialog shows the general element properties.

### Element type

The selected elements' type is shown. Beam elements can be changed into truss elements or slabs into shell elements, for example. When changing areas in solid elements (VQ83), new nodes are created at an interval corresponding to the element thickness  $dz$ . Solid elements cannot be changed into other types of elements.

### Length, Angle

Display of beam or cable length [m], input value for the rotation of the beam section around the beam axis [°].

### Area

Display the element area [m<sup>2</sup>].

### Solid

Display the element volume [m<sup>3</sup>].

### Angle for elements, internal forces and reinforcement

Angle of the local coordinate systems in [°].

### Cable prestress

Input value of the cable prestress [kN].

### Nodes

Display the node coordinates.

### Joints

The joint properties of beam elements can be specified separately for each degree of freedom. When selected, the corresponding degree of freedom is defined as jointed. The arrow indicates the selected end of the beam.

### Joint springs

Spring rigidity [MN/m, MNm] with which the respective beam end (a = start, b = end) is elastically attached to the node. If no value is defined, the respective joint property applies.

## Eccentricity

Beam elements can have different eccentricities ( $u \equiv$  local  $x$ ,  $v \equiv$  local  $y$ ,  $w \equiv$  local  $z$ ) at their start and end in their local coordinate system. Area elements only have the eccentricity  $w$  in their local  $z$  direction. The element reactions refer to the eccentric element axis, on which possible joints also are located. The nodes are still in their original location.

## Equivalent Beam Length

For the buckling check with the equivalent beam method, the equivalent beam length can be defined for beams made of timber according to EN 338 and EN 14080.

### y-axis, z-axis

The specification of an appropriate equivalent beam length under consideration of support and load conditions is subject to the discretion and the responsibility of the user. If the value 0 is entered no check is carried out for the corresponding direction.

### Note

If a structural member with constant cross-section is divided into multiple beam elements, the buckling length decisive for the whole structural member shall be applied to all elements. A computational determination of buckling lengths and buckling eigenmodes by the FEM program is possible for selected load combinations (see description of *Load group*).



## Section

The section describes the following properties of an element:

- Section form
- Specifications for determination of shear stresses
- Material
- Bedding
- Specifications for reinforced concrete design
- Specifications for thermal analysis
- Fire exposure for the simplified check according to EN 1995-1-2

Each section is uniquely identified by a number between 1 and 9999. In general a section is not assigned to a single element but rather a group of elements.

### Number

Select the section. You can assign the number freely for a newly generated section. Sections can subsequently be renumbered using the context menu for the subitem *Structure Description / Section properties* of the data base.

### Section Type

- *Area*
- *Beam*
- *Polygon*
- Steel section with profile number.
  - IPE*: Medium I girders according to DIN 1025 Part 5 and Euronorm 19-57.
  - HEA*: Wide I girders, light variant according to Euronorm 53-62 or girders of the IPBI series according to DIN 1025 Part 3.
  - HEB*: Wide I girders according to Euronorm 53-62 or girders of the IPB series according to DIN 1025 Part 2.
  - HEM*: Wide I girders, reinforced variant according to Euronorm 53-62 or girders of the IPBv series according to DIN 1025 Part 4.
  - I*: Narrow I girders according to DIN 1025 Part 1.
  - U*: U steel according to DIN 1026.
  - T*: T steel according to DIN 1024.
- *Parameter*: Parameterized steel profile with I-, L-, T-, U-, tube- or box-shaped cross-section geometry.
- *Library*: Section library with steel sections.
- *Database*: Profile from the user database (*Anwend.pro*).
- *Rectangle*
- *Tension member*
- *Composite*
- *Solid*

**Material Type**

- *<Database>*: User-defined material and creep coefficients from the system file *lgraph.dat*.
- *B25 to B55*: Concrete qualities according to DIN 1045:1988, Table 1.
- *C12/15 to C100/115*: Concrete qualities according to DIN 1045-1 or ENV 1992-1-1:1991 (*C55/67* or higher only for design according to DIN 1045-1).
- *LC12/13 to LC60/66*: Concrete qualities according to DIN 1045-1.
- *C12/15-EN to C90/105-EN*: Concrete qualities according to EN 1992-1-1.
- *C12/15-EN-D to C100/115-EN-D, LC12/13-EN-D to LC80/88-EN-D*: Concrete qualities according to DIN EN 1992-1-1.
- *CX*: Concrete according to DIN 1045-1 with user-defined properties.
- *CX-EN*: Concrete according to EN 1992-1-1 with user-defined properties.
- *CX-EN-D*: Concrete according to DIN EN 1992-1-1 with user-defined properties.
- *ÖC16/20 to ÖC50/60*: Concrete qualities according to OENORM B 4710-1.
- *ÖNB40*: Concrete quality according to OENORM B 4200-10.
- *ÖNBeton*: Concrete according to OENORM with user-defined properties.
- *SC12/15 to SC50/60*: Concrete qualities according to SIA262 or SN EN206-1.
- *SLC12/13 to SLC50/55*: Concrete qualities according to SIA262 or SN EN206-1.
- *SCX*: Concrete according to SIA262 or SN EN206-1 with user-defined properties.
- *S235, S275, S355, S450*: Construction steel according to DIN 18800 T1, Table 1.
- *S235-EN to S500-EN*: Construction steel according to EN 1993-1-1, Tab. 3.1 or EN 10025-2.
- *NH1, NH2, NH3*: Softwood grade I, II and III according to DIN 1052:1988.
- *BSH1, BSH2*: Glued laminated lumber grade I and II according to DIN 1052:1988.
- *C14 to C50, D18 to D80*: Solid timber according to EN 338.
- *GL20h to GL32h, GL20c to GL32c*: Glued laminated timber according to EN 14080.
- *Frei*: User-defined material.
- *Beton*: User-defined concrete.
- *Stahl*: User-defined steel.

**New**

Create a new section.

**Copy**

Copy the current section.

**Delete**

Delete the current section.

**Label**

The section can be labeled with any text.

Depending on the section and material type, further dialogs are available:

- *Form*
- *Shear stresses*
- *Material*
- *Bedding*
- *Reinforcing steel*
- *DIN 1045-1*
- *EN 1992-1-1*
- *BS EN 1992-1-1*
- *DIN EN 1992-1-1*
- *OENORM EN 1992-1-1*
- *SS EN 1992-1-1*
- *OENORM B 4700*
- *SIA 262*
- *DIN TR 102*
- *EN 1992-2*
- *DIN EN 1992-2*
- *OENORM EN 1992-2*
- *Recalculation Guideline*
- *EN 1995-1-1*
- *Thermal analysis*

**Area Section****Height dz**

Section height of the area elements [m].

**Orthotropy dzy/dz**

The orthotropy factor is used to reduce the element thickness for the calculation of the stiffness in the local y direction of the element.

**Torsion-free**

If selected, the torsion stiffness of the element stiffness matrix is set to zero. In this case, the mixed-term members are ignored in the differential equation of the slab proportion:

$$\frac{\partial^2 m_{xy}}{\partial x \partial y} = 0$$

The torsion moments  $m_{xy}$ , however, only reach zero if the internal force system and the element system relevant to the stiffness are identical.

**Slab rigidity factor**

The normal slab rigidity is multiplied by this value (applies for slab and shell elements).

**Polygon Section****A**

Display of the calculated section area [m<sup>2</sup>].

**I<sub>x</sub>**

Torsion moment of inertia [m<sup>4</sup>].

As regards the application in solid construction, a very small stiffness is suggested by the program. For dynamic calculations, if the torsion stiffness is defined as too small, this will generally lead to unwanted eigenmodes (torsion vibrations), so a realistic value should be used.

**Compute I<sub>x</sub>**

Calculate the torsion moment of inertia of the polygon. The result is entered into the text field for  $I_x$ .

**Factor for I<sub>x</sub>**

The torsion moment of inertia  $I_x$  is multiplied by this factor.

**I<sub>y</sub>, I<sub>z</sub>, I<sub>yz</sub>**

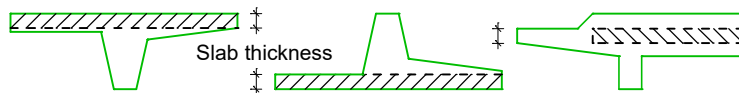
Display the calculated moments of inertia [m<sup>4</sup>].

**Ignore I<sub>yz</sub> in beam stiffness**

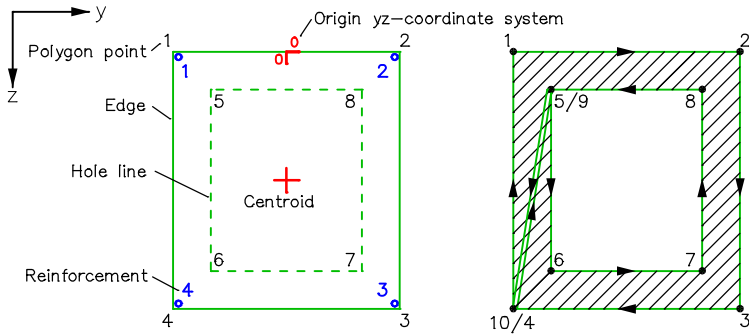
In the static and dynamic analysis  $I_{yz}$  is ignored.

**Reduce dead load**

Use this option to subtract the slabs' share from the dead load. The effective width is determined from the extremal dimensions at the top and bottom side in the horizontal direction (y axis). The slab thickness results from the smaller height when comparing the left and right edge of the polygon. This option is only allowed for section polygons in the form of a slab or edge beam with horizontal top or bottom sides. In a dynamic analysis the net mass is reduced by a corresponding factor.

**Edit...**

The graphical editing of the section geometry is carried out in a separate window. The section is defined with edge and hole bordering lines.



Internal storage takes place using a polygon that is calculated based on the by-pass method. The number of polygon points is limited to 999.

The edge and hole bordering points are automatically numbered such that the leftmost point is assigned number 1. If there are several eligible points, the one with the smallest z value is chosen.

After the polygon points have been entered in the database, the order of the points that has been determined there is also valid for the graphical representation for as long as the polygon remains unmodified.

**Reinforcem...**

Properties of each reinforcing steel layer of the selected sections.

**Cross-section class**

For polygonal sections and sections from the user database made of construction steel according to EN 10025-2 or the material type *Stahl*, the classification into the cross-section class according to EN 1993-1-1, Table 5.2, is carried out by the user.

**File Menu of the Editing Window**

**New**

Start a new section polygon.

**Insert**

Insert a section polygon, steel structure or database profile in the current polygon.

**Store in database...**

Save the current section polygon in the user database *Anwend.pro*.

**Import**

Import a drawing of a project file or a DXF file.

**Export**

Export the section polygon as a DXF file.

**Polygon Menu of the Editing Window**

Some of the following functions are also available for composite sections.

 **Line**

Enter an individual edge or hole bordering line.

 **Circle**  **Tube**  **Rectangle**  **Downstand Beam**  **Upstand Beam**

These section types are created automatically after the relevant parameters have been provided. Optionally, reinforcing steel can be simultaneously defined at the specified edge distance.

 **Reinforcing steel**

The reinforcing steel for the bending design can be freely positioned in the section. For your convenience, a guide line with the specified edge distance is displayed. The displayed reinforcing steel number helps to identify the steel layers during the results output. The steel properties (e.g. yield strength  $f_{yk}$ , base reinforcement) are preset. For marked reinforcing steel, properties can be edited in the context menu.

 **Label**

Switch the polygon's label on or off.

 **Refresh**

Refresh the polygon's displayed label.

 **Settings**

Set the type and scope of label, as well as edge distance for the input of the reinforcing steel.

**Take over**  **edge** /  **hole**

Adopt the edge and/or hole bordering lines from other lines, arcs and circles, for example, after the DXF import of a section drawing.

 **Section properties**

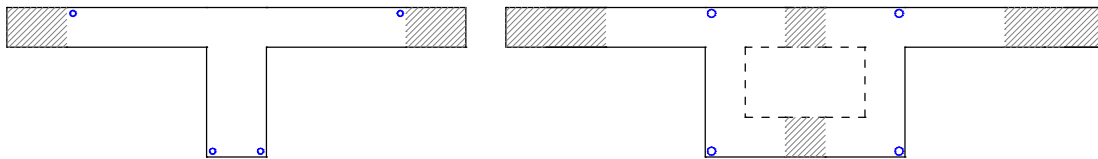
Show detailed section properties (e.g. shear center, resistance momenta, unit warping  $\omega$ ).

**Effective width**

Definition of the effective flange width by selection of the areas outside of the effective width. For this purpose the section is automatically divided into triangles. The selected areas are not taken into account for the calculation of the centroid coordinates and the bending moments of inertia.

For reinforced concrete sections the following procedure is recommended:

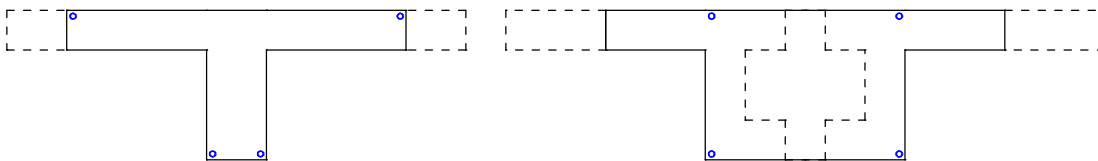
- Input of the full polygon with polygon points at the boundaries of the effective width
- Selection of the areas outside of the effective width
- Positioning of the concrete steel layers within the effective width



*Hatched representation of the section parts outside of the effective width*

The check in the ultimate and fatigue limit states are carried out with the cross-section areas within the effective width. For the crack and stress checks in the SLS, the normal forces are related to the total cross-section (Rossner/Graubner (2012), p. 228). In a *nonlinear system analysis* the entire concrete cross-section is used.

Alternatively the section polygon can be defined reducing the section dimensions based on the effective width. In the bridge checks according to DIN TR 102 and DIN EN 1992-2 the effective area of the full section can optionally be defined for the longitudinal force in the SLS checks. For the check of the principal stresses or shear stresses on box girders the first method is to be preferred, because in this case the shear flow is not discontinuous.



*Section polygons reduced to the effective width*

**Reinforcement Properties**
**Yield strength  $f_{yk}$** 

Yield strength of the reinforcing steel [MN/m<sup>2</sup>].

**Base reinforcement**

The base reinforcement is the initial reinforcement for the design. It will never be reduced by the program. By entering different base reinforcements, a relationship can be established among the steel sections to be calculated.

**Fix base reinforcement**

A fixed steel layer are not increased during the design.

**Zv0**

This force is used to take into account an initial strain  $\varepsilon_0 = Z_{v0} / (E \cdot A)$  for calculation of the reinforcing steel stresses.

**Rolling-kind**

In the nonlinear structural analysis for fire scenarios the parameters of the stress-strain curves of the reinforcing steel according to EN 1992-1-1, Figure 3.3 and Table 3.2a depend on the temperature and the rolling-kind:

- *Cold-formed*
- *Hot-rolled*

**Rectangle Section**

For this section type no reinforced concrete check can be carried out.

**Width dy, Height dz**

Expansion of the rectangle in the local y or z direction of the beams [m].

**Beam Section**

No checks can be performed for this section type.

**A**

Section area [m<sup>2</sup>].

**Ix, Iy, Iz, Iyz**

Moments of inertia [m<sup>4</sup>].

**Longitudinal bearing**

- *Standard*: The beams transmit compressive and tensile forces.
- *Only pressure*: The beams only transmit compressive forces.
- *Only tension*: The beams only transmit tensile forces.

The use of compressive or tensile beams leads to a nonlinear calculation. In this case the principle of superposition no longer applies.

**Tension Member**

This section type describes elements without compressive strength and should only be assigned to truss or cable elements. The use of this section type leads to a nonlinear calculation. In this case the superposition principle is no longer valid. No checks can be performed for tension members.

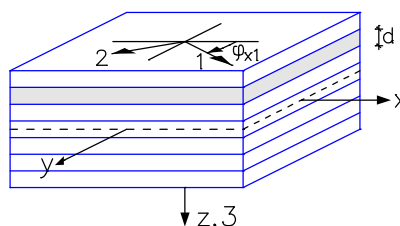
**A**

Section area [m<sup>2</sup>].




**Laminate**

This section type can be used for shell elements. It allows the calculation of shell structures made of laminates with their stresses. A laminate section has multiple layers, which are listed from top to bottom, each with the following material properties:

- E-Modulus E1 [MN/m<sup>2</sup>]
- E-Modulus E2 [MN/m<sup>2</sup>]
- G-Modulus G12 [MN/m<sup>2</sup>]
- G-Modulus G13 [MN/m<sup>2</sup>]
- G-Modulus G23 [MN/m<sup>2</sup>]
- Poisson's ratio mue21
- Angle phi.x1 [°]
- Thermal expansion coefficient alpha.t [1/K]
- Gamma [kN/m<sup>3</sup>]

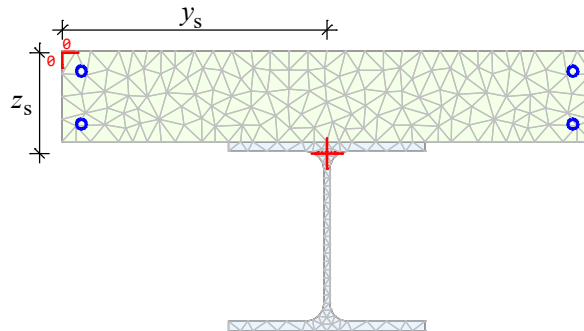


Laminate with layers, element coordinate system (x, y, z) and coordinate system of one laminate layer

If laminate layers are selected, the user-defined material properties can be edited and saved to the system file (*lgraph.dat*) using the button . With the buttons  or  laminate layers can be added or deleted. If laminate layers are selected, new layers are inserted above, otherwise at the end of the list. Optionally, the calculation can be performed without shear bond between the laminate layers.

## Composite Section

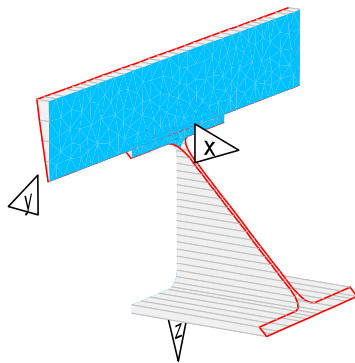
This type of section is available within the finite element analysis for beam elements and design objects. The connected composite section is divided into section parts, which can have different material properties. In the section dialog, a composite section is composed of already defined sections (menu: *Section / Insert*).



After correct definition, an automatic mesh generation is available for this purpose, which is required for calculating the section properties.

The beam axis is located in the center of gravity of the ideal section.

The section analysis includes the determination of the unit warping from shear force and torsion according to the finite element method. This yields the ideal torsional moment of inertia  $I_x$  and the elastic stresses from the internal forces of the section.



After an elastic structure calculation, the stress curve from load and residual stress states can be displayed at any point of the structure using the context menu (Section stress).

To display superposed residual stress states (e.g. from temperature or creep), result markers for stresses must be used. Stress determination is carried out independently of standards.

This applies in particular to stress states from creep and shrinkage.

The section values are **ideal** values and refer to the smallest E-modulus ( $E_{\min}$ ) or G-modulus of the composite section. The section values  $A$ ,  $I_y$ ,  $I_z$ ,  $I_{yz}$  as well as the ideal center of gravity coordinates  $y_s$  and  $z_s$  are calculated using the following formulas:

$A = 1/E_{\min} \cdot \sum E_i \cdot A_i$	Composite section area
$y_s = 1/(A \cdot E_{\min}) \cdot \sum E_i \cdot y_{s,i} \cdot A_i$	Y ordinate of the centroid
$z_s = 1/(A \cdot E_{\min}) \cdot \sum E_i \cdot z_{s,i} \cdot A_i$	Z ordinate of the centroid
$I_y = 1/E_{\min} \cdot \sum E_i \cdot [I_{y,i} + (z_s - z_{s,i})^2 \cdot A_i]$	Second-degree area moment around y
$I_z = 1/E_{\min} \cdot \sum E_i \cdot [I_{z,i} + (y_s - y_{s,i})^2 \cdot A_i]$	Second-degree area moment around z
$I_{yz} = 1/E_{\min} \cdot \sum E_i \cdot [I_{yz,i} + (y_s - y_{s,i}) \cdot (z_s - z_{s,i}) \cdot A_i]$	Deviation moment of inertia

Within a **nonlinear system analysis**, the composite section can also be used. An existing concrete or prestressing steel reinforcement is taken into account here. The strengths and the stress-strain relationships of the partial sections are taken from the respective material standards. Here always the entire cross-section is taken into account. However, nonlinear shear stress distributions cannot be determined.

The corresponding section stresses are stored at the locations of the result markers.

Furthermore, the composite section can also be used for single design.

**I<sub>x</sub>**  
 Torsion moment of inertia [m<sup>4</sup>].  
 As regards the application in solid construction, a very small stiffness is suggested by the program. For dynamic calculations, if the torsion stiffness is defined as too small, this will generally lead to unwanted eigenmodes (torsional vibrations), so a realistic value should be used (*Compute I<sub>x</sub>*).

**Compute I<sub>x</sub>**  
 Calculate the torsion moment of inertia of the composite section. The result is entered into the text box for I<sub>x</sub>.

**Ignore I<sub>yz</sub> in member stiffness**  
 In the static and dynamic analysis I<sub>yz</sub> is ignored.

**Edit...**  
 The graphical editing of the section geometry is done in a separate window.

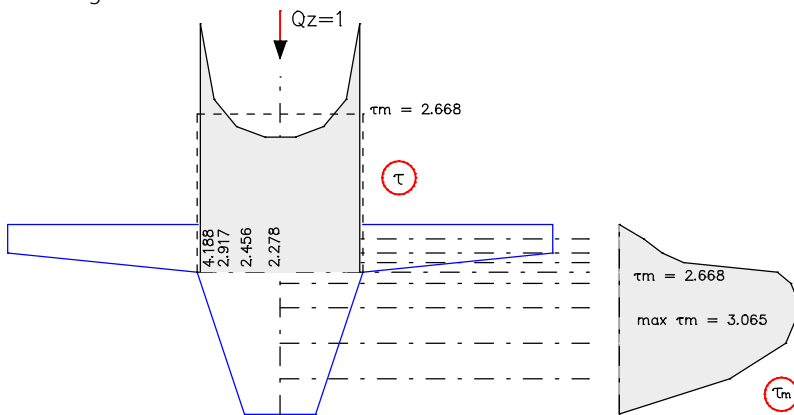
For the handling of composite sections, the **examples of Two-span steel composite girder** and **Composite column** may be helpful.

### Shear Stresses

By default the linear-elastic shear stresses are calculated according to the theory of thick-walled profiles. As a result, in the fillet area of rolled sections or in blunt corners of welded profiles larger stresses occur than those according to the theory of thin-walled profiles.

Specifying the maximum sheet thickness (open profiles:  $\tau_{\max} = M_x / I_x \cdot t_{\max}$ ) or the torsion resistance moment (box profiles:  $\tau_{\max} = M_x / W_t$ ) allows you to limit the torsion shear stresses to the maximum value according to the theory of thin-walled profiles.

An average across the section width can be selected for shear stresses from lateral force.



*Intersections and shear stress distribution from lateral force at the section polygon according to the theory of thick-walled profiles ( $\tau$ ) and after averaging ( $\tau_m$ )*



## Material

### Properties for all Materials

#### E-Modulus

Modulus of elasticity [MN/m<sup>2</sup>].

Note: The modulus of elasticity is part of the stress-strain function according to EN 1992-1-1, Equation (3.14), and SIA 262, Equation (28). The function has a pole that can move into the decisive strain range, if the given modulus of elasticity is reduced. As a consequence, results of the nonlinear analysis and the reinforced concrete and prestressed concrete checks can be affected.

#### Poisson's ratio

Poisson's ratio  $\nu$  for area and solid elements.

#### G-Modulus

Shear modulus for beams [MN/m<sup>2</sup>]. For area and solid elements, the G-modulus is given by  $G = E / [2 \cdot (1 + \nu)]$ .

#### Gamma

Specific weight of the material [kN/m<sup>3</sup>]. This is used for the *Dead load* load type as well as for determination of the mass matrix during dynamic structure analysis.

#### alpha.t

Coefficient of thermal expansion [1/K].

#### Material linear

With this switch linear elastic material behavior is assumed for this cross-section during the nonlinear structural analysis. In general, a nonlinear analysis can only be performed for area, solid, polygon, database and steel sections. For all other section types and for the material types *Beton* and *Timber* a linear material behavior is always assumed.

### Frei (General)

For the material type *Frei* (free) the corresponding strength limits must additionally be specified for a nonlinear system analysis.

#### Compression

Yield strength for compressive stresses  $f_{y,compression}$  (positive value) [MN/m<sup>2</sup>].

#### Tension strength

Yield strength for tensile stresses  $f_{y,tension}$  (positive value) [MN/m<sup>2</sup>].

#### Material model, Yield criterion

For solid sections of the material type *Frei*, different material models (damage, plasticity or combined model) can be chosen. For area sections the *Raghava* or the *Rankine* yield criterion can be chosen.

### Stahl (General)

#### fyk

Characteristic yield strength [MN/mm<sup>2</sup>].

### Timber EN 338, EN 14080

#### Apparent density

Apparent density of timber [kg/m<sup>3</sup>].

#### Service Class

Service class for timber checks.

**CX (DIN 1045-1), CX-EN-D (DIN EN 1992-1-1), CX-EN (EN 1992-1-1)**

For the user-defined concrete material types *CX*, *CX-EN-D* and *CX-EN* additional concrete parameters according to DIN 1045-1, Table 9 or EN 1992-1-1, Table 3.1 are defined in addition to the  $\alpha$  values. Non-editable values are determined by linear interpolation with  $f_{ck}$  in the corresponding table.

**fck**

Characteristic value of compressive cylinder strength of concrete at 28 days [MN/m<sup>2</sup>].

**alpha or alpha.cc**

Coefficient which takes into account the long term effects on the compressive strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

**fctm**

Mean value of axial tensile strength of concrete [MN/m<sup>2</sup>].

**alpha.ct**

Coefficient which takes into account the long term effects on the tensile strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

**Lightweight Concrete****Apparent density**

Apparent density of lightweight concrete [kg/m<sup>3</sup>].

**Concrete SIA 262****ke for E-Mod.**

Factor for calculation of the modulus of elasticity according to SIA 262, Section 3.1.2.3.3.

For the material type *SCX* the concrete properties are defined additionally according to SIA 262, Table 3.

**fck**

Characteristic cylinder compressive strength [MN/m<sup>2</sup>].

**fck,cube**

Characteristic cube compressive strength [MN/m<sup>2</sup>].

**fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

**ÖNBeton (OENorm B 4700)**

For this material type the concrete properties are defined additionally according to OENORM B 4700, Table 4.

**fcwk**

Characteristic cube compressive strength, nominal strength of the strength class [MN/m<sup>2</sup>].

**fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

**fctk**

Characteristic concrete tensile strength [MN/m<sup>2</sup>].

**taud**

Calculation value of shear stress [MN/m<sup>2</sup>].

## Creep Coefficients

The time-dependent material characteristics for the creep and shrinkage behavior enter into the calculation of the *Creep and shrinkage (Ks)* load case. The resulting concrete strains are calculated with the displayed modulus of elasticity.

### Creep coefficient phi (t,t0)

Creep coefficient  $\varphi(t, t_0)$

### Relaxation coefficient rho

Relaxation coefficient according to Trost (usually  $\rho=0.80$ ).

### Shrinkage coefficient epsilon cs(t,ts)\*1.e5.

The shrinkage coefficient  $\varepsilon_{cs}(t, t_0)$  multiplied with  $1 \cdot 10^5$ . A negative value must be entered for a contraction.

### Compute coefficients

The button causes the coefficients  $\varphi(t, t_0)$  and  $\varepsilon_{cs}(t, t_0)$  to be calculated from  $t_0$ ,  $t_s$ ,  $t$ , *Cement curing*, *Air humidity RH*, *Effective height  $h_0$*  and optionally  $\gamma_{lt}$  according to the standard apposite to the selected material type. The creep coefficient  $\varphi(t, t_0)$  is calculated for DIN 1045-1 according to Book 525, Eq. (H.9-6), for OENORM B 4750 acc. to Eq. (7) and for EN 1992-1-1 acc. to Annex B, Eq. (B.1). The total shrinkage strain  $\varepsilon_{cs}(t, t_0)$  is calculated for DIN 1045-1 according to Book 525, Eq. (H.9-20), for OENORM B 4750 acc. to Eq. (10) and for EN 1992-1-1 acc. to Eq. (3.8). EN 1992-2, Chapter B.103, for high strength concretes  $\geq C55/67$  is not taken into account.

### Load start t0

Age of concrete at the time of loading in days.

### Drying out start ts

Age of concrete at the beginning of drying shrinkage in days.

### Concrete age at date t

Age of concrete in days at the moment considered.

### Cement curing

The speed of cement curing for assignment to the apposite cement class.

### Air humidity RH

Relative humidity of the ambient environment in %.

### Effective thickness h0

Effective section thickness with  $h_0 = 2 \cdot A_c / u$  in m, where  $A_c$  is the cross-sectional area and  $u$  is the perimeter of the member in contact with the atmosphere.

### Factor gamma.lt

Safety factor  $\gamma_{lt}$  for the long-term extrapolation of the delayed strains according to EN 1992-2, Chapter B.105. This factor shall be specified by the user and is taken into account for concrete material according to EN 1992-1-1 by multiplying with the creep coefficient and the dry shrinkage. For concrete material according to the national annex Germany, it is only taken into account in dry shrinkage.

### Note:

The mentioned formulas from the standards cannot be used to determine the creep coefficient for any given period from  $t_1$  to  $t_2$  after the time of loading. Approximately this value can be calculated in the following way:

$$\varphi(t_2, t_1) = \varphi(t_2, t_0) - \varphi(t_1, t_0)$$

This procedure can analogously be used for the shrink coefficient.

## Bedding

### Bedding modulus x, y, z

Bedding moduli for the element directions [MN/m<sup>3</sup>]. The bedding moduli of beams can be linearly variable between the beginning (a) and the end (e) of the element.

### Bedding width

For beam sections, each bedding modulus must have the bedded width [m] specified.

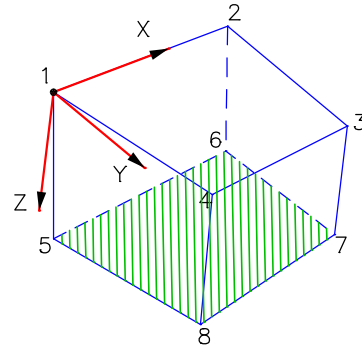
### Compressive or tensile strength (nonlinear system analysis)

The strengths [MN/m<sup>2</sup>] defined here are only valid for nonlinear system analysis. This can be used to realize a bilinear bedding characteristic in the compressive and tensile section of the elements (see also *Foundation models*). For beams the strengths can be linearly variable between the beginning (a) and the end (e) of the element.

Note:

For solid elements *VQ83*, the bedded surface is defined by the nodes 5 to 8.

For tetrahedron elements *VT10* an explicit selection of the bedded surfaces is necessary. The local direction for the bedding modulus z is always perpendicular to the selected surface. The bedding direction x of the surface 1 of the tetrahedron points from node 1 to node 2. It runs parallel to the to the surface 1 for the other three surfaces. The user cannot modify the directions for tetrahedron elements and is urgently recommend to choose the same value for the bedding modulus in x and y direction.



## Reinforcing Steel of an Area Section

### Selection of layers

Select steel layers by clicking them in the graphic.

### New layer

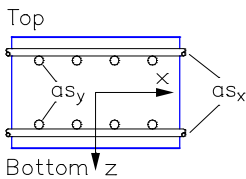
Add a new steel layer.

### Delete layer

Delete selected steel layer.

### Edge distance in x or y direction

Axis distance of the steel layer to the section edge in the x or y direction [m].



### Base reinforcement

Minimum section of the reinforcement for the bending design. This can also be used to establish a relationship between the steel sections to be determined [cm<sup>2</sup>/m].

### Fix base reinforcement

The base reinforcement is not changed during the design.

### Quality

- 420S: Reinforcing rod with  $f_{yk} = 420$  MN/m<sup>2</sup>.
- 500A: Reinforcing rod with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- 500M: Reinforcing meshes with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- *General information:* Freely definable steel quality [MN/m<sup>2</sup>].

### Yield strength

Yield strength  $f_{yk}$  [MN/m<sup>2</sup>].

### Rolling-kind

In the nonlinear structural analysis for fire scenarios the parameters of the stress-strain curves of the reinforcing steel according to EN 1992-1-1, Figure 3.3 and Table 3.2a depend on the temperature and the rolling-kind:

- *Cold-formed*
- *Hot-rolled*

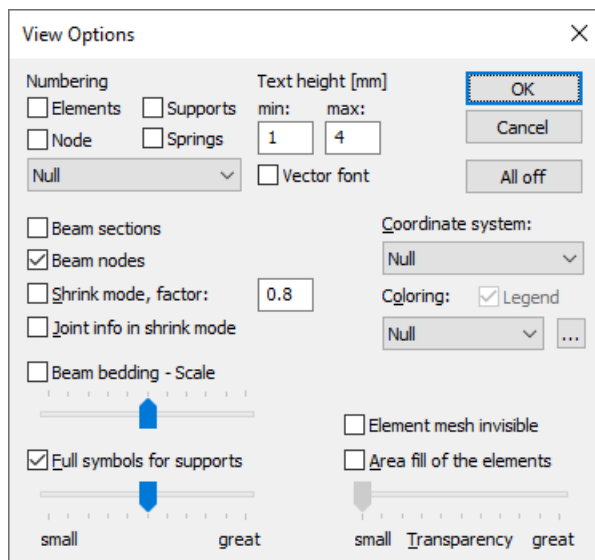
## Element Info

This function shows the following properties of an element graphically at the element and in the status bar:

- Element number
- Element type
- Node numbers
- Element system
- Section
- Material
- Joints

## View

This menu item is used to display and check the structure properties.



### Numbering

You can label elements, nodes, supports and springs with their numbers. Material or section numbers can also be added to elements. The display of the numbers can be limited to values > 1. Alternatively, the section name can be added to beam sections.

### Text height

Set minimum and maximum text heights [mm].

### Beam section

The beam section is drawn at the middle of the beam.

### Beam nodes

The beam nodes are marked by a symbol.

**Shrink mode**

Enable or disable shrunken element representation. Most of the input functions are inactive in this mode.

**Shrink factor**

Define the shrink factor for the shrink mode.

**Joint info in shrink mode**

For beams existing beam end joints are labeled in the shrink mode. Joint springs are marked with an asterisk '\*'.

**Beam bedding**

The bedding moduli are plotted along the beam axes with the specified scale.

**Element mesh invisible**

If the surfaces of the area and solid elements are colored (e.g. filling of element surfaces, section representation), no element edges are displayed.

**Area fill of the elements**

The surfaces of the area and solid elements are filled with the selected color. If transparency is deactivated (*small*), the element edges are displayed in black.

**Transparency**

The degree of transparency of area and solid elements can be adjusted.

**Full symbols for supports**


The properties of the supports are shown in detail. The scroll bar changes the size of the icons in use.

Icons	$u_x$	$u_y$	$u_z$	$\varphi_x$	$\varphi_y$	$\varphi_z$
	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	Free	Free	Free
	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>
	Free	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	Free	<b>Fixed</b>
	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	Free	<b>Fixed</b>	<b>Fixed</b>
	Free	Free	<b>Fixed</b>	Free	Free	<b>Fixed</b>
	<b>Fixed</b>	<b>Fixed</b>	Free	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>

**Coordinate system**

Turn on coordinate system for reinforcement, elements, supports or internal forces. When displaying support reactions in the form of *support lines*, their coordinate system is displayed.

**Coloring**

The objects are drawn in different colors, depending on the selected property (e.g. *material*, *section*). If necessary, the color assignment can be modified with the  button. The possible objects that are distinguished by color include elements, design objects and model objects. The properties *As top*, *bottom* and *asx,y 1./2. layer* refer to the base reinforcement of the cross-section. Color 254 is used for *Undefined*.

**Legend**

Enable/disable the color legend.

**All off**

Turn off all display options.

## Mesh Check

The element mesh can be checked with this menu item.

### Node distance

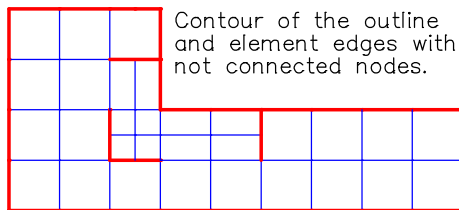
The function checks if there are further nodes in the neighborhood of a node within the given tolerance. After confirmation, the so-called duplicate nodes are marked by red arrows and you are queried if you want to merge them. In this case their coordinates are averaged. Nodes that are not merged should have a distance of at least 1 mm for numerical reasons.

### Element distance

Checks if any elements lie on one another, for instance, due to redundant element entry. The elements that are found are marked by red arrows.

### Contour

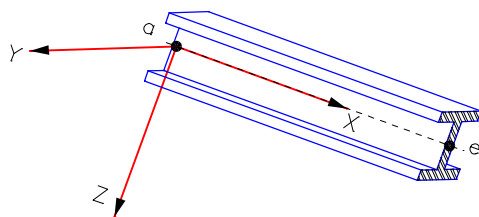
All free element edges are highlighted red. This helps you spot non-cohesive mesh areas (slits, for example). The contour can be changed into a continuous line, if desired.



## Element System

The local coordinate systems of selected elements can be changed using this menu item.

### Beams



+x, -x	By exchanging the start and end nodes of the beam, the positive or negative x axis points to the specified point.
+z, -z	The spatial orientation of the section is defined by a point in the local xz plane. The point defines the direction of the positive or negative z axis. As an option, the direction can also be reset to the default orientation.
y, z	The orientation of the selected axis is defined by specifying two points.
Angle	The beam is rotated about the beam axis (local x axis).

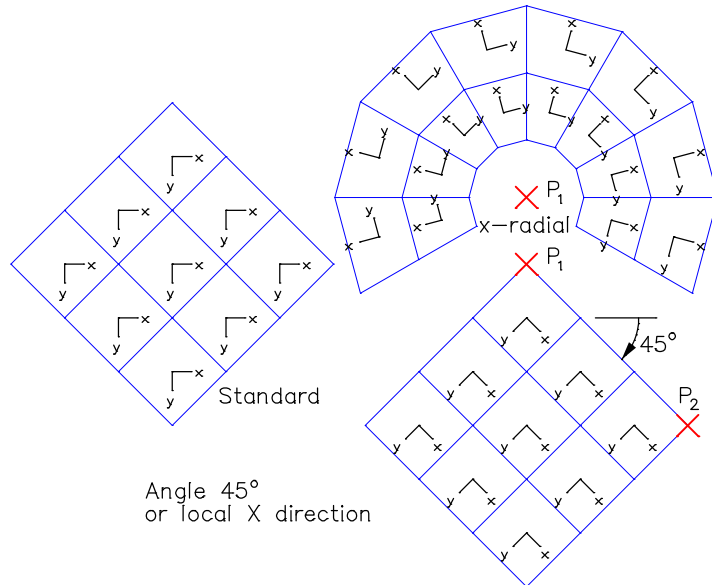
### Area or solid elements

<i>Elements</i>	Rotation of the element system for area and solid elements. As regards shell elements, the direction of the local z axis can also be reversed. A change of the element system affects the impact of orthotropic material properties, elastic bedding and local element loads.
<i>Internal forces</i>	Change the internal force system for area and solid elements. The change of the internal force orientation only takes effect after recalculating the load cases.
<i>Reinforcement</i>	Angular deviation of the reinforcement assembly $\alpha_{sy}$ from the $90^\circ$ direction for area elements. The x direction of the reinforcement system is always identical to the x direction of the internal force system. The specification of an angular deviation is only necessary for inclined reinforcements. The change of the reinforcement direction only takes effect after recalculation.

Options for modification of the coordinate systems:

x, y	In 2D mode, two points are entered and projected perpendicular to the image plane into the element plane to determine the direction of the selected axis. In 3D mode, the projection direction can either be indicated by a third point or can be defined in the direction of the <i>element normals</i> .
x- or y-radial	Specification of a middle point for radial orientation (for circular slabs, for example).

- x and y, x and z* The x direction and the xy plane or xz plane of the internal force system are defined by specifying three points.
- Radial* Specification of a rotation axis for radial orientation (e.g. for cylindrical solids).
- Angle* The rotation angle defines the deviation from the standard orientation in degrees. The rotation occurs around the z axis of the area or solid element.
- Standard* Reset to the standard orientation.
- +z, z* The positive or negative z axis is oriented towards the indicated point.



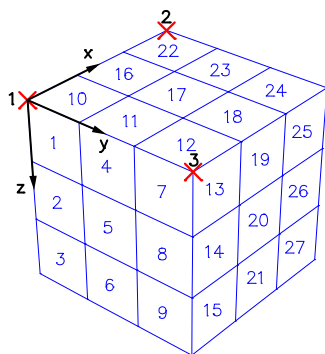
Orientation of local systems (in this case internal force systems)

## Number Sort

This menu item is used to change element and node numbers.

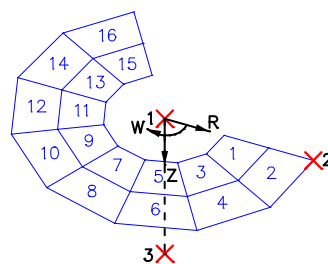
### Input

- Sort* Sort node or element numbers geometrically.
- Define* Define new start values and increments for nodes and element numbers.
- Edit* Edit node and element numbers.



Sorting of the element numbers in the perpendicular coordinate system:

1. Sorting middle point
  2. Direction of sorting, local x axis
  3. Direction of sorting, Point in the local xy plane
- Order: X-Y-Z



Sorting of the element numbers in the polar coordinate system:

1. Sorting middle point
  2. Direction of sorting, local x axis
  3. Direction of sorting, Point in the local xz plane
- Order: Angle-Z-Radius



## Supports

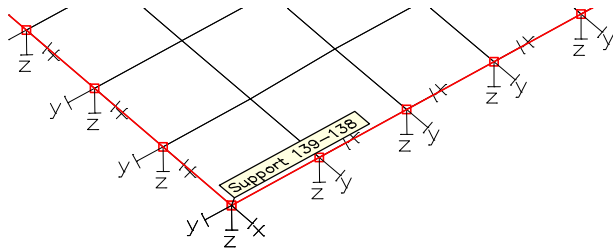
This function is used to define supports. A support describes the support conditions of a system node. The supported nodes are labeled with simple or detailed red icons (see *View*). The support system defines the directions of the degrees of freedom and can deviate from the global coordinate system.

The individual degrees of freedom can be supported in a fixed or elastic manner. Additionally, the loss of the support for tensile forces (tensile failure) for the directions  $x$ ,  $y$ , or  $z$  can be defined with the following options:

- *Load case controlled*: The tensile failure is only active in the load cases where the *No support reaction < 0* option has been chosen for the *Theory or Load group* load type.
- *Always*: The tensile failure is always active.
- *Never*: The tensile failure is never active.

### Line support

Use this button to input line supports. A line support is defined geometrically through start and end points. For all nodes that lie between these points and form element edges individual supports will be created. Already existing point supports are ignored. The local  $x$  axis of the support line points from start to end point, the local  $z$  axis normally corresponds to the global coordinate system. The orientation of the line support can be visualized by means of the coordinate systems of its supports. Please note that a support can belong to multiple line supports (on a slab corner, for example).



Line supports can only be edited as a whole with the following context menu functions:

- Properties
- Regenerate
- Break Up (conversion into point support)
- Reverse Z Direction

When calculating the structure, the properties of the individual supports that make up the line support are decisive. If necessary, the specified spring rigidities are internally converted into individual springs by multiplication with the respective bearing widths. These are provided in a table for verification purposes.

### Point support

Start the input of supports through the selection of nodes. The subsequent editing is performed using the context menu functions:

- Support
- Orientation

### Wall

Start the input of line supports through the selection of walls.

### Info












Show the support system with the bound degrees of freedom for a selected point or line support.

Note: Only point and line supports can be selected when the function is active. This improves the handling of these objects.

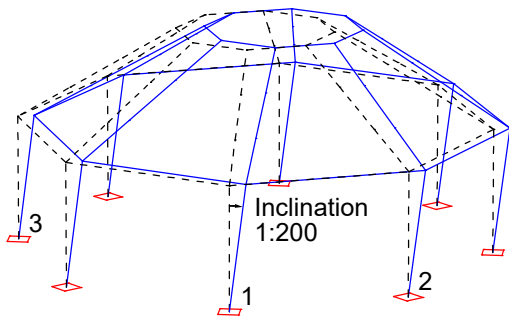
## Predeformation

This function is used to enter and edit stress-free node displacements for a structure (only for frameworks). As part of the system analysis, these node displacements are added to the initial node coordinates and may serve for example as a geometric imperfection for calculations according to the second-order theory. Therefore, the calculated nodal deformations refer to the already predeformed structure. Because of these reasons only beam and nodal loads should be used as load types for predeformed structures. To be considered for calculations, the predeformation must be activated within the *load group* definition.

The following options allow you to define predeformations or subsequently change them, if necessary.

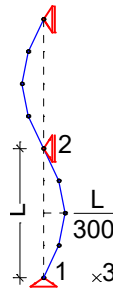
-  Create a copy of the current predeformation.
-  Add the selected predeformation to the current predeformation.
-  Start the representation options dialog.
-  End the predeformation display.
-  Create a new predeformation.
-  Display an existing predeformation.
-  Change the number of the current predeformation.
-  Delete the predeformation.
-  Edit the predeformation.
-  Start the precurvature input.
-  Start the predeformation input.

Absolute input of the predeformation refers to the initial system, relative input refers to the already deformed system.



Predeformation of a hall construction

1: 1<sup>st</sup> point of base line, 2: 2<sup>nd</sup> point of base line, 3: Point in base plane



Precurvature of a column

The handling is also shown in the video "[https://download.infograph.de/video\\_de/Vorverformung%20definieren.mp4](https://download.infograph.de/video_de/Vorverformung%20definieren.mp4)" (German language).

## Moving Nodes

This function is used to change the location of selected nodes. In the case of a single element node, simply specify the new position. If multiple element nodes are to be moved, you should choose the *Group* or *Elements* option.

### Group

Select the element nodes to be moved.

### Elements

Select the elements, whose nodes are to be moved.

### Displacement vector

Movement through displacement. The displacement vector is defined by two points.

### Rotate

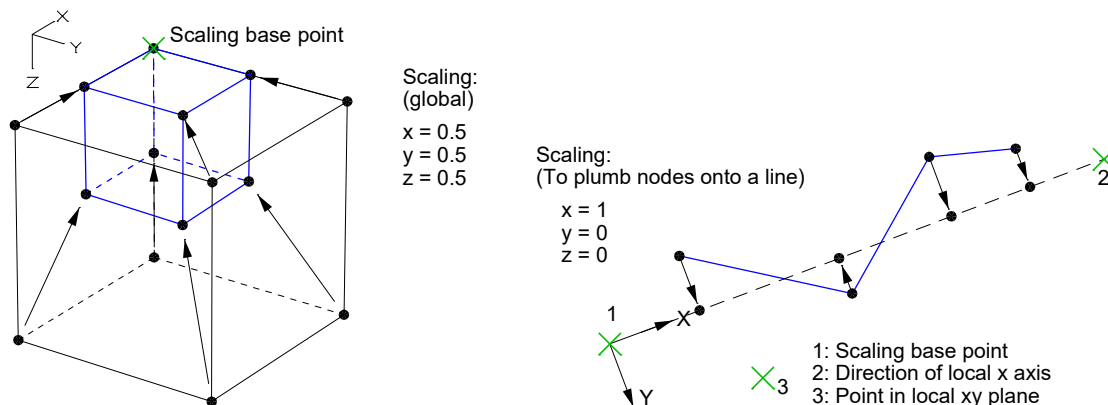
Movement through rotation. A rotation point is defined in 2D mode while a rotation axis is defined in 3D mode.

## Mirror

Movement through mirroring. A mirror axis is defined in 2D mode while a mirror plane is defined in 3D mode.

## Scaling

Movement through scaling in the global or local reference system. You can align nodes on an axis or plane by specifying a scaling factor of zero.



The program checks whether the forms of the adjacent elements remain valid after the node displacements. To avoid numerical problems, the program expects an element size of more than 1 mm. If desired, displaced nodes that coincide with non-selected nodes can be merged with these.

## Finding Nodes or Elements

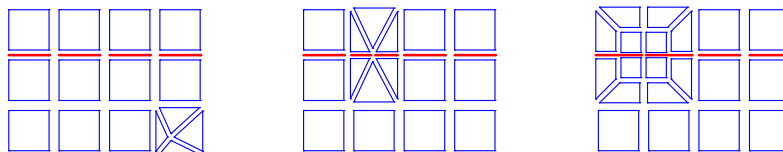
If you are looking for a node with a particular number, the functions *Select Nodes* and *Move Nodes* can be of help. Enter the node number you are looking for into this function. If you are looking for an element with a particular number, use the functions *Element Info* or *Select*. The element number can be specified in both functions. If the element is not identifiable even though it is selected, you can query further information with the *Element Properties* function. The selected objects' color and layer are displayed in the toolbar.

## Condense

The *Condense* menu item allows you to modify an existing element mesh.

### Point

The element mesh is condensed around the specified point. The type of condensation depends on whether the point is within an element, on element surfaces or edges, on beams or on existing nodes. The condensation point is given a new node, if necessary.



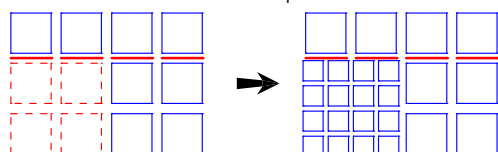
Point condensations on element meshes with beams (representation in shrink mode)

### Halve

Selected area and solid elements (VQ83) are divided into two. You will be asked to indicate the desired direction of the division.

### Quarter

Selected area elements are quartered. Selected solid elements (VQ83) will have their surfaces divided into four parts.



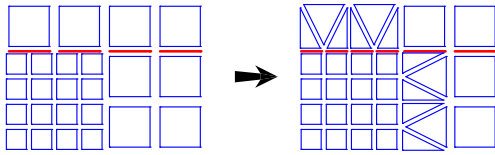
Element mesh with quartered area elements (representation in shrink mode)

**Divide**

Selected beams are divided. Enter the desired number of parts.

**Adapt**

Selected area and solid elements are divided at nodes inside or at their edges, so that the elements are connected to that node. Selected beam elements are divided at nodes on their axis or at intersections with other beam elements.



*Adapted element mesh with beams (representation in shrink mode)*

**Merge**

All selected triangular elements are merged into square elements, when possible.

**Triangulate**

All selected rectangular elements are split into two triangular elements.

**Link Elements**

Kinematic dependencies between the global degrees of freedom of two element nodes can be defined with the *Link Elements* function. This is done by expressing the nodal degrees of freedom of a dependent node through the degrees of freedom of the reference node by way of an equation of condition.

for example  $u_{z,dep.} = f_1 \cdot u_{x,ref.} + f_2 \cdot u_{y,ref.} + f_3 \cdot u_{z,ref.} + f_4 \cdot \phi_{x,ref.} + f_5 \cdot \phi_{y,ref.} + f_6 \cdot \phi_{z,ref.}$

with

$f_1, f_2, f_3$  Factors for the displacements,

$f_4, f_5, f_6$  Factors for the rotations.

You can link any number of dependent nodes to a reference node. Nodes with supports may not be defined as dependent nodes. Nodes of slab, plain and solid elements have three degrees of freedom. Nodes of shells and beams, however, have six degrees of freedom. The reference node of a link element must have all degrees of freedom that the dependent node has. The link elements are displayed with dotted lines. Additionally, an arrow points to the reference node.

The following types of link elements exist:

- *Stiff*: Stiff link element (abbreviation: SS).
- *Jointed*: One-sided jointed link element with a ball joint at the dependent node (abbreviation: GS).
- *ux, ... phi.z*: Dependent degree of freedom of the user-defined equation of condition. The factors for the displacements and rotations of the equation of condition have to be defined.

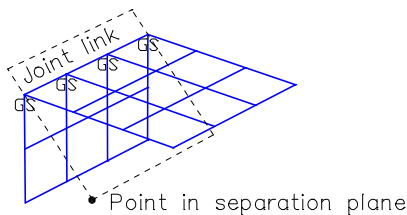
**Point link**

After selecting the dependent nodes and the reference node, you must define the link element type.

**Joint link**

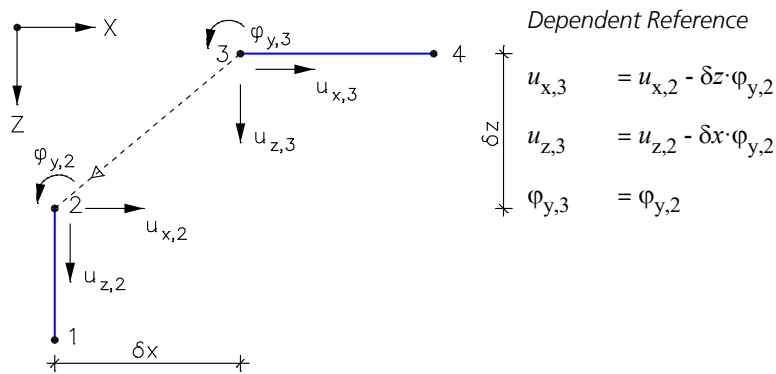
This option separates area elements along a common element edge that must be defined and creates link elements between the original (reference) and newly created (dependent) nodes.

In the 3D representation a point must be given in the separation plane next to the joint. The link elements are created between the nodes of the elements on the one side and those on the other side of the separation plane.



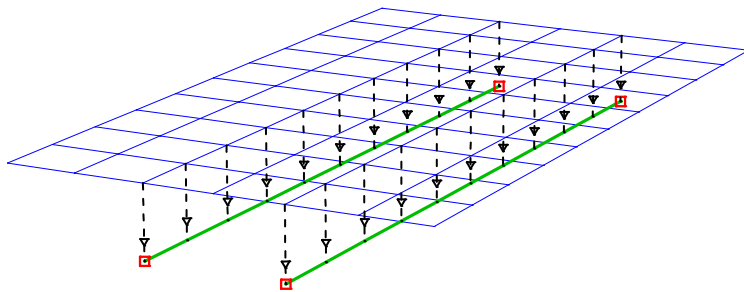
## Examples

### Equations of condition for the stiff link elements in the xz plane.

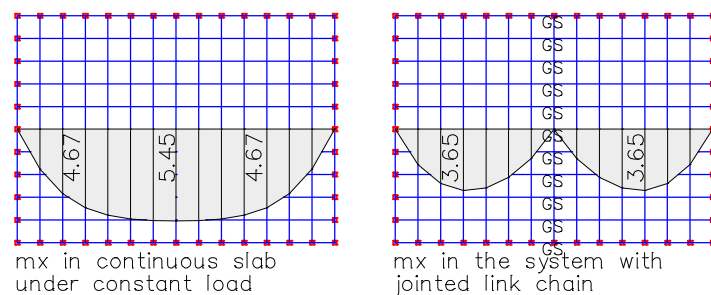
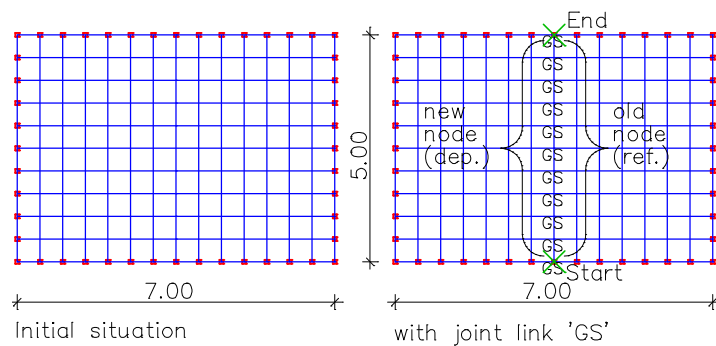


### Eccentrically linked girders

Link elements can also be used to take into account the eccentricities of girders. The following figure shows how the connection of shell elements with 3D beams can be used to model a double-webbed T-beam.



### Joint link



Modeling of a joint groove in a floor slab with the help of link elements

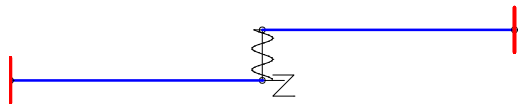
## Spring Elements

This element describes a spring between two nodes. Its x direction points from the first to the second node. The z direction is defined by the user. If both nodes are at the same location, the x direction must also be defined. The resulting Cartesian coordinate system defines the six components of the spring. A linear or nonlinear spring characteristic can be defined for each of the components. As opposed to the beams, the distance between the nodes does not affect the stiffness of the springs.

If a friction coefficient  $\mu$  is defined for the y or z direction and the friction coefficient is *inactive* for the other of the two transverse directions, a maximum friction force of  $\mu \cdot N_x$  can act in the spring for a compressive force  $N_x$ . If friction coefficients are defined for both directions, the maximum friction force for a compressive force  $N_x$  in the spring results from the yield surface of the elliptical cone, which is defined by the two friction coefficients and the compressive force. If there is a tensile force in the spring, the friction force is zero. An input of friction coefficients is only recommended in connection with a non-linear spring characteristic for contact in x direction.

Spring elements can be used in static and dynamic calculations. Hysteresis, which may occur in the dynamic analysis, is not taken into account at this time. For nonlinear spring characteristics, make sure you use a realistic load combination (see *Load group*) as a subsequent load case superposition is not allowed here. Nonlinear spring characteristic can be taken into account in dynamic analyses using the direct time step integration (see example '*Linked two-mass oscillator > linear increase with breaking spring*').

Viscous damping in the local x direction of the spring can be additionally defined for the direct time step integration.



Structure with spring element

**Spring element**

Number: 1    Label: Sample spring

Characteristics:

Comp.	Label	Linear	Stiffness	Friction
ux	Contact	No		
uy		Yes	100	Inactive
uz		Yes	100	0.2
phi.x		Yes	0	
phi.y		Yes	0	
phi.z		Yes	0	

Geometry

	x	y	z
Node 4	2.5	0	-1
Node 1	2.5	0	-1.05
x-direction	0	0	-1

Viscous damping (ux): 0 [kNs/m]

**Spring characteristic**

Values:

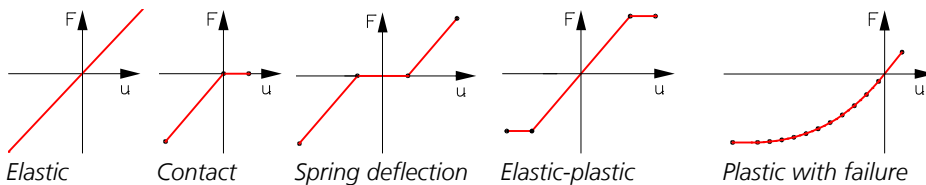
	ux [m]	Fx [MN]
1	0.1	-6
2	-0.05	-1
3	0	0
4	0.05	0
*		

Label: Contact

OK    Cancel

A nonlinear spring characteristic can consist up of horizontal and inclined sections. If a deformation results outside of the defined region of the characteristic, it is assumed that the spring has failed for this component and is thus calculated with a reaction of 0. The characteristics of a spring element can be saved in the *Igraph.dat* database for use in other structures.

A number of possible load-displacement curves are shown below.

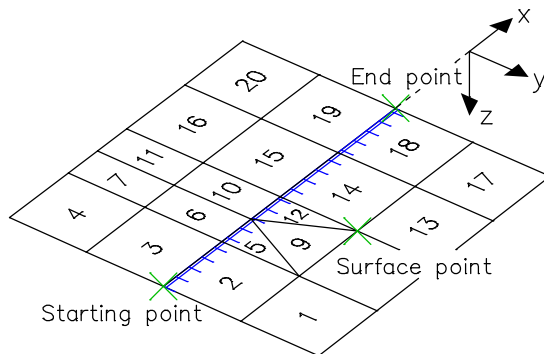


## Line Hinges

This *Line hinges* function is used to define line-shaped joints on area elements. The line hinge is geometrically delineated through its start and end point as well as a point in the plane of the affected elements. During the FEM calculation, the elements that are assigned to a line hinge are determined and internally provided with corresponding mechanisms.

It only affects elements that are in the plane defined by the three points and have nodes on the line hinge. If the line hinge intersects an element, it is not considered.

The local x axis points from its start to its end point. The local y axis corresponds to the orientation icon of the line hinge and points to the direction of the elements attached in a jointed manner. This gives the local z axis perpendicular to the xy plane.



**Line Hinge** ✕

<b>Local displacements</b>		<b>Contact / Friction</b>	
X: Hinge	...	Inactive	▼
Y: 1000 MN/m <sup>2</sup>	...	0.200	▼
Z: 250 MN/m <sup>2</sup>	...	Contact	▼
<b>Local twists</b>			
X: 10 MNm/rad/m	...	Database	
Y: Hinge	...		
Z: Fix	...		
GUID: 0pkbMHDdn855BSTQBcwJ9R		Points: 0/-1/0	
Label:		2/-1/0	
		1/-0.5/0	
		OK	
		Cancel	

The GUID can be used to search for the line hinge (see *Find GUID*).

The elements affected by a line hinge are listed in the *Finite elements* listing along with the element list.

### Line hinges

```

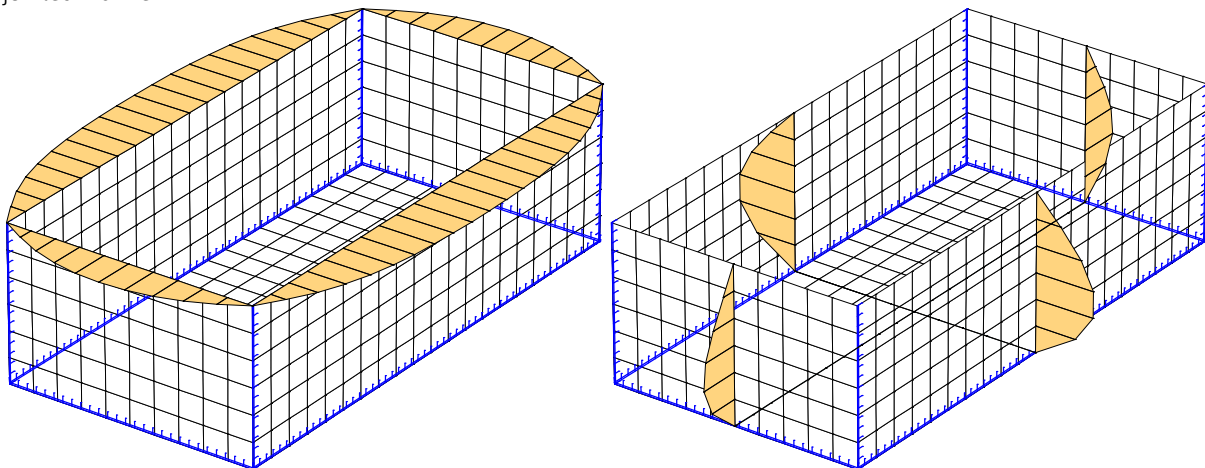
No. 1
Starting point: 0.000 / -1.000 / 0.000 [m]
End point : 2.000 / -1.000 / 0.000 [m]
Surface point : 1.000 / -0.500 / 0.000 [m]
Spring stiffness : Cu.x =1.0e-09 Cu.y =1.0e+03 Cu.z =2.5e+02 [MN/m2]
                  Cphi.x=1.0e+01 Cphi.y=1.0e-09 Cphi.z=1.0e+06 [MNm/rad/m]
Friction/contact: (ux) inactive (uy) 0.200 [-] (uz) Contact [-]
Linked elements with mechanisms
2 5 9 12 14 18

```

Instead of Fix or Hinge spring rigidities can be defined for the properties of a line hinge. They apply to the following degrees of freedom.

For the properties of a line hinge, a spring stiffness or characteristic curve can be defined separately for each degree of freedom instead of *Fix* or *Hinge*. If a spring stiffness or characteristic curve is defined for the local y or z direction of shell structures, the option *Contact* (compression only) can be selected for one of these directions. If *Contact* is selected, a friction coefficient can optionally be defined for each of the other directions if they have a spring stiffness or characteristic curve. This then defines the elliptical yield surface. The options *Contact* or *Characteristic* lead to a nonlinear calculation model and thus to an iteration in the load case calculation.

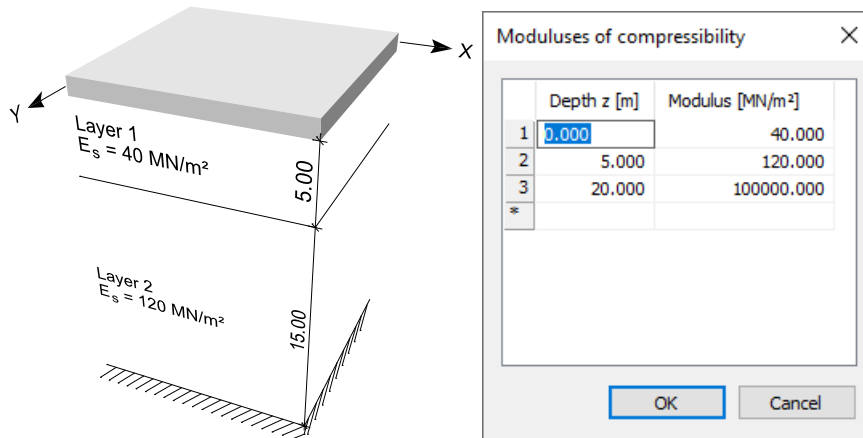
The following example shows an elastically bedded container under liquid pressure, whose corners are all attached in a jointed manner.



Internal forces  $m_x$  and  $m_y$  as a result of liquid pressure

## Moduli of Compressibility

The model of the layered elastic halfspace (modulus of compressibility method) requires the layer thicknesses as well as their moduli of compressibility. The depth  $z$  of the soil layers is counted from the foundation level (foundation slab) of the structure. The first soil layer thus begins at  $z = 0$ , the last layer reaches up to  $z = \infty$ . The ultimate depth of the halfspace can be defined with a very high modulus of compressibility in the last soil layer. The value can be entered in the Analysis settings or in the 'Structure description' folder in the database.



Example of the layering of the halfspace below a foundation slab

Requirements for the analysis according to the modulus of compressibility method:

- A calculation is only possible for area and solid elements.
- All bedded areas must have the same global  $z$  component.
- The affected elements must be assigned a material with a bedding modulus in  $z$  direction. It is needed as the start value for the calculation.
- The Poisson's ratio of the half space is zero.
- Systems in which the subsidence hollow and the deformation of the foundation body have different signs due to the load can not be calculated exactly with the iteration method used.

## Arranging Elements

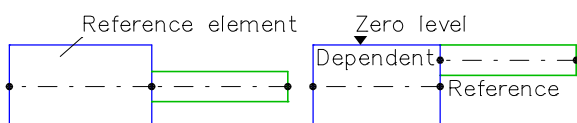
This function is used to align selected elements with a reference element. The local  $z$  axes of the elements must point in the same direction. The alignment can be accomplished in two ways:

- Use of eccentricities
- Use of link elements

The orientation is performed through a displacement in the local  $z$  direction until the zero levels correspond to those of the reference element. For area elements their top is considered the zero level. For beams the zero level is defined at  $z=0$  in the section polygon. Optionally, the level may be defined as follows:

- Beams:  $z=0$ , area el.: Top (default)
- Beams:  $z=0$ , area el.: Bottom
- Top face
- Bottom face

If eccentricities are used, the offset is realized via corresponding eccentricities in the element. This means, no new nodes are created. If link elements are used, the program possibly creates new nodes and additional stiff link elements between the nodes, because of the offset. The top node of the link element becomes the dependent node, as long as it does not have a support defined to it. Link elements that already exist are retained.



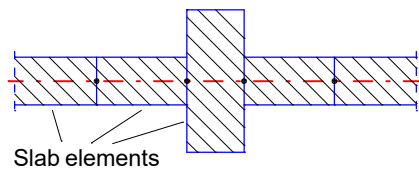
Elements before and after the alignment (default)



## Models for the Downstand Beam

There are many models for representing downstand beams. A selection of these models is introduced below to illustrate their effect on calculating a slab.

1. Modeling of the downstand beam using stronger slab elements:



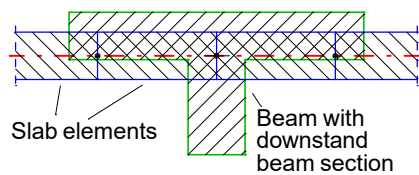
Advantages

- Simple model
- Automatic design

Disadvantages

- Eccentricity is not taken into account

2. Modeling of the downstand beam through a beam series with T-beam section:



Advantages

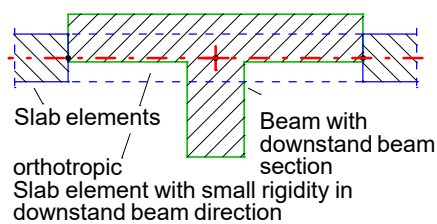
- Easy input
- Automatic design

Disadvantages

- Redundant (double) stiffness of the slab in the area of the effective width

The resulting internal forces distribute themselves according to the relationship of the stiffnesses in the slab and downstand beam. As a result, reinforcement is determined for the downstand beam as well as for the slab during the automatic design. The error is negligible for large differences in stiffness between T-beam and slab.

3. Modeling of the downstand using a beam series with T-beam section and orthotropic slab elements in the area of the effective width:



Advantages

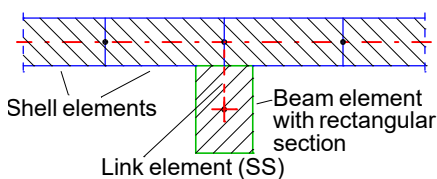
- Exact distribution of the stiffnesses and resulting internal forces
- Automatic design

Disadvantages

- Complicated modeling

When reducing the slab rigidity in the downstand beam direction (orthotropy), the internal forces, and as such the reinforcement, shift correspondingly. The load performance of the slab remains unchanged in the lateral direction.

4. Modeling of the downstand beam with shell elements and linked beams:



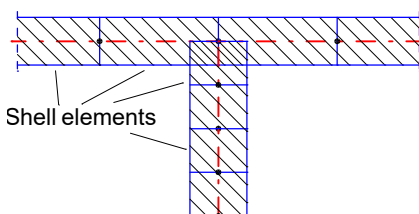
Advantages

- Effective width results automatically
- Correct static model within the scope of FEM analysis

Disadvantages

- Design of the T-beam section is only possible with the help of a design object
- Shell and link elements or eccentricities are necessary

5. Modeling of the downstand beam using shell elements:



Advantages

- Effective width results automatically
- Correct static model within the scope of FEM analysis

Disadvantages

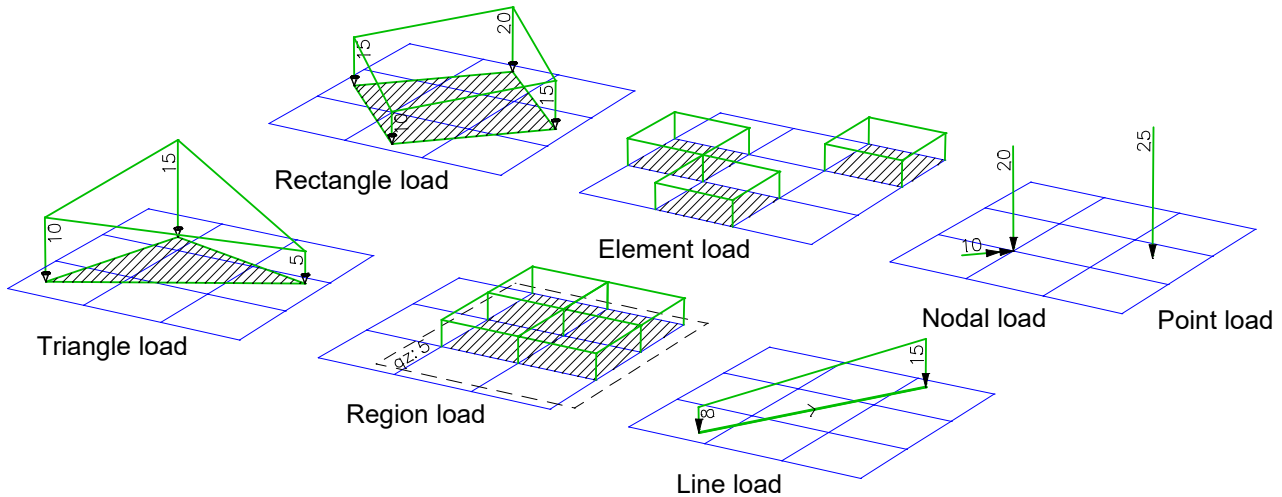
- Design of the T-beam section is only possible with the help of a design object
- Shell elements are necessary

## Load

The following chapter describes load input as well as the different load types.












The specified loads are represented with icons. In 3D mode, point, line, and area loads are scaled according to their load ordinates. For point loads the resulting load direction is drawn. The orientation of line loads and line moments is indicated by an arrow.

Beam loads generally act on the centroidal axis of the beams. For asymmetrical sections the effect of the eccentricity of the shear center on the fixed-end reactions is therefore not accounted for.



### Load types of area elements

A load case is made up of one or multiple loads. The following options allow you to define load cases or subsequently change them, if necessary.

-  Create a copy of the current load case.
-  Generate load cases according to the rules set out by the *Generate* function. This leads to the duplication of either all or only the selected loads. With the load type *Dynamic train load*, the driving speed can be increased.
-  Import load data, support reactions, static pseudo loads of an earthquake calculation based on the response spectrum method or static pseudo loads for a nonlinear pushover analysis. Optionally, all load data existing in the current load case can be removed. When importing the support reactions, only the selected global components of the support forces and no moments are imported. For support reactions from combinations, the local reaction  $max R_z$  is imported with its associated components  $R_x$  and  $R_y$ . For line supports, their support reactions can be imported as point loads or as averaged constant line or trapezoidal loads.
-  Start the representation options dialog.
-  End the load representation.
-  Create a new load case.
-  Open the previous load case.
-  Open the next load case.
-  Show an existing load case.
-  Change the number and name of the current load case.
-  Start the load input.

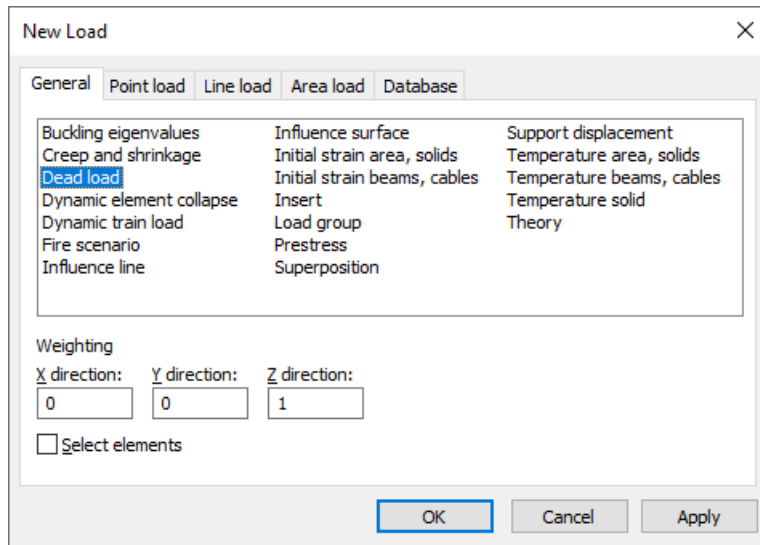
Load cases can be renumbered in groups within the database tree.

Double-clicking on an existing load object or the context function *Load* opens a dialog in which changes to the selected load objects can be made. With the context function *Scale load* the load values of multiple selected loads can be scaled with a factor.

When entering free point and line loads, the load directions of slab and plain structures are restricted to those directions in which the existing area elements can take loads. All load directions can be allowed for loading beam elements.

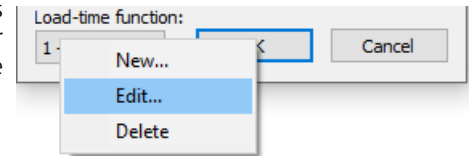
## Input

The dialog box is used to input load data within a load case.



The load types are explained on the following pages. For easier identification they are labeled with load abbreviations.

For many load types a dialog for further load data appears after the load has been entered graphically. In this dialog a load-time curve can be defined for most load types for a non-linear system analysis or a load-related dynamic time step calculation. If no curve is selected, the load is constant.



## General

### Dead Load

The entire structure is stressed by its dead load (load abbreviation: *EG*).

#### Input

*Weighting X, Y,  
Z direction*

The weighting factors allow for defining the direction of effect as well as scaling the dead load. They refer to the global coordinate system.

### Dead Load of an Element Selection

Selected elements are stressed by their dead load (load abbreviation: *EGEL*).

#### Input

*Weighting X, Y,  
Z direction*

The weighting factors allow for defining the direction of effect as well as scaling the dead load. They refer to the global coordinate system.

### Influence Surface

This load type (load abbreviation: *EFF*) can be used to determine influence surfaces for internal forces of area elements.

#### Input

*Field point*

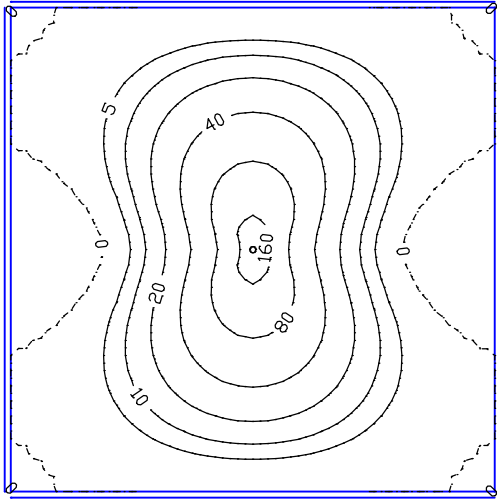
Coordinate x, y, z [m].

*Influence surface*

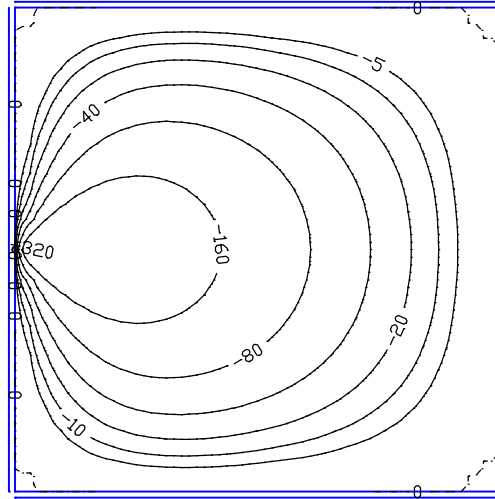
"nx", "ny", "nxy", "mx", "my", "mxy"

The ordinates of the resulting influence surface are expressed as deformation in the direction of the load to be analyzed.

Example: Square slab with four fixed edges



Influence surface "mx"  
Field point: Midspan



Influence surface "mx"  
Field point: Middle of the left edge

## Influence Line

Selected elements are stressed by their dead load (load abbreviation: *EGEL*).

### Input

*Weighting X, Y, Z direction* The weighting factors allow for defining the direction of effect as well as scaling the dead load. They refer to the global coordinate system.

## Inserting of Load Case Data

This load type (load abbreviation: *EINF*) allows you to insert load data from other load cases in the current load case in a weighted manner. The *Superposition* load type may not be present in the inserted load cases.

### Input

*First load case* First load case that is inserted.  
*Last load case* Last load case that is inserted.  
*Weighting factor* Factor by which to multiply all load data.

## Buckling Eigenvalues

Using this load type (load abbreviation: *PKI*), the buckling eigenvalues, buckling eigenmodes and the buckling lengths are determined for the load case in question. It cannot be used in connection with the iterative equation solver.

### Input

*Number* Number of desired buckling eigenvalues or buckling eigenmodes (maximum 9).

## Creep and Shrinkage

The creep and shrinkage load case is calculated using this load type (load abbreviation: *KS*). During calculation the creep and shrinkage coefficients defined in the material properties are taken into consideration.

### Permanent load case

Load case of the creep-generating permanent load. This load case combines the dead load, additional load, and prestressing load cases with the help of the *superposition* load type. As the load cases are calculated in ascending order and the internal forces of the permanent load case are needed for creep and shrinkage, the load case number of the permanent load case should be smaller than the number of the current creep and shrinkage load case.

Optionally, for tendons with bond the redistribution of internal forces between concrete and prestressing steel can be taken into consideration.

### Support Displacement

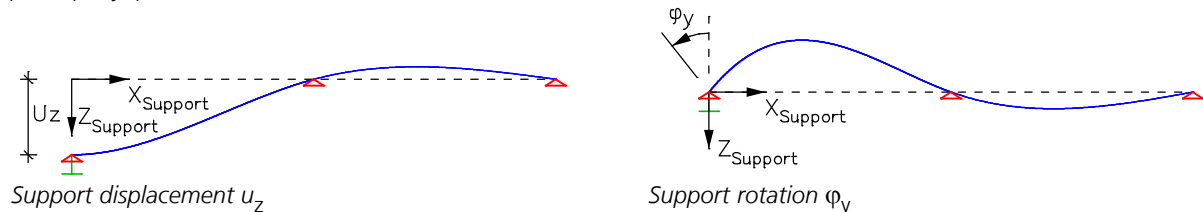
The support displacement acts on supported nodes in the direction of the bound degrees of freedom (load abbr.: *DS*).

#### Input

Selection of the affected supports.

$u_x, u_y, u_z$  Displacements [mm].

$\phi_i.x, \phi_i.y, \phi_i.z$  Rotations [rad/1000].



### Superposition of Load Cases

Through superposition (load abbreviation: *SUP*) the existing results of different load cases can be added and weighted. Other load types are not permitted in the load case in question. Because the load cases are analyzed in ascending order, the load numbers of the load cases to be superposed should be smaller than the number of the current load case.

#### Input

*First load case* First load case to be superposed.

*Last load case* Last load case to be superposed.

*Weighting factor* Factor by which to multiply all results.

### Theory

This load type (load abbreviation: *TH*) is used to specify the calculation theory for the load case in question.

#### Input

*Geometrical theory* 1. / 2. / 3. order: Calculation according to geometric first-, second- or third-order theory.

*Number of iterations* Maximum number of iterations without observing the error threshold. At number "1" the termination is done program controlled.

*Error* Error threshold for the relative size of the remaining residuum in [%].  
Only values  $\leq 1\%$  are permissible.

*No soil pressure > 0* Disregard tensile stresses in the foundation area. For a nonlinear system analysis the tension strength specified for nonlinear bedding is always used.

*No support reaction < 0* Disregard tensile forces in the supports.

When using this load type, make sure to pay attention to the notes provided in the *Load group* section.

## Temperature on Area and Solid Elements

The temperature load is defined in two different ways:

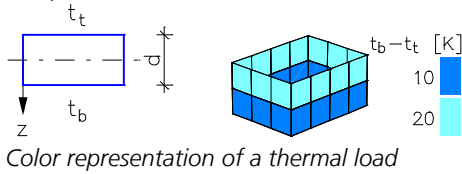
- With the *element load on area and solid elements*, the temperature load is assigned directly to elements (load abbreviation: *TMPF*).
- The *free load for area elements* is described geometrically (load abbreviation: *GD1+TMPD*). The assignment to the area elements only is done during the FEM calculation, if the center of gravity of the element is within the free load. The distance between the load and element plane must not be greater than half the thickness of the element. Optionally, the load area can be defined via selected model faces.

### Input

Type of load input:

- *Free load for area elements*
- *Free load on model faces*
- *Element load on area and solid elements*

*tm* Temperature as a difference to the installation temperature [K].  
*tb - tt* Temperature difference between the bottom and top side of the section (only for area elements) [K].



Color representation of a thermal load

The load type *free temperature on solid elements* is to be used to describe the temperature load for solid elements geometrically.

## Temperature on Beams and Cables

The temperature load is defined in two different ways:

- With the *element load on beams and cables*, the temperature load is assigned directly to elements (load abbreviation: *TMPS*).
- The *free load on beams* is described by the start and end point of the temperature load (load abbreviation: *LKO+TMPL*). The assignment to the beams only is done during the FEM calculation, if the center of the beam element is within the free load. Optionally, the load geometry can be defined via linear edges or selected beams.

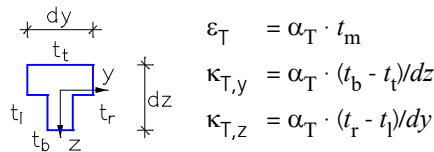
### Input

Type of load input:

- *Free load on beams*
- *Selection of linear edges for free load on beams*
- *Selection of beams for free load on beams*
- *Element load on beams, cables*

*tm* Temperature as a difference to the installation temperature [K].  
*tb - tt* Temperature difference between the bottom and top side of the section [K].  
*tr - tl* Temperature difference between the right and left side of the section [K].

For the calculation the following coherences apply:

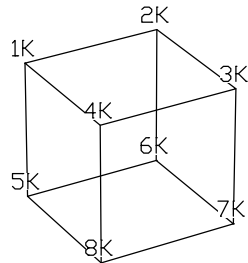


This load type is inadmissible for the section type *Beam*, because the dimensions  $d_y$  and  $d_z$  are not defined. Alternatively the load type *Initial Strain* can be used.

## Free Temperature on Solid Elements

This load type (load abbreviation: *TMPV*) describes an octagonal spatial temperature field with linear temperature curves along its edges. The temperature of the elements is interpolated between the values at the corners of the temperature field during the FEM calculation. In this process the solid element *VQ83* gets the temperature that is determined at its centroid. For the solid element *VT10* (tetrahedron) a linear variable temperature curve is determined between its four corners, if all element nodes are within the temperature field. All temperatures are defined to be the difference to the installation temperature in [K].

### Load data load case 1: Temperature field



Temperature solid (TMPV)					
No.	Pt.	t [K]	x [m]	y [m]	z [m]
1	1	1	0.000	0.000	0.000
	2	2	10.000	0.000	0.000
	3	3	10.000	10.000	0.000
	4	4	0.000	10.000	0.000
	5	5	0.000	0.000	10.000
	6	6	10.000	0.000	10.000
	7	7	10.000	10.000	10.000
	8	8	0.000	10.000	10.000

## Initial Strain

### Initial Strain on Area and Solid Elements

This load type (load abbreviation: *VORD*) allows you to describe special cases of thermal loads on area and solid elements. The definition is based on general strain and curvature parameters that relate to the coefficient of expansion  $\alpha_T$ .

#### Input

Selection of the stressed elements.

$Eps.x, Eps.y, Eps.xy$  Strain or shear [K] based on  $\alpha_T$ .

$Kap.x, Kap.y, Kap.xy$  Curvature or torsion [K/m] based on  $\alpha_T$ .

with

$$Eps = \varepsilon_0 / \alpha_T$$

$$Kap = \kappa_0 / \alpha_T$$

Only curvatures can be entered in slab structures, only strains can be entered in plain stress structures.

#### Uniform temperature load on area elements:

$$Eps.x = Eps.y = t_m$$

$$Eps.xy = 0$$

with

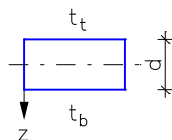
$t_m$  = temperature as a difference to the installation temperature.

#### Non-uniform temperature load on area elements:

$$Kap.x = Kap.y = (t_b - t_t)/d$$

$$Kap.xy = 0$$

with



$t_b - t_t$  = Temperature difference between the bottom and top side of the section.

$d$  = Section height.

#### Uniform temperature load on solid elements:

$$Eps.x = Eps.y = Eps.xy = t_m$$

$$Kap.x = Kap.y = Kap.xy = 0$$

with

$t_m$  = Temperature as a difference to the installation temperature.

## Initial Strain on Beams and Cable Elements

This load type (load abbreviation: *VORD*) allows you to describe special cases of thermal loads on beams and cable elements. The definition is based on general strain and curvature parameters that relate to the coefficient of expansion  $\alpha_T$ .

### Input

Selection of the stressed beams and cable elements.

$Eps.x$	Strain [K] based on $\alpha_T$ .
$Kap.x, Kap.y, Kap.z$	Torsion or curvature [K/m] based on $\alpha_T$ .
with	
$Eps$	$= \varepsilon_0 / \alpha_T$
$Kap$	$= \kappa_0 / \alpha_T$

The parameters are defined as follows for the temperature load:

$Eps.x$	$= t_m$	
$Kap.y$	$= (t_b - t_t) / dz$	
$Kap.z$	$= (t_r - t_l) / dy$	

with

$t_m$	Temperature as a difference to the installation temperature,
$t_b - t_t$	Temperature difference between the bottom and top side of the section,
$t_r - t_l$	Temperature difference between the right and left side of the section,
$dy, dz$	Section dimensions.

## Prestressing

The prestressing load type (load abbreviation: *VSPG*) activates the specified tendon groups within the load case in question. The statically determined bending moments  $M_0$  of prestressing can be determined for beams with the lever arm, which results either by taking the total or the effective flange width (cf. DAFStb, Book 600, Chapter 5.3.2.1; Book 525, Chapter 7.3.1 (1) or DIN 1075, Chapter 5.1.3.1). For area and solid elements, the option "Entire flange width for  $M_0$ " is not applicable. The load case should not contain any other load data, so that the effect of the prestressing is clearly distinguishable from the other stresses.

## Dynamic Train Load

The train load type (load abbreviation: *ZUG*) is used to describe a train crossing. When performing a dynamic calculation (time step integration), the locations of the axle loads, which change over time, are determined. Multiple trains can be considered simultaneously, allowing you, for example, to analyze the effect trains traveling in opposite directions have on each other. The axle loads of the trains always act in the global z direction.

### Input

<i>Track</i>	The track of a train is represented on the structure through a polygon. This must lie within the section dimensions of the elements. Area and solid elements as well as beams with polygon sections are considered here.
<i>Label</i>	Freely selectable.
<i>No.</i>	Freely selectable.
<i>Train</i>	The load of a train is defined by axle loads [kN] and corresponding distances [m] from the start of the train. <ul style="list-style-type: none"> <li>• <i>ICE 1, ICE 2, ...</i>: Standard trains with preset axle loads and distances.</li> <li>• <i>User-defined</i>: Freely definable train.</li> </ul>
$v$	Speed of the train [km/h].
<i>Start time</i>	Time until the departure of the train on the track [s]. This can be used to generate a time offset between different trains.



## Dynamic Element Collapse

The load type (load abbreviation: *KOLL*) defines the failure of elements at a specific point of time. During a direct time step integration the effective stiffnesses of the selected elements at the time of collapse are reduced with the *factor for effective stiffness*. The element dead load is reduced accordingly. For different elements different times of collapse can be taken into account.

## Predeformation

This load type (load abbreviation: *VV*) is used to predeform the structure. To do so, enter the predeformation number. This load type is only available for frameworks exclusively. It should only be used in combination with beam or nodal loads.

## Fire Scenario

This load type (load abbreviation: *FIRE*) indicates that this load case will be calculated with respect to the selected fire scenario. It is only taken into account for nonlinear system analyses (for fire scenarios).

## Thermal Action

This load type (load abbreviation: *TRMO*) is used to calculate solid structures under consideration of the temperature distribution due to a thermal action. If an instationary thermal action was selected, additionally the desired time step is to be set. The thermal strains result from the material properties with the help of the installation temperature and the coefficient of thermal expansion  $\alpha_T$ .

## Load Group

This load type (load abbreviation: *GRL*) is especially suited for preparing load cases for nonlinear calculation. It allows you to compile loads from existing load cases by retrieving the necessary load factors and to activate the desired calculation theory. The load group is used to combine the existing *Insert*, *Theory*, *Predeformation* and *Buckling eigenvalues* load types into a single load type.

Load groups can also be modified via a dialog in the section *Loads* of the data base. The *Superposition* load type may not be present in the inserted load cases.

### General

<i>Theory</i>	<i>1. / 2. / 3. order theory</i> : Calculation according to geometric first-, second- or third-order theory. <i>Buckling eigenvalues</i> : Calculation of buckling eigenvalues and buckling eigenmodes.
<i>Error threshold</i>	Error threshold for the relative size of the remaining residuum in [%]. Only values $\leq 1\%$ are permissible.
<i>No soil pressure &gt; 0</i>	Disregard tensile stresses in the foundation area. For a nonlinear system analysis the tension strength specified for nonlinear bedding is always used.
<i>No support reaction &lt; 0</i>	Disregard tensile forces in the supports.
<i>Add. global load factor</i>	Additional factor for all loads.
<i>Number of eigenvalues</i>	Number of desired buckling eigenvalues or buckling eigenmodes (maximum 9).
<i>Predeformation</i>	Number of the predeformation.
<i>Load cases</i>	Select load cases to add them to the selection list.
<i>Load factor</i>	The individual load cases are weighted by a factor.
<i>Apply</i>	Change the weighting factor for selected load cases.
<i>Pushover</i>	Selection of the pushover case corresponding to the selected horizontal loads ( <i>modal</i> or <i>constant</i> in "+x", "-x", "+y" or "-y" direction). This option is only available if corresponding results from the dynamic pushover calculation are available.

### Concrete creeping, shrinkage

In combination with the nonlinear analysis a load case related definition of the creep and shrinkage coefficients is available for beam and area elements made of standardized concrete material with nonlinear material behavior. A description of the calculation method can be found in chapter *Nonlinear structural analysis*.

*Consider concrete creeping, shrinkage in the nonlinear analysis*

Enables concrete creeping and shrinkage for the cross-sections, which are listed in the table. For first-time activation available cross-sections are added automatically to the table. Already defined coefficients are taken over.

*Refresh*

Calculate the creep and shrinkage coefficients for every activated line.

*All sec. identical*

Takes over the input values of the 1st table entry for the further cross-sections.

*phi*

Creep coefficient  $\varphi(t, t_0)$ .

*Calculated*

Yes: The creep and shrinkage coefficients are determined according to the material with the parameters  $t_0$  and  $t$  as well as other values from the cross-section definition.

*t0, t*

Load start and concrete age in days.

*Factor*

Optional factor for the calculated creep coefficient.

This is used for example for the determination of  $\varphi_{ef}$  according to EN 1992-1-1 from the calculated coefficient.

*eps.cs\*1e5*

The shrinkage coefficient  $\varepsilon_{cs}(t, t_0)$  multiplied with  $1 \cdot 10^5$ . A negative value must be entered for a contraction.

When using this load type, the following should be observed.

#### Calculation according to the second- or third-order theory

Before carrying out a calculation according to the second- or third-order theory, make sure the system is kinematically stable according to the first-order theory. A failure in stability due to beam buckling or shell buckling is recognized during calculation according to second-order theory by a singularity of the system of equations or of an element stiffness matrix. The calculation aborts in this case and informs you about the location and the failed degree of freedom. In borderline cases it may be necessary to set the error threshold to '0' in order to determine the failure state. When using the iterative equation solver, a failure of the system cannot be directly determined.

#### Error threshold

All nonlinear problems are solved iteratively. The length of the error vector, the residuum, is used to control the iteration. It is shown for each iteration step in the output bar. The iteration is terminated when the remaining residuum in relation to the initial residuum has reached the specified error threshold. The initial residuum is the length of the load vector. If the load case definition also includes terms such as temperature, support displacement or shrinkage, the initial residuum can become very large. To avoid a premature end of iteration, the error threshold preset to 1% should be reduced in this case. If the error threshold cannot be reached, the calculation program issues a message.

#### Loss of tensile bedding and tensile support

Unrealistic load combinations (for instance, a missing dead load) can lead to loss of all supports or loss of all bedding (see also *Foundation models*). If the system becomes relocatable due to this, the calculation is aborted because of the singularity. The relocatable degree of freedom is indicated, provided the iterative equation solver was not selected.

#### Calculation according to the modulus of compressibility method

When choosing the modulus of compressibility method, the iterative calculation of the subsidence hollow is performed automatically with an error threshold of 1%. A smaller value can be defined in the load case, and the loss of tensile bedding can also be selected.

#### Analysis of cable structures

If a structure contains cable elements, these are automatically calculated according to the theory of large deformations with an error threshold of 1%. All other elements are calculated according to the first-order theory, unless the second-order theory has been specified in the load case. The error threshold defined there also applies to the cable elements. If the system becomes unstable due to the loss of a cable element, the calculation is terminated due to the singular system of equations, provided the iterative equation solver was not selected.

#### Consideration of tension and compression beams

For structures with tension and compression beams, the iterative determination of the equilibrium state is automatically carried out with an error threshold of 1%. In the load case a smaller value can be specified. Instabilities due to beam failure lead to a singular system of equations and an abortion of the calculation, provided the iterative equation solver was not selected.

### Load combination

In order to combine the decisive actions for calculations according to the deflection theory, with the exclusion of tensile bedding or loss of tensile support, transfer the affected load cases to the selection list and assign them load factors.

For calculations according to DIN 18800, Part 2, or EN 1993-1-1 the following different cases exist:

- Calculation according to the geometric second-order theory with linear material law according to elasticity theory and subsequent elastic-elastic or elastic-plastic check. The internal forces are to be determined according the standard with  $\gamma_M$ -fold design values of the actions. To do so, the load cases of the selection list are to be weighted with the factor  $\gamma_F$ . Additionally,  $\gamma_M$  is to be specified as the global load factor.
- Ultimate limit state check according to the geometric second-order theory with nonlinear material law according to the plasticity theory. In this case  $\gamma_M$  is considered on the resistance side according to the standard. The internal forces calculation is to be carried out with the design values of the actions. Therefore, only  $\gamma_F$  needs to be provided as a weighting factor.

### Prohibition of superposition

Calculations with cable elements or tension and compression beams, according to the second-order theory, excluding the tensile bedding or with loss of tensile support, are performed on the basis of nonlinear iteration. So in these cases the superposition principle is no longer valid, meaning the results may not be additively superposed with other results. Further processing using load case combinations is only permissible if these only contain mutually exclusive actions.

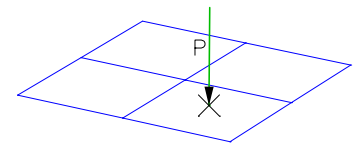
## Point Loads

### Free Point Load

The load is assigned to elements during the FEM calculation. For area elements the load must be within the section. The load is assigned to beam elements when it is within the rectangle enclosing the cross-section. For solid elements it can be anywhere within the element.

#### Input

<i>Global orientation</i>	Point load in the global system (load abbreviation: <i>PKO+PG</i> ).
<i>Local orientation</i>	Point load in the element system (load abbreviation: <i>PKO+PL</i> ).
<i>Load point</i>	Coordinates $x, y, z$ [m].
$P_x, P_y, P_z$	Load components in the directions of the selected coordinate system [kN].



The *convert* context function can be used to convert free point loads into *nodal loads* or *point loads on beams*.

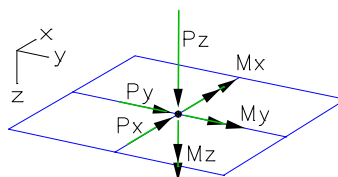
### Nodal Load

Nodal loads act in the global system (load abbreviation: *KNL*).

#### Input

Selection of the stressed element nodes.

$P_x, P_y, P_z$	Forces [kN].
$M_x, M_y, M_z$	Moments [kNm].



The *convert* function can be used to convert nodal loads into *free point loads*.

### Point Load on Beams

#### Input

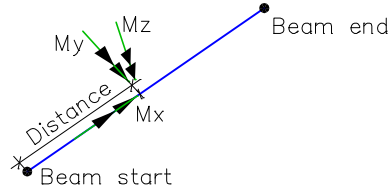
<i>Global orientation</i>	Point load in the global system (load abbreviation: <i>SFG</i> ).
<i>Local orientation</i>	Point load in the beam system (load abbreviation: <i>SFL</i> ).
<i>Load point</i>	Load point on the beam axis. The distance from the beam start is saved.
$P_x, P_y, P_z$	Load components in the directions of the selected coordinate system [kN].

The *convert* context function can be used to convert point loads on beams into *free point loads*.

### Point Moment on Beams

**Input**

- Load point* Load point on the beam axis. The distance from the beam start is saved (load abbreviation: *SML*).
- Mx, My, Mz* Load components in the beam system [kNm].

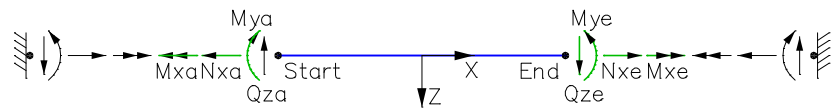


### Fixed-End Reactions of Beams

The fixed-end reactions at the beam start (load abbreviation: *SOA*) or beam end (load abbreviation: *SOE*) must be input separately.

**Input**

- Selection of the stressed beams.
- Nx, Qy, Qz* Normal and lateral force [kN].
- Mx, My, Mz* Moments [kNm].



*Fixed-end reactions at the beam start and end.*

### Line Loads

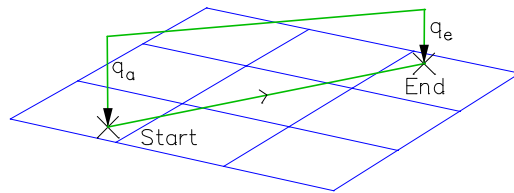
#### Free Line Load - Global

The load is assigned to elements section by section during the FEM calculation. For beam elements the load must lie exactly on the beam axis and for area elements in the element plane. For solid elements it can also lie within the element (load abbreviation: *LKO+LG*).

Optionally, the load geometry can be defined using selected lines and arcs. In the case of arcs, the line loads are approximated to the arc by 20 ° large equivalent secants.

**Input**

- Starting point* Coordinate x, y, z [m].
- End point* Coordinate x, y, z [m].
- qxa, qxe* Load ordinates in x direction [kN/m].
- qya, qye* Load ordinates in y direction [kN/m].
- qza, qze* Load ordinates in z direction [kN/m].



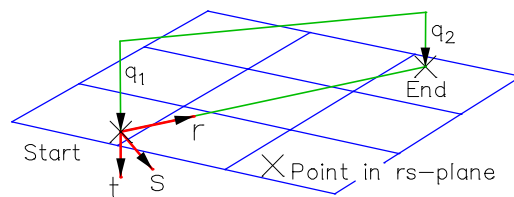
#### Free Line Load - Local

The load is assigned to elements section by section during the FEM calculation. For beam elements the load must lie exactly on the beam axis and for area elements in the element plane. For solid elements it can also lie within the element (load abbreviation: *LKO+LL*).

Optionally, the load geometry can be defined using selected lines and arcs (see also *Free Line Load - global*).

**Input**

- Starting point* Coordinate x, y, z [m].
- End point* Coordinate x, y, z [m].
- Point in rs-plane* Coordinate x, y, z [m].
- Orientation* r, s, t (see figure).
- q1* Load ordinates at the start in selected direction [kN/m].
- q2* Load ordinates at the end in selected direction [kN/m].



The *convert* context function can be used to convert free line loads into *line loads on beams*.

## Local Line Load on Area and Solid Elements in the Element Direction

This load type (load abbreviation: *LKO+LLA*) differs from the local free line load because it acts in the direction of the coordinate systems of the affected elements. It cannot be entered graphically. Its input values correspond to the global free line load.

## Line Load on Beams and Cable Elements

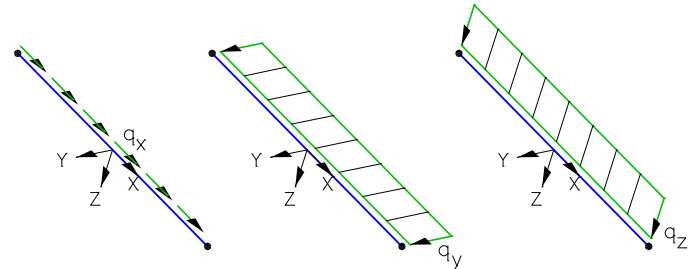
### Input

Selection of the stressed beams and cable elements.

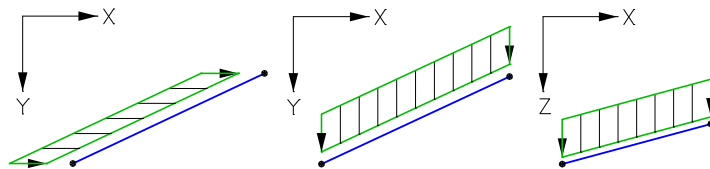
*Global orientation* Line load on beams and cables in the global system (load abbreviation: *SLG*).

*Local orientation* Line load on beams in the local coordinate system (load abbreviation: *SLL*).

$q_x, q_y, q_z$  Load components in the directions of the selected coordinate system [kN/m].



Local line load on beams



Global line load of beams or cable elements ( $q_x, q_y, q_z$ )

## Trapezoidal Loads on Beams

### Input

Selection of the stressed beams.

*Starting point* Load start.

*End point* Load end.

*Global x, y, z* Trapezoidal load in the global system (load abbreviation: *STA + GX, GY, GZ*).

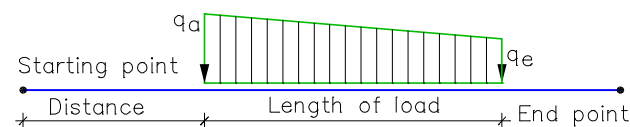
*Local x, y, z* Trapezoidal load in the beam system (load abbreviation: *STA + LX, LY, LZ*).

*Projection x, y, z* Projection of the trapezoidal load in the global direction (load abbreviation: *STA + PX, PY, PZ*).

*Starting ordinate* Load ordinate at the load start in the directions of the selected coordinate system [kN/m].

*End ordinate* Load ordinate at the load end in the directions of the selected coordinate system [kN/m].

The load is automatically distributed over multiple beams for a trapezoidal load. The distances from the beam starts, the load lengths and the interpolated load ordinates are automatically determined for the selected beams. This allows you to describe a trapezoidal load for an entire beam series with a single input.



The *convert* context function can be used to convert line and trapezoidal loads on beams into *free line loads*.

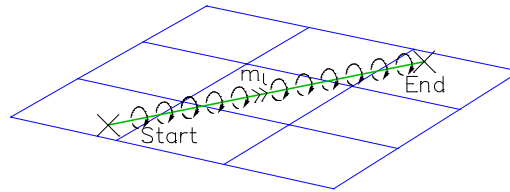
## Free Line Moments on Beams and Area Elements

The load is assigned to elements section by section during the FEM calculation. The direction of rotation of the line moment (load abbreviation: *LKO+LM*) is defined by its start and end point. The load must lie exactly on the beam axis or in the element plane.

Optionally, the load geometry can be defined using selected lines and arcs (see also *Free Line Load - global*).

### Input

*Starting point* Start coordinate  $x_a, y_a, z_a$  [m].  
*End point* End coordinate  $x_e, y_e, z_e$  [m].  
*ml* Line moment [kNm/m].

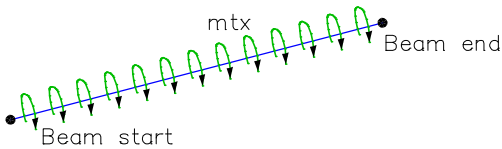


## Line Torsional Moment on Beams

### Input

Selection of the stressed beams (load abbreviation: *SMT*).

*mtx* Line torsional moment [kNm/m].



Line torsional moment about the beam axis

## Area Loads

### Area Element Load

#### Input

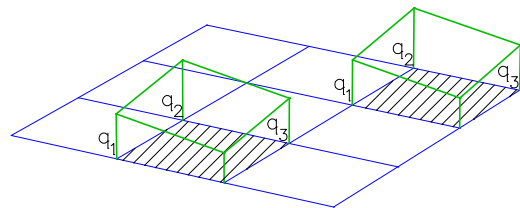
*Global orientation* Area load in the global system (load abbreviation: *FGX, FGY, FGZ*).

*Local orientation* Area load in the element system (load abbreviation: *FLX, FLY, FLZ*).

*Projection orientation* Projective area load (load abbreviation: *FPX, FPY, FPZ*).

*X direction,*  
*Y direction,*  
*Z direction* Direction of effect of the load in the selected coordinate system.

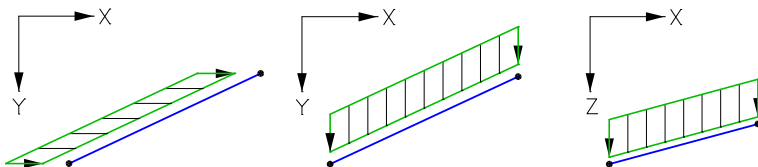
After selecting the elements to be loaded, the load ordinates ( $q_1, q_2, q_3$  [kN/m<sup>2</sup>]) are to be defined at the first three element nodes in the direction of the selected coordinate system.



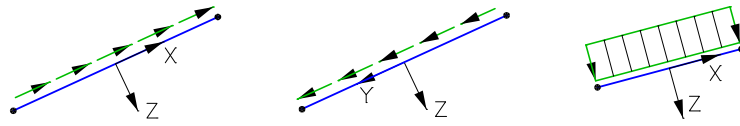
Global area load in 3D representation

*X,Y,Z direction* Trapezoidal load between the start (*1st point*) and the end (*2nd point*) of a line to be defined.

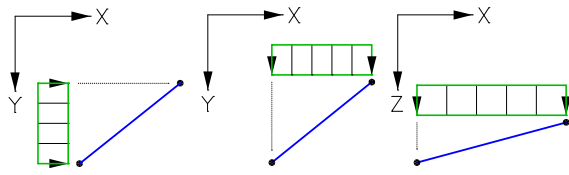
After selecting the elements to be loaded and the start and end point of the line, the load ordinates ( $q_x, q_y, q_z$  [kN/m<sup>2</sup>]) are to be defined at the *1st* and *2nd* point in the directions of the selected coordinate system.



Area load in global system ( $x, y, z$ )



Area load in element system (x, y, z)



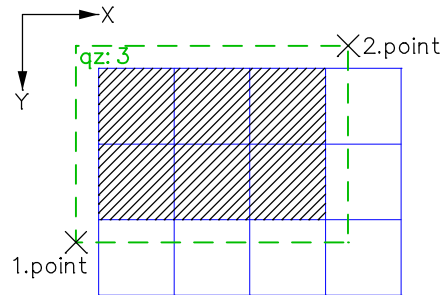
Projective area load (x, y, z)

### Uniform Region Load on Area Elements

The regional load (load abbreviation: *GKO+GG*) acts in the global direction of the area elements. The load geometry is delineated with the corner points of the enclosing window. The area must be parallel to the xy plane. All area elements that lie completely within the load area are stressed.

**Input**

- 1st point      Coordinates of the first corner x, y, z [m].
- 2nd point      Coordinates of the second corner x, y, z [m].
- qx, qy, qz      Global uniformly distributed load [kN/m<sup>2</sup>].



### Free Area Load - Rectangle

This load type can be defined as a constant or variable load. The load area is assigned to elements section by section during the FEM calculation. The assignment is done in two different ways:

- For the load orientation *local t projective* the load is projected on elements in the load direction *t*. The direction *t* of the load is defined by its direction of rotation (point 1 → 2 → 3). Only elements, which are not hidden by overlaying elements in direction of projection, can be affected by this load type. This also applies to a partial overlap. The reference area is always the defined load area.
- The other load orientations are assigned to the directly affected elements. Therefore for area elements the distance between load and element plane must not be greater than half the thickness of the element. The load plane should be parallel to the element plane. For solid elements the load can be anywhere within the element.

**Input**

1st point 1st coordinate x, y, z [m].

2nd point 2nd coordinate x, y, z [m].

3rd point 3rd coordinate x, y, z [m].

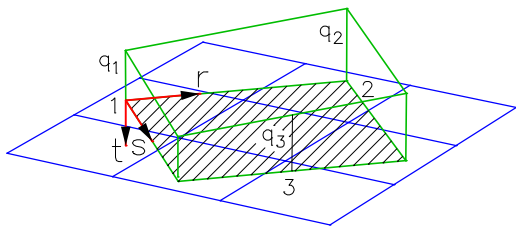
Uniform load

*qx, qy, qz* Global load ordinates [kN/m<sup>2</sup>] (load abbreviation: GR1+GR2).

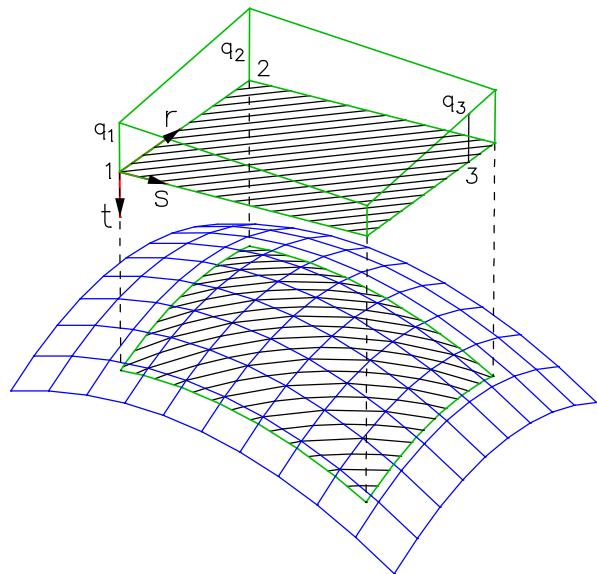
Variable

*Orientation* The load orientation of the trapezoidal load is global, projected or local to the load contact area (*r, s, t*) and thus independent of the element system.

*q1, q2, q3* Load ordinates of the three points [kN/m<sup>2</sup>] (load abbreviation: GR1+GRT).



Load orientation "local t"



Load orientation "local t projective"

### Free Area Load - Polygon

This load type can be defined as a constant or variable load. The load area is assigned to elements section by section during the FEM calculation. The assignment is done in two different ways:

- For the load orientation *local t projective* the load is projected on elements in the load direction *t*. The direction *t* of the load is defined by its direction of rotation (point 1 → 2 → 3). Only elements, which are not hidden by overlaying elements in direction of projection, can be affected by this load type. This also applies to a partial overlap. The reference area is always the defined load area.
- The other load orientations are assigned to the directly affected elements. Therefore for area elements the distance between load and element plane must not be greater than half the thickness of the element. The load plane should be parallel to the element plane. For solid elements the load can be anywhere within the element.

**Input**

Load area Corner points of the load area [m].

Uniform load

*qx, qy, qz* Global load ordinates [kN/m<sup>2</sup>] (load abbreviation: GD1+GD2).

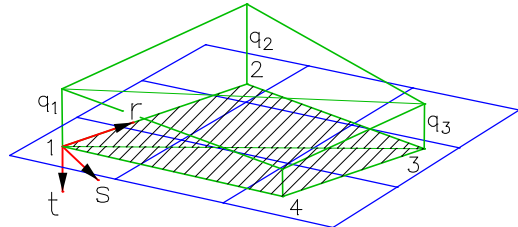
Variable

*Orientation* The load orientation of the trapezoidal load is global, projected or local to the load contact area (*r, s, t*) and thus independent of the element system.

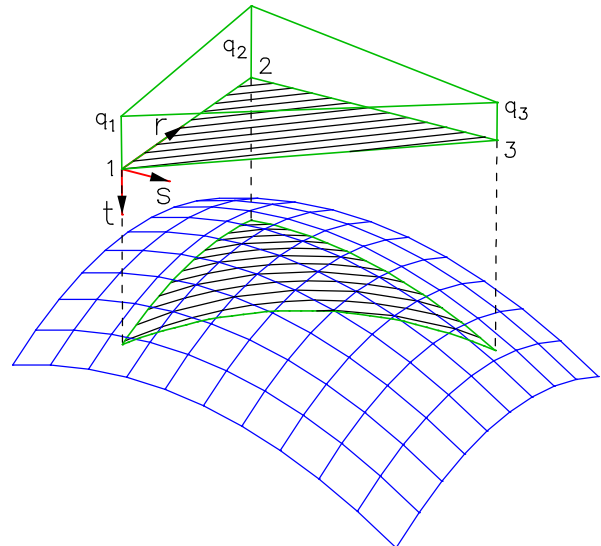
*q1, q2, q3* Load ordinates of the three points [kN/m<sup>2</sup>] (load abbreviation: GD1+GDT).



- Start** Start defining a new area with different load ordinates. The input of the current area is terminated.
- Close** Close the current area. The extended dialog allows you to enter further areas with similar load ordinates within the same plane.
- Back** Undo the last point entered.



Load orientation "local t"



Load orientation "local t projective"

### Uniform Load on Model Faces

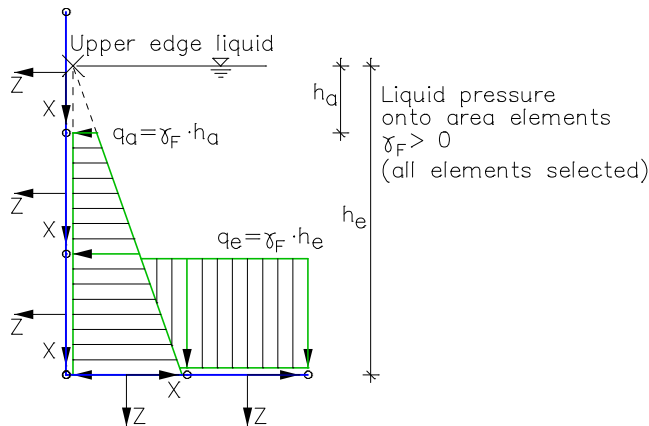
After the selection of model faces their areas are divided into triangles and corresponding area loads are created. The loads (load abbreviation: *GD1+GDT*) are independent of the model faces and elements. The load direction can be global, projective or perpendicular towards the model face (*local t*). The option "Generate load cases for subareas" enables a grid-shaped partition of the load area into several load cases.

### Liquid Pressure on Area Elements

The liquid pressure (load abbreviation: *FD*) stresses all selected area elements that are fully below the liquid level. It acts in the local z direction of the elements.

#### Input

- Upper edge liquid* Z coordinate of the liquid level [m].
- Specific weight* Specific weight  $\gamma_F$  of the liquid [kN/m<sup>3</sup>].



## Area Load Onto Beams

This load type can be used to distribute an area load to selected beams. The load area can be any polygonal plane area. Selected beams, which are not within the load plane, are projected to this plane before the beam loads are calculated. Thereby the load is only assigned to the beams which lie within the load area. The load distribution areas are calculated using the Voronoi diagram.

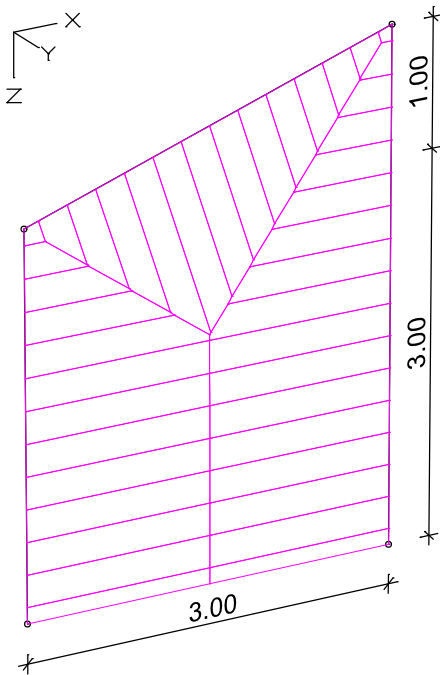
After the load definition has been finished, only the resulting beam loads can be edited.

### Input

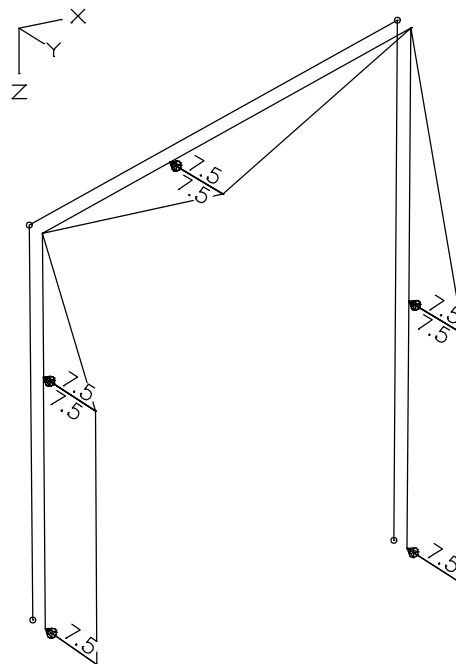
*Selection of the stressed beams.*

- Load geometry* Definition of the corner points of the load area [m].
- Load orientation* The load can act in the direction of the global coordinate system based on the real load area (Global x, y,z) or based on the projected load area (Projection x,y,z) or it can act normal to the load plane.
- Load* If variable load distribution is selected, the load is assigned to beams proportionately to their load distribution area. Alternatively, the load can be assigned to all beams as a constant line load.
- Load size* The area load [kN/m<sup>2</sup>] can be constant or variable. For variable area loads the load ordinates are to be specified for the first three points of the load area.
- Properties* With this function the properties of the area load can be modified after the beam loads have been created. Besides the load sums of the area load are compared with those of the beams.
- Next* Further loads can be defined.

### Example



Load area and load distribution (Voronoi-Diagram) for 5 kN/m<sup>2</sup> constant area load



Resulting beam loads [kN/m]

## Database

The dialog page *Database* allows the creation and the import of user-defined load combinations. Therefor point, line and area loads, which are independent of nodes and elements, can be used. They are stored in the file *lgraph.dat*.

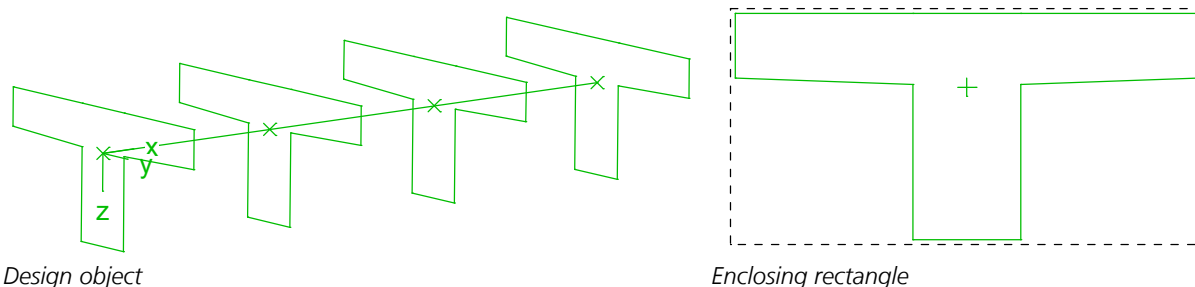
## Design objects

### Purpose

In solid construction calculation models are used that are based on a combination of beam and area elements or, less commonly, solid elements. These models make possible, for instance, the representation of construction stages and deliver a detailed picture of the stress acting on the structure. They cannot, however, be used directly for reinforced concrete or prestressed concrete checks. The reason for this is that the checks are usually section oriented. For instance, for the checks at the ultimate limit state, several elements must be merged into one object with respect to geometry and stresses. This is done using so-called *Design objects* (except design program for DIN 1045:1988).

### Definition

A design object has the geometry of a line or a *NURBS curve*. The curve between start and end point defines the object axis and at the same time the local x axis. The local z axis typically corresponds to the global z axis, although it can be defined at an angle of tilt. Sections can be defined at any point along the object axis. The sections are perpendicular to the axis. At the point of each section the FE program integrates the stresses and internal forces of the elements that lie within the enclosing rectangle of the section. The internal forces can be related to the axis of the design object or to the centers of gravity of the cross-sections. If the internal forces are related to the axis of the design object, a reasonable position of the reference axis of the design object is important, especially for a torsion design. The coordinate system of the design object determines the orientation of the internal forces.

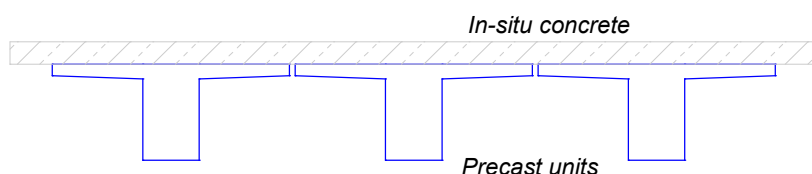


*Design object*

*Enclosing rectangle*

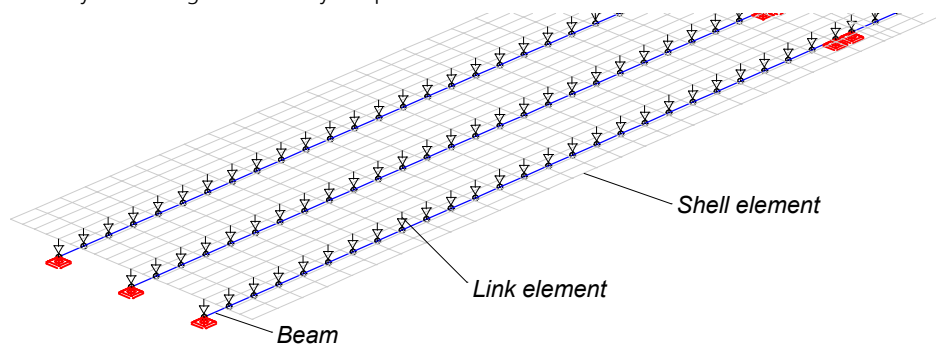
### Example

The following illustration shows the section of a simple two-span bridge, built with precast units and a subsequently applied in-situ concrete slab.



*Bridge section*

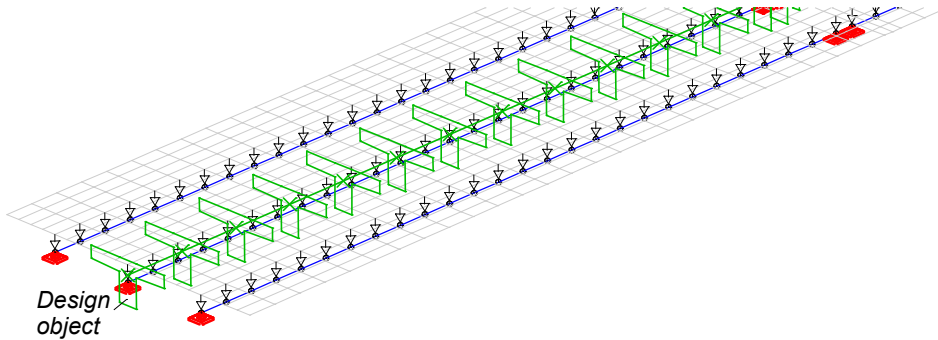
A possible structure model is shown in the next figure. The precast units are modeled using beam elements and the roadway slab using eccentrically coupled shell elements.



*Structure model*

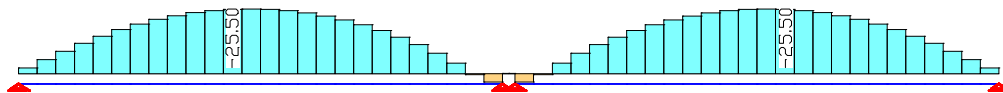
This system delivers the internal forces in the precast units and the in-situ concrete slab. The internal forces for the design of the central composite girder are to be determined as the next step. For this purpose a linear design object is defined. In this

example 13 sections are chosen for each design object.

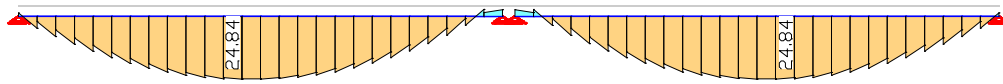


Structure model with design object

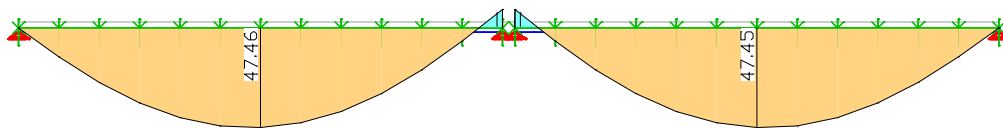
The following illustrations show several internal forces in the area of the central girder.



$n_x$  in the prismatic shell elements above the central girder [kN/m]



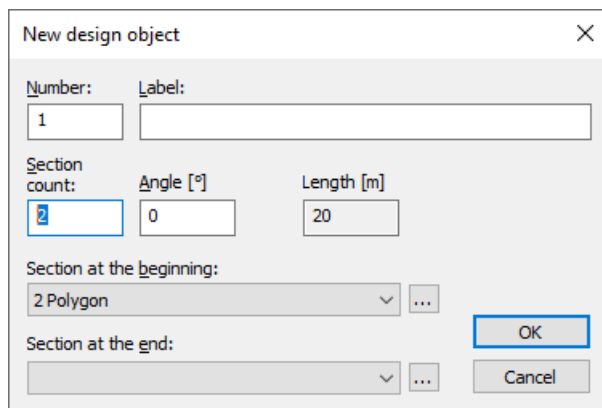
$M_y$  in the beam elements of the central girder [kNm]



$M_y$  in the design objects [kNm]

## Input

The input function is called up from the *Structure* menu (*Design Objects*). After the geometry has been defined, the following dialog appears for entering the additional object properties. The angle of tilt refers to an additional rotation of the object about its longitudinal axis compared with the standard orientation. When in standard orientation, the local y axis runs parallel to the global XY plane.



The video "[https://download.infograph.de/video\\_de/Bemessungsobjekt%20eingeben.mp4](https://download.infograph.de/video_de/Bemessungsobjekt%20eingeben.mp4)" (German language) shows the handling of design objects.

## Editing

To edit a design object, select it and open the following dialog using the context menu. The location of the sections is determined by the point on the axis of the design object  $x_i$  (0 to 1) or  $x$  and the section offset ( $y$ ,  $z$ ). Additionally, a subsequent rotation about the object axis can be applied. A graphical function for manipulating the location of the section is available using '*Structure.Design objects.Move cross section*'.

Design object immediately after input

Design object ✕

Number:  Label:

X:  Y:  Z:  Angle [°]:

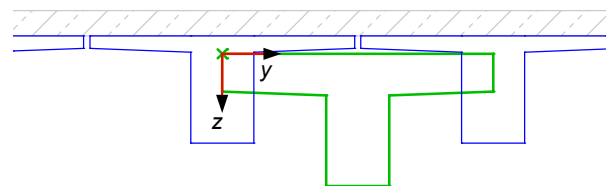
Start point:

End point:

Pos. of cross-sections in local coordinates:

	$x_i$ [-]	$x$ [m]	$y$ [m]	$z$ [m]	Angle [°]	Cross sect
1*	0	0.000	0.000	0.000	0.00	2 Polygon
2	0.0833333	1.600	0.000	0.000	0.00	2 Polygon
3	0.166667	3.200	0.000	0.000	0.00	2 Polygon

Relate the internal forces of all design objects to the cross-section centroid



Design object after moving cross-sections

Design object ✕

Number:  Label:

X:  Y:  Z:  Angle [°]:

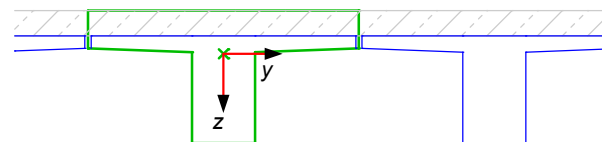
Start point:

End point:

Pos. of cross-sections in local coordinates:

	$x_i$ [-]	$x$ [m]	$y$ [m]	$z$ [m]	Angle [°]	Cross sect
1*	0	0.000	-1.075	-0.342	0.00	2 Polygon
2	0.0833333	1.600	-1.075	-0.342	0.00	2 Polygon
3	0.166667	3.200	-1.075	-0.342	0.00	2 Polygon

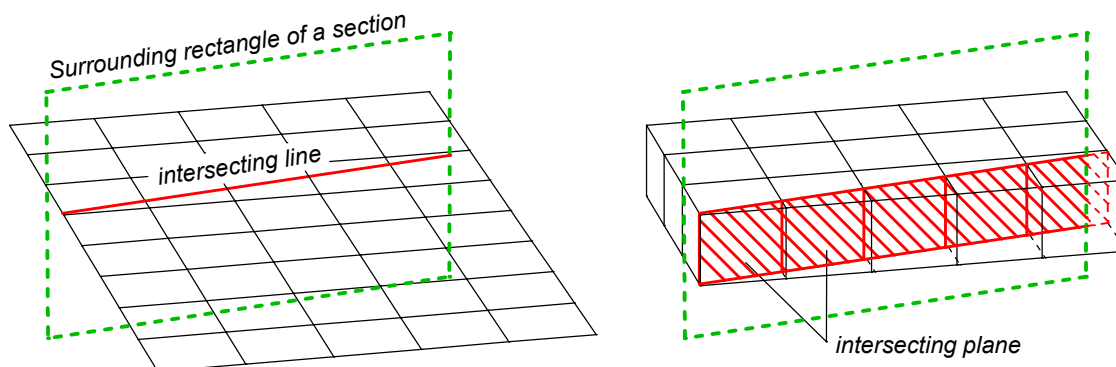
Relate the internal forces of all design objects to the cross-section centroid



## Integration

During the integration of the internal forces or stresses at the location of a section, the following elements are considered:

- Beams whose axis pierce through the enclosing rectangle of the section.
- Area elements whose lines of intersection with the section plane lie within the enclosing rectangle. Thereby also the parts of cut elements are taken into account.
- Solid elements whose planes of intersection with the section plane lie within the enclosing rectangle. Thereby also subareas are taken into account.



Handling of the area and solid elements during integration.

In the standard case the integrated internal forces refer to the coordinate system of the design object. Optionally, the internal forces can be related to the cross-sectional centroids. During the calculation of the statically determined and undetermined part of prestressing the tendon groups, who have an angle smaller than  $45^\circ$  with the design object axis at the intersection point of the enclosing rectangle, are taken into account.

## Analysis

The *Analysis* menu is used to make program-specific settings and to launch the analysis programs.



*Settings...*



*Batch...*

*Construction Stages (Batch calculation of all Construction Stages)*



*Statics (linear material laws)*

*Nonlinear analysis*

*Dynamics*

*Thermal Analysis*

*Section Temperatures*

*Load Case Combination*

*BS EN 1992-1-1 Design*

*DIN 1045-1 Design*

*DIN EN 1992-1-1 Design*

*EN 1992-1-1 Design*

*OENORM B 4700 Design*

*OENORM EN 1992-1-1 Design*

*SIA 262 Design*

*SS EN 1992-1-1 Design*

*Single Design*

*DIN Technical Report Bridge Checks*

*EN 1992-2 Bridge Checks*

*DIN EN 1992-2 Bridge Checks*

*OENORM EN 1992-2 Bridge Checks*

*Recalculation Guideline Bridge Checks*

*DIN 18800 Steel Checks*

*EN 1993-1-1 Steel Checks*

*DIN 1052 Timber Checks*

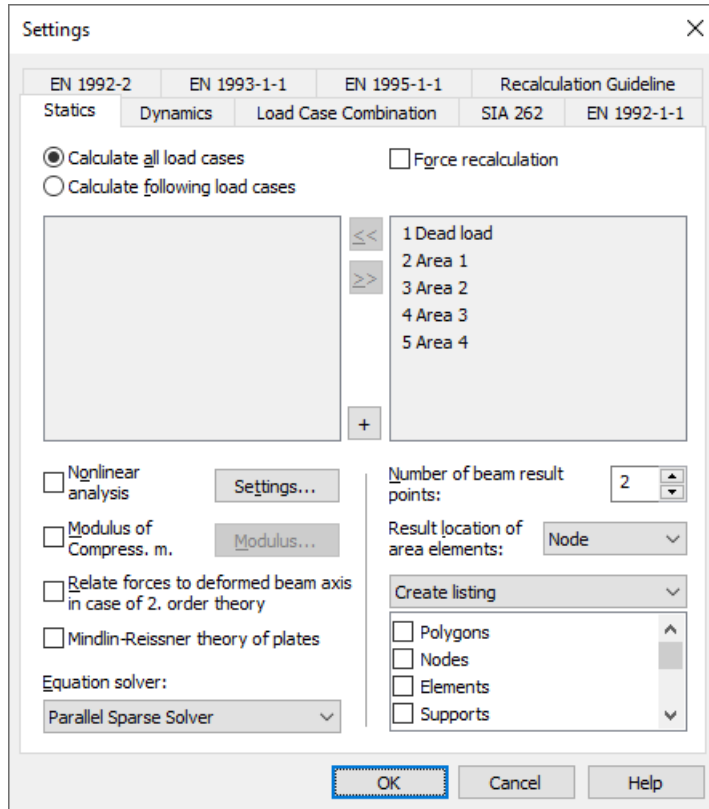
*EN 1995-1-1 Timber Checks*

*Selected Elements*

*Stop Analysis*

During the analysis the program's progress is shown in the output bar along with any possible warnings and error messages.

## Settings Statics



### Calculate all load cases

All load cases are calculated (default).

### Calculate following load cases

The load cases from the left list box are calculated.

### Force recalculation

If this switch is activated, all of the selected load cases will be recalculated. Otherwise, the calculation program checks whether a recalculation is necessary.

### Nonlinear analysis

Instead of statics with linear material laws, a nonlinear system analysis is performed. The *Settings* button opens the settings dialog for the ultimate limit state, the serviceability and the fire scenario check. Load cases selected in the setting dialog (nonlinear analysis) are calculated instead of the selection made here. If the modulus of compressibility method or the Mindlin-Reissner plate theory is selected, this option is not available.

### Modulus of compressibility method

The *Modulus of compressibility method* is used as the subsoil model. The *Modulus of compressibility* button opens the dialog for entering the moduli of compressibility. The bedding modulus method is always used for dynamic calculations and the nonlinear system analysis.

### Relate forces to deformed beam axis in case of 2. order theory

For calculations according to the second-order theory, the internal forces and stresses are based on the beam chord of the deformed system.

### Mindlin-Reissner plate theorie

Generally, area elements with an element approach according to the '*discrete Kirchhoff theory*' (DKT) are used. Optionally, an element approach according to '*Mindlin-Reissner*' (MR) can be used. This option is not available in the nonlinear system analysis.

### Equation solver

- *Standard*: The equation system is solved directly.
- *Iterative*: The equation system is solved iteratively.
- *Substructure Standard*: The equation system is solved using the substructure technique. The elements are allocated to the substructures according to the layer they belong to. The layer number corresponds to the number of the substructure. The elements of a substructure do not have to be placed in an interconnected area.
- *Substructure Info*: The analysis program creates an information log which helps you perform an effective substructure division.
- *Parallel Sparse Solver*: The equation system is solved directly with additional memory optimization and parallel processing.

### Number of beam result locations

The internal forces are calculated at equidistant points in the beam. Two result locations are used by default (start and end node). For bedded beams the reactions are only calculated in the nodes.

### Result location area elements

In the static and dynamic analysis, the internal forces of area elements or the stresses of solid elements are calculated at the Gauss points in the element and interpolated or extrapolated to the selected locations. The following locations can be selected:

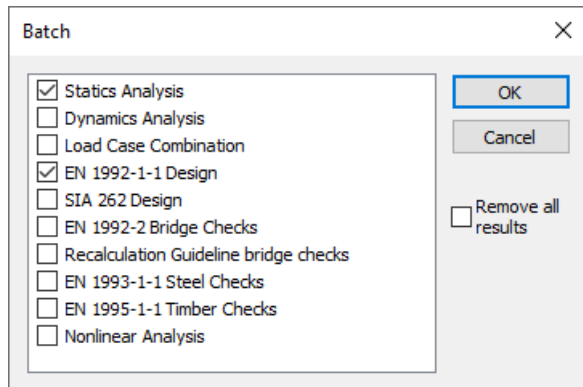
- *Nodes*: Internal force calculation in the element nodes (default).
- *Centroid*: Internal force calculation in the centroids of the elements.
- *Side middle*: Internal force calculation in the side middles of the elements.

### Listing

- *No listing*: No log is generated by the calculation program.
- *Create listing*: A log is generated during the calculation according to the selection list.

## Batch

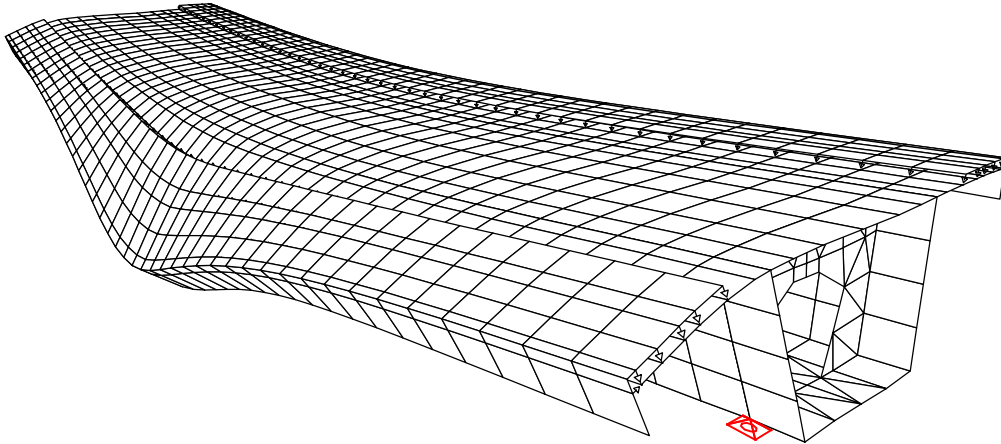
Use this menu item to run all selected analyses as a batch job.





## Results

The following chapter describes the representation of the analysis results. Results can be shown graphically as well as in tables. Their selection takes place in the database or with the *Results* function of the *Structure* menu.



*Superelevated representation of the deformations of a box girder bridge*












### Sum of installed loads and support reactions

LC.	Label	Fx [kN]	Fy [kN]	Fz [kN]
1	Dead load	0.000	0.000	15172.098
	Support reactions	-0.000	-0.000	15172.098
11	Cap load location 1	0.000	-0.000	295.201
	Support reactions	-0.000	-0.000	295.201
12	Cap load location 2	0.000	0.000	295.200
	Support reactions	-0.000	0.000	295.200

*Tabular output of the load sums*

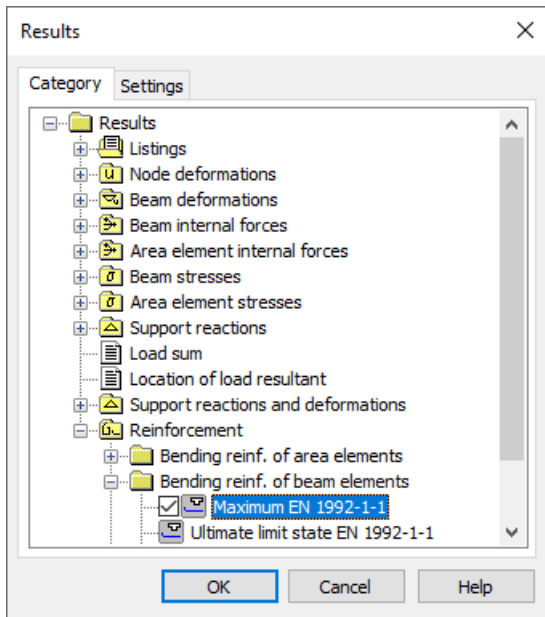
## Result Bar



The result bar includes the following buttons for representing results:

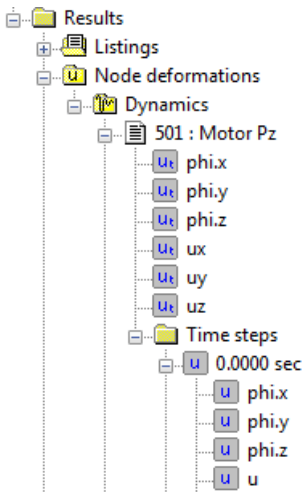
-  Start the dialog for result representation.
-  Turn off result representation.
-  Result representation of the previous load case or load case combination.
-  Result representation of the next load case or load case combination.
-  Ordinate selection for manual labeling.
-  Enable result representation for the selected elements.
-  Disable result representation for the selected elements.
-  Enter section.
-  Delete section.
-  Display or insert result markers for fire scenario stresses or for composite section stresses. During the analysis the section stress distribution of every time step is stored for the marked locations in the beam element. The results can be displayed by double-clicking the corresponding marker.
-  Output the ordinate value at the marked point in the status bar. If internal forces or reinforcement are displayed on the sections, their integral will also be shown over the section length.

## Result Categories

The following dialog shows possible result categories.



The load case and combination results as well as all presentable result components are listed in the subsection of the respective category. After making a selection, the *Settings* tab offers views described below with additional options. The symbol in front of the item identifies the type of view ( e.g. : table view, : graphical view of internal forces).



Results of a time step calculation or a fire scenario analysis for a selected time step are shown like the results of a load case. The progression of results can be represented for a selected location in a diagram. Alternatively, the results can be animated for the whole system (*colored, deformation figure, result graph*), if a superordinate component is chosen.

## Deformation Figure

### Scale

The scale can be set automatically or manually. The node displacements are multiplied by the distortion factor and added to the node coordinates. For load cases the bending curve of beams is displayed as well.

### Display original system in dashed manner

Switch on/off the dashed non-deformed system during deformation display.

### Label

- *Auto*: Automatically add labels to the nodes listing the length of the displacement vector [mm].
- *Off*: Switch off labeling.

### Limit value for labeling

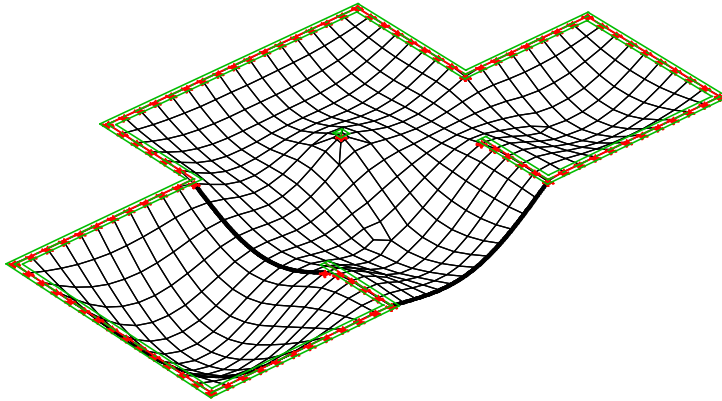
Only displacements exceeding the limit value are labeled.

**Text height [mm]**

The unit depends on the output device.

**Decimal places**

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.



*Superelevated deformation figure*

**Colored, Isosurfaces**

For the representation type *Colored* the elements are colored according to their results. For the representation type *Isosurfaces* results of solid elements are represented by unicolored surfaces of the same value.

**Number of colors**

Set the number of colors for automatic color grading.



**Color shift**

Shift of the automatic color grading.

**Grading**

Enable/disable manual color grading. Color and number values can be changed.

Functions of the context menu ( right mouse button):

<i>Save as Presetting</i>	Saves the current grading as presetting for other projects
<i>Load Presetting</i>	Loads the saved presetting
<i>Load Colors of View</i>	Sets the grading according to the current color representation
<i>Database...</i>	Management of different gradings in the <i>lgraph.dat</i> file
<i>Insert Row</i> 	Adds another one above the selected color level
<i>Delete</i> 	Deletes the selected color level
<i>Set All Values...</i>	Fills the list with the help of <i>start value</i> , <i>end value</i> and <i>grading</i>

**Element mesh**

Enable/disable the element mesh during the colored depiction of results.

**Legend**

Enable/disable the color legend.

**Mean values**

The displayed results are averaged in the results location if all adjacent elements have the same properties.

**Dividing line**

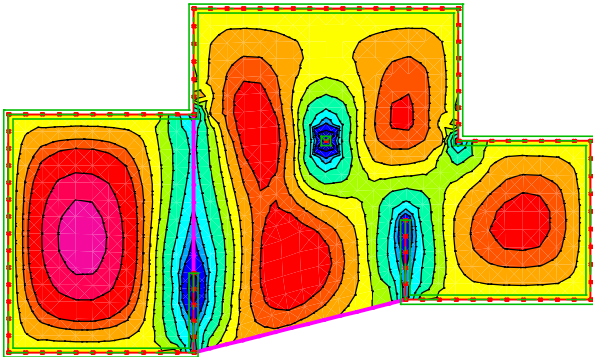
Enable/disable dividing lines at the color transitions.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

### Deformation figure

Enable/disable the deformation figure of a load case.



Colored representation of the internal forces  $m_x$

## Isolines

In the area elements contours are shown according to their results. The contours are created through linear interpolation from node values. If the calculation took place in the side middles and/or in the elements' centroids, the results are extrapolated on the element nodes. The node values are averaged if all adjacent elements have the same properties.

### Number of lines

Set the number of contours for automatic grading.

### Text height

Set the text height for the labels.



### Decimal places

Set the decimal places for automatic grading.

### Grading

Enable/disable manual contour grading. Color and number values can be changed.

Functions of the context menu ( right mouse button):

Save as Presetting	Saves the current grading as presetting for other projects
Load Presetting	Loads the saved presetting
Load Colors of View	Sets the grading according to the current color representation
Database...	Management of different gradings in the <i>Igraph.dat</i> file
Insert Row 	Adds another one above the selected contour
Delete 	Deletes the selected contour
Set All Values...	Fills the list with the help of <i>start value</i> , <i>end value</i> and <i>grading</i>

### Colored

Enable/disable the coloring of the manual contour grading.

### Element mesh

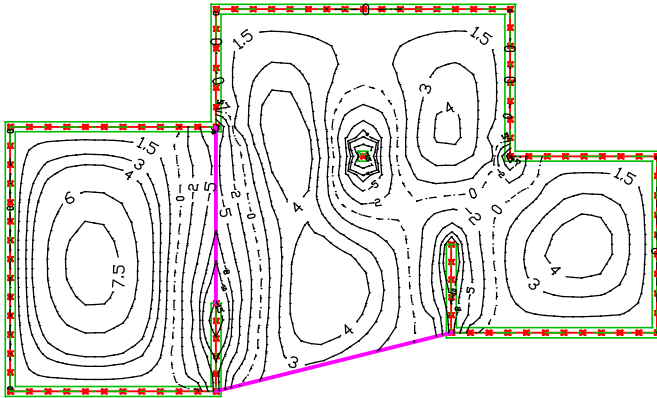
Enable/disable the element mesh when displaying the contours.

### Extreme values

Enable/disable the local extreme values.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.



Contour representation of the internal forces  $m_x$

## Numeric

This function is used to display all results in the result location numerically.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

### Reinforcement subtraction [cm<sup>2</sup>/m] x-direction or y-direction

The set reinforcement is subtracted from the computed reinforcement. Only the remaining reinforcement is displayed. An additional button enables you to modify the offered list of reinforcement subtractions.

### Lower boundary for presentation

Only results exceeding the limit value are output.

### Mean of values

The displayed results are averaged at the results location if all adjacent elements have the same properties.

### Mean

- *No*: The displayed results are output at the result location.
- *Node*: The displayed results are averaged at the result location, if all adjacent elements have the same properties.
- *Element*: The displayed results are the averaged values of the respective element.

### Text height [mm]

The unit depends on the output device.

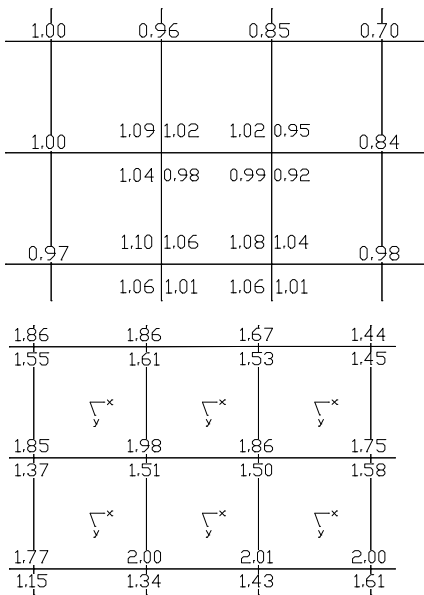
### Decimal places

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

### Coordinate system in the elements

Selection when displaying a longitudinal reinforcement:

- *If not equal to global system*: The reinforcement system is only displayed, if it is not global.
- *Off*: The reinforcement system is **not** displayed.
- *On*: The reinforcement system is **always** displayed.



Numeric representation of the principle stresses  $\sigma_1$  in an area with differing element properties with activated averaging.

Numeric representation of the lower reinforcement layer in x direction (top number) and y direction (lower number) and output of the reinforcement system.

## Section View

For area and solid elements the results are applied through user-defined intersections. After selecting the representation type *Section* the or icon in the results bar can be used to enter or delete intersections.

### Section

Choose a saved group of intersections. The selected group is deleted using the *Del* key.

### Save...

Save the existing intersections by assigning them a name. Changes to a loaded group are indicated by an asterisk.

### Scale

The scale of the ordinate display can be set automatically or manually.

### Label

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.
- *Everywhere*: Label everywhere.
- *Selectable*: Choose manual labeling with the help of the results bar.

### Limit value for labeling

Only ordinates exceeding the limit value are labeled.

### Show integral

Show the integral of the results for every section.

### Decimal places

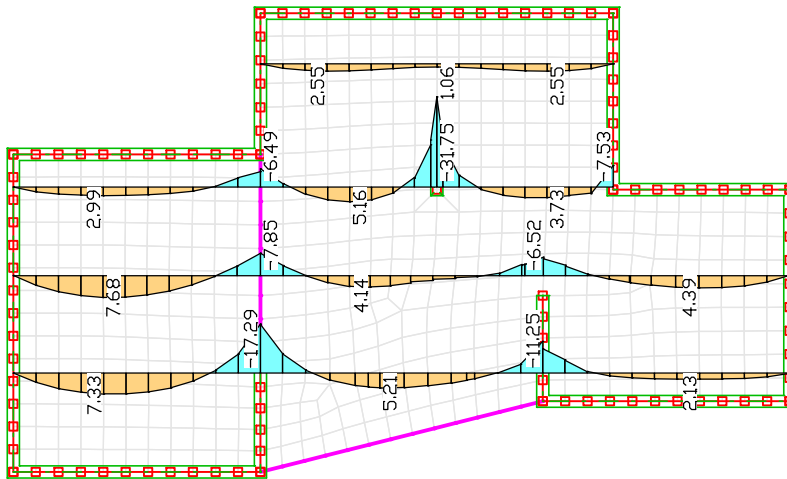
- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

### Text height [mm]

The unit depends on the output device.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.



Section representation of the internal forces  $m_x$

The video "[https://download.infograph.de/video\\_de/Ergebnisdarstellung%20Schnittkurve.mp4](https://download.infograph.de/video_de/Ergebnisdarstellung%20Schnittkurve.mp4)" (German language) shows the handling of section curves.

## Vectors

The extremal principal stresses, internal forces or strains of area elements are displayed vectorially in the centroid in the direction of action. In addition, the strain  $\epsilon_{\text{crack}}$  in the crack direction can be plotted.

The principal stresses of solid elements are displayed vectorially in the centroid. For load case results they are average values, for combination results they are extreme values of the element.

The extremal resulting lateral forces  $q_r$  of a load case can also be represented vectorially in the element plane.

### Colored

Positive values are drawn red and negative values blue. If the b/w representation is active the values of the first direction are drawn with a solid line and the values of the second direction with a dashed line.

### Scale

The scale of the vectors can be set automatically or manually.

### Label

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.

### Limit value for labeling

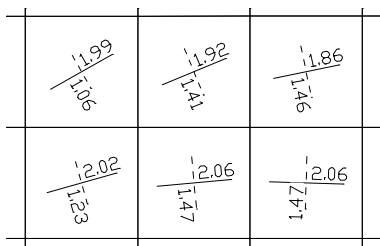
Only ordinates exceeding the limit value are labeled.

### Decimal places

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.



### Text height [mm]

The unit depends on the output device.



Vector representation of the extremal principal stresses

## Solid Section

For solid elements the results are applied through user-defined intersections. After selecting the representation type *Solid section* the  or  icon in the results bar can be used to enter or delete intersections.

### Section

Choose a saved group of intersections. The selected group is deleted using the *Del* key.

### Save...

Save the existing intersections by assigning them a number. Changes to a loaded group are indicated by an asterisk.

### Number of colors

Set the number of colors for automatic color grading.

### Color shift

Shift of the automatic color grading.

### Grading

Enable/disable manual color grading. Color and number values can be changed. The grading can be saved as a template for other projects with the right mouse button. The context option *Database* allows you to manage different color gradings in the *Igraph.dat* file.

The  icon is used to add another color grade above the selected one.

The  icon is used to delete the selected color grade.

### Legend

Enable/disable the color legend.

### Mean values

The displayed results are averaged in the results location if all adjacent elements have the same properties.

### Dividing line

Enable/disable dividing lines at the color transitions.

## Support Lines

The basis for all representations are the support reactions [kN or kNm] computed in the supported nodes. These are each defined in the support system. For the linear display of the support reactions, support lines are created. The support lines are made up of the successive supported element edges. The support reactions are transformed into the coordinate system of the support lines and distributed along the respective bearing width.

The coordinate system of the support lines is defined as follows:

- The x axis shows the direction of the support lines.
- The z axis depends on the orientation of the supports.

### Mean

Alternatively, the ordinates of the linear representation can be averaged in the following ways:

- *Linear*: The forces and moments are in equilibrium with the representation without averaging (trapezoidal form).
- *Constant*: The forces are in equilibrium with the representation without averaging (rectangular form).

As different load cases are often involved in the extremal reactions of the individual supports in load case combinations, averaging can, in certain situations, result in an unrealistic sum of the support forces.

### Scale

The scale of the ordinate display can be set automatically or manually.



**Label**

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.
- *Everywhere*: Label all ordinates.
- *Selectable*: Choose manual labeling with the help of the results bar.

**Limit value for labeling**

Only ordinates exceeding the limit value are labeled.

**Show integral**

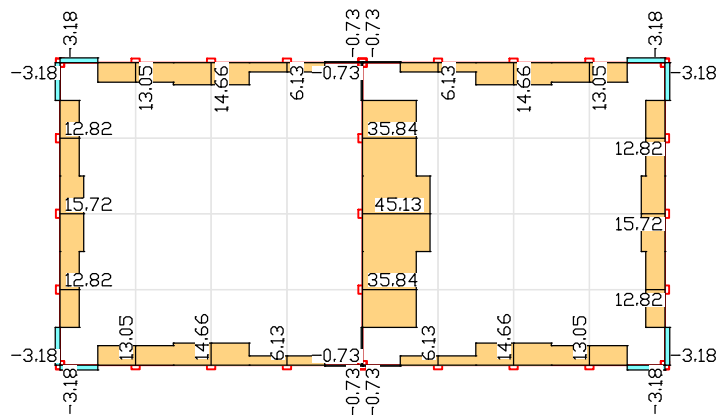
Show the integral of the results for every support line.

**Decimal places**

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

**Text height [mm]**

The unit depends on the output device.



*Linear representation of the support reactions*

**Result Graphs**

The results are plotted along either the object axis for beams and design objects or the secants of the spline for tendon groups. A linear progress of results is assumed between the result locations.

**Scale**

The scale of the ordinate display can be set automatically or manually.

**Label**

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.
- *Everywhere*: Label all ordinates.
- *Selectable*: Choose manual labeling with the help of the results bar.

**Limit value for labeling**

Only ordinates exceeding the limit value are labeled.

**Decimal places**

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

**Text height [mm]**

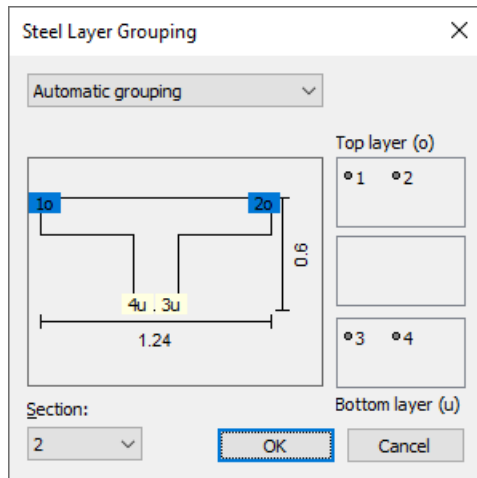
The unit depends on the output device.

### Show ordinates in global z direction

This selection refers to the results for tendon groups that are displayed by default in the principal bending plane of the spline.

### Steel layer grouping...

Define the grouping of the steel layers with this function.



During automatic grouping, all steel layers with a z component smaller than the z component of the centroid are displayed on the upper side of the elements, all other layers on the lower side.

For manual grouping, steel layers can be moved between the list boxes using the mouse. The steel layers of the middle list box are not displayed.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

## Line Hinges

The results are presented along the line hinges. The representations are based on the spring forces determined in the nodes. When displaying the spring forces, these are transformed into the coordinate system of the line hinges and distributed over the respective reference width.

### Scale

The scale of the ordinate display can be set automatically or manually.

### Label

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.
- *Everywhere*: Label all ordinates. Line hinges
- *Selectable*: Choose manual labeling with the help of the results bar.

### Limit value for labeling

Only ordinates exceeding the limit value are labeled.

### Show integral

Show the integral of the results for every Line hinges.

### Decimal places

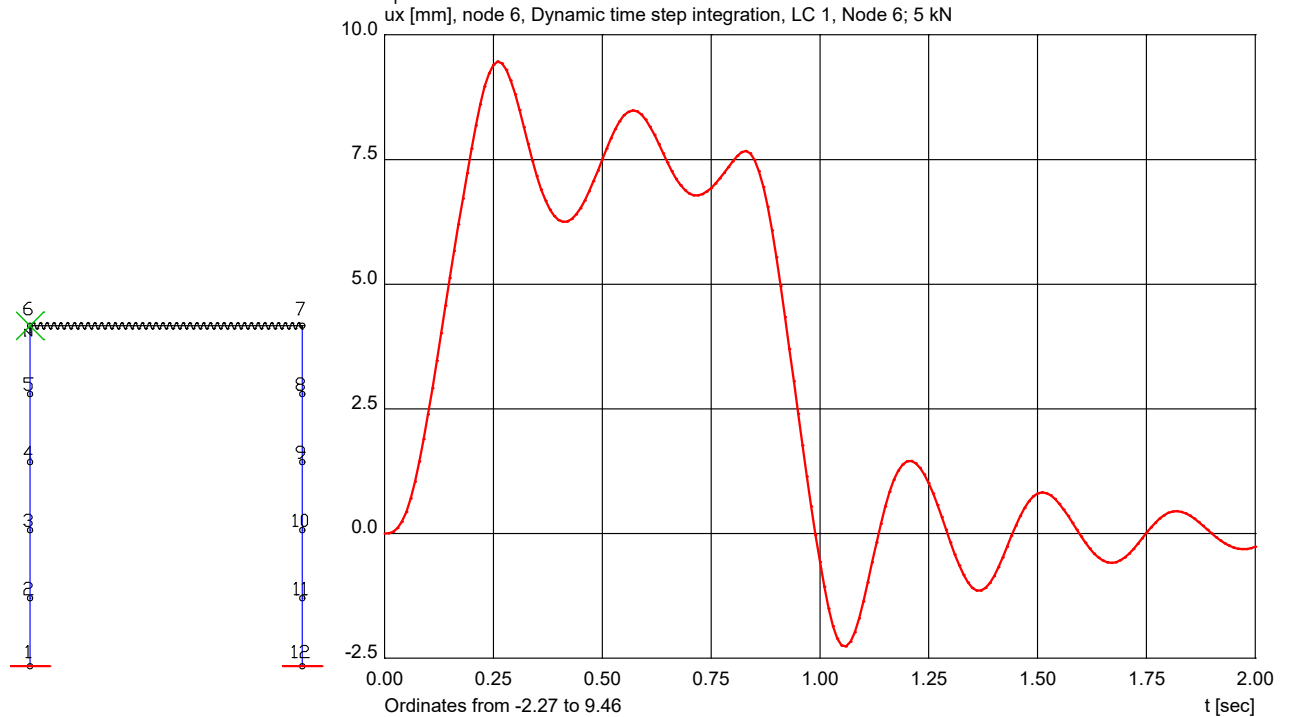
- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

### Text height [mm]

The unit depends on the output device.

## Result Diagram

The results of dynamic calculations or a thermal analysis are shown in a diagram. For the results of a time step integration, the desired result location has to be specified on the structure.



System response  $u_x$  to a instationary excitation

With the context option *Copy diagram values* of the diagram the illustrated pairs of values can be copied to the clipboard.

## Section Stresses

The stress curve is illustrated on a beam section in a dialog window. This can be accessed via the context menu of the beams when displaying result graphs for stresses or internal forces. The stress curve can also be displayed at points with result markers for composite section stresses. Only the stresses at the markers include the internal stresses in the cross-section.

### Load case

Select the calculated load cases and combinations.

### Set

Select the internal force set for the stress determination. The absolute extreme values of the stresses from all internal force sets are displayed using *Extreme value*.

### Beam

Number of the selected beam.

### Location

Select the result location in the beam.

### Section

Description of the displayed section.

### Nx, Qy, Qz, Mx, My, Mz

Internal forces at the selected result location, from which the displayed stress curve results. Due to the internal stresses in the cross-section, the stresses cannot always be calculated from the internal forces, so they are not displayed for composite section stresses on result markers.

## Stress

The stresses are generally calculated while taking the linear elastic material behavior into consideration. For results from a nonlinear system analysis, the stresses of the plastic state are displayed on polygon sections and steel sections made of construction steel (S235 to S450 or S235-EN to S450-EN) or *Stahl*. The stored stresses of a nonlinear system analysis can also be displayed on result markers for composite section stresses.

- $\sigma_x$ : Longitudinal stresses from bending moments and longitudinal force.
- $\sigma_v$ : Comparison stresses according to the Huber-von Mises yield criterion.
- $\sigma_{1,2}$ : Main stresses from bending moments, longitudinal force, lateral forces and torsion.
- $\tau_r$ : Shear stresses from lateral forces and torsion parallel to the edge.
- $\tau_{xy}$ : Shear stresses from lateral forces and torsion in y direction.
- $\tau_{xz}$ : Shear stresses from lateral forces and torsion in z direction.

## Label, Text height

Labeling of extreme values. With the right mouse button the labels for chosen ordinate values can be optionally activated or deactivated.

## Print

Print the section stresses. Optionally, the size of the printout (*Print size: height, width*) can be defined.

## ->Print list

Insert the current view in the printing list.

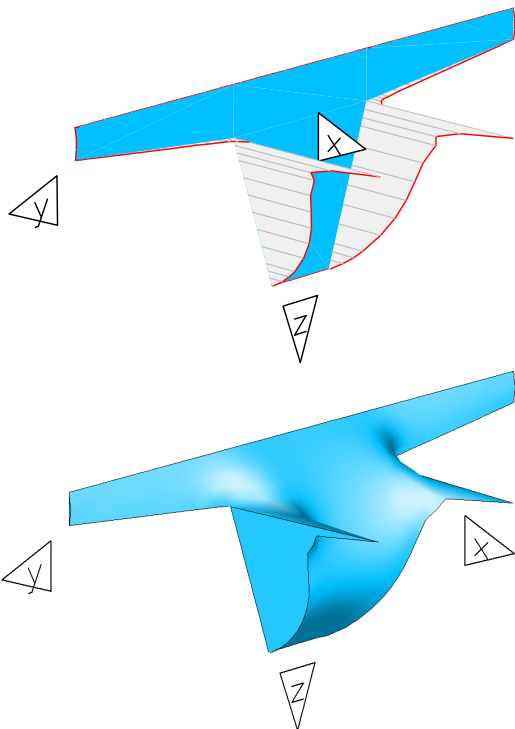
## Representation

The stresses can be represented either as lines along the edge or as a 3D face.

The view can be rotated with the help of the mouse. The area displayed can be moved by holding down the Shift key. Use the Ctrl key to change the size of the zoom section. If you place the mouse pointer over an ordinate and wait a moment, the stress present at that location is displayed.

## Section stress $\tau_{xz}$ (Elastic)

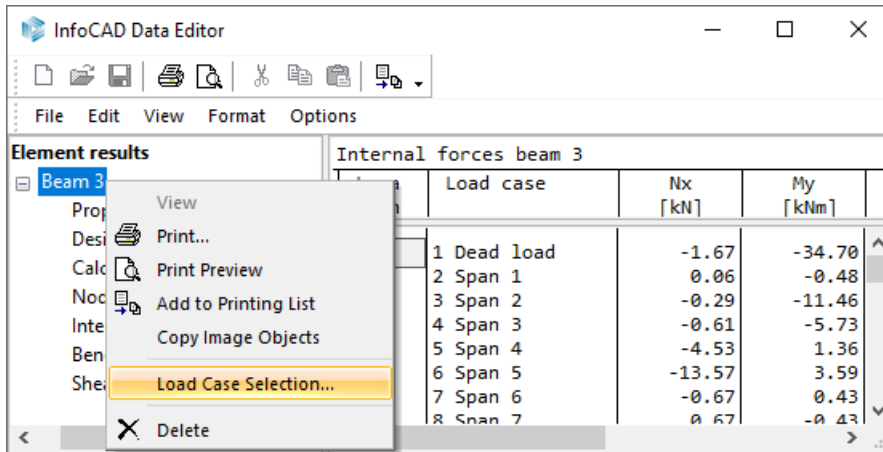
Beam 1, result point 1, section Polygon - C20/25  
 Load case 1  
 $N_x / Q_y / Q_z = 0.00 / -0.00 / 5865.00$  kN;  $M_x / M_y / M_z = -0.00 / -58650.00 / 0.00$  kNm  
 Range of values of the stresses: -0.00157631 bis 4.18786 MN/m<sup>2</sup>.



Shear stress  $\tau_{xz}$  represented as lines along the edge and as 3D face

## Result List

The result list is output for selected elements in the data editor. It can be accessed over the context menu for the elements.



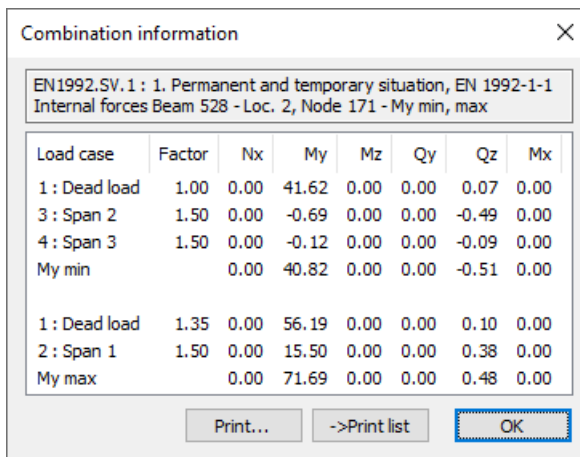
The output can be limited to certain load cases using the *Load Case Selection* option of the context menu.

## Combination Information

The *Combination information* option can be used to determine the load cases involved in a combination result with their respective weightings. This function is available during the display of combination results. After the selection of the desired result location with the right mouse button, the *Combination information* function appears in the context menu.

The following functions are available in the context menu of the dialog:

- *Copy (clipboard)*
- *Create load group...*
- *Create superposition load case*
- *Associated deformations...*
- *Associated support forces...*



### Last Combination

Shows result values corresponding to the last *Combination information*.

## Reinforcement Export

The displayed bending reinforcement of the area elements (1st and 2nd layer of both directions) is written to a file for transfer to construction programs.

The following formats are available:

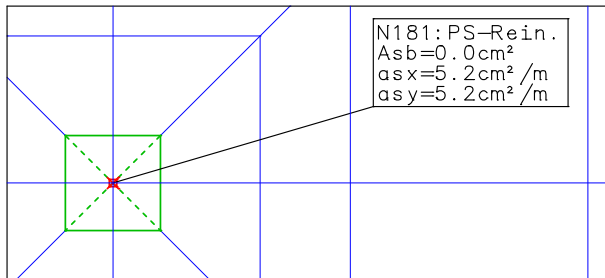
- ALLPLOT transfer files (\*.asf)
- STRAKON transfer files (\*.cfe)
- ISBCAD transfer files (\*.fem)
- ZEICON transfer files (fax\*)

These formats require a plane area structure. For other structures it makes sense to export the reinforcement for one plane after another. If the selected elements are not on the same plane, a projection plane has to be defined. If required, also a local coordinate system has to be defined.

Averaged values are transferred for calculation at the nodes or side middles.

## Punching Shear Check

The punching shear check can be accessed from the *Structure* menu and is performed interactively for selected element nodes. The following figure shows a punching object on a column. The required bending and shear reinforcement is noted.



*Punching object*

### Input

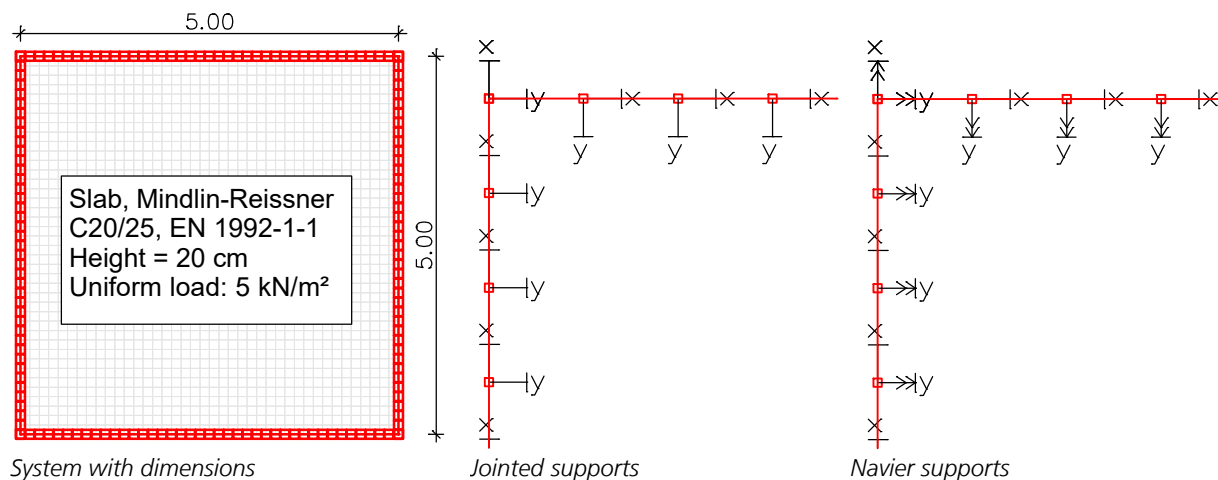
- Off* Turns off the representation of punching objects.
- Listing* Log and show punching shear check for all punching objects. The log is stored in the database.
- Enter* Create a punching object at the selected node and perform the punching shear check.

The standard used for the check is chosen according to the concrete class used. A selection dialog appears if this cannot be identified automatically. More information can be found in the respective chapters about Reinforced Concrete Design.

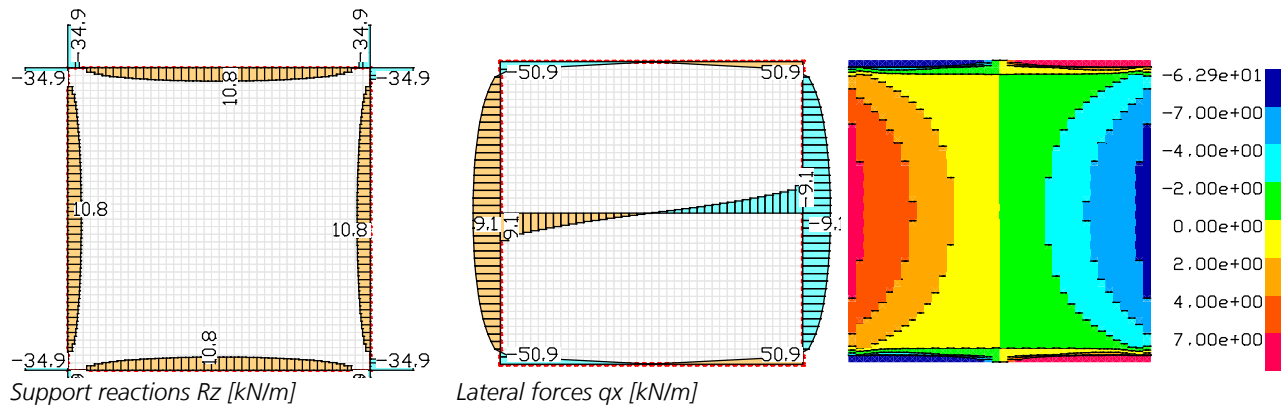
# Examples

## Slab With Mindlin-Reissner Elements

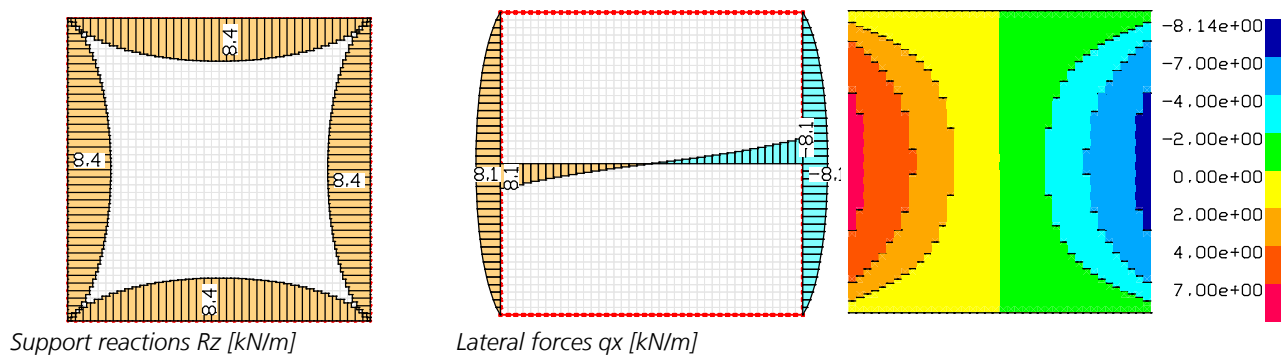
This example shows the lateral force behavior of the Mindlin-Reissner elements. The type of support has a decisive effect on the progress of the lateral forces. The following pictures show comparably the results of a jointed and a Navier support.



### Jointed supports

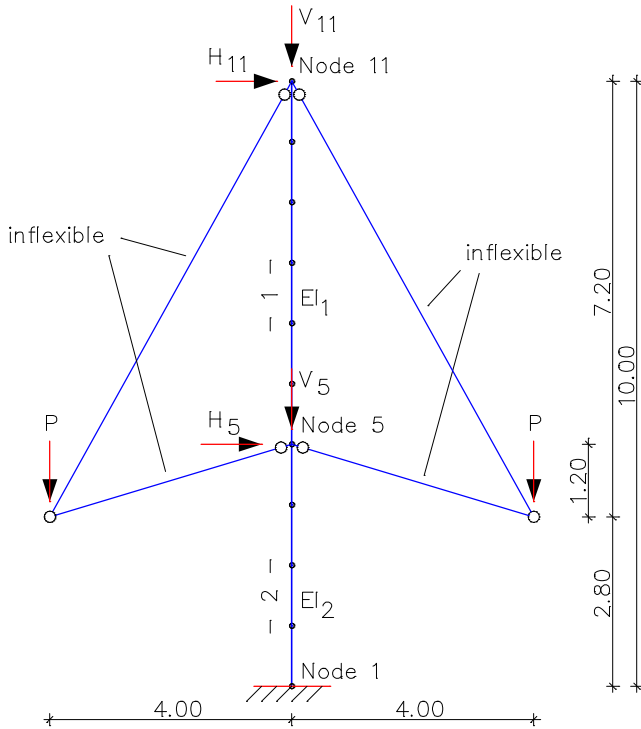


### Navier supports



# Buckling Eigenmodes of a Column

The illustrated structure was analyzed by *Rubin and Aminbaghai* (1997). To detect buckling eigenmodes of structures, the load type *Pki* should be entered in the corresponding load case.



- Case 1:  
 $EI_1 = 15000 \text{ kNm}^2$ ,  $EI_2 = 20000 \text{ kNm}^2$   
 $H_{11} = 3 \text{ kN}$ ,  $V_{11} = 200 \text{ kN}$   
 $H_5 = 2 \text{ kN}$ ,  $V_5 = 300 \text{ kN}$   
 $P = 400 \text{ kN}$
- Case 2:  
 $EI_1 = EI_2 = 15000 \text{ kNm}^2$   
 $H_{11} = 3 \text{ kN}$ ,  $V_{11} = 0 \text{ kN}$   
 $H_5 = 2 \text{ kN}$ ,  $V_5 = 0 \text{ kN}$   
 $P = 400 \text{ kN}$

**Material properties**

No.	Mat. type	EModule [MN/m <sup>2</sup> ]	GModule [MN/m <sup>2</sup> ]	Poisson ratio	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]
1	S235	210000	81000	0.30	1.200e-05	78.500
2	S235	210000	81000	0.30	1.200e-05	78.500
3	S235	210000	81000	0.30	1.200e-05	78.500

**Section properties**

- No. 1 ST  
 $A = 1.000e+00 \text{ [m}^2\text{]}$   
 $I_x = 1.000e+00 \text{ [m}^4\text{]}$ ,  $I_y = 7.140e-05 \text{ [m}^4\text{]}$ ,  $I_z = 1.000e+00 \text{ [m}^4\text{]}$
- No. 2 ST  
 $A = 1.000e+00 \text{ [m}^2\text{]}$   
Case 1:  $I_x = 1.000e+00 \text{ [m}^4\text{]}$ ,  $I_y = 9.520e-05 \text{ [m}^4\text{]}$ ,  $I_z = 1.000e+00 \text{ [m}^4\text{]}$   
Case 2:  $I_x = 1.000e+00 \text{ [m}^4\text{]}$ ,  $I_y = 7.140e-05 \text{ [m}^4\text{]}$ ,  $I_z = 1.000e+00 \text{ [m}^4\text{]}$
- No. 3 ST  
 $A = 1.000e+00 \text{ [m}^2\text{]}$   
 $I_x = 1.000e+00 \text{ [m}^4\text{]}$ ,  $I_y = 1.000e+00 \text{ [m}^4\text{]}$ ,  $I_z = 1.000e+00 \text{ [m}^4\text{]}$

**Node description**

No.	x [m]	y [m]	z [m]
1	0.000	0.000	0.000
2	0.000	0.000	-1.000
3	0.000	0.000	-2.000
4	0.000	0.000	-3.000
5	0.000	0.000	-4.000
6	0.000	0.000	-5.000
7	0.000	0.000	-6.000
8	0.000	0.000	-7.000
9	0.000	0.000	-8.000
10	0.000	0.000	-9.000
11	0.000	0.000	-10.000
12	4.000	0.000	-2.800
13	-4.000	0.000	-2.800

**Beam elements**

No.	Ele-ment type	Start Node	End Node	Angle [°]	CrS	Mat	Joints				Length [m]	Layer
							Q xyz	M xyz	Q xyz	M xyz		
1	RS	1	2	-0.00	2	2	0	0	0	0	1.000	2
2	RS	2	3	-0.00	2	2	0	0	0	0	1.000	2
3	RS	3	4	-0.00	2	2	0	0	0	0	1.000	2
4	RS	4	5	-0.00	2	2	0	0	0	0	1.000	2
5	RS	5	6	-0.00	1	1	0	0	0	0	1.000	2
6	RS	6	7	-0.00	1	1	0	0	0	0	1.000	2
7	RS	7	8	-0.00	1	1	0	0	0	0	1.000	2
8	RS	8	9	-0.00	1	1	0	0	0	0	1.000	2
9	RS	9	10	-0.00	1	1	0	0	0	0	1.000	2
10	RS	10	11	-0.00	1	1	0	0	0	0	1.000	2
11	RF	5	12	-0.00	3	3	0	0	0	0	4.176	2
12	RF	12	11	-0.00	3	3	0	0	0	0	8.237	2
13	RF	5	13	-0.00	3	3	0	0	0	0	4.176	2
14	RF	13	11	-0.00	3	3	0	0	0	0	8.237	2

**Supports**

Node no.	Angle [°]	x	y	z	ux	uy	uz	phi.x	phi.y	phi.z	Ten-sion loss
1	0.0	0.0	0.0	0.0	F	F	F	F	F	F	L:x-y-z
2	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
3	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
4	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
5	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
6	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
7	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
8	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
9	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
10	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
11	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
12	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z
13	0.0	0.0	0.0	0.0	-	F	-	F	-	F	L:x-y-z



**Load data Load case 1 (Case 1)**

```

KNL : Nodal load
Node
from to      Px      Py      Pz      Mx      My      Mz
      [kN]      [kNm]
      5      5      2.00   0.00  300.00  0.00   0.00   0.00
      11     11     3.00   0.00  200.00  0.00   0.00   0.00
      12     13     0.00   0.00  400.00  0.00   0.00   0.00
PKI : Buckling eigenvalues and mode shapes
      Number of eigenvalues 4
    
```

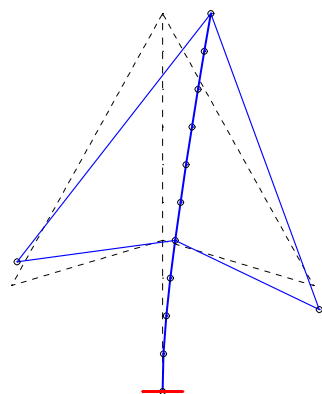
**Load data Load case 2 (Case 2)**

```

KNL : Nodal load
Node
from to      Px      Py      Pz      Mx      My      Mz
      [kN]      [kNm]
      5      5      2.00   0.00   0.00   0.00   0.00   0.00
      11     11     3.00   0.00   0.00   0.00   0.00   0.00
      12     13     0.00   0.00  400.00  0.00   0.00   0.00
PKI : Buckling eigenvalues and mode shapes
      Number of eigenvalues 4
    
```

The stability analysis delivers, in complete agreement with the above publication, the following buckling eigenmodes and results:

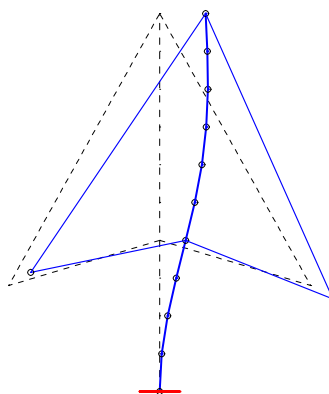
Case 1



Buckling load factor = 2.14

Buckling lengths:  $sk_{\text{group 1}} = 7.724 \text{ m}$   
 $sk_{\text{group 2}} = 8.425 \text{ m}$

Case 2



Buckling load factor = 3.58

Buckling lengths:  $sk_{\text{group 1}} = 6.559 \text{ m}$   
 $sk_{\text{group 2}} = 7.185 \text{ m}$

**Excerpt from the finite elements log**

**Load factors of the mode shapes load case 2**

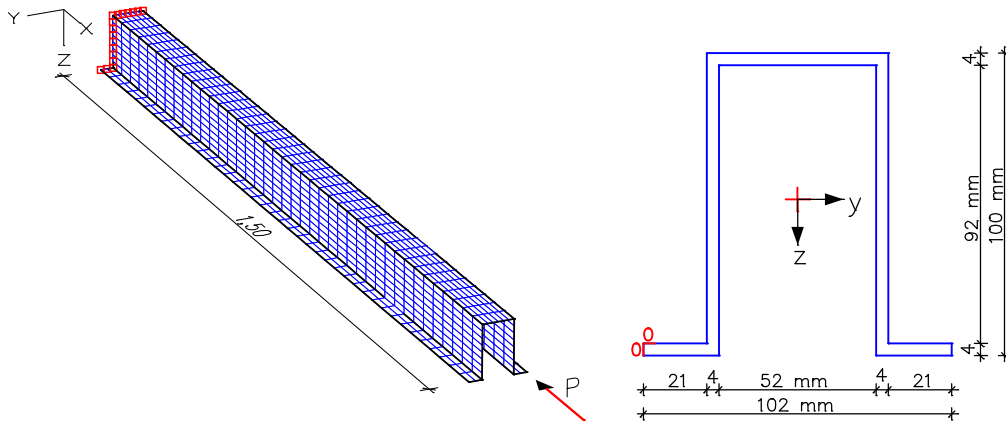
No.	load factor	rel. tol.
1	3.5836e+00	6.2366e-07
2	7.0151e+00	6.2249e-07
3	1.9320e+01	4.1153e-07
4	3.2522e+01	6.4108e-07

**Buckling length's load case 2**

Beam	Pki [kN]	EIy [kNm <sup>2</sup> ]	sk.y [m]	EIz [kNm <sup>2</sup> ]	sk.z [m]
1	-2866.85	1.4994e+04	7.185	2.1000e+08	850.270
2	-2866.85	1.4994e+04	7.185	2.1000e+08	850.270
3	-2866.85	1.4994e+04	7.185	2.1000e+08	850.270
4	-2866.85	1.4994e+04	7.185	2.1000e+08	850.270
5	-3440.22	1.4994e+04	6.559	2.1000e+08	776.187
6	-3440.22	1.4994e+04	6.559	2.1000e+08	776.187
7	-3440.22	1.4994e+04	6.559	2.1000e+08	776.187
8	-3440.22	1.4994e+04	6.559	2.1000e+08	776.187
9	-3440.22	1.4994e+04	6.559	2.1000e+08	776.187
10	-3440.22	1.4994e+04	6.559	2.1000e+08	776.187

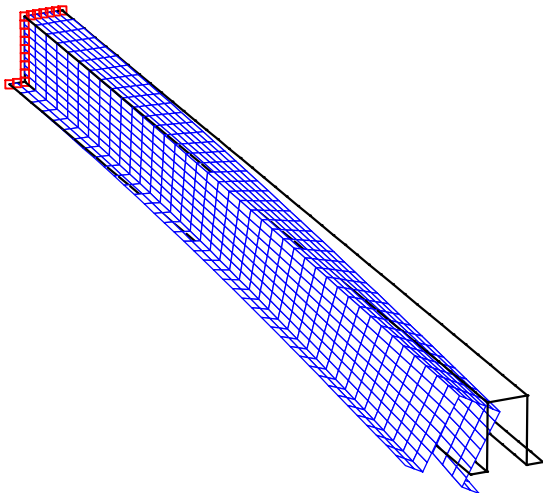
## Lateral Torsional Buckling of a Shell Structure

The following shell model of a 1.5 m long compression beam is supported on one side in a fixed manner according to Euler case 1. The profile and system dimensions are illustrated below. The system is made of S235 construction steel.

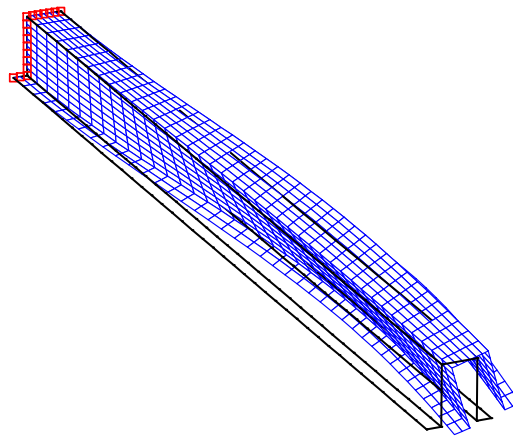


According to Peterson, the ideal buckling load is (1980, formula according to Steinbach)  $P_{Ki} = 56.4$  kN. The load P is represented by a line load along the element edges on the face. Its load sum is equivalent to 1 kN.

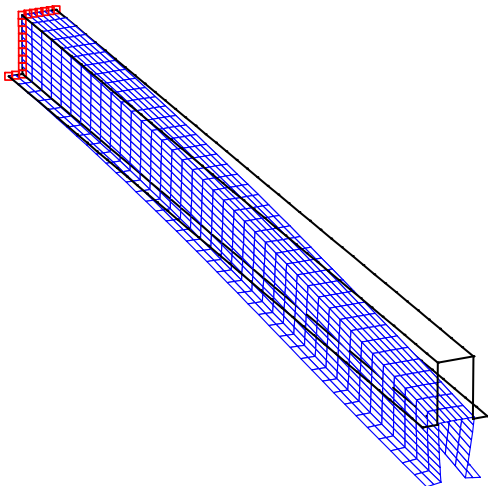
For the system detailed above, the FEM calculation delivers the following results for the first 4 buckling eigenmodes. Influences such as shear strain and warping are taken into account.



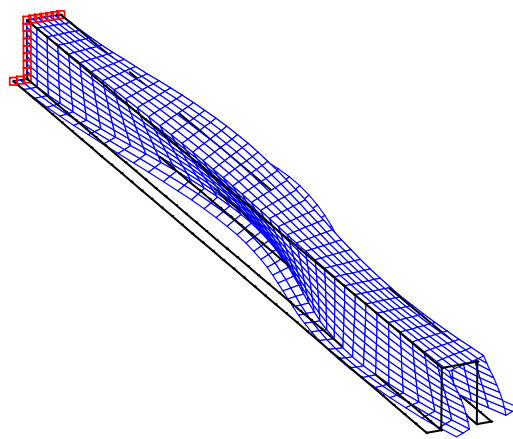
Buckling eigenmode 1, load factor = 54.14  
(torsional buckling)



Buckling eigenmode 2, load factor = 190.52  
(buckling around the z axis of the section)



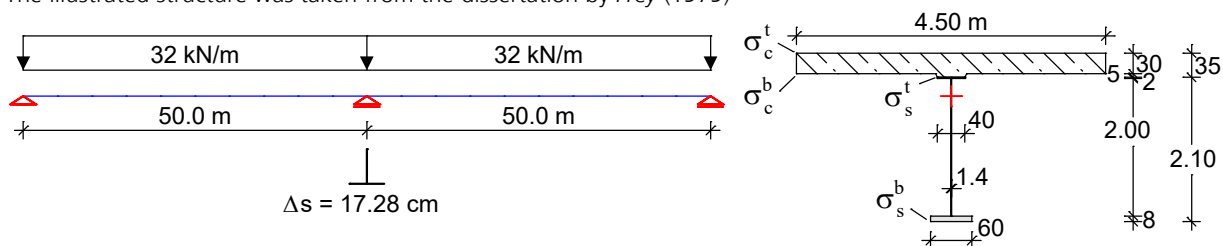
Buckling eigenmode 3, load factor = 345.88  
(buckling around the y axis of the section)



Buckling eigenmode 4, load factor = 422.59  
(buckling around the z axis, second eigenmode)

## Two-span Steel Composite Girder

The illustrated structure was taken from the dissertation by Frey (1979)

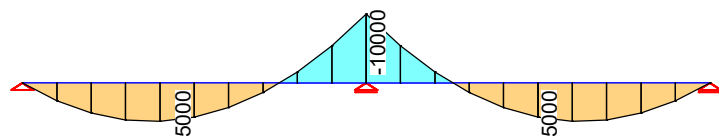


System with dimensions, load and cross-section

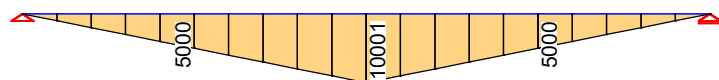
### Material properties and cross-section dimensions Load

Steel:	St 37; $E_s = 210 \cdot 10^3 \text{ MN/m}^2$	1st load case,	permanent load: $g = 32 \text{ kN/m}$
Concrete:	B 35; $E_b = 34 \cdot 10^3 \text{ MN/m}^2$	2nd load case,	lowering of the center support: $\Delta s = 17.28 \text{ cm}$
	Creep coefficient: $\varphi_\infty = 3.9$	3rd load case,	creep causing permanent load: $g + \Delta s$
	Shrinkage coefficient: $\epsilon_{cs,\infty} = -20 \cdot 10^{-5}$	4th load case,	creep and shrinkage: Creep of permanent load (LC 3)
	Relaxation coefficient: $\rho = 0.8$	5th load case,	superposition of LC 3 and LC 4

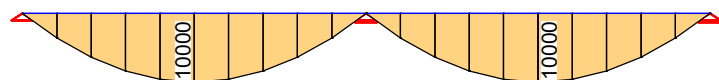
### Results



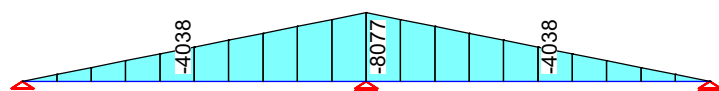
Load case 1, permanent load: Internal forces  $M_y$



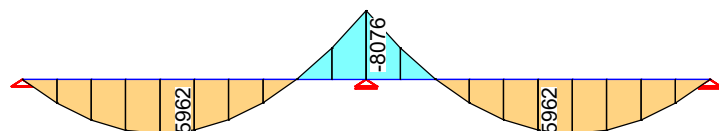
Load case 2, lowering of the center support:  $\Delta s = 17.28 \text{ cm}$ : Internal forces  $M_y$



Load case 3, creep causing permanent load:  $g + \Delta s$ : Internal forces  $M_y$



Load case 4, creep and shrinkage: Internal forces  $M_y$



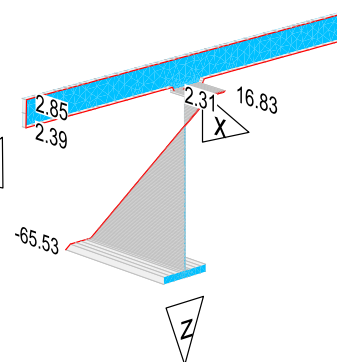
Load case 5, superposition of LC 3 and LC 4: Internal forces  $M_y$

Stresses at the center support

Load case	$\sigma_c^t$	$\sigma_c^b$	$\sigma_s^t$	$\sigma_s^b$
1. g	4.40	2.28	11.91	-79.53
2. $\Delta s$	-4.40	-2.28	-11.91	79.53
3. $g + \Delta s$	0	0	0	0
4. CS	2.85	2.39	16.83	-65.53
Frey	2.87	2.40	17.09	-66.03

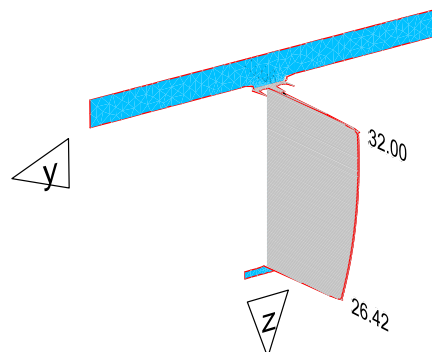
### Section stress $\sigma_{x,x}$ (Elastic)

Beam 11,  $x_i = 0.00$ , section 3 - Composite  
Load case 5 Superposition of LC 3 and LC 4  
Range values of the stresses: -65.5261 to 16.8312 MN/m<sup>2</sup>



### Section stress $\tau_{x,z}$ (Elastic)

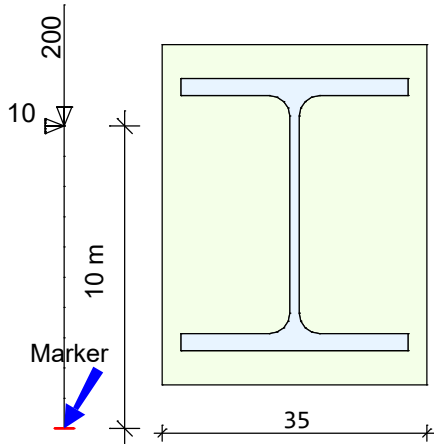
Beam 11,  $x_i = 0.00$ , section 3 - Composite  
Load case 5 Superposition of LC 3 and LC 4  
Range values of the stresses: -0.270943 to 31.9979 MN/m<sup>2</sup>



# Composite Column

The illustrated structure of a 10 m high composite column with normal force and bending is used as an example to show the calculation model and its results. To obtain the composite section stresses (internal stresses) due to creep and shrinkage, a corresponding result marker was set at the fixing point. The cross-sectional stresses at this marker are shown below.

## DEAD LOAD



Concrete class: C35/45-EN

Steel girder: HEB 360

Load: Dead load, imposed load 200 kN and horizontal load 10 kN

### Ideal cross-section values

#### Steel girder

$$A_s = 1.81 \cdot 10^{-2} \text{ m}^2$$

$$I_{y,s} = 4.319 \cdot 10^{-4} \text{ m}^4$$

$$E_s = 210000 \text{ MN/m}^2$$

#### Concrete

$$A_c = 0.13938 \text{ m}^2$$

$$I_{y,c} = 2.2246 \cdot 10^{-3} \text{ m}^4$$

$$E_c = 34000 \text{ MN/m}^2$$

Creep value:  $\varphi_\infty = 1.55$

Shrinkage value:

$$\varepsilon_{cs,\infty} = -27.8 \cdot 10^{-5}$$

Relaxation parameter:

$$\rho = 0.8$$

#### Composite Section

$$A = 0.25117 \text{ m}^2$$

$$I_y = 4.8922 \cdot 10^{-3} \text{ m}^4$$

Clamped column with rolled girder in concrete

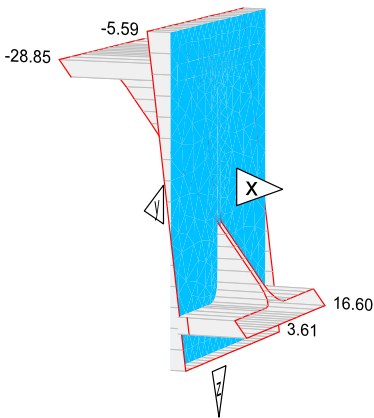
## Load

- 1st load case, permanent load: dead load, imposed load and horizontal load (see upper figure)
- 2st load case, creep and shrinkage according to Trost under permanent load (LC 1)
- 3st load case, superposition of LC 1 and LC 2

## Results at the fixing point

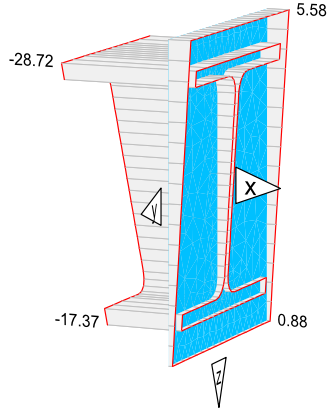
### Section stress $\sigma_{x,x}$ (Elastic)

Beam 10, xi = 1.00, section 3 - Composite  
Load case 1 Permanent load  
Range values of the stresses: -28.8495 to 16.6008 MN/m<sup>2</sup>



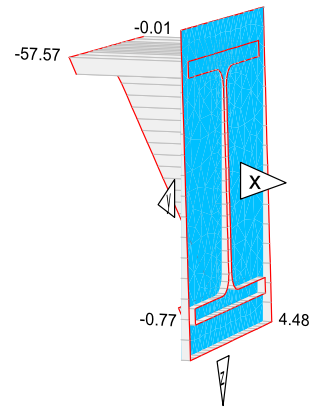
### Section stress $\sigma_{x,x}$ (Elastic)

Beam 10, xi = 1.00, section 3 - Composite  
Load case 2 Creep and shrinkage  
Range values of the stresses: -28.7185 to 5.57815 MN/m<sup>2</sup>



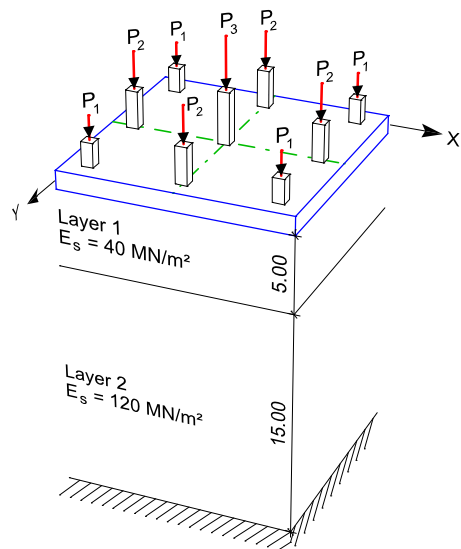
### Section stress $\sigma_{x,x}$ (Elastic)

Beam 10, xi = 1.00, section 3 - Composite  
Load case 3 Sup 1+2  
Range values of the stresses: -57.568 to 4.48297 MN/m<sup>2</sup>



## Silo Foundation Based on the Modulus of Compressibility Method

The use of this method for slab calculations is illustrated based on the example of a silo surface foundation (H.Ahrens and Winselmann, 1984). The four silo cells are led over nine constant area loads into the 0.8 m thick, 10 x 10 m foundation slab that rests on the layered subsoil. The slab measurements and the load contact areas can be seen in the illustration.



$$P_1 = 520 \text{ kN}$$

$$P_2 = 800 \text{ kN}$$

$$P_3 = 1120 \text{ kN}$$

Slab: C 20/25, EN 1992-1-1

Element thickness  $d = 0.8 \text{ m}$

### Foundation slab with load

The slab elements must be assigned a material with a bedding modulus in z direction ( $k_{bz}$ ). It is needed as a start value for the calculation. The ultimate depth (rock) of the halfspace can be given a very high modulus of compressibility (100000 MN/m<sup>2</sup>, for example) in the last soil layer.

### Material properties

No.	Material type	EModule [MN/m <sup>2</sup> ]	GModule [MN/m <sup>2</sup> ]	Poisson ratio	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]
1	C20/25-EN	30000	12500	0.20	1.000e-05	25.000

### Bedding properties

Sec. No.	k <sub>bx</sub> -a	k <sub>bx</sub> -e	k <sub>by</sub> -a	k <sub>by</sub> -e	k <sub>bz</sub> -a	k <sub>bz</sub> -e	b <sub>x</sub>	b <sub>y</sub> [m]	b <sub>z</sub>
1	0	0	0	0	10	10	1.00	1.00	1.00

### Soil properties for the stiffness module method

The calculation is based on the modulus of compression method

Layer no.	from z [m]	Module Es [MN/m <sup>2</sup> ]
1	0.00	40.00
2	5.00	120.00
3	20.00	100000.00

The last layer extends up to  $z = \text{infinite}$ .

### Section properties

No. 1 FL  $d = 0.8$

**Load data Load case 1 (Silo load)**

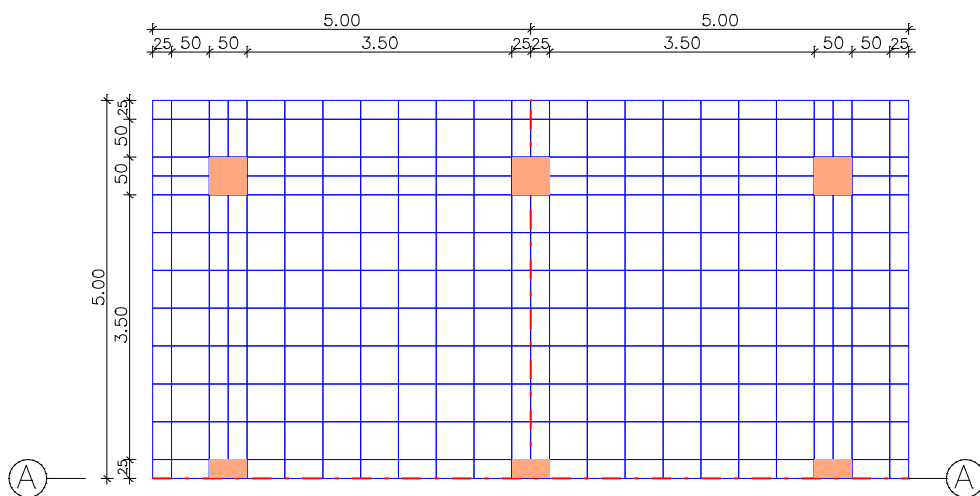
GR2 : Uniformly distributed load - rectangular area (global)

	x [m]	y	z	qx [kN/m <sup>2</sup> ]	qy	qz
1. Point:	0.750	0.750	0.000	0.00	0.00	2080.00
2. Point:	1.250	0.750	0.000			
3. Point:	1.250	1.250	0.000			
1. Point:	4.750	0.750	0.000	0.00	0.00	3200.00
2. Point:	5.250	0.750	0.000			
3. Point:	5.250	1.250	0.000			
1. Point:	8.750	0.750	0.000	0.00	0.00	2080.00
2. Point:	9.250	0.750	0.000			
3. Point:	9.250	1.250	0.000			
1. Point:	0.750	4.750	0.000	0.00	0.00	3200.00
2. Point:	1.250	4.750	0.000			
3. Point:	1.250	5.250	0.000			
1. Point:	4.750	4.750	0.000	0.00	0.00	4480.00
2. Point:	5.250	4.750	0.000			
3. Point:	5.250	5.250	0.000			
1. Point:	8.750	4.750	0.000	0.00	0.00	3200.00
2. Point:	9.250	4.750	0.000			
3. Point:	9.250	5.250	0.000			
1. Point:	0.750	8.750	0.000	0.00	0.00	2080.00
2. Point:	1.250	8.750	0.000			
3. Point:	1.250	9.250	0.000			
1. Point:	4.750	8.750	0.000	0.00	0.00	3200.00
2. Point:	5.250	8.750	0.000			
3. Point:	5.250	9.250	0.000			
1. Point:	8.750	8.750	0.000	0.00	0.00	2080.00
2. Point:	9.250	8.750	0.000			
3. Point:	9.250	9.250	0.000			

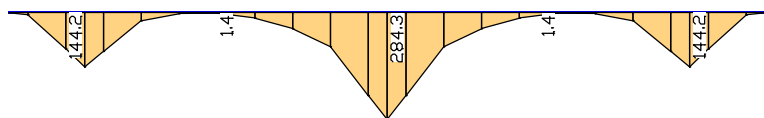
TH : Calculation theory 1. order

**Global equilibrium control load case 1**

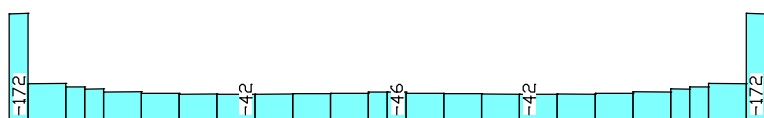
	Rx [kN]	Ry	Rz
Load :	0.00	0.00	6400.00
Support reaction:	0.00	0.00	0.00
Bedding forces :	0.00	0.00	6400.00
Sum :	0.00	0.00	0.00



Element mesh and dimensions (illustration limited to half of the system)



Moment curve  $m_x$  [kNm/m] in section AA



Soil pressure  $\sigma_z$  [kN/m<sup>2</sup>] in section AA

# Pile Foundation

The structure shown corresponds to the example B9 of the document *Recommendations of the Piles Working Group (2012)*. The internal load-bearing capacity must be verified. For this purpose, the ground resistance to be applied and the stress on the pile must first be determined. Three different models for the ground resistance are compared. Subsequently, a compression member design according to EN 1992-1-1 is performed. The example is intended to explain, among other things, the handling of the *edge* property pile-beam.

**Action:**

Permanent action (G):

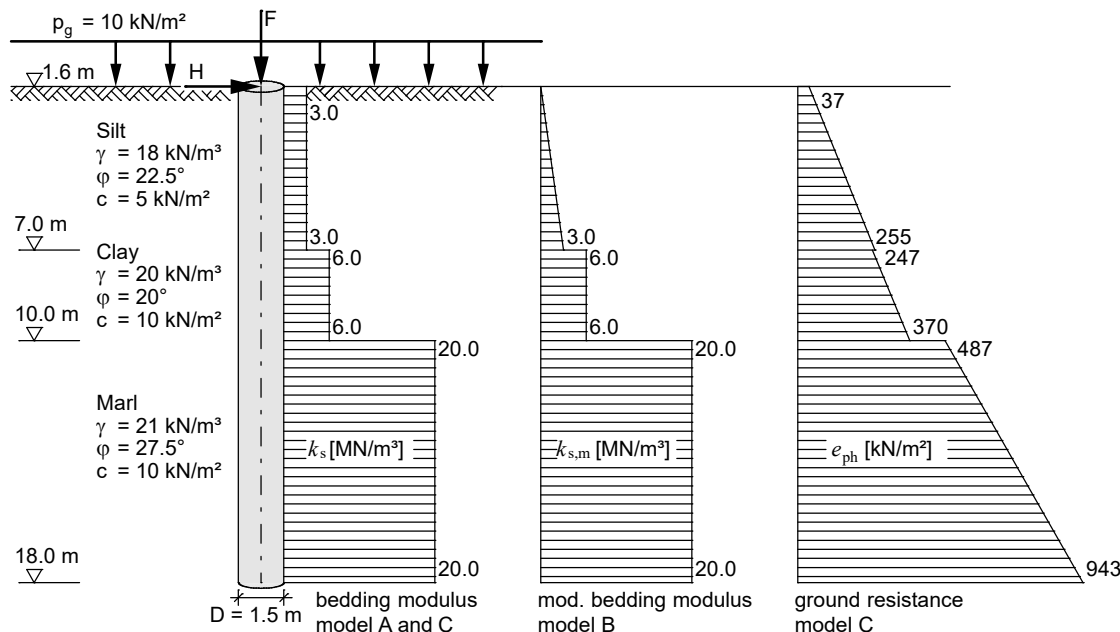
$$F_{G,k} = 3333 \text{ kN} \quad H_{G,k} = 600 \text{ kN}$$

Variable action (Q):

$$F_{Q,k} = 2000 \text{ kN} \quad H_{Q,k} = 400 \text{ kN}$$

**Calculation models for the ground resistance:**

- A. Linear model with constant bedding modulus per layer.
- B. Linear model with modified increasing bedding modulus in the first ground layer.
- C. Non-linear model with constant bedding modulus per layer and with a maximum ground resistance (strength) that varies over the depth.



System with characteristic actions, subsoil structure, bedding modulus and ground resistance

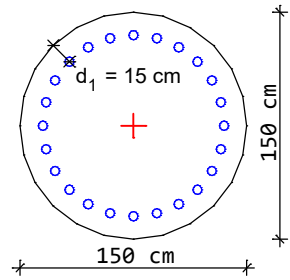
The screenshot shows the 'Edge' dialog box with the following settings: Meaning: Pile beam, Support: (empty), Pitch: 5, Angle: 0. Section at the beginning: 1 Polygon Pile. Bedding profile: 1. Bedding profile, z0 [m]: 1.6, Show symbol (is valid for all objects): checked. Layer of elements: 10: Beam. Label: (empty). BIM-ID: (empty). The 'Bedding profile' dialog box shows the following table:

No.	Label	Thickn. [m]	Bedding [MN/m³]			Strength [MN/m²]		
			x	y	z	x	y	z
1	1. Ground layer - Silt	5.4	0	3	3	0	0.037	0.037
2	2. Ground layer - Clay	3	0	6	6	0	0.247	0.247
3	3. Ground layer - Marl	8	0	20	20	0	0.487	0.487
			0	20	20	0	0.943	0.943

Drawing object edge with its properties for model A and C

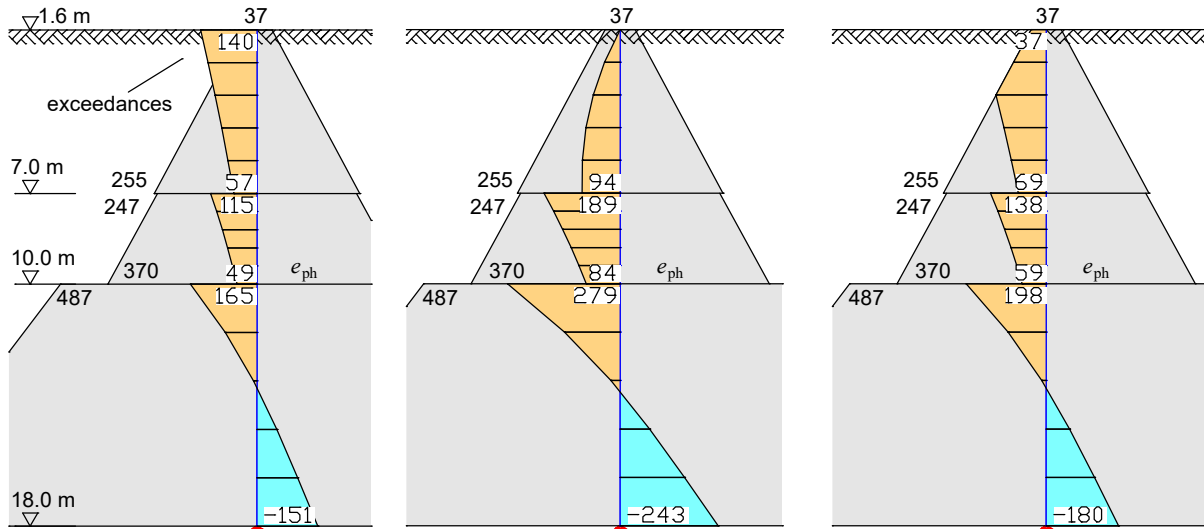
The cross-section defined for the edge (1 Polygon Pile) is a polygon according to the right figure made of C25/30 and with a bedding width for the y- and z-direction of the cross-section of 1.5 m. The bedding moduli as well as the soil strengths are defined there within the bedding profile. In this example, loads in vertical direction (local x-direction of the pile) are to be absorbed only via the support at the lower end and not via surface friction.

Since a linear calculation model was chosen in models A and B, characteristic loads can be calculated in the structural analysis and the load case results can be superpositioned and weighted for the checks. With the results determined in this way, the checks according to EN 1992-1-1 have to be carried out and the soil pressures have to be checked manually.

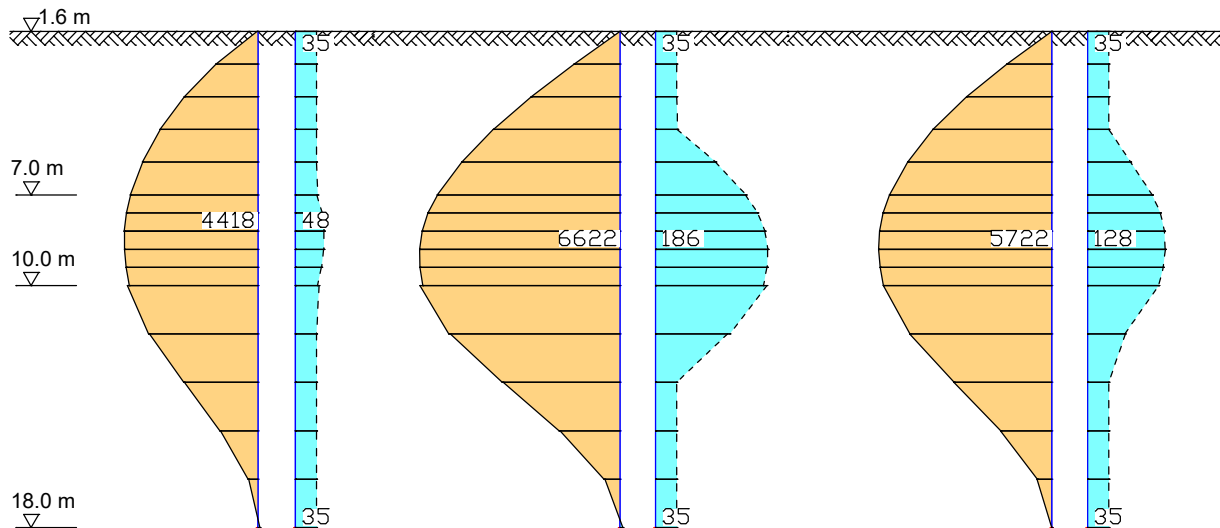


In order to be able to consider the strength of the soil in model C, a bilinear soil behavior is assumed. This requires a non-linear system analysis and that the relevant load situation is already calculated for the load cases. In this example a corresponding load case is defined by a load group.

Since the bilinear soil behavior (bedding modulus-bedding width until reaching the strength) is to be considered, but not the nonlinear material behavior of the concrete and the reinforcing steel, the material property *Material linear* is chosen.



Soil pressure  $\sigma_z$  of model A, B and C due to G+Q [kN/m²].



Bending moments  $M_y$  or longitudinal reinforcement  $A_s$  of model A, B and C due to 1.35-G+1.5-Q [kNm or cm²].

It can be seen that in model A the soil pressure in the upper part of the pile is clearly exceeded. In model B, no exceedances occur in the 2nd ground layer. However, due to the low bedding modulus in the uppermost ground layer the largest bending moments  $M_y$  and thus the largest required reinforcement  $A_s$  of the three models occur. Model C maintains the required soil pressures and gives better reinforcement results than model B.



## Construction Stages, Creep Redistribution

Following the creation and process of construction stages are described. For every construction stage a separate project file is created for which all calculation options are available.

At a simple example, the calculation of creep redistribution as a result of a system change is explained with the help of construction stages.

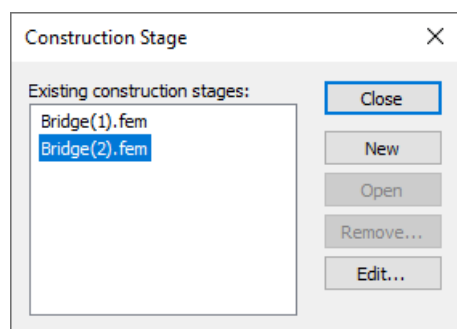
### Construction stages

The system 'inherits' all the properties from one construction stage to the next. During this process, the subsequent stage stores all the information about its predecessor. This information prevents the redundant use of load cases, elements and similar items when the subsequent stage is being processed.

When calculating a construction stage, the results of the preceding stage are copied and then processed so that they can be combined or superposed with the results from the current file.

Procedure:

1. First the original system for the first construction stage is to be entered.
2. To save the file a file name ending in (1) is to be used. For example, bridge(1).FEM (this activates the *Construction Stage...* function).
3. The necessary calculations and checks for this project are to be performed before editing the next construction stage.
4. With the function *Construction Stage...* and the dialog option *New* the second construction stage (here: Bridge(2).fem) is automatically generated and activated.
5. Now the desired system modifications or additions can be made.
6. When the internal force calculation begins, the results from the previous system are copied and made available for load case combinations and superpositions, as if they are calculated for the current construction stage.
7. With this method additional construction stages can be created.



It can be switched between the individual construction stages by simply double-clicking them.

The function *New* can be used to attach or add a subsequent stage after the current construction stage.

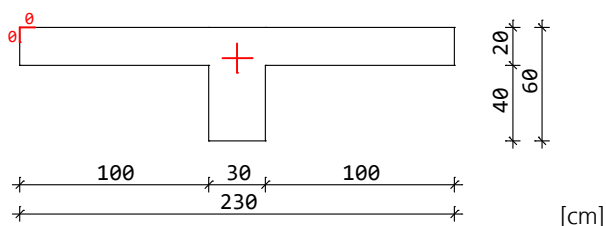
The preceding stage has to be fully processed before generating the next construction stage. Later changes to a construction stage concerning information inherited by the subsequent stage must be added to the successor using the *Edit...* option.

Exceptions:

1. Changes within a load case. To update the results, the construction stage and the subsequent stage have to be recalculated in sequential order.
2. Changes to the element properties.

### Creep redistribution as a result of a system change

The original system, construction stage 1, consists of 2 single-span beams with a length of 4.00 m and 6.00 m. Material: C20/25 acc. to EN 1992-1-1, Section: T-beam.



For the original system the following is calculated (results see below):

**Load case 1:** Dead load

**Load case 2:** Creep  $t_0 = 10$  to  $t_1 = 30$  days; with  $\varphi(t_1, t_0) = \varphi(30, 10) = 0.87$ ; creep-generating continuous load: LC 1

Now the construction stage 2 is generated and the system changes are implemented here. By removing the joint, a continuous model is created. The additional load is applied and calculated:

**Load case 3:** Additional load 10 kN/m

The internal forces and deformations after 30 days result from the dead load + creep + the additional load:

**Load case 4:** Superposition LC 1, 2 and 3 (superposition of the results)

The creep redistribution up to the final state is calculated by:

**Load case 5:** Creep  $t_1 = 30$  to final state;  $\Delta\varphi(\infty, 30) = \varphi(\infty, 10) - \varphi(30, 10) = 2.51 - 0.87 = 1.64$ ; creep-generating continuous load: LC 4

The final state results from load case 4 and 5:

**Load case 6:** Final state (superposition of the results LC 4+LC 5)

The *superposition* load type is especially suitable for adding the results of different load cases (which can derive from different construction stages).

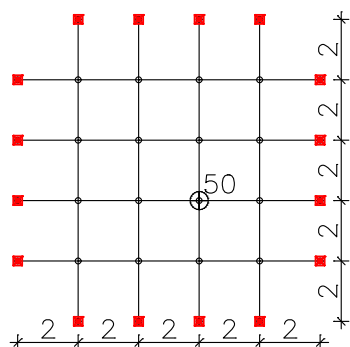
	Construction stage	Internal forces $M_y$	Deformations (1000-fold superelevated)
Load case 1: Dead load	Bridge(1).fem		
Load case 2: Creep $\varphi(30, 10)$			
Load case 3: Additional load	Bridge(2).fem		
Load case 4: LF 1+2+3			
Load case 5: Creep $\Delta\varphi(\infty, 30)$			
Load case 6: Final state			

For comparison:

Monolithic model		
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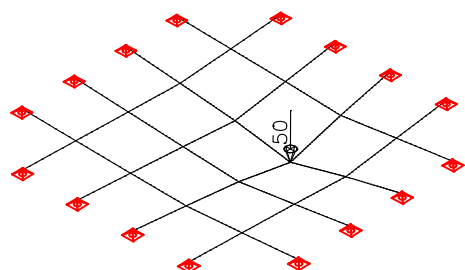
## Cable Mesh

This example shows a 2D cable mesh. As can be seen in the following illustrations, normal forces result in the cables because of the point load active vertical to the mesh.

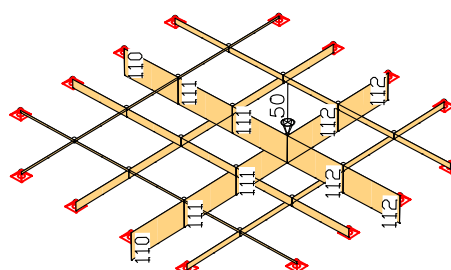


System measurements [m] with load [kN]

Cable section: 1.625 cm<sup>2</sup>  
 E-modulus: 150000 MN/m<sup>2</sup>  
 Prestressing: 0.0 kN



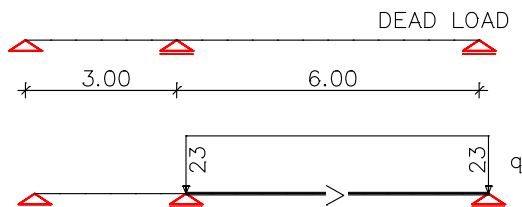
Deformation figure superelevated by a factor of 2



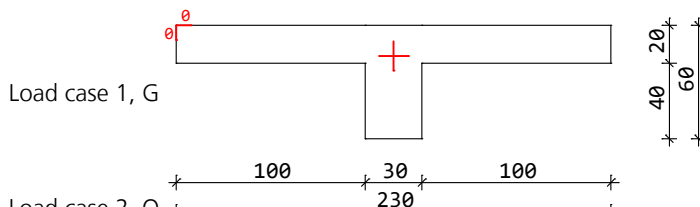
Internal forces Nx [kN]

## Two-Span Girder with Failing Tension Supports

This example deals with a two-span girder under permanent and variable loads. As can be seen in the following figures, a linear calculation under the load situation "G+1.5·Q" results in a tensile force in the left support in this example. For comparison, a similar load group was calculated with the option "No support reactions < 0".



System measurements [m] with load [kN/m]



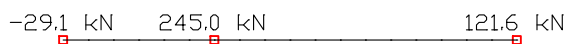
Section [cm] of C20/25 acc. to EN 1992-1-1

### Load data Load case 11 (G+1.5·Q, linear)

EINF: Insertion of load data  
 Load case 1 to 1 weighting factor 1.000  
 Load case 2 to 2 weighting factor 1.500  
 TH : Calculation theory 1. order

### Load data Load case 12 (G+1.5·Q, Fail of tensile support)

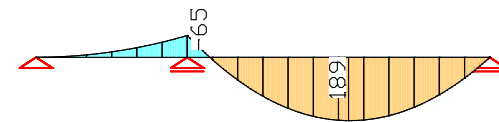
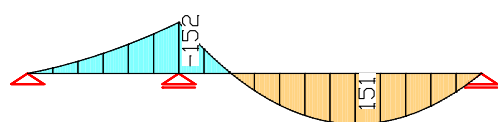
EINF: Insertion of load data  
 Load case 1 to 1 weighting factor 1.000  
 Load case 2 to 2 weighting factor 1.500  
 TH : Calculation theory 1. order  
 Support reactions without tensile forces



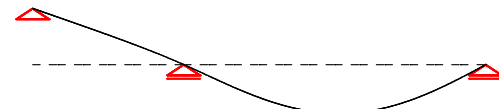
Support reactions Rz



Internal forces My [kN]

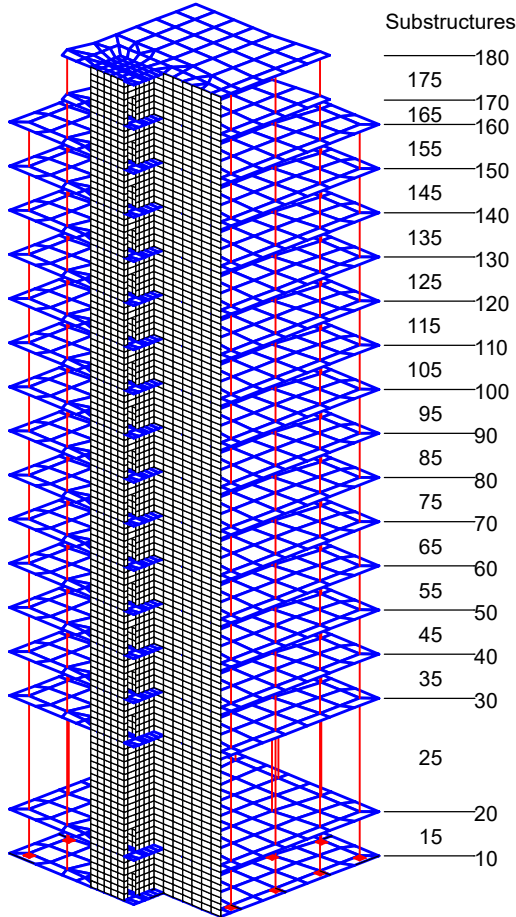


Deformation figure superelevated by a factor of 500



## High-Rise Based on the Substructure Technique

The following example shows a 17-floor high-rise, which was calculated as a global system as part of a dynamic analysis. For this reason the floor slabs were discretized relatively roughly. A much finer division for a static analysis would have been possible without an appreciable increase in the calculation time, as the number of global nodes would not have increased. The substructural division was performed by assigning each floor slab or vertical element of a floor to one substructure. The numbering is illustrated in the figure.



### System characteristics overall system

Nodes	882
Elements	35
Unknown variables	5292
Bandwidth	600
Stiffness matrix	17.3 MB

### Assembly of the substructures

Sub-structure	global nodes	local nodes	Elements	Bandwidth	Stiffness matrix
10	115	50	146	84	1.7 MB
15	198	100	243	150	3.7 MB
20	115	50	146	84	1.7 MB
25	561	100	617	300	13.0 MB
30	115	50	146	84	1.7 MB
35	198	100	243	150	4.0 MB
40	115	50	146	84	1.7 MB
45	198	100	243	150	3.7 MB
50	115	50	146	84	1.7 MB
55	198	100	243	150	3.7 MB
60	115	50	146	84	1.7 MB
65	198	100	243	150	3.7 MB
70	115	50	146	84	1.7 MB
75	198	100	243	150	3.7 MB
80	115	50	146	84	1.7 MB
85	198	100	243	150	3.7 MB
90	115	50	146	84	1.7 MB
95	198	100	243	150	3.7 MB
100	115	50	146	84	1.7 MB
105	198	100	243	150	3.7 MB
110	115	50	146	84	1.7 MB
115	198	100	243	150	3.7 MB
120	115	50	146	84	1.7 MB
125	198	100	243	150	3.7 MB
130	115	50	146	84	1.7 MB
135	198	100	243	150	3.7 MB
140	115	50	146	84	1.7 MB
145	198	100	243	150	3.7 MB
150	115	50	146	84	1.7 MB
155	198	100	243	150	3.7 MB
160	115	50	146	84	1.7 MB
165	198	82	234	150	3.5 MB
170	73	41	98	60	936.4 KB
175	198	82	234	150	3.5 MB
180	73	41	98	60	936.4 KB

Structure model of a high-rise with substructural division

### System characteristics (standard equation solver)

Nodes	6597
Elements	7019
Unknown variables	39582
Bandwidth	2184
Stiffness matrix	429.5 MB

### Information protokoll

The information log from the substructurization shows that, except for substructure 25, the information can be condensed into small work units. The data for the entire system illustrates the advantage of substructuring. The required memory for the calculation of a load case is 130 MB in this application.

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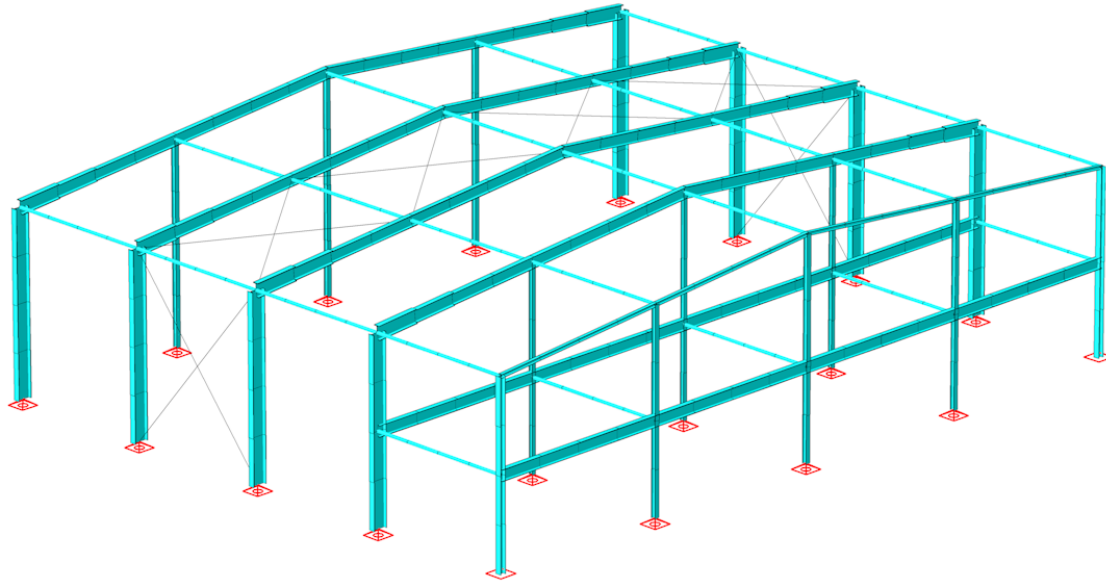
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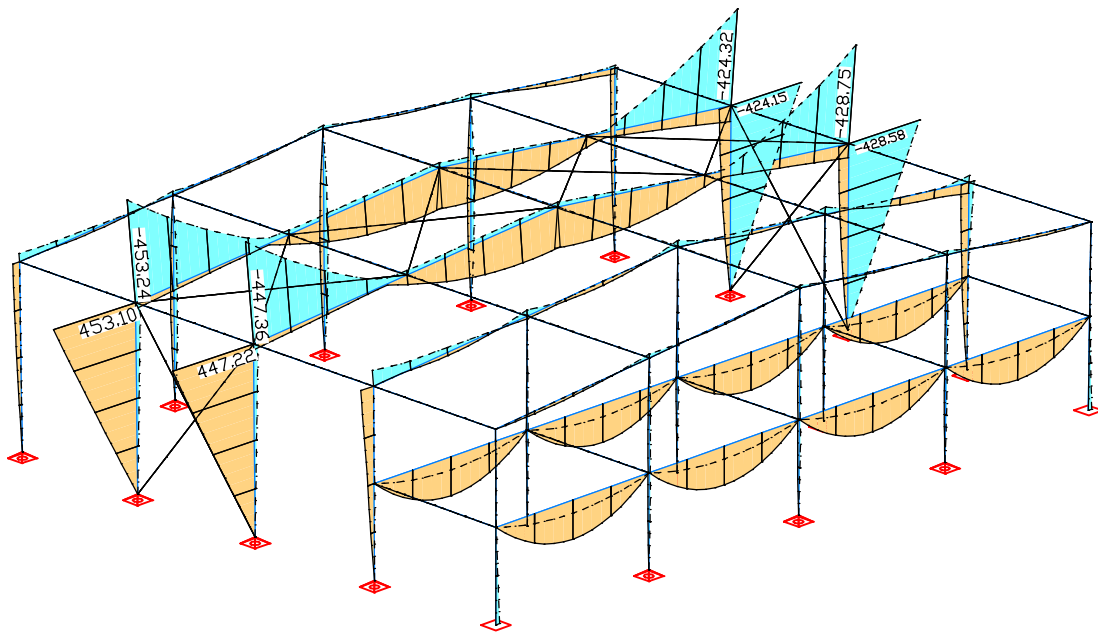
## 3D Frame

### Basics

The *3D Frame* program system is designed for the analysis of 2D and 3D framework and truss constructions. The following example image shows a beam model of a hall construction.



*Static system with activated section representation*


















*Bending moments min,max  $M_y$  [kNm]*

Notes on the bases of calculation are in chapter '*Finite Elements / Basics*'.

## Structure Description

The following *Structure* menu functions can be used for system input and the graphical representation of the results:

-  *Element Input*
-  *Element Properties...*
-  *Element Info*
-  *View...*
-  *Mesh Check*
-  *Element System*
-  *Number Sort*
-  *Supports*
-  *Load*
-  *Predeformation*
-  *Moving Nodes*
-  *Condense*
-  *Link Elements*
-  *Print Input Data...*
-  *Results...*

The general procedure for data input, analysis and data representation of a structure is described in the document *Introductory\_Examples.pdf* installed in the program directory.

## Element Input

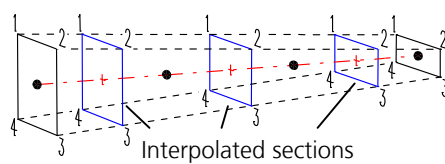
This menu item is used to input beams.

### Input

*Starting point* Input a beam from starting to end point.

*Properties* The beam properties dialog is called up with this option.

*Generate* The entered line is split into  $n$  beams of the same length. In conjunction with the polygon, square and beam section types, you can specify different start and end sections. The program then interpolates the selected number of sections. This enables the modeling of a haunch. The position of the polygon points are interpolated for polygon sections. For this to work, the number and arrangement of the polygon points must match one another.



When entering beams in 3D mode, a point in the local xz plane is queried. For 2D input, the program uses the default orientation.



## Element Properties

This dialog is used to edit the properties of selected beams or preset the properties for new beams yet to be created. The dialog consists of the following pages:

- *General or Joints*
- *Eccentricity*
- *Equivalent beam length*
- *Section*
- *Deformation Check*

### General

This dialog shows the general beam properties.

#### Nodes

Display the node coordinates.

#### Length

Display the beam length [m].

#### Angle

Rotation of the local coordinate system about the x axis with respect to the default orientation in [°].

#### Joints

The joint properties can be specified separately for each degree of freedom. When selected, the corresponding degree of freedom is defined as jointed. The arrow indicates the selected end of the beam. A beam with two ball joints is treated like a truss element.

#### Joint springs

Spring rigidity [MN/m, MNm] with which the respective beam end (a = start, b = end) is elastically attached to the node. If no value is defined, the respective joint property applies.

### Eccentricity

Beam elements can have different eccentricities ( $u \equiv$  local x,  $v \equiv$  local y,  $w \equiv$  local z) at their start and end in their local coordinate system. The beam reactions refer to the eccentric beam axis, on which possible joints also are located. The nodes are still in their original location.

### Equivalent Beam Length

For the buckling check with the equivalent beam method, the equivalent beam length can be defined for beams made of timber according to EN 338 and EN 14080.

#### y-axis, z-axis

The specification of an appropriate equivalent beam length under consideration of support and load conditions is subject to the discretion and the responsibility of the user. If the value 0 is entered no check is carried out for the corresponding direction.

#### Note

If a structural member with constant cross-section is divided into multiple beam elements, the buckling length decisive for the whole structural member shall be applied to all elements. A computational determination of buckling lengths and buckling eigenmodes by the FEM program is possible for selected load combinations (see description of *Load group*).

## Section

The section describes the following properties of a beam:

- Section form
- Specifications for determination of shear stresses
- Material
- Bedding
- Specifications for reinforced concrete design
- Specifications for thermal analysis
- Fire exposure for the simplified check according to EN 1995-1-2

Each section is uniquely identified by a number between 1 and 9999. In general a section is not assigned to a single beam but rather a group of beams.

### Number

Select the section. You can assign the number freely for a newly generated section. Sections can subsequently be renumbered using the context menu for the subitem *Structure Description / Section properties* of the data base.

### Section Type

- *Beam*
- *Polygon*
- Steel section with profile number.
  - IPE*: Medium I girders according to DIN 1025 Part 5 and Euronorm 19-57.
  - HEA*: Wide I girders, light variant according to Euronorm 53-62 or girders of the IPBI series according to DIN 1025 Part 3.
  - HEB*: Wide I girders according to Euronorm 53-62 or girders of the IPB series according to DIN 1025 Part 2.
  - HEM*: Wide I girders, reinforced variant according to Euronorm 53-62 or girders of the IPBv series according to DIN 1025 Part 4.
  - I*: Narrow I girders according to DIN 1025 Part 1.
  - U*: U steel according to DIN 1026.
  - T*: T steel according to DIN 1024.
- *Parameter*: Parameterized steel profile with I-, L-, T-, U-, tube- or box-shaped cross-section geometry.
- *Library*: Section library with steel sections.
- *Database*: Profile from the user database (*Anwend.pro*).
- *Rectangle*
- *Tension member*
- *Composite*

**Material Type**

- *<Database>*: User-defined material coefficients from the system file *lgraph.dat*.
- *B25* to *B55*: Concrete qualities according to DIN 1045:1988, Table 1.
- *C12/15* to *C100/115*: Concrete qualities according to DIN 1045-1 or ENV 1992-1-1:1991 (*C55/67* or higher only for design according to DIN 1045-1).
- *LC12/13* to *LC60/66*: Concrete qualities according to DIN 1045-1.
- *C12/15-EN* to *C90/105-EN*: Concrete qualities according to EN 1992-1-1.
- *C12/15-EN-D* to *C100/115-EN-D*, *LC12/13-EN-D* to *LC80/88-EN-D*: Concrete qualities according to DIN EN 1992-1-1.
- *CX*: Concrete according to DIN 1045-1 with user-defined properties.
- *CX-EN*: Concrete according to EN 1992-1-1 with user-defined properties.
- *CX-EN-D*: Concrete according to DIN EN 1992-1-1 with user-defined properties.
- *ÖC16/20* to *ÖC50/60*: Concrete qualities according to OENORM B 4710-1.
- *ÖNB40*: Concrete quality according to OENORM B 4200-10.
- *ÖNBeton*: Concrete according to OENORM with user-defined properties.
- *SC12/15* to *SC50/60*: Concrete qualities according to SIA262 or SN EN206-1.
- *SLC12/13* to *SLC50/55*: Concrete qualities according to SIA262 or SN EN206-1.
- *SCX*: Concrete according to SIA262 or SN EN206-1 with user-defined properties.
- *S235*, *S275*, *S355*, *S450*: Construction steel according to DIN 18800 T1, Table 1.
- *S235-EN* to *S500-EN*: Construction steel according to EN 1993-1-1, Tab. 3.1 or EN 10025-2.
- *NH1*, *NH2*, *NH3*: Softwood grade I, II and III according to DIN 1052:1988.
- *BSH1*, *BSH2*: Glued laminated lumber grade I and II according to DIN 1052:1988.
- *C14* to *C50*, *D18* to *D80*: Solid timber according to EN 338.
- *GL20h* to *GL32h*, *GL20c* to *GL32c*: Glued laminated timber according to EN 14080.
- *Frei*: User-defined material.
- *Beton*: User-defined concrete.
- *Stahl*: User-defined steel.

**New**

Create a new section.

**Copy**

Copy the current section.

**Delete**

Delete the current section.

**Label**

The section can be labeled with any text.

Depending on the section and material type, further dialogs are available:

- *Form*
- *Shear stresses*
- *Material*
- *Bedding*
- *DIN 1045-1*
- *EN 1992-1-1*
- *BS EN 1992-1-1*
- *DIN EN 1992-1-1*
- *OENORM EN 1992-1-1*
- *SS EN 1992-1-1*
- *OENORM B 4700*
- *SIA 262*
- *EN 1995-1-1*
- *Thermal analysis*

## Polygon Section

### A

Display of the calculated section area [m<sup>2</sup>].

### I<sub>x</sub>

Torsion moment of inertia [m<sup>4</sup>].

With regards to use in solid construction, a very small stiffness is suggested by the program.

### Compute I<sub>x</sub>

Calculate the torsion moment of inertia of the polygon. The result is entered into the text field for  $I_x$ .

### Factor for I<sub>x</sub>

The torsion moment of inertia  $I_x$  is multiplied by this factor.

### I<sub>y</sub>, I<sub>z</sub>, I<sub>yz</sub>

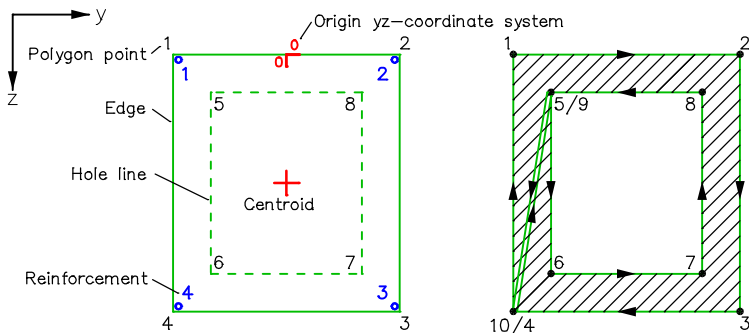
Display the calculated moments of inertia [m<sup>4</sup>].

### Ignore I<sub>yz</sub> in beam stiffness

In the static and dynamic analysis  $I_{yz}$  is ignored.

### Edit...

The graphical editing of the section geometry is carried out in a separate window. The section is defined with edge and hole bordering lines.



Internal storage takes place using a polygon that is calculated based on the by-pass method. The number of polygon points is limited to 999.

The edge and hole bordering points are automatically numbered such that the leftmost point is assigned number 1. If there are several eligible points, the one with the smallest z value is chosen.

After the polygon points have been entered in the database, the order of the points that has been determined there is also valid for the graphical representation for as long as the polygon remains unmodified.

### Reinforcem...

Properties of each reinforcing steel layer of the selected sections.

### Cross-section class

For polygonal sections and sections from the user database made of construction steel according to EN 10025-2 or the material type *Stahl*, the classification into the cross-section class according to EN 1993-1-1, Table 5.2, is carried out by the user.

## File Menu of the Editing Window

### New

Start a new section polygon.

### Insert

Insert a section polygon, steel structure or database profile in the current polygon.

### Store in database...

Save the current section polygon in the user database *Anwend.pro*.

**Import**

Import a drawing of a project file or a DXF file.

**Export**

Export the section polygon as a DXF file.

**Polygon Menu of the Editing Window****Line**

Enter an individual edge or hole bordering line.

**Circle****Tube****Rectangle****Downstand Beam****Upstand Beam**

These section types are created automatically after the relevant parameters have been entered. Optionally, reinforcing steel can be simultaneously defined in the specified edge distance.

**Reinforcing steel**

The reinforcing steel for the bending design can be freely positioned in the section. A guide line with the set edge distance is displayed as a visual aid. The displayed reinforcing steel number helps to identify the steel layers during output of the results. The steel properties (e.g. yield strength  $f_{yk}$ , base reinforcement) are preset. For marked reinforcing steel, properties can be edited in the context menu.

**Label**

Switch the labeling of the polygon on or off.

**Refresh**

Refresh the displayed polygon labeling.

**Settings**

Set the type and scope of labeling as well as edge distance for the input of the reinforcing steel.

**Take over  edge /  hole**

Adopt the edge and/or hole bordering lines from other lines, arcs and circles, for example, after the DXF import of a section drawing.

**Section properties**

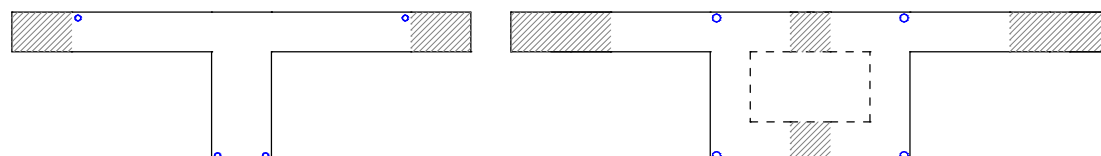
Show detailed section properties (e.g. shear center, resistance momenta, unit warping  $\omega$ ).

**Effective width**

Definition of the effective flange width by selection of the areas outside of the effective width. For this purpose the section is automatically divided into triangles. The selected areas are not taken into account for the calculation of the centroid coordinates and the bending moments of inertia.

For reinforced concrete sections the following procedure is recommended:

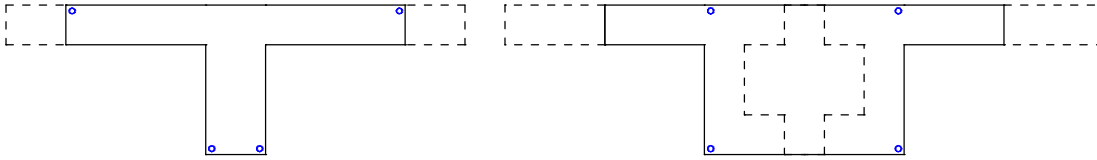
- Input of the full polygon with polygon points at the boundaries of the effective width
- Selection of the areas outside of the effective width
- Positioning of the concrete steel layers within the effective width



*Hatched representation of the section parts outside of the effective width*

The check in the ultimate and fatigue limit states are carried out with the cross-section areas within the effective width. For the crack and stress checks in the SLS, the normal forces are related to the total cross-section (Rossner/Graubner (2012), p. 228). In a *nonlinear system analysis* the entire concrete cross-section is used.

Alternatively, the section polygon can be defined reducing the section dimensions based on the effective width.



Section polygons reduced to the effective width

## Reinforcement Properties

### Yield strength $f_{yk}$

Yield strength of the reinforcing steel [MN/m<sup>2</sup>].

### Base reinforcement

The base reinforcement is the initial reinforcement for the design. It will never be reduced by the program. By entering different base reinforcements, a relationship can be established among the steel sections to be calculated.

### Fix base reinforcement

A fixed steel layer are not increased during the design.

### Zv0

This force is used to take into account an initial strain  $\varepsilon_0 = Z_{v0} / E \cdot A$  for calculation of the reinforcing steel stresses.

### Rolling-kind

In the nonlinear structural analysis for fire scenarios the parameters of the stress-strain curves of the reinforcing steel according to EN 1992-1-1, Figure 3.3 and Table 3.2a depend on the temperature and the rolling-kind:

- *Cold-formed*
- *Hot-rolled*

## Rectangle Section

For this section type no reinforced concrete check can be carried out.

### Width $d_y$ , Height $d_z$

Expansion of the rectangle in the local y or z direction of the beams [m].

## Beam Section

No checks can be performed for this section type.

### A

Section area [m<sup>2</sup>].

### $I_x$ , $I_y$ , $I_z$ , $I_{yz}$

Moments of inertia [m<sup>4</sup>].

### Longitudinal bearing

- *Standard*: The beams transmit compressive and tensile forces.
- *Only pressure*: The beams only transmit compressive forces.
- *Only tension*: The beams only transmit tensile forces.

The use of compressive or tensile beams leads to a nonlinear calculation. In this case the principle of superposition no longer applies.

## Tension Member

This section type describes beams without compressive strength. They should be attached flexibly, if possible. The use of this section type leads to a nonlinear analysis. In this case, the superposition principle is no longer valid. No check can be performed for tension members.

### A

Section area [m<sup>2</sup>].

## Composite Section

This section type can only be used for single designs.

The section properties of the composite section are calculated with the following formulas:

$A_v = 1/E_{\min} \cdot \sum E_i \cdot A_i$	Composite section area
$z_{s,v} = 1/(A_v \cdot E_{\min}) \cdot \sum E_i \cdot z_{s,i} \cdot A_i$	Z ordinate of the centroid
$y_{s,v} = 1/(A_v \cdot E_{\min}) \cdot \sum E_i \cdot y_{s,i} \cdot A_i$	Y ordinate of the centroid
$I_{y,v} = 1/E_{\min} \cdot \sum E_i \cdot [I_{y,i} + (z_{s,v} - z_{s,i})^2 \cdot A_i]$	Second-degree area moment around y
$I_{z,v} = 1/E_{\min} \cdot \sum E_i \cdot [I_{z,i} + (y_{s,v} - y_{s,i})^2 \cdot A_i]$	Second-degree area moment around z
$I_{yz,v} = 1/E_{\min} \cdot \sum E_i \cdot [I_{yz,i} + (y_{s,v} - y_{s,i}) \cdot (z_{s,v} - z_{s,i}) \cdot A_i]$	Deviation moment of inertia

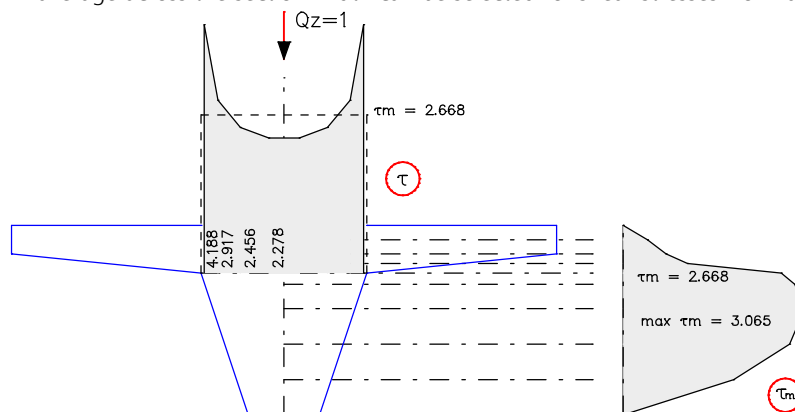
## Shear Stresses

By default the linear-elastic shear stresses are calculated according to the theory of thick-walled profiles.

As a result, in the fillet area of rolled sections or in blunt corners of welded profiles larger stresses occur than those according to the theory of thin-walled profiles.

Specifying the maximum sheet thickness (open profiles:  $\tau_{\max} = M_x / I_x \cdot t_{\max}$ ) or the torsion resistance moment (box profiles:  $\tau_{\max} = M_x / W_t$ ) allows you to limit the torsion shear stresses to the maximum value according to the theory of thin-walled profiles.

An average across the section width can be selected for shear stresses from lateral force.



Intersections and shear stress distribution from lateral force at the section polygon according to the theory of thick-walled profiles ( $\tau$ ) and after averaging ( $\tau_m$ )

## Material

### Properties for all Materials

#### E-modulus

Modulus of elasticity [MN/m<sup>2</sup>].

Note: The modulus of elasticity is part of the stress-strain function according to EN 1992-1-1, Equation (3.14), and SIA 262, Equation (28). The function has a pole that can move into the decisive strain range, if the given modulus of elasticity is reduced. As a consequence, results of the nonlinear analysis and the reinforced concrete checks can be affected.

#### G-Modulus

Shear modulus [MN/m<sup>2</sup>].

#### Gamma

Specific weight of the material [kN/m<sup>3</sup>]. This is used for the *Dead load* load type.

#### alpha.t

Coefficient of thermal expansion [1/K].

#### Material linear

With this switch linear elastic material behavior is assumed for this cross-section during the nonlinear structural analysis. In general, a nonlinear analysis can only be performed for polygon sections, database sections and steel sections. For all other section types and for the material types *Beton* and *Timber* a linear material behavior is always assumed.

### CX (DIN 1045-1), CX-EN-D (DIN EN 1992-1-1), CX-EN (EN 1992-1-1)

For the user-defined concrete material types *CX*, *CX-EN-D* and *CX-EN* additional concrete parameters according to DIN 1045-1, Table 9 or EN 1992-1-1, Table 3.1 are defined in addition to the  $\alpha$  values. Non-editable values are determined by linear interpolation with  $f_{ck}$  in the corresponding table.

#### fck

Characteristic value of compressive cylinder strength of concrete at 28 days [MN/m<sup>2</sup>].

#### alpha or alpha.cc

Coefficient which takes into account the long term effects on the compressive strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

#### fctm

Mean value of axial tensile strength of concrete [MN/m<sup>2</sup>].

#### alpha.ct

Coefficient which takes into account the long term effects on the tensile strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

### Lightweight Concrete

#### Apparent density

Apparent density of lightweight concrete [kg/m<sup>3</sup>].

### Concrete SIA 262

#### ke for E-Mod.

Factor for calculation of the modulus of elasticity according to SIA 262, Section 3.1.2.3.3.

For the material type *SCX* the concrete properties are defined additionally according to SIA 262, Table 3.

#### fck

Characteristic cylinder compressive strength [MN/m<sup>2</sup>].

#### fck.cube

Characteristic cube compressive strength [MN/m<sup>2</sup>].



**fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

**ÖNBeton (OENorm B 4700)**

For this material type the concrete properties are defined additionally according to OENORM B 4700, Table 4.

**fcwk**

Characteristic cube compressive strength, nominal strength of the strength class [MN/m<sup>2</sup>].

**fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

**fctk**

Characteristic concrete tensile strength [MN/m<sup>2</sup>].

**taud**

Calculation value of shear stress [MN/m<sup>2</sup>].

**Frei (General)**

For the material type *Frei* (free) the corresponding strength limits must additionally be specified for a nonlinear system analysis.

**Compression**

Yield strength for compressive stresses  $f_{y,compression}$  (positive value) [MN/m<sup>2</sup>].

**Tension strength**

Yield strength for tensile stresses  $f_{y,tension}$  (positive value) [MN/m<sup>2</sup>].

**Stahl (General)****fyk**

Characteristic yield strength [MN/mm<sup>2</sup>].

**Timber EN 338, EN 14080****Apparent density**

Apparent density of timber [kg/m<sup>3</sup>].

**Service Class**

Service class for timber checks.

## Bedding

### Bedding modulus x, y, z

Bedding moduli for beam directions [MN/m<sup>3</sup>]. The bedding moduli can be linearly variable between the beginning (a) and the end (e) of the beam.

### Bedding width

For each bedding modulus an bedded width [m] is to be specified.

### Compressive or tensile strength (nonlinear system analysis)

The strengths [MN/m<sup>2</sup>] defined here are only valid for nonlinear system analysis. This can be used to realize a bilinear bedding characteristic in the compressive and tensile section (see also *Foundation models*). The strengths can be linearly variable between the beginning (a) and the end (e) of the beam.

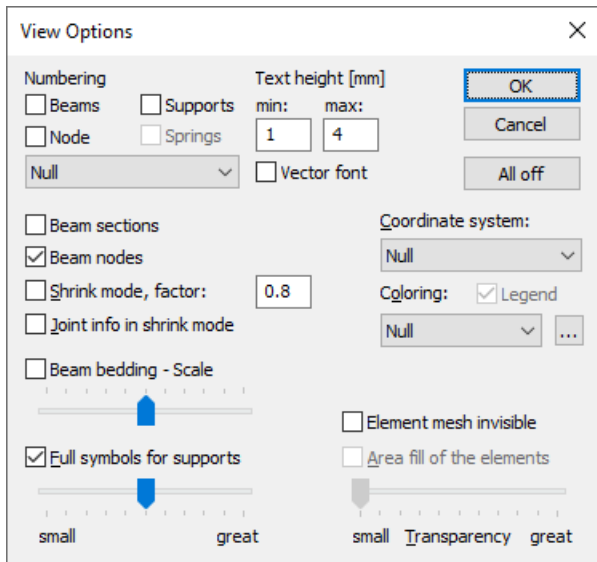
## Element Info

This function shows the following beam properties graphically at the beam and in the status bar:

- Beam number
- Node numbers
- Beam system
- Section
- Material
- Joints

## View

This menu item is used to display and check the structure properties.



### Numbering

You can label beams, nodes and supports with their numbers. Material or section numbers can also be added to beams. The display of the numbers can be limited to values > 1. Alternatively, the section name can be added to beams.

### Text height

Set minimum and maximum text heights [mm].

### Beam section

The beam section is drawn at the middle of the beam.

### Beam nodes

The beam nodes are marked by a symbol.

**Shrink mode**

Enable or disable shrunken beam representation. Most of the input functions are inactive in this mode.

**Shrink factor**

Define shrink factor for the shrink mode.

**Joint info in shrink mode**

In shrink mode the existing beam end joints are labeled. Joint springs are labeled with '\*'.

**Beam bedding**

The bedding moduli are plotted along the beam axes with the specified scale.

**Full symbols for supports**


The properties of the supports are shown in detail. The scroll bar changes the size of the icons in use.

Icons	$u_x$	$u_y$	$u_z$	$\varphi_x$	$\varphi_y$	$\varphi_z$
	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	Free	Free	Free
	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>
	Free	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	Free	<b>Fixed</b>
	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>	Free	<b>Fixed</b>	<b>Fixed</b>
	Free	Free	<b>Fixed</b>	Free	Free	<b>Fixed</b>
	<b>Fixed</b>	<b>Fixed</b>	Free	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>

**Coordinate system**

Activates the coordinate system for beams or supports.

**Coloring**

The objects are drawn in different colors, depending on the selected property (e.g. *material*, *section*). If necessary, the color assignment can be modified with the  button. The possible objects that are distinguished by color include elements, design objects and model objects. The properties *As top*, *bottom* refer to the base reinforcement of the cross-section. Color 254 is used for *Undefined*.

**Legend**

Enable/disable the color legend.

**All off**

Turn off all display options.

**Mesh Check**

With this menu item beams and nodes can be checked.

**Node distance**

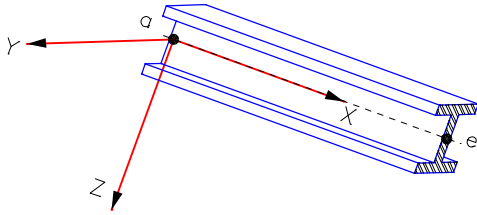
The function checks if there are further nodes in the neighborhood of a node within the given tolerance. After confirmation, the so-called duplicate nodes are marked by red arrows and you are queried if you want to merge them. In this case their coordinates are averaged.

**Beam distance**

Checks if any beams lie on one another, for instance, due to redundant beam entry. The beams that are found are marked by red arrows.

## Element System

This menu item is used to change the coordinate system of selected beams.



+x, -x	By exchanging the start and end nodes of the beam, the positive or negative x axis points to the specified point.
+z, -z	The spatial orientation of the section is defined by a point. The point defines the direction of the positive or negative z axis. As an option, the direction can also be reset to the default orientation.
y, z	The orientation of the selected axis is defined by specifying two points.
Angle	The beam is rotated about the beam axis (local x axis).

## Number Sort

This menu item is used to change beam and node numbers.

### Input

Sort	Sort node or beam numbers geometrically.
Define	Define new start values and increments for node and beam numbers.
Edit	Edit node and beam numbers.

## Supports

This function is used to define supports. A support describes the support conditions of a system node. The individual degrees of freedom can be supported in a fixed or elastic manner. The support system defines the directions of the degrees of freedom and can deviate from the global coordinate system. The supported nodes are labeled with simple or detailed red icons (see *View*).

### Create

Start the input of supports through the selection of nodes. The subsequent editing takes place using the context menu functions:

- Support
- Orientation

### Info












Displays the support system with the bound degrees of freedom for a selected support.

Note: When the function is active only supports can be selected. This improves the handling of these objects.

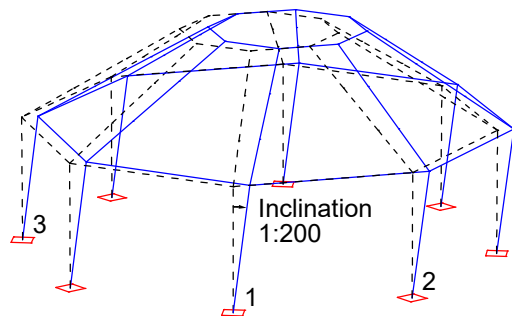
## Predeformation

This function is used to enter and edit stress-free node displacements for a structure. As part of the system analysis, these node displacements are added to the initial node coordinates and may serve for example as a geometric imperfection for calculations according to the second-order theory. Therefore, the calculated nodal deformations refer to the already predeformed structure. To be considered for calculations, the predeformation must be activated within the *load group* definition.

The following options allow you to define predeformations or subsequently change them, if necessary.

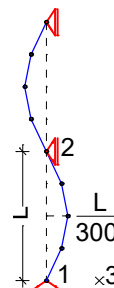
-  Create a copy of the current predeformation.
-  Add the selected predeformation to the current predeformation.
-  Start the representation options dialog.
-  End the predeformation display.
-  Create a new predeformation.
-  Display an existing predeformation.
-  Change the number of the current predeformation.
-  Delete the predeformation.
-  Edit the predeformation.
-  Start the precurvature input.
-  Start the predeformation input.

Absolute input of the predeformation refers to the initial system, relative input refers to the already deformed system.



*Predeformation of a hall construction*

1: 1<sup>st</sup> point of base line, 2: 2<sup>nd</sup> point of base line, 3: Point in base plane



*Precurvature of a column*

The handling is also shown in the video "[https://download.infograph.de/video\\_de/Vorverformung%20definieren.mp4](https://download.infograph.de/video_de/Vorverformung%20definieren.mp4)" (German language).

## Moving Nodes

This function is used to change the location of selected nodes. In the case of a single beam node, simply specify the new position. If multiple beam nodes are to be moved, you should choose the *Group* or *Beams* option.

### Group

Select the beam nodes to be moved.

### Beams

Select the beams, whose nodes are to be moved.

### Displacement vector

Movement through displacement. The displacement vector is defined by two points.

**Rotate**

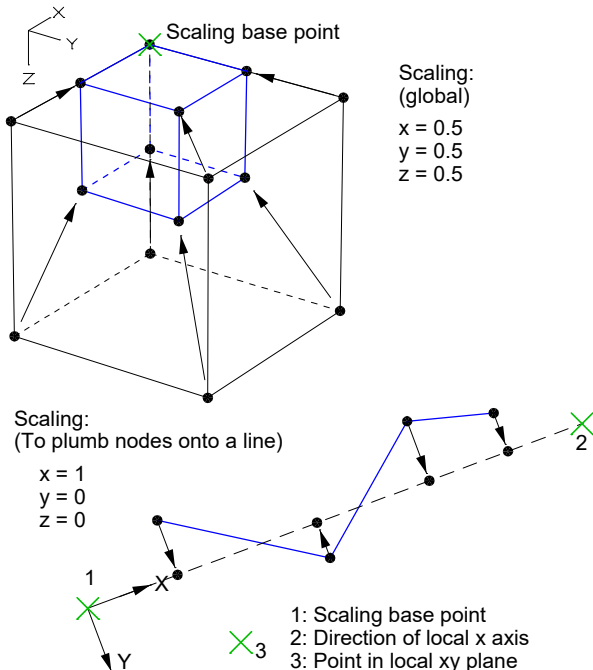
Movement through rotation. A rotation point is defined in 2D mode while a rotation axis is defined in 3D mode.

**Mirror**

Movement through mirroring. A mirror axis is defined in 2D mode while a mirror plane is defined in 3D mode.

**Scaling**

Movement through scaling in the global or local reference system. You can align nodes on an axis or plane by specifying a scaling factor of zero.



If desired, displaced nodes that coincide with non-selected nodes can be merged with these.

**Finding Nodes or Beams**

If you are looking for a node with a particular number, the functions *Select Nodes* and *Move Nodes* can be of help. Enter the node number you are looking for into this function.

If you are looking for a beam with a specific number, use the functions *Element Info* or *Select*. The beam number can be entered in both functions. If the beam is not identifiable even though it is selected, you can query further information using the *Element Properties* function. The selected objects' color and layer are displayed in the toolbar.

**Condense**

This function is used to condense existing beams.

**Point**

At the point specified a node is inserted on the beam axis.

**Divide**

Selected beams are divided. Enter the desired number of parts.

**Adapt**

The selected beams are checked for whether any nodes touch their axes. If necessary, the beams in question are divided at these nodes.

## Arranging Beams

This function is used to align selected beams with a reference beam. The local z axes of the beams must point in the same direction. The alignment can be accomplished in two ways:

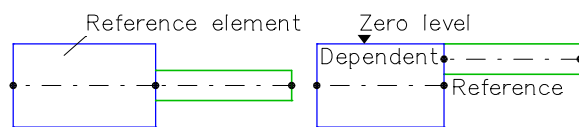
- Use of eccentricities
- Use of link elements

The orientation is performed through displacement in the local z direction until the zero levels correspond to those of the reference beam.

The zero level is at  $z = 0$  in the section polygon. Optionally, the level may be defined as follows:

- $z=0$  (default)
- Top face
- Bottom face

If eccentricities are used, the offset is realized via corresponding eccentricities in the beam element. This means, no new nodes are created. If link elements are used, the program possibly creates new nodes and additional stiff link elements between the nodes, because of the offset. The top node of the link element becomes the dependent node, as long as it does not have a support defined to it. Link elements that already exist are retained.



Beams before and after the alignment (default)

## Link Elements

Kinematic dependencies between the global degrees of freedom of two beam nodes can be defined with the *Link Elements* function. This is done by expressing the nodal degrees of freedom of a dependent node through the degrees of freedom of the reference node by way of an equation of condition.

for example  $u_{z,dep.} = f_1 \cdot u_{x,ref.} + f_2 \cdot u_{y,ref.} + f_3 \cdot u_{z,ref.} + f_4 \cdot \varphi_{x,ref.} + f_5 \cdot \varphi_{y,ref.} + f_6 \cdot \varphi_{z,ref.}$

with

$f_1, f_2, f_3$  Factors for the displacements,

$f_4, f_5, f_6$  Factors for the rotations.

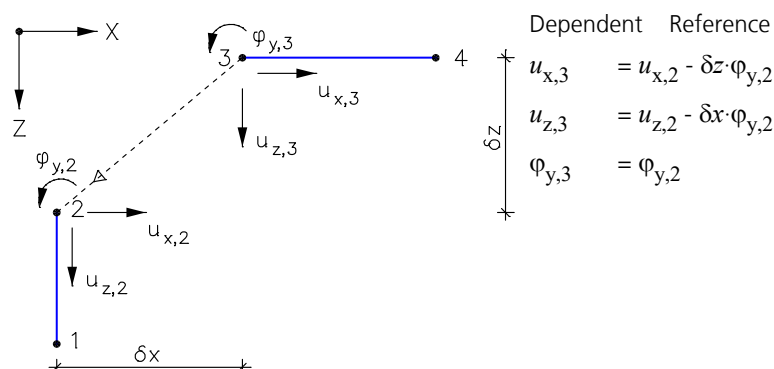
You can link any number of dependent nodes to a reference node. Nodes with supports may not be defined as dependent nodes. The link elements are displayed with dotted lines. Additionally, an arrow points to the reference node.

After selecting the dependent nodes and the reference node, you must define the link element type.

### Type

- *Stiff*: Stiff link element (abbreviation: *SS*).
- *Jointed*: One-sided jointed link element with a ball joint at the dependent node (abbreviation: *GS*).
- $ux, \dots, phi.z$ : Dependent degree of freedom of the user-defined equation of condition. The factors for the displacements and rotations of the equation of condition have to be defined.

### Equations of condition for the stiff link elements in the xz plane.














# Load

The following chapter describes load input as well as the different load types.

The specified loads are represented with icons. In 3D mode, point and line loads are scaled according to their load ordinates. For point loads the resulting load direction is drawn. The orientation of line loads and line moments is indicated by an arrow.

Beam loads generally act on the centroidal axis of the beams. For asymmetrical sections the effect of the eccentricity of the shear center on the fixed-end reactions is not accounted for.

A load case is made up of one or multiple loads. The following options allow you to define load cases or subsequently change them, if necessary.

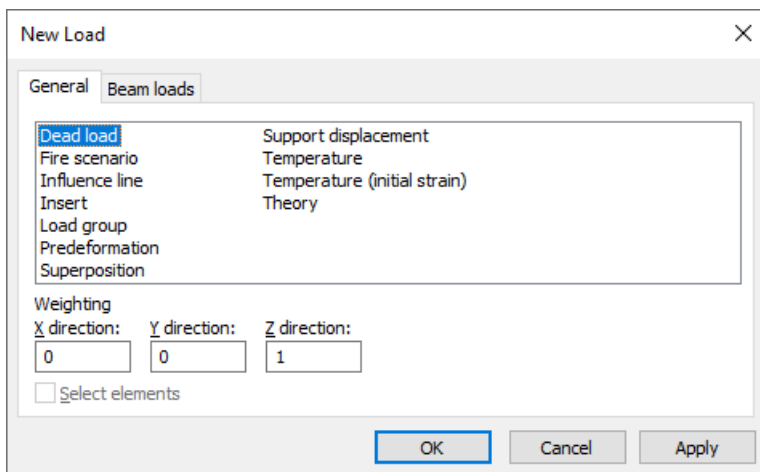
-  Create a copy of the current load case.
-  Generate load cases according to the rules set out by the *Generate* function. This leads to the duplication of either all or only the selected loads.
-  Import load data.
-  Start the representation options dialog.
-  End the load representation.
-  Create a new load case.
-  Open the previous load case.
-  Open the next load case.
-  Show an existing load case.
-  Change the number and name of the current load case.
-  Start the load input.

Load cases can be renumbered in groups within the database tree.

Double-clicking on an existing load object or the context function *Load* opens a dialog in which changes to the selected load objects can be made. With the context function *Scale load* the load values of multiple selected loads can be scaled with a factor.

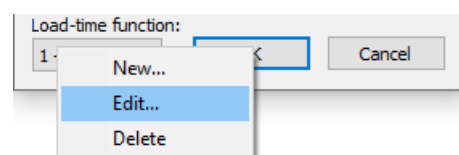
# Input

The dialog box is used to input load data within a load case.



The load types are explained on the following pages. For easier identification they are labeled with load abbreviations.

For many load types a dialog for further load data appears after the load has been entered graphically. In this dialog a load-time curve can be defined for most load types for a non-linear system analysis. If no curve is selected, the load is constant.





## Dead Load

The entire structure is stressed by its dead load (load abbreviation: *EG*).

### Input

*Weighting X, Y, Z direction* The weighting factors allow for defining the direction of effect as well as scaling the dead load. They refer to the global coordinate system.

## Influence Line

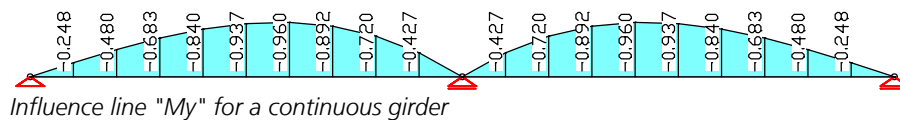
### Input

Selection of the desired beam (load abbreviation: *EFL*).

*Load point* Load point in the beam. The distance from the beam start is saved.

*Influence line* "Nx", "Qy", "Qz", "Mx", "My", "Mz".

The ordinates of the resulting influence line are expressed as deformation in the direction of the load to be analyzed.



## Inserting of Load Case Data

This load type (load abbreviation: *EINF*) allows you to insert load data from other load cases in the current load case in a weighted manner. The *Superposition* load type may not be present in the inserted load cases.

### Input

*First load case* First load case that is inserted.

*Last load case* Last load case that is inserted.

*Weighting factor* Factor by which to multiply all load data.

## Support Displacement

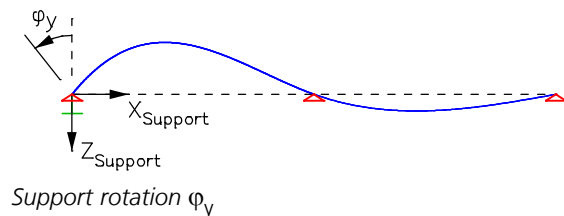
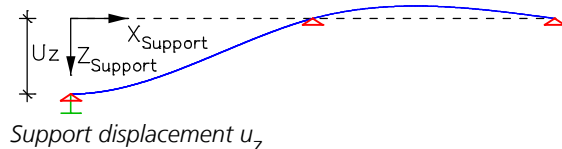
The support displacement acts on supported nodes in the direction of the bound degrees of freedom (load abbr.: *DS*).

### Input

Selection of the affected supports.

*ux, uy, uz* Displacements [mm].

*phi.x, phi.y, phi.z* Rotations [rad/1000].



## Superposition of Load Cases

Through superposition (load abbreviation: *SUP*) the existing results of different load cases can be added and weighted. Other load types are not permitted in the load case in question. Because the load cases are analyzed in ascending order, the load numbers of the load cases to be superposed should be smaller than the number of the current load case.

### Input

<i>First load case</i>	First load case to be superposed.
<i>Last load case</i>	Last load case to be superposed.
<i>Weighting factor</i>	Factor by which to multiply all results.

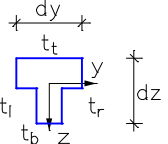
## Temperature

### Input

Selection of the stressed beams (load abbreviation: *TEMP*).

<i>tm</i>	Temperature as a difference to the installation temperature [K].
<i>tb - tt</i>	Temperature difference between the bottom and top side of the section [K].
<i>tr - tl</i>	Temperature difference between the right and left side of the section [K].

For the calculation the following coherences apply:



$$\varepsilon_T = \alpha_T \cdot t_m$$

$$\kappa_{T,y} = \alpha_T \cdot (t_b - t_t)/dz$$

$$\kappa_{T,z} = \alpha_T \cdot (t_r - t_l)/dy$$

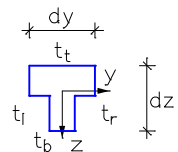
This load type is inadmissible for the section type *Beam*, because the dimensions  $d_y$  and  $d_z$  are not defined. Alternatively the load type *Initial Strain* can be used.

## Temperature, Initial Strain

### Input

Selection of the stressed beams (load abbreviation: *TMPA*).

<i>tm</i>	Temperature as a difference to the installation temperature [K].
$(tb - tt)/dz$	Temperature difference between bottom and top side of the section with respect to the section height [K/m].
$(tr - tl)/dy$	Temperature difference between right and left side of the section with respect to the section width [K/m].



Section with thermal load

## Theory

This load type (load abbreviation: *TH*) is used to specify the calculation theory for the load case in question.

### Input

<i>Geometrical theory</i>	1. / 2. / 3. order: Calculation according to geometric first-, second- or third-order theory.
<i>Number of iterations</i>	This value is not used at the moment.
<i>Error</i>	Error threshold for the relative size of the remaining residuum in [%]. Only values $\leq 1\%$ are permissible.
<i>No soil pressure &gt; 0</i>	Disregard tensile stresses in the foundation area. For a nonlinear system analysis the tension strength specified for nonlinear bedding is always used.

When using this load type, make sure to pay attention to the notes provided in the *Load group* section.

## Predeformation

This load type (load abbreviation: *VV*) is used to predeform the structure. To do so, enter the predeformation number.

## Fire Scenario

This load type (load abbreviation: *FIRE*) indicates that this load case will be calculated with respect to the selected fire scenario. It is only taken into account for nonlinear system analyses (for fire scenarios).

## Load Group

This load type (load abbreviation: *GRL*) is especially suited for preparing load cases for nonlinear calculation. It allows you to compile loads from existing load cases by retrieving the necessary load factors and to activate the desired calculation theory.

The load group is used to group the existing load types *Insert*, *Theory* and *Predeformation* into a load type.

Load groups can also be modified via a dialog in the section *Loads* of the data base. The *Superposition* load type may not be present in the inserted load cases.

### General

<i>Theory</i>	1. / 2. / 3. <i>order theory</i> : Calculation according to geometric first-, second- or third-order theory.
<i>Error threshold</i>	Error threshold for the relative size of the remaining residuum in [%]. Only values $\leq 1\%$ are permissible.
<i>No soil pressure &gt; 0</i>	Disregard tensile stresses in the foundation area. For a nonlinear system analysis the tension strength specified for nonlinear bedding is always used.
<i>Add. global load factor</i>	Additional factor for all loads.
<i>Predeformation</i>	Number of the predeformation.
<i>Load cases</i>	Select load cases to add them to the selection list.
<i>Load factor</i>	The individual load cases are weighted by a factor.
<i>Apply</i>	Change the weighting factor for selected load cases.

### Concrete creeping, shrinkage

In combination with the nonlinear analysis a load case related definition of the creep and shrinkage coefficients is available for standardized concrete material with nonlinear material behavior. A description of the calculation method can be found in chapter *Nonlinear structural analysis*.

*Consider concrete creeping, shrinkage in the nonlinear analysis*

	Enables concrete creeping and shrinkage for the cross-sections, which are listed in the table. For first-time activation available cross-sections are added automatically to the table. Already defined coefficients are taken over.
<i>Refresh</i>	Calculate the creep and shrinkage coefficients for every activated line.
<i>All sec. identical</i>	Takes over the input values of the 1st table entry for the further cross-sections.
<i>phi</i>	Creep coefficient $\varphi(t, t_0)$ .
<i>Calculated</i>	Yes: The creep and shrinkage coefficienta are determined from the further parameters.
<i>t0, t</i>	Load start and concrete age in days.
<i>Factor</i>	Optional factor for the calculated creep coefficient. This is used for example for the determination of $\varphi_{ef}$ according to EN 1992-1-1 from the calculated coefficient.
<i>eps.cs*1e5</i>	The shrinkage coefficient $\varepsilon_{cs}(t, t_0)$ multiplied with $1 \cdot 10^5$ . A negative value must be entered for a contraction.

When using this load type, the following should be observed.

### Calculation according to the second- or third-order theory

Before carrying out a calculation according to the second- or third order theory, make sure the system is kinematically stable according to the first-order theory. A failure in stability due to beam buckling is recognized during calculation according to the second-order theory by a singularity of the system of equations or of a beam stiffness matrix. The calculation aborts in this case and informs you about the location and the failed degree of freedom. In borderline cases it may be necessary to set the error threshold to '0' in order to determine the failure state.

### Error threshold

All nonlinear problems are solved iteratively. The length of the error vector, the residuum, is used to control the iteration. It is shown for each iteration step in the output bar. The iteration is ended when the remaining residuum has reached the specified error threshold in relation to the initial residuum. The calculation results have, on average, a relative error of the same extent. If the error threshold cannot be reached, the calculation program issues a corresponding message.

### Loss of tensile bedding

Unrealistic load combinations (for instance, a missing dead load) can lead to loss of all bedding (see also *Foundation models*). If the system becomes relocatable due to this, then the calculation is aborted because of a singular system of equations. The relocatable degree of freedom is provided.

### Consideration of tension and compression beams

For structures with tension and compression beams, the iterative determination of the equilibrium state is automatically carried out with an error threshold of 1%. In the load case a smaller value can be specified. Instabilities due to beam failure lead to a singular system of equations, in which case the calculation will abort.

### Load combination

In order to combine the decisive actions for calculations according to the second-order theory or with loss of tensile bedding, transfer the corresponding load cases to the selection list and assign them load factors.

For calculations according to DIN 18800, Part 2, or EN 1993-1-1 the following different cases exist:

- a) Calculation according to the geometric second-order theory with linear material law according to elasticity theory and subsequent elastic-elastic or elastic-plastic check. The internal forces are to be determined according to the standard with  $\gamma_M$ -fold design values of the actions. To do so, the load cases of the selection list are to be weighted with the factor  $\gamma_F$ . Additionally,  $\gamma_M$  is to be specified as the global load factor.
- b) Ultimate limit state check according to the geometric second-order theory with nonlinear material law according to the plasticity theory. In this case  $\gamma_M$  is considered on the resistance side according to the standard. The internal forces calculation is to be carried out with the design values of the actions. Therefore, only  $\gamma_F$  needs to be provided as a weighting factor.

### Prohibition of superposition

Calculations with tension and compression beams, according to the second-order theory or excluding tensile bedding are performed on the basis of nonlinear iteration. So in these cases the superposition principle is no longer valid, meaning the results may not be additively superposed with other results. Further processing using load case combinations is only permissible if these only contain mutually exclusive actions.

## Point Load

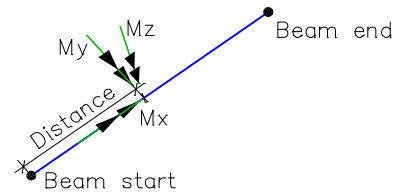
### Input

<i>Global orientation</i>	Point load in the global system (load abbreviation: <i>FG</i> ).
<i>Local orientation</i>	Point load in the beam system (load abbreviation: <i>FL</i> ).
<i>Load point</i>	Load point on the beam axis. The distance from the beam start is saved.
<i>Px, Py, Pz</i>	Load components in the directions of the selected coordinate system [kN].

## Point Moment

### Input

*Load point* Load point on the beam axis. The distance from the beam start is saved (load abbreviation: *ML*).  
*Mx, My, Mz* Load components in the beam system [kNm].



Point moment on the beam

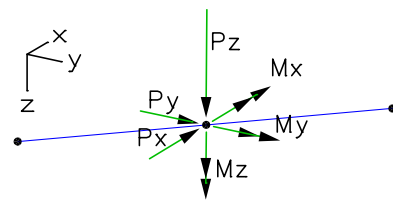
## Nodal Load

Nodal loads act in the global system (load abbreviation: *KNL*).

### Input

Selection of the stressed beam nodes.

*Px, Py, Pz* Forces [kN].  
*Mx, My, Mz* Moments [kNm].



## Line Load

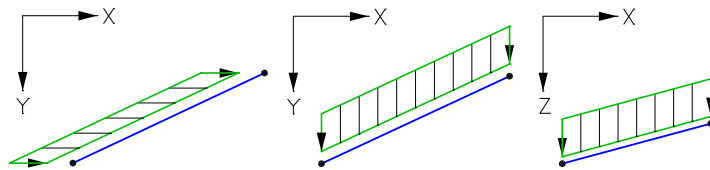
### Input

Selection of the stressed beams.

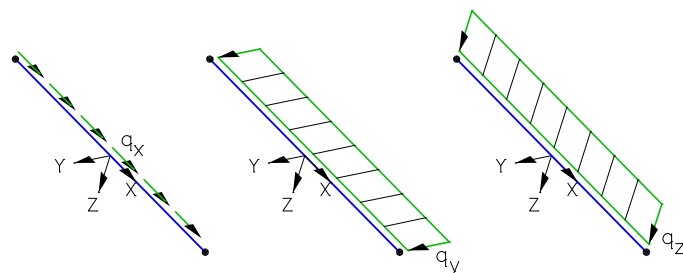
*Global orientation* Line load in the global system (load abbreviation: *LG*).

*Local orientation* Line load in the beam system (load abbreviation: *LL*).

*qx, qy, qz* Load components in the directions of the selected coordinate system [kN/m].



Global line load (*qx, qy, qz*)



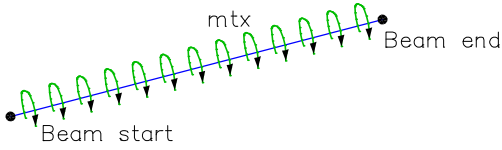
Local line load

## Line Torsional Moment

### Input

Selection of the stressed beams (load abbreviation: *MT*).

*mtx* Line torsional moment [kNm/m].



Line torsional moment about the beam axis

## Trapezoidal Load

### Input

Selection of the stressed beams.

*Starting point* Load start.

*End point* Load end.

*Global x, y, z* Trapezoidal load in the global system (load abbreviation: *TA + GX, GY, GZ*).

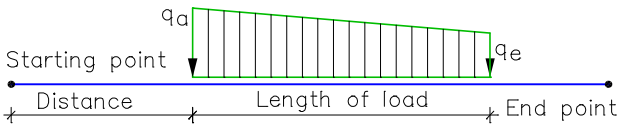
*Local x, y,* Trapezoidal load in the beam system (load abbreviation: *TA + LX, LY, LZ*).

*Projection x, y, z* Projection of the trapezoidal load in the global system (load abbreviation: *TA + PX, PY, PZ*).

*Starting ordinate* Load ordinate at the load start in the directions of the selected coordinate system [kN/m].

*End ordinate* Load ordinate at the load end in the directions of the selected coordinate system [kN/m].

The load is automatically distributed over multiple beams for a trapezoidal load. The distances from the beam starts, the load lengths, and the interpolated load ordinates are automatically determined for selected beams. This allows you to describe a trapezoidal load for an entire beam series with a single input.



Trapezoidal load on beam

## Fixed-End Reactions

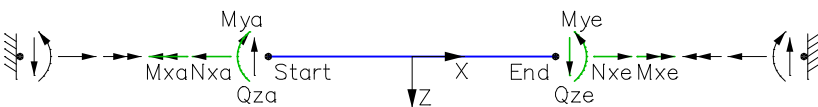
The fixed-end reactions at the beam start (load abbreviation: *SOA*) or beam end (load abbreviation: *SOE*) must be input separately.

### Input

Selection of the stressed beams.

*Nx, Qy, Qz* Normal and lateral force [kN].




*Mx, My, Mz* Moments [kNm].



Fixed-end reactions at the beam start and end.

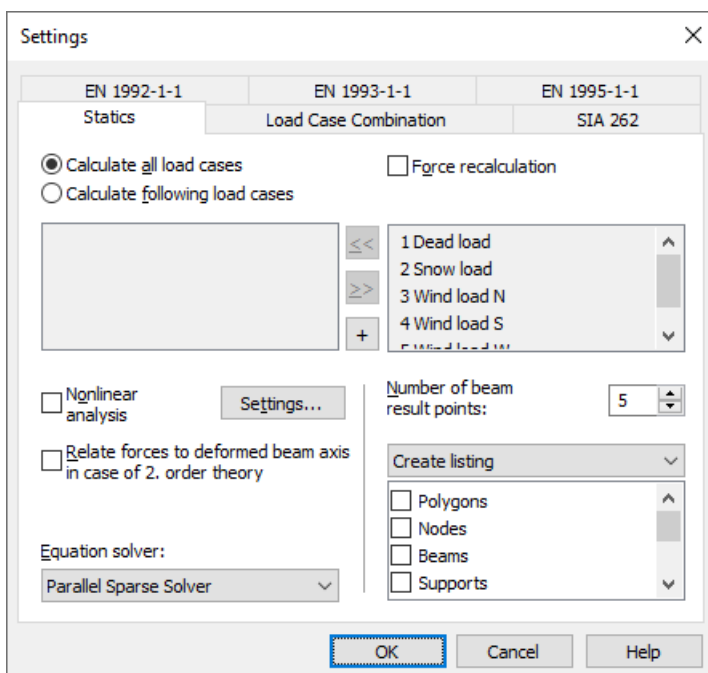
# Analysis

The *Analysis* menu is used to make program-specific settings and to launch the analysis programs.

-  *Settings...*
-  *Batch...*
  - Construction Stages (Batch calculation of all Construction Stages)*
-  *Statics (linear material laws)*
  - Nonlinear analysis*
  - Section Temperatures*
  - Load Case Combination*
  - BS EN 1992-1-1 Design*
  - DIN 1045-1 Design*
  - DIN EN 1992-1-1 Design*
  - EN 1992-1-1 Design*
  - OENORM B 4700 Design*
  - OENORM EN 1992-1-1 Design*
  - SIA 262 Design*
  - SS EN 1992-1-1 Design*
  - Single Design*
  - DIN 18800 Steel Checks*
  - EN 1993-1-1 Steel Checks*
  - DIN 1052 Timber Checks*
  - EN 1995-1-1 Timber Checks*
  - Selected Elements*
  - Stop Analysis*

During the analysis the program's progress is shown in the output bar along with any possible warnings and error messages.

## Settings Statics



### Calculate all load cases

All load cases are calculated (default).

### Calculate following load cases

The load cases from the left list box are calculated.

### Force recalculation

If this switch is activated, all of the selected load cases will be recalculated. Otherwise, the calculation program checks whether a recalculation is necessary.

### Relate forces to deformed beam axis in case of 2. order theory

For calculations according to the second-order theory, the internal forces and stresses are based on the beam chord of the deformed system.

### Equation solver

- *Standard*: The equation system is solved directly.
- *Iterative*: The equation system is solved iteratively.
- *Substructure Standard*: The equation system is solved using the substructure technique. The elements are allocated to the substructures according to the layer they belong to. The layer number corresponds to the number of the substructure. The elements of a substructure do not have to be placed in an interconnected area.
- *Substructure Info*: The analysis program creates an information log which helps you perform an effective substructure division.
- *Parallel Sparse Solver*: The equation system is solved directly with additional memory optimization and parallel processing.

### Nonlinear analysis

Instead of statics with linear material laws, a nonlinear system analysis is performed. The *Settings* button opens the settings dialog for the ultimate limit state, the serviceability and the fire scenario check. Load cases selected in the setting dialog (nonlinear analysis) are calculated instead of the selection made here.

### Number of beam result locations

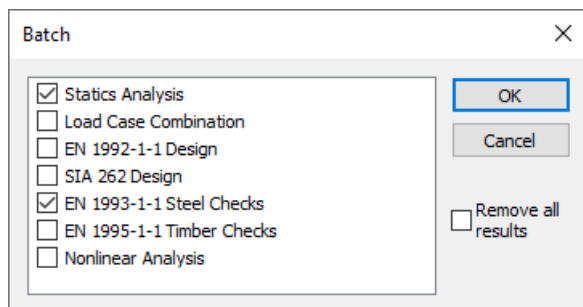
The internal forces are calculated at equidistant points in the beam. By default five result locations are used. For bedded beams the reactions are only calculated in the nodes.

### Listing

- *No listing*: The calculation program generates no log.
- *Create listing*: A log is generated during the calculation according to the selection list.

## Batch

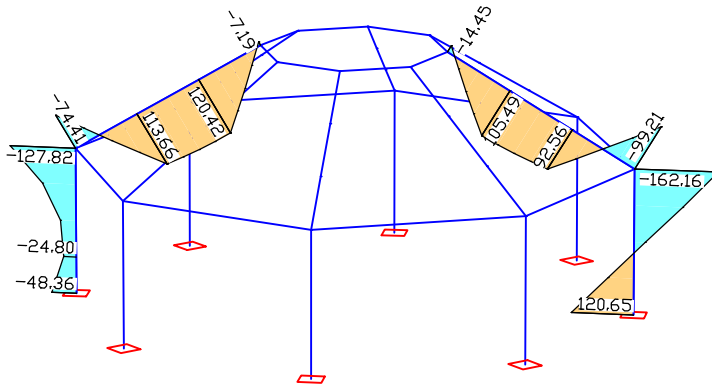
Use this menu item to run all selected analyses as a batch job.





## Results

The following chapter describes the representation of the analysis results. Results can be shown graphically as well as in tables. Their selection takes place in the database or with the *Results* function of the *Structure* menu.



Selected internal forces representation  $M_y$  of a hall construction










### Sum of installed loads and support reactions

LC.	Label	Fx [kN]	Fy [kN]	Fz [kN]
1	Dead load	-0.000	0.000	1224.149
	Support reactions	0.000	-0.000	1224.149
2	Wind	82.426	34.142	0.000
	Support reactions	82.426	34.142	-0.000
3	Deflection theorie	123.639	51.213	1837.119
	Support reactions	123.639	51.213	1837.119

Tabular output of the load sums

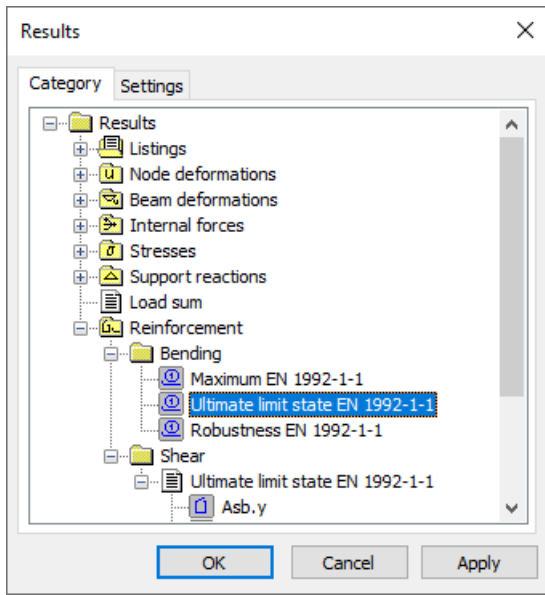
## Result Bar



The result bar includes the following buttons for representing results:

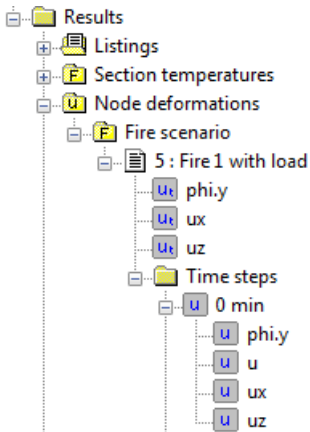
-  Start the dialog for result representation.
-  Turn off result representation.
-  Result representation of the previous load case or load case combination.
-  Result representation of the next load case or load case combination.
-  Ordinate selection for manual labeling.
-  Enable result representation for the selected beams.
-  Disable result representation for the selected beams.
-  Insert result marker for fire scenario stresses. During the analysis the section stress distribution of every time step is stored for the marked locations. The results can be displayed by double-clicking the corresponding marker.
-  Output the ordinate value at the marked point in the status bar.

## Result Categories

The following dialog shows possible result categories.



The load case and combination results as well as all presentable result components are listed in the subsection of the respective category. After making a selection, the *Settings* tab offers views described below with additional options. The symbol in front of the item identifies the type of view ( e.g. : table view, : graphical view of internal forces).



Results of a fire scenario analysis for a selected time step are shown like the results of a load case. The progression of results can be represented for a selected location in a diagram. Alternatively, the results can be animated for the whole system (*colored, deformation figure, result graph*), if a superordinate component is chosen.

## Deformation Figure

### Scale

The scale can be set automatically or manually. The node displacements are multiplied by the distortion factor and added to the node coordinates. For load cases the bending curve is displayed as well.

### Display original system in dashed manner

Switch on/off the dashed non-deformed system during deformation display.

### Label

- *Auto*: Automatically add labels to the nodes listing the length of the displacement vector [mm].
- *Off*: Disable labeling.

### Limit value for labeling

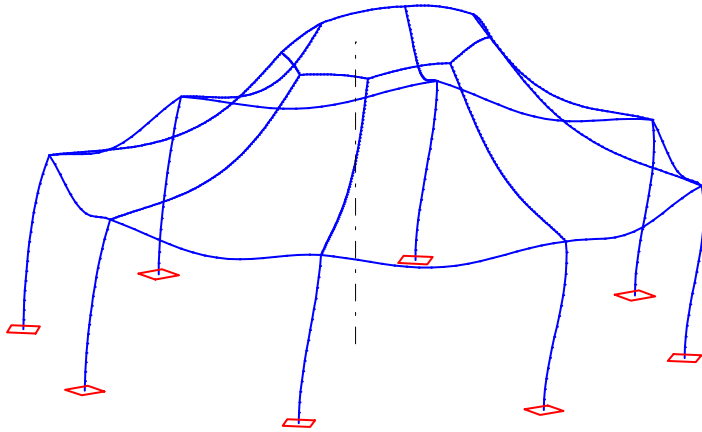
Only displacements exceeding the limit value are labeled.

### Text height [mm]

The unit depends on the output device.

### Decimal places

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.



Superelevated deformation figure

### Numeric

This view displays the support reactions or deformations numerically.

#### Lower boundary for presentation

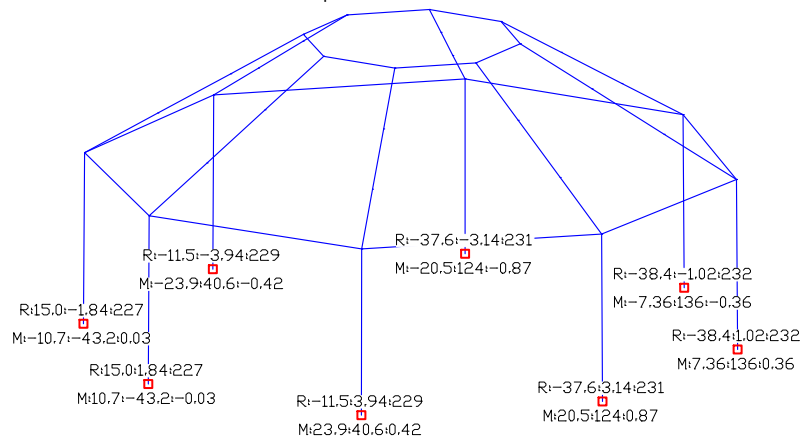
Only results exceeding the limit value are output.

#### Text height [mm]

The unit depends on the output device.

### Decimal places

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.



Display of the support reactions with the components  $R_x$ ,  $R_y$ ,  $R_z$  [kN] and  $M_x$ ,  $M_y$ ,  $M_z$  [kNm]

## Result Graphs

The results are plotted along the beam axis. A linear progress of results is assumed between the result locations.

### Scale

The scale of the ordinate display can be set automatically or manually.

### Label

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.
- *Everywhere*: Label all ordinates.
- *Selectable*: Choose manual labeling with the help of the result bar.

### Limit value for labeling

Only ordinates exceeding the limit value are labeled.

### Decimal places

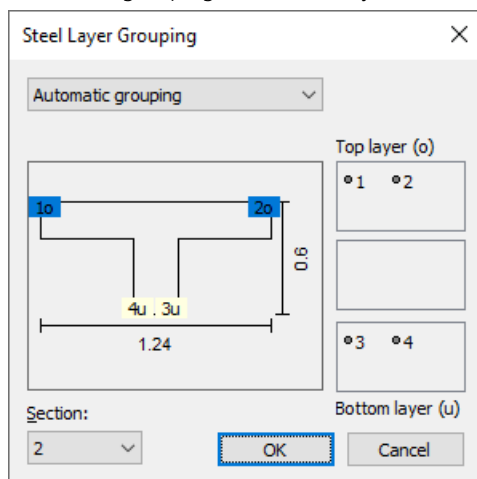
- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

### Text height [mm]

The unit depends on the output device.

### Steel layer grouping...

Define the grouping of the steel layers with this function.



During automatic grouping, all steel layers with a z component smaller than the z component of the centroid are displayed on the upper side of the elements, all other layers on the lower side.

For manual grouping, steel layers can be moved between the list boxes using the mouse. The steel layers of the middle list box are not displayed.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

## Colored

For the representation type *Colored* the beams are colored according to their results.

### Number of colors

Set the number of colors for automatic color grading.



### Color shift

Shift of the automatic color grading.

### Grading

Enable/disable manual color grading. Color and number values can be changed.

Functions of the context menu ( right mouse button):

<i>Save as Presetting</i>	Saves the current grading as presetting for other projects
<i>Load Presetting</i>	Loads the saved presetting
<i>Load Colors of View</i>	Sets the grading according to the current color representation
<i>Database...</i>	Management of different gradings in the <i>Igraph.dat</i> file
<i>Insert Row</i> 	Adds another one above the selected color level
<i>Delete</i> 	Deletes the selected color level
<i>Set All Values...</i>	Fills the list with the help of <i>start value</i> , <i>end value</i> and <i>grading</i>

### Legend

Enable/disable the color legend.

### Dividing line

Enable/disable dividing lines at the color transitions.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

### Deformation figure

Enable/disable the deformation figure of a load case.



*Colored representation of the internal forces  $M_y$*

## Section Stresses

The stress curve is illustrated on a section in a dialog window. This can be accessed via the context menu of the beams when displaying result graphs for stresses or internal forces.

### Load case

Select the calculated load cases and combinations.

### Set

Select the internal force set for the stress determination. The absolute extreme values of the stresses from all internal force sets are displayed using *Extreme value*.

### Beam

Number of the selected beam.

### Location

Select the result location in the beam.

### Section

Description of the displayed section.

**Nx, Qy, Qz, Mx, My, Mz**

Internal forces at the selected result location, from which the displayed stress curve results.

**Stress**

The stresses are generally calculated while taking the linear elastic material behavior into consideration. For results from a nonlinear system analysis, the stresses of the plastic state are displayed on polygon sections and steel sections made of construction steel (S235 to S450 or S235-EN to S450-EN) or *Stahl*.

- $\sigma_x$ : Longitudinal stresses from bending moments and longitudinal force.
- $\sigma_v$ : Comparison stresses according to the Huber-von Mises yield criterion.
- $\sigma_{1,2}$ : Main stresses from bending moments, longitudinal force, lateral forces and torsion.
- $\tau_r$ : Shear stresses from lateral forces and torsion parallel to the edge.
- $\tau_{xy}$ : Shear stresses from lateral forces and torsion in y direction.
- $\tau_{xz}$ : Shear stresses from lateral forces and torsion in z direction.

**Label, Text height**

Labeling of extreme values. With the right mouse button the labels for chosen ordinate values can be optionally activated or deactivated.

**Print**

Print the section stresses. Optionally, the size of the printout (*Print size: height, width*) can be defined.

**->Print list**

Insert the current view in the printing list.

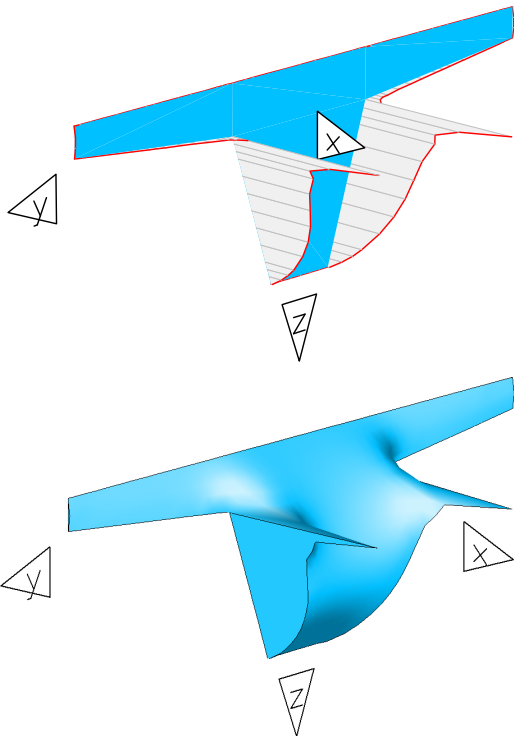
**Representation**

The stresses can be represented either as lines along the edge or as a 3D face.

The view can be rotated with the help of the mouse. The area displayed can be moved by holding down the Shift key. Use the Ctrl key to change the size of the zoom section. If you place the mouse pointer over an ordinate and wait a moment, the stress present at that location is displayed.

**Section stress  $\tau_{xz}$  (Elastic)**

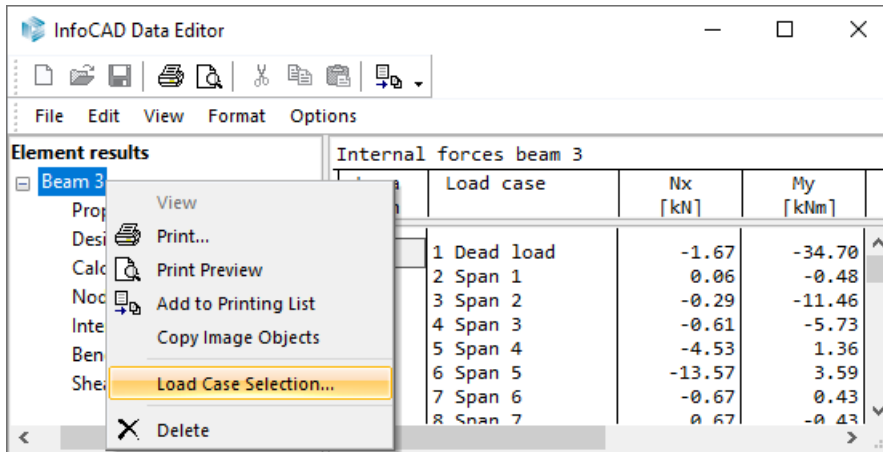
Beam 1, result point 1, section Polygon - C20/25  
 Load case 1  
 $N_x / Q_y / Q_z = 0.00 / -0.00 / 5865.00$  kN;  $M_x / M_y / M_z = -0.00 / -58650.00 / 0.00$  kNm  
 Range of values of the stresses: -0.00157631 bis 4.18786 MN/m<sup>2</sup>.



Shear stress  $\tau_{xz}$  represented as lines along the edge and as 3D face

## Result List

The result list is output for selected beams in the data editor. It can be accessed via the beam context menu.



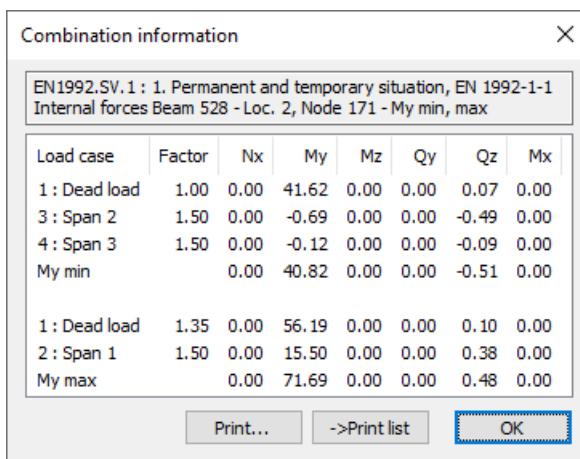
The output can be limited to certain load cases using the *Load Case Selection* option of the context menu.

## Combination Information

The *Combination information* option can be used to determine the load cases involved in a combination result with their respective weightings. This function is available during the display of combination results. After the selection of the desired result location with the right mouse button, the *Combination information* function appears in the context menu.

The following functions are available in the context menu of the dialog:

- *Copy (clipboard)*
- *Create load group...*
- *Create superposition load case*
- *Associated deformations...*
- *Associated support forces...*



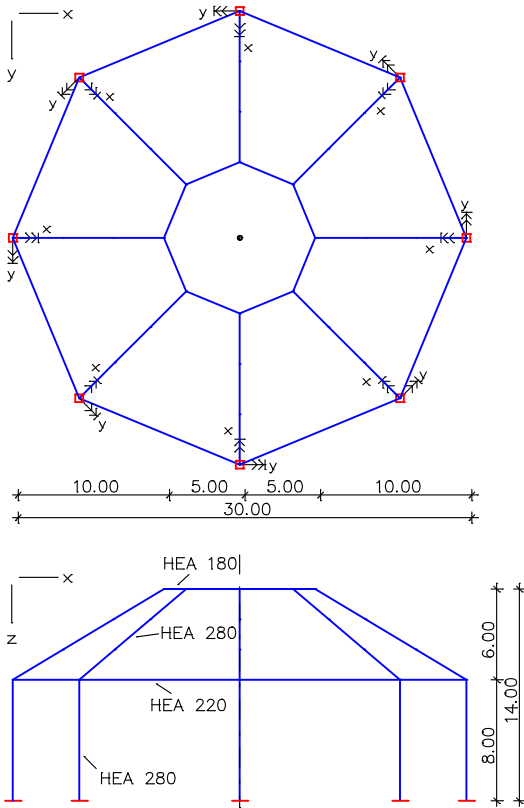
### Last Combination

Shows result values corresponding to the last *Combination information*.

## Example

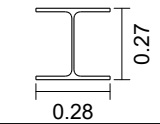
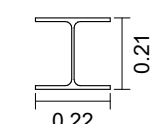
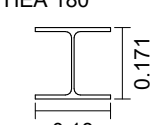
### Steel Hall According to the Second-Order Theory

In the following example a 3D steel hall (S235, EN 10025-2) is analyzed according to the second-order theory.



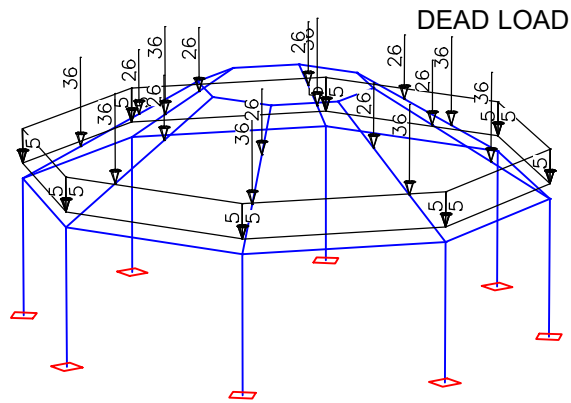
Static system

#### Section properties

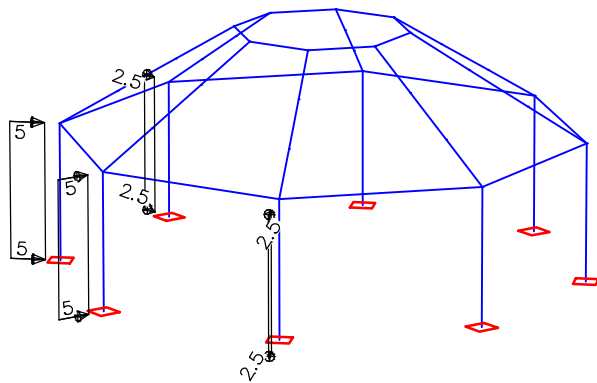
1	HEA 280 	Centroid [m] Area [m <sup>2</sup> ] Moments of inertia [m <sup>4</sup> ] Main axis angle [Grad]	ys = 0.000 A = 9.7300e-03 Ix = 6.2400e-07 Iy = 1.3670e-04 Iz = 4.7600e-05 Phi = 0.000	zs = 0.000 I1 = 1.3670e-04 I2 = 4.7600e-05 Iyz = 0.0000e+00
2	HEA 220 	Centroid [m] Area [m <sup>2</sup> ] Moments of inertia [m <sup>4</sup> ] Main axis angle [Grad]	ys = 0.000 A = 6.4300e-03 Ix = 2.8600e-07 Iy = 5.4100e-05 Iz = 1.9500e-05 Phi = 0.000	zs = 0.000 I1 = 5.4100e-05 I2 = 1.9500e-05 Iyz = 0.0000e+00
3	HEA 180 	Centroid [m] Area [m <sup>2</sup> ] Moments of inertia [m <sup>4</sup> ] Main axis angle [Grad]	ys = 0.000 A = 4.5300e-03 Ix = 1.4900e-07 Iy = 2.5100e-05 Iz = 9.2500e-06 Phi = 0.000	zs = 0.000 I1 = 2.5100e-05 I2 = 9.2500e-06 Iyz = 0.0000e+00



**Load data Load case 1 (Dead load)**

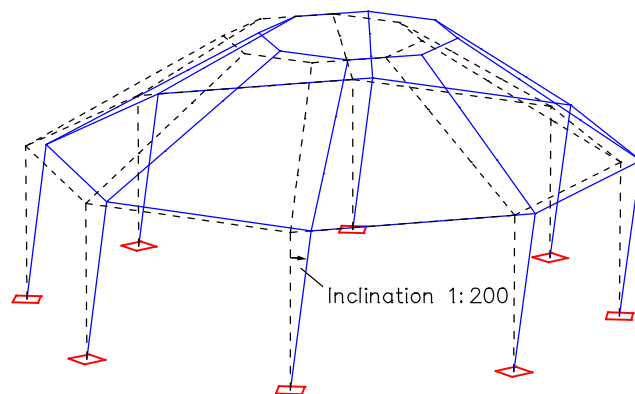


**Load data Load case 2 (Wind)**



**Load data Load case 3 (second-order theory)**

The illustrated predeformation is taken into account for the analysis according to the second-order theory.



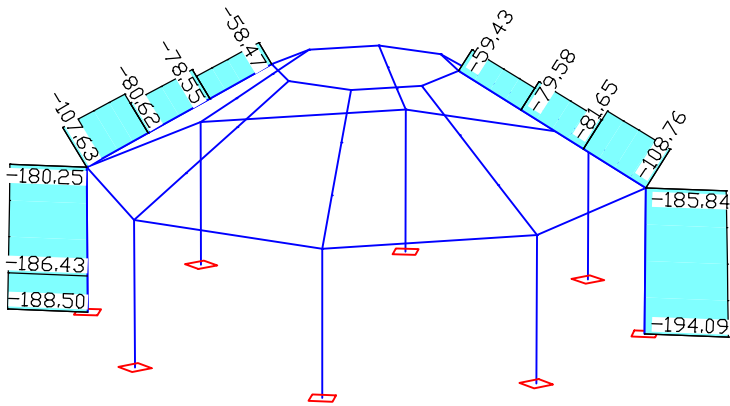
**Load data load case 3: Second-order theory**

Load group (GRL)  
 Theory: 2. order theory  
 No soil pressure > 0: No; No support reac. < 0: No; Error threshold [%]: 1.00  
 Additional global load factor: 1.10; Predeformation: 1  
 Consider concrete creeping in the nonlinear analysis: No

Selected load cases

No.	Label	Factor
1	Dead load	1.35
2	Wind	1.5

Selected internal forces  $M_y$  [kNm] from load case 3



Selected internal forces  $N_x$  [kN] from load case 3

**Design per EN 1993-1-1:2005/A1:2014**

According to Clause 6.2.1 of the standard, for classes 1, 2, 3 and 4 the elastic resistance of the cross-section is verified.

For Class 4 cross-sections the check is carried out with effective properties as per EN 1993-1-5, Clause 4.3.

The actions are combined acc. to EN 1990, Eq. (6.10), using the partial and combination factors according to EN 1990:2021.

**Designing occurs for the extreme values of actions.**

**Terms in Printout**

- Sigma.x, Sigma.v: Normal stresses, comparison stresses acc. to Eq. (6.1).
- Tau.xy, Tau.xz : Shear stresses from lateral forces  $Q_y$ ,  $Q_z$  and tors. moment  $M_x$ .
- A : Sectional area.
- $I_x, I_y, I_z, I_{yz}$  : Moments of inertia.
- y, z : Location in the sectional polygon [m].

**Stress Calculation**

The normal stresses and shear stresses are calculated for homogeneous polygon sections from normal force, bending moments, lateral force and torsion. The shear characteristics are calculated acc. to the Boundary-Element-Method. The calculation points for all stresses are edge points of the cross-section.

**Cross-Section Classification**

The classification occurs for standard profiles and parameter cross-sections acc. to EN 1993-1-1, Table 5.2. T-Profiles are handled as outstand flanges. User-defined polygonal cross-sections are classified always into class 3.

**Partial Safety Factors for Steel**

	gamma.M0
Permanent and temporary comb.	1,00
Accidental combination	1,00

**Characteristic Material Properties**

Yield stresses for steel acc. to EN 1993-1-1, Table 3.1 for  $t \leq 40$  mm [MN/m<sup>2</sup>]. Limiting stresses according to Eq. (6.19) and Eq. (6.42).

		Perm. C.		Accid. C.	
Material $f_{yk}$	Sigma.Rd	Tau.Rd	Sigma.Rd	Tau.Rd	
S235-EN 235	235,00	135,68	235,00	135,68	

**Cross-Section Properties**

Cross-section	Material	A[mm <sup>2</sup> ]	Iy[m <sup>4</sup> ]	Iz[m <sup>4</sup> ]	Ix[m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
1 HEA 280	S235-EN	9730	1.3670e-04	6.2400e-07		
2 HEA 220	S235-EN	6430	5.4100e-05	2.8600e-07		
3 HEA 180	S235-EN	4530	1.9500e-05	0.0000e+00		
			2.5100e-05	1.4900e-07		
			9.2500e-06	0.0000e+00		

**EN 1993-1-1 actions**

**Standard design group**

**G - Dead load**

$\gamma_{sup} / \gamma_{inf} = 1,35 / 1$

Load cases

-----  
1 Dead load

**QW - Wind load**

$\gamma_{sup} / \gamma_{inf} = 1,5 / 0$

Combination coefficients for: Superstructures

Wind loads  
 $\psi_{0.0} / \psi_{1.1} / \psi_{2.2} = 0,6 / 0,2 / 0$

Load cases 1. Variant, exclusive

-----  
2 Wind

**Fd - Design values of actions**

Load cases

-----  
3 Second-order theory

**1. Permanent and temporary situation**

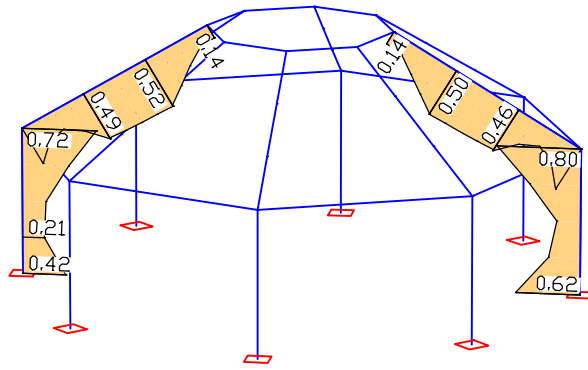
Final state

G - Dead load  
QW - Wind load

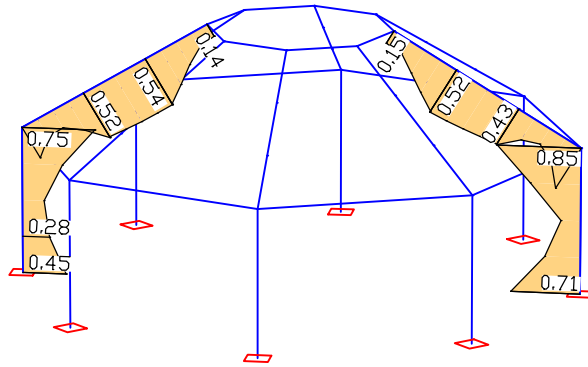
**2. Permanent and temporary situation**

Final state

Fd - Design values of action



Utilization; 1. Permanent and temporary situation

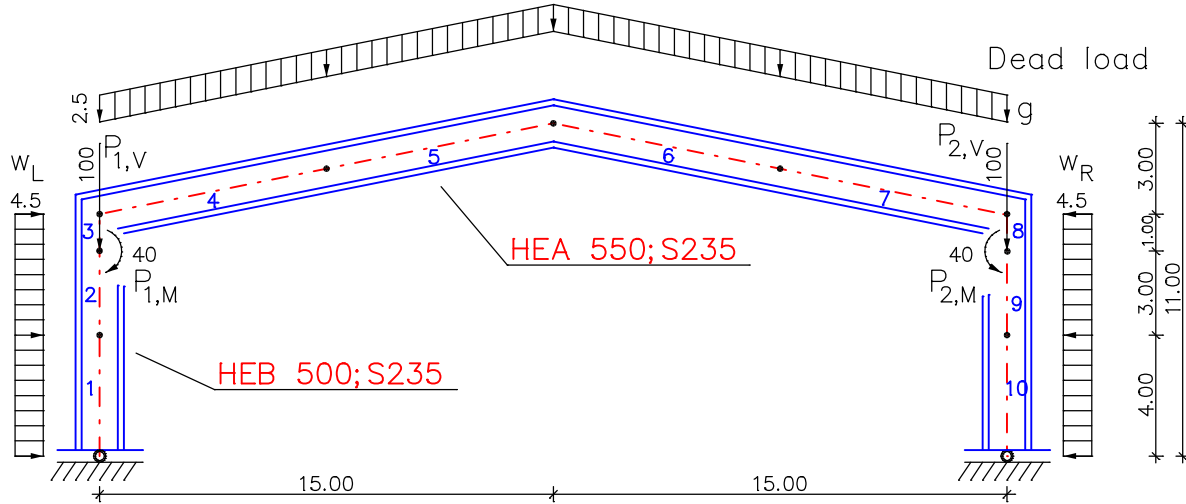


Utilization; 2. Permanent and temporary situation

# 2D Frame

## Basics

The 2D Frame program system is designed for calculating 2D beam constructions. The following example image shows the beam model of a hall frame.



Static system of a hall frame

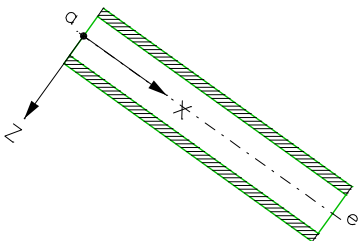
Notes on the bases of calculation are in chapter 'Finite Elements / Basics'.

## Coordinate Systems

A framework model consists of the following structure objects with their own (local) coordinate systems. All coordinate systems are right-handed systems.

### Beams

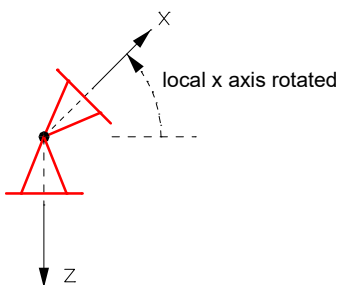
The following illustration depicts the local coordinate system of a beam.



The local beam coordinates have their origin in the beam's start node (a). The local x axis runs from (a) to (e), the beam's end node.

### Supports
















Normally, the support system corresponds to the global coordinate system. The support system can be changed by rotating the local axes.



Support system with rotated x axis

## Structure Description

The following *Structure* menu functions can be used for system input and the graphical representation of the results:

-  *Element Input*
-  *Element Properties...*
-  *Element Info*
-  *View...*
-  *Mesh Check*
-  *Element System*
-  *Number Sort*
-  *Supports*
-  *Load*
-  *Predeformation*
-  *Moving Nodes*
-  *Condense*
-  *Link Elements*
-  *Print Input Data...*
-  *Results...*

The general procedure for data input, analysis and data representation of a structure is described in the document *Introductory\_Examples.pdf* installed in the program directory.

## Element Input

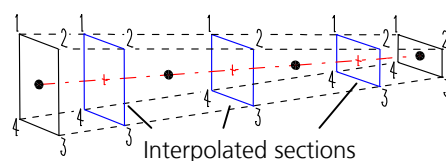
This menu item is used to input beams.

### Input

*Starting point* Input a beam from start to end point.

*Properties* The beam properties dialog is called up with this option.

*Generate* The entered line is split into  $n$  beams of the same length. In conjunction with the polygon, square and beam section types, you can specify different start and end sections. The program then interpolates the selected number of sections. This enables the modeling of a haunch. The position of the polygon points are interpolated for polygon sections. For this to work, the number and arrangement of the polygon points must match one another.



## Element Properties

This dialog is used to edit the properties of selected beams or preset the properties for new beams yet to be created. The dialog consists of the following pages:

- *General or Joints*
- *Eccentricity*
- *Equivalent beam length*
- *Section*
- *Deformation Check*

### General

This dialog shows the general beam properties.

#### Nodes

Display the node coordinates.

#### Length

Display the beam length [m].

#### Joints

The joint properties can be specified separately for each degree of freedom. When selected, the corresponding degree of freedom is defined as jointed. The arrow indicates the selected end of the beam.

#### Joint springs

Spring rigidity [MN/m, MNm] with which the respective beam end (a = start, b = end) is elastically attached to the node. If no value is defined, the respective joint property applies.

### Eccentricity

Beam elements can have different eccentricities ( $u \equiv$  local x,  $w \equiv$  local z) at their start and end in their local coordinate system. The beam reactions refer to the eccentric beam axis, on which possible joints also are located. The nodes are still in their original location.

### Equivalent Beam Length

For the buckling check with the equivalent beam method, the equivalent beam length can be defined for beams made of timber according to EN 338 and EN 14080.

#### y-axis, z-axis

The specification of an appropriate equivalent beam length under consideration of support and load conditions is subject to the discretion and the responsibility of the user. If the value 0 is entered no check is carried out for the corresponding direction.

#### Note

If a structural member with constant cross-section is divided into multiple beam elements, the buckling length decisive for the whole structural member shall be applied to all elements. A computational determination of buckling lengths and buckling eigenmodes by the FEM program is possible for selected load combinations (see description of *Load group*).

## Section

The section describes the following properties of a beam:

- Form
- Specifications for determination of shear stresses
- Material
- Bedding
- Specifications for reinforced concrete design
- Specifications for thermal analysis
- Fire exposure for the simplified check according to EN 1995-1-2

Each section is uniquely identified by a number between 1 and 9999. In general, a section is not assigned to a single beam but rather to a group of beams.

### Number

Select the section. You can assign the number freely for a newly generated section. Sections can subsequently be renumbered using the context menu for the subitem *Structure Description / Section properties* of the data base.

### Section Type

- *Beam*
- *Polygon*
- Steel section with profile number.
  - IPE*: Medium I girders according to DIN 1025 Part 5 and Euronorm 19-57.
  - HEA*: Wide I girders, light variant according to Euronorm 53-62 or girders of the IPBI series according to DIN 1025 Part 3.
  - HEB*: Wide I girders according to Euronorm 53-62 or girders of the IPB series according to DIN 1025 Part 2.
  - HEM*: Wide I girders, reinforced variant according to Euronorm 53-62 or girders of the IPBv series according to DIN 1025 Part 4.
  - I*: Narrow I girders according to DIN 1025 Part 1.
  - U*: U steel according to DIN 1026.
  - T*: T steel according to DIN 1024.
- *Parameter*: Parameterized steel profile with I-, L-, T-, U-, tube- or box-shaped cross-section geometry.
- *Library*: Section library with steel sections.
- *Database*: Profile from the user database (*Anwend.pro*).
- *Rectangle*
- *Tension member*
- *Composite*

**Material Type**

- *<Database>*: User-defined material coefficients from the system file *lgraph.dat*.
- *B25 to B55*: Concrete qualities according to DIN 1045:1988, Table 1.
- *C12/15 to C100/115*: Concrete qualities according to DIN 1045-1 or ENV 1992-1-1:1991 (*C55/67* or higher only for design according to DIN 1045-1).
- *LC12/13 to LC60/66*: Concrete qualities according to DIN 1045-1.
- *C12/15-EN to C90/105-EN*: Concrete qualities according to EN 1992-1-1.
- *C12/15-EN-D to C100/115-EN-D, LC12/13-EN-D to LC80/88-EN-D*: Concrete qualities according to DIN EN 1992-1-1.
- *CX*: Concrete according to DIN 1045-1 with user-defined properties.
- *CX-EN*: Concrete according to EN 1992-1-1 with user-defined properties.
- *CX-EN-D*: Concrete according to DIN EN 1992-1-1 with user-defined properties.
- *ÖC16/20 to ÖC50/60*: Concrete qualities according to OENORM B 4710-1.
- *ÖNB40*: Concrete quality according to OENORM B 4200-10.
- *ÖNBeton*: Concrete according to OENORM with user-defined properties.
- *SC12/15 to SC50/60*: Concrete qualities according to SIA262 or SN EN206-1.
- *SLC12/13 to SLC50/55*: Concrete qualities according to SIA262 or SN EN206-1.
- *SCX*: Concrete according to SIA262 or SN EN206-1 with user-defined properties.
- *S235, S275, S355, S450*: Construction steel according to DIN 18800 T1, Table 1.
- *S235-EN to S500-EN*: Construction steel according to EN 1993-1-1, Tab. 3.1 or EN 10025-2.
- *NH1, NH2, NH3*: Softwood grade I, II and III according to DIN 1052:1988.
- *BSH1, BSH2*: Glued laminated lumber grade I and II according to DIN 1052:1988.
- *C14 to C50, D18 to D80*: Solid timber according to EN 338.
- *GL20h to GL32h, GL20c to GL32c*: Glued laminated timber according to EN 14080.
- *Frei*: User-defined material.
- *Beton*: User-defined concrete.
- *Stahl*: User-defined steel.

**New**

Create a new section.

**Copy**

Copy the current section.

**Delete**

Delete the current section.

**Label**

The section can be labeled with any text.

Depending on the section and material type, further dialogs are available:

- *Form*
- *Shear stresses*
- *Material*
- *Bedding*
- *DIN 1045-1*
- *EN 1992-1-1*
- *BS EN 1992-1-1*
- *DIN EN 1992-1-1*
- *OENORM EN 1992-1-1*
- *SS EN 1992-1-1*
- *OENORM B 4700*
- *SIA 262*
- *EN 1995-1-1*
- *Thermal analysis*



## Polygon Section

### A

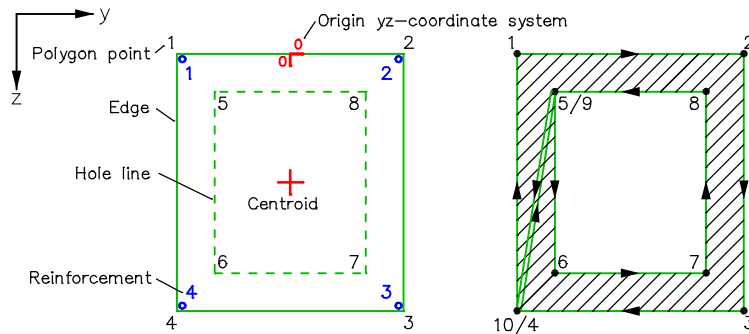
Display of the calculated section area [m<sup>2</sup>].

### I<sub>y</sub>

Display of the calculated bend moment of inertia [m<sup>4</sup>].

### Edit...

The graphical editing of the section geometry is carried out in a separate window. The section is defined with edge and hole bordering lines.



Internal storage takes place using a polygon that is calculated based on the by-pass method. The number of polygon points is limited to 999.

The edge and hole bordering points are automatically numbered such that the leftmost point is assigned number 1. If there are several eligible points, the one with the smallest z value is chosen.

After the polygon points have been entered in the database, the order of the points that has been determined there is also valid for the graphical representation for as long as the polygon remains unmodified.

### Reinforcem...

Properties of each reinforcing steel layer of the selected sections.

### Cross-section class

For polygonal sections and sections from the user database made of construction steel according to EN 10025-2 or the material type *Stahl*, the classification into the cross-section class according to EN 1993-1-1, Table 5.2, is carried out by the user.

## File Menu of the Editing Window

### New

Start a new section polygon.

### Insert

Insert a section polygon, steel structure or database profile in the current polygon.

### Store in database...

Save the current section polygon in the user database *Anwend.pro*.

### Import

Import a drawing of a project file or a DXF file.

### Export

Export the section polygon as a DXF file.

## Polygon Menu of the Editing Window



**Line**

Enter an individual edge or hole bordering line.



**Circle**



**Tube**



**Rectangle**



**Downstand Beam**



**Upstand Beam**

These section types are created automatically after the relevant parameters have been entered. Optionally, reinforcing steel can be simultaneously defined in the specified edge distance.



**Reinforcing steel**

The reinforcing steel for the bending design can be freely positioned in the section. A guide line with the set edge distance is displayed as a visual aid. The displayed reinforcing steel number helps to identify the steel layers during output of the results. The steel properties (e.g. yield strength  $f_{yk}$ , base reinforcement) are preset. For marked reinforcing steel, properties can be edited in the context menu.



**Label**

Switch the labeling of the polygon on or off.



**Refresh**

Refresh the displayed polygon labeling.



**Settings**

Set the type and scope of labeling as well as edge distance for the input of the reinforcing steel.

**Take over**  **edge** /  **hole**

Adopt the edge and/or hole bordering lines from other lines, arcs and circles, for example, after the DXF import of a section drawing.



**Section properties**

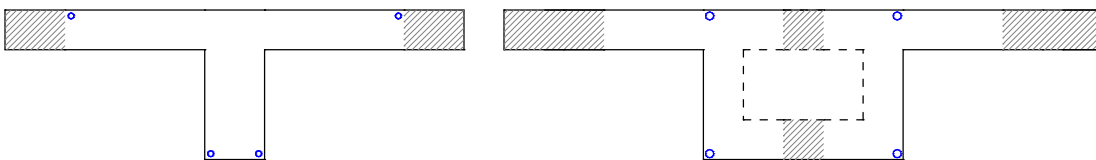
Show detailed section properties (e.g. shear center, resistance momenta, unit warping  $\omega$ ).

### Effective width

Definition of the effective flange width by selection of the areas outside of the effective width. For this purpose the section is automatically divided into triangles. The selected areas are not taken into account for the calculation of the centroid coordinates and the bending moments of inertia.

For reinforced concrete sections the following procedure is recommended:

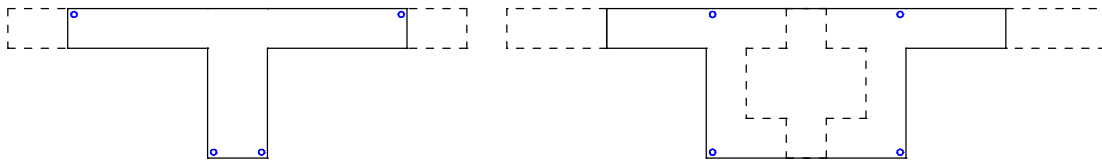
- Input of the full polygon with polygon points at the boundaries of the effective width
- Selection of the areas outside of the effective width
- Positioning of the concrete steel layers within the effective width



*Hatched representation of the section parts outside of the effective width*

The check in the ultimate and fatigue limit states are carried out with the cross-section areas within the effective width. For the crack and stress checks in the SLS, the normal forces are related to the total cross-section (Rossner/Graubner (2012), p. 228). In a *nonlinear system analysis* the entire concrete cross-section is used.

Alternatively, the section polygon can be defined reducing the section dimensions based on the effective width.



Section polygons reduced to the effective width

## Reinforcement Properties

### Yield strength $f_{yk}$

Yield strength of the reinforcing steel [MN/m<sup>2</sup>].

### Base reinforcement

The base reinforcement is the initial reinforcement for the design. It will never be reduced by the program. By entering different base reinforcements, a relationship can be established among the steel sections to be calculated.

### Fix base reinforcement

A fixed steel layer are not increased during the design.

### Zv0

This force is used to take into account an initial strain  $\varepsilon_0 = Z_{v0} / E \cdot A$  for calculation of the reinforcing steel stresses.

### Rolling-kind

In the nonlinear structural analysis for fire scenarios the parameters of the stress-strain curves of the reinforcing steel according to EN 1992-1-1, Figure 3.3 and Table 3.2a depend on the temperature and the rolling-kind:

- *Cold-formed*
- *Hot-rolled*

## Rectangle Section

For this section type no reinforced concrete check can be carried out.

### Width $d_y$ , Height $d_z$

Expansion of the rectangle in the local y or z direction of the beams [m].

## Beam Section

No checks can be performed for this section type.

### A

Section area [m<sup>2</sup>].

### I<sub>y</sub>

Bend moment of inertia [m<sup>4</sup>].

### Longitudinal bearing

- *Standard*: The beams transmit compressive and tensile forces.
- *Only pressure*: The beams only transmit compressive forces.
- *Only tension*: The beams only transmit tensile forces.

The use of compressive or tensile beams leads to a nonlinear calculation. In this case the principle of superposition no longer applies.

## Tension Member

This section type describes beams with no compressive strength. They should be attached flexibly, if possible. The use of this section type leads to a nonlinear calculation. This means the superposition principal is no longer valid. No checks can be performed for tension members.

### A

Section area [m<sup>2</sup>].

## Composite Section

This section type can only be used for single designs.

The section properties of the composite section are calculated with the following formulas:

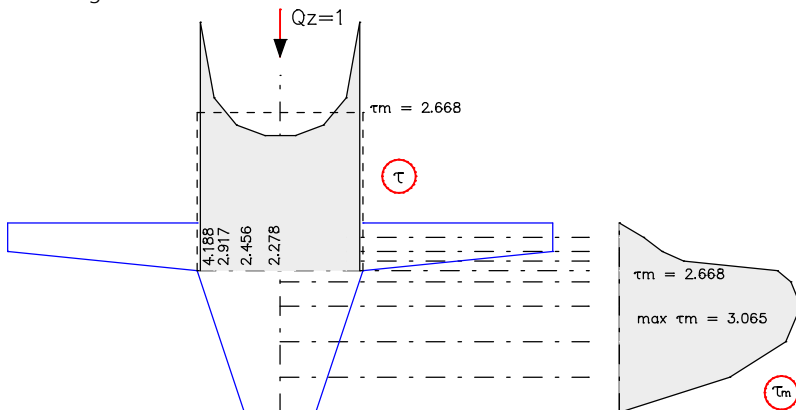
$A_v = 1/E_{\min} \cdot \sum E_i \cdot A_i$	Composite section area
$z_{s,v} = 1/(A_v \cdot E_{\min}) \cdot \sum E_i \cdot z_{s,i} \cdot A_i$	Z ordinate of the centroid
$y_{s,v} = 1/(A_v \cdot E_{\min}) \cdot \sum E_i \cdot y_{s,i} \cdot A_i$	Y ordinate of the centroid
$I_{y,v} = 1/E_{\min} \cdot \sum E_i \cdot [I_{y,i} + (z_{s,v} - z_{s,i})^2 \cdot A_i]$	Second-degree area moment around y
$I_{z,v} = 1/E_{\min} \cdot \sum E_i \cdot [I_{z,i} + (y_{s,v} - y_{s,i})^2 \cdot A_i]$	Second-degree area moment around z
$I_{yz,v} = 1/E_{\min} \cdot \sum E_i \cdot [I_{yz,i} + (y_{s,v} - y_{s,i}) \cdot (z_{s,v} - z_{s,i}) \cdot A_i]$	Deviation moment of inertia

## Shear Stresses

By default the linear-elastic shear stresses are calculated according to the theory of thick-walled profiles.

As a result, in the fillet area of rolled sections or in blunt corners of welded profiles larger stresses occur than those according to the theory of thin-walled profiles.

An average across the section width can be selected for shear stresses from lateral force.



Intersections and shear stress distribution from lateral force at the section polygon according to the theory of thick-walled profiles ( $\tau$ ) and after averaging ( $\tau_m$ )

## Material

### Properties for all Materials

#### E-Modulus

Modulus of elasticity [MN/m<sup>2</sup>].

Note: The modulus of elasticity is part of the stress-strain function according to EN 1992-1-1, Equation (3.14), and SIA 262, Equation (28). The function has a pole that can move into the decisive strain range, if the given modulus of elasticity is reduced. As a consequence, results of the nonlinear analysis and the reinforced concrete checks can be affected.

**G-Modulus**

Shear modulus [MN/m<sup>2</sup>].

**Gamma**

Specific weight of the material [kN/m<sup>3</sup>]. This is used for the *Dead load* load type.

**alpha.t**

Coefficient of thermal expansion [1/K].

**Material linear**

With this switch linear elastic material behavior is assumed for this cross-section during the nonlinear structural analysis. In general, a nonlinear analysis can only be performed for polygon sections, database sections and steel sections. For all other section types and for the material types *Beton* and *Timber* a linear material behavior is always assumed.

**CX (DIN 1045-1), CX-EN-D (DIN EN 1992-1-1), CX-EN (EN 1992-1-1)**

For the user-defined concrete material types *CX*, *CX-EN-D* and *CX-EN* additional concrete parameters according to DIN 1045-1, Table 9 or EN 1992-1-1, Table 3.1 are defined in addition to the  $\alpha$  values. Non-editable values are determined by linear interpolation with  $f_{ck}$  in the corresponding table.

**fck**

Characteristic value of compressive cylinder strength of concrete at 28 days [MN/m<sup>2</sup>].

**alpha or alpha.cc**

Coefficient which takes into account the long term effects on the compressive strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

**fctm**

Mean value of axial tensile strength of concrete [MN/m<sup>2</sup>].

**alpha.ct**

Coefficient which takes into account the long term effects on the tensile strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

**Lightweight Concrete****Apparent density**

Apparent density of lightweight concrete [kg/m<sup>3</sup>].

**Concrete SIA 262****ke for E-Mod.**

Factor for calculation of the modulus of elasticity according to SIA 262, Section 3.1.2.3.3.

For the material type *SCX* the concrete properties are defined additionally according to SIA 262, Table 3.

**fck**

Characteristic cylinder compressive strength [MN/m<sup>2</sup>].

**fck,cube**

Characteristic cube compressive strength [MN/m<sup>2</sup>].

**fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

## ÖNBeton (OENorm B 4700)

For this material type the concrete properties are defined additionally according to OENORM B 4700, Table 4.

### **fcwk**

Characteristic cube compressive strength, nominal strength of the strength class [MN/m<sup>2</sup>].

### **fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

### **fctk**

Characteristic concrete tensile strength [MN/m<sup>2</sup>].

### **taud**

Calculation value of shear stress [MN/m<sup>2</sup>].

## Frei (General)

For the material type *Frei* (free) the corresponding strength limits must additionally be specified for a nonlinear system analysis.

### **Compression**

Yield strength for compressive stresses  $f_{y,\text{compression}}$  (positive value) [MN/m<sup>2</sup>].

### **Tension strength**

Yield strength for tensile stresses  $f_{y,\text{tension}}$  (positive value) [MN/m<sup>2</sup>].

## Stahl (General)

### **fyk**

Characteristic yield strength [MN/mm<sup>2</sup>].

## Timber EN 338, EN 14080

### **Apparent density**

Apparent density of timber [kg/m<sup>3</sup>].

### **Service Class**

Service class for timber checks.

## Bedding

### **Bedding modulus x, z**

Bedding moduli for beam directions [MN/m<sup>3</sup>]. The bedding moduli can be linearly variable between the beginning (a) and the end (e) of the beam.

### **Bedding width**

For each bedding modulus an bedded width [m] is to be specified.

### **Compressive or tensile strength (nonlinear system analysis)**

The strengths [MN/m<sup>2</sup>] defined here are only valid for nonlinear system analysis. This can be used to realize a bilinear bedding characteristic in the compressive and tensile section (see also *Foundation models*). The strengths can be linearly variable between the beginning (a) and the end (e) of the beam.

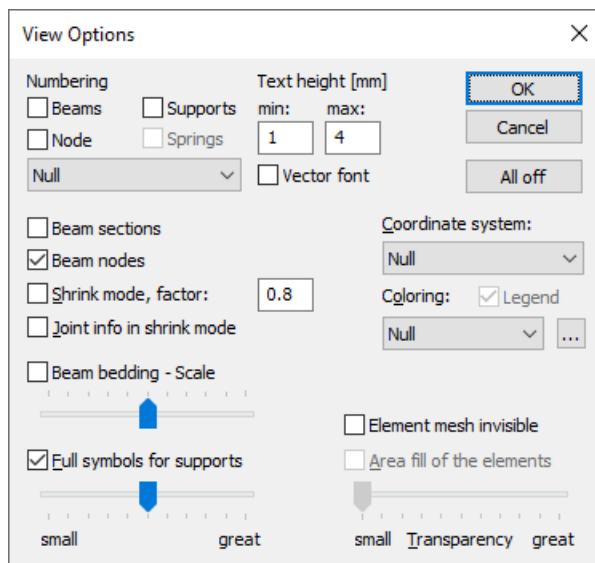
## Element Info

This function shows the following beam properties graphically at the beam and in the status bar:

- Beam number
- Node numbers
- Beam system
- Section
- Material
- Joints

## View

This menu item is used to display and check the structure properties.



### Numbering

You can label beams, nodes and supports with their numbers. Section numbers can also be added to beams. The display of the numbers can be limited to values  $> 1$ . Alternatively, the section name can be added to beam.

### Text height

Set minimum and maximum text heights [mm].

### Beam section

The beam section is drawn at the middle of the beam.

### Beam nodes

The beam nodes are marked by a symbol.

### Shrink mode

Enable or disable shrunken beam representation. Most of the input functions are disabled in this mode.

### Shrink factor

Define the shrink factor for the shrink mode.

### Joint info in shrink mode





In shrink mode the existing beam end joints are labeled. Joint springs are marked with an asterisk '\*'.

### Beam bedding

The bedding moduli are plotted along the beam axes with the specified scale.

### Full symbols for supports


The properties of the supports are shown in detail. The scroll bar changes the size of the icons in use.

Icons	$u_x$	$u_z$	$\varphi_y$
	<b>Fixed</b>	<b>Fixed</b>	Free
	<b>Fixed</b>	<b>Fixed</b>	<b>Fixed</b>
	Free	<b>Fixed</b>	Free
	<b>Fixed</b>	Free	<b>Fixed</b>

### Coordinate system

Activate the coordinate system for beams or supports.

### Coloring

The objects are drawn in different colors, depending on the selected property (e.g. *material*, *section*). If necessary, the color assignment can be modified with the  button. The possible objects that are distinguished by color include elements, design objects and model objects. The properties *As top*, *bottom* refer to the base reinforcement of the cross-section. Color 254 is used for *Undefined*.

### Legend

Enable/disable the color legend.

### All off

Turn off all display options.

## Mesh Check

This menu item is used to check beams and nodes.

### Node distance

The function checks if there are further nodes in the neighborhood of a node within the given tolerance. After confirmation, the so-called duplicate nodes are marked by red arrows and you are queried if you want to merge them. In this case their coordinates are averaged.

### Beam distance

Checks if any beams lie on one another, for instance, due to redundant beam entry. The beams that are found are marked by red arrows.

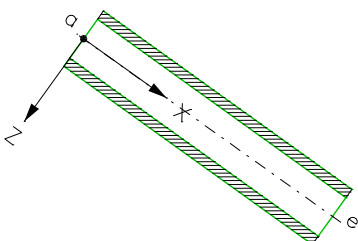
## Element System

This menu item is used to change the coordinate system of selected beams.

### Input

+X, -X, +Z, -Z

By exchanging the start and end nodes of the beam, the positive or negative axis is directed at the specified point.





## Number Sort

This menu item is used to change beam and node numbers.

### Input

<i>Sort</i>	Sort node or beam numbers geometrically.
<i>Define</i>	Define new start values and increments for node and beam numbers.
<i>Edit</i>	Edit node and beam numbers.

## Supports

This function is used to define supports. A support describes the support conditions of a system node. The individual degrees of freedom can be supported in a fixed or elastic manner. The direction of the degrees of freedom can deviate from the global coordinate system (described by an angle). The supported nodes are labeled with simple or detailed red icons (see *View*).

### Create

Start the input of supports through the selection of nodes. The subsequent editing is performed using the context menu functions:

- Support
- Orientation

### Info

Display the support system with the bound degrees of freedom for a selected support.

Note: When the function is active only supports can be selected. This improves the handling of these objects.

## Predeformation

This function is used to enter and edit stress-free node displacements for a structure. As part of the system analysis, these node displacements are added to the initial node coordinates and may serve for example as a geometric imperfection for calculations according to the second-order theory. Therefore, the calculated nodal deformations refer to the already predeformed structure. To be considered for calculations, the predeformation must be activated within the *load group* definition.

The following options allow you to define predeformations or subsequently change them, if necessary.



Create a copy of the current predeformation.



Add the selected predeformation to the current predeformation.



Start the representation options dialog.



End the predeformation display.



Create a new predeformation.



Display an existing predeformation.



Change the number of the current predeformation.



Delete the predeformation.



Edit the predeformation.

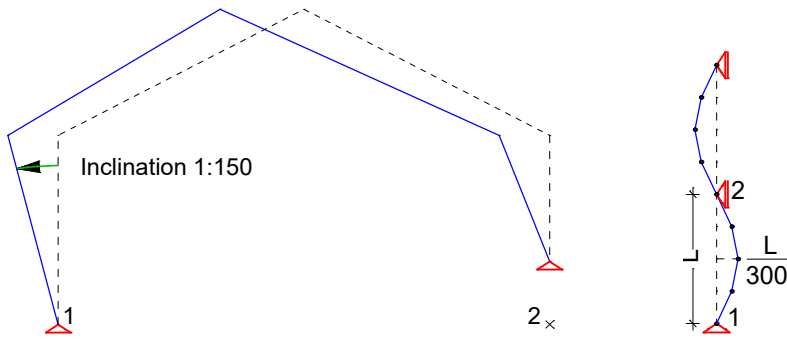


Start the precurvature input.



Start the predeformation input.

Absolute input of the predeformation refers to the initial system, relative input refers to the already deformed system.



Predeformation of the selected node of a frame

Precurvature of a column

1: 1<sup>st</sup> point of base line, 2: 2<sup>nd</sup> point of base line

The handling is also shown in the video "[https://download.infograph.de/video\\_de/Vorverformung%20definieren.mp4](https://download.infograph.de/video_de/Vorverformung%20definieren.mp4)" (German language).

## Moving Nodes

This function is used to change the location of selected nodes. In the case of a single beam node, simply specify the new position. If multiple beam nodes are to be moved, you should choose the *Group* or *Beams* option.

### Group

Select the beam nodes to be moved.

### Beams

Select the beams, whose nodes are to be moved.

### Displacement vector

Movement through displacement. The displacement vector is defined by two points.

### Rotate

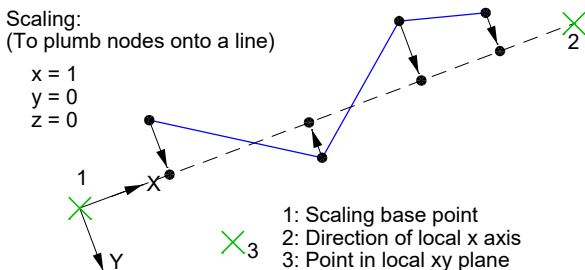
Movement through rotation.

### Mirror

Movement through mirroring.

### Scaling

Movement through scaling in the global or local reference system. You can align nodes on an axis by specifying a scaling factor of zero.



If desired, displaced nodes that coincide with non-selected nodes can be merged with these.

## Finding Nodes or Beams

If you are looking for a node with a particular number, the functions *Select Nodes* and *Move Nodes* can be of help. Enter the node number you are looking for into this function.

If you are looking for a beam with a specific number, use the functions *Element Info* or *Select*. The beam number can be entered in both functions. If the beam is not identifiable even though it is selected, you can query further information using the *Element Properties* function. The selected objects' color and layer are displayed in the toolbar.

## Condense

This function is used to condense existing beams.

### Point

At the point specified a node is inserted on the beam axis.

### Divide

Selected beams are divided. Enter the desired number of parts.

### Adapt

The selected beams are checked for whether any nodes touch their axes. If necessary, the beams in question are divided at these nodes.

## Link Elements

Kinematic dependencies between the global degrees of freedom of two beam nodes can be defined with the *Link Elements* function. This is done by expressing the nodal degrees of freedom of a dependent node through the degrees of freedom of the reference node by way of an equation of condition.

for instance  $u_{x,dependent} = f_1 \cdot u_{x,reference} + f_2 \cdot u_{z,reference} + f_3 \cdot \varphi_{y,reference}$

with

$f_1, f_2$  Factors for the displacements,

$f_3$  Factor for the rotation.

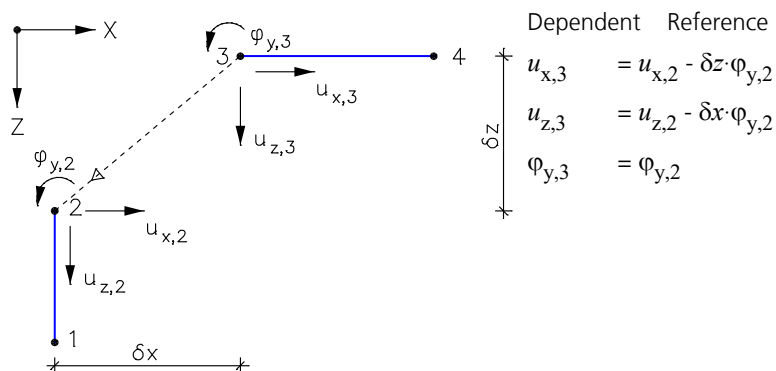
You can link any number of dependent nodes to a reference node. Nodes with supports may not be defined as dependent nodes. The link elements are displayed with dotted lines. Additionally, an arrow points to the reference node.

After selecting the dependent nodes and the reference node, you must define the link element type.

### Type

- *Stiff*: Stiff link element (abbreviation: SS).
- *Jointed*: One-sided jointed link element with a ball joint at the dependent node (abbreviation: GS).
- *ux, uz, phi.y*: Dependent degree of freedom of the user-defined equation of condition. The factors for the displacements and rotations of the equation of condition have to be defined.

### User-defined equations of condition for a stiff link element



## Arranging Beams

This function is used to align selected beams with a reference beam. The local z axes of the beams must point in the same direction. The alignment can be accomplished in two ways:

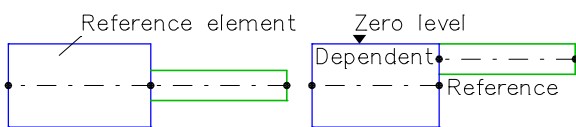
- Use of eccentricities
- Use of link elements

The orientation is performed through displacement in the local z direction until the zero levels correspond to those of the reference beam.

The zero level is at  $z=0$  in the section polygon. Optionally, the level may be defined as follows:

- $z=0$  (default)
- Top face
- Bottom face

If eccentricities are used, the offset is realized via corresponding eccentricities in the beam element. This means, no new nodes are created. If link elements are used, the program possibly creates new nodes and additional stiff link elements between the nodes, because of the offset. The top node of the link element becomes the dependent node, as long as it does not have a support defined. Already existing link elements are retained.














Beams before and after the alignment (default)

## Load

The following chapter describes load input as well as the different load types. The specified loads are represented with icons.

Beam loads generally act on the centroidal axis of the beams. For asymmetrical sections the effect of the eccentricity of the shear center on the fixed-end reactions is not accounted for.

A load case is made up of one or multiple loads. The following options allow you to define load cases or subsequently change them, if necessary.

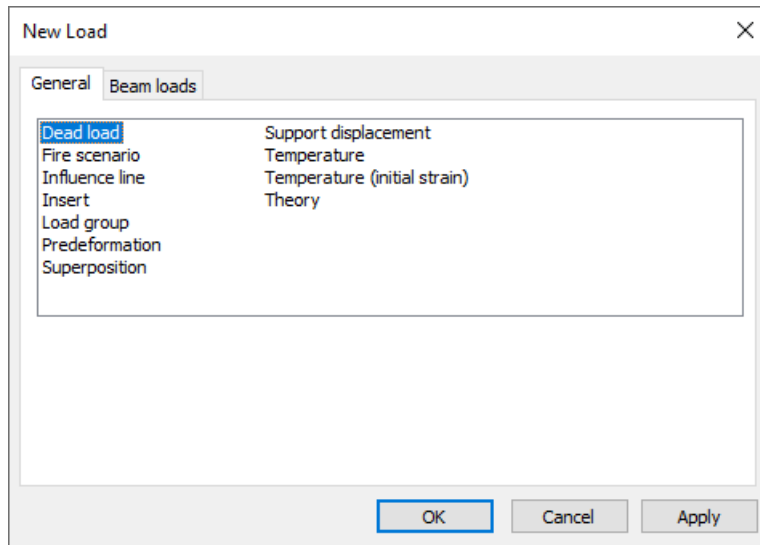
-  Create a copy of the current load case.
-  Generate load cases according to the rules set out by the *Generate* function. This leads to the duplication of either all or only the selected loads.
-  Import load data.
-  Start the representation options dialog.
-  End the load representation.
-  Create a new load case.
-  Open the previous load case.
-  Open the next load case.
-  Show an existing load case.
-  Change the number and name of the current load case.
-  Start the load input.

Load cases can be renumbered in groups within the database tree.

Double-clicking on an existing load object or the context function *Load* opens a dialog in which changes to the selected load objects can be made. With the context function *Scale load* the load values of multiple selected loads can be scaled with a factor.

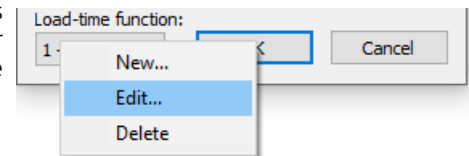
## Input

The dialog box is used to input load data within a load case.



The load types are explained on the following pages. For easier identification they are labeled with load abbreviations.

For many load types a dialog for further load data appears after the load has been entered graphically. In this dialog a load-time curve can be defined for most load types for a non-linear system analysis. If no curve is selected, the load is constant.



## Dead Load

The entire structure is stressed by its dead load in the global z direction (load abbreviation: *EG*).

## Influence Line

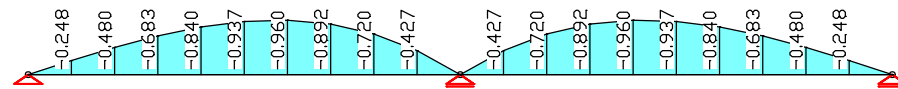
### Input

Selection of the desired beam (load abbreviation: *EFL*).

*Load point* Load point in the beam. The distance from the beam start is saved.

*Influence line* "Nx", "Qz", "My".

The ordinates of the resulting influence line are expressed as deformation in the direction of the load to be analyzed.



Influence line "My" for a continuous girder

## Inserting of Load Case Data

This load type (load abbreviation: *EINF*) allows you to insert load data from other load cases in the current load case in a weighted manner. The *Superposition* load type may not be present in the inserted load cases.

### Input

*First load case* First load case that is inserted.

*Last load case* Last load case that is inserted.

*Weighting factor* Factor by which to multiply all load data.

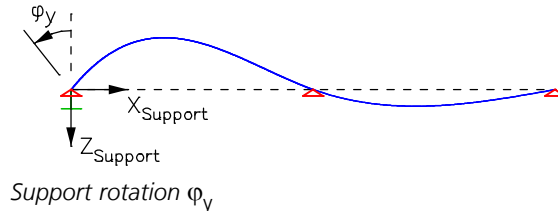
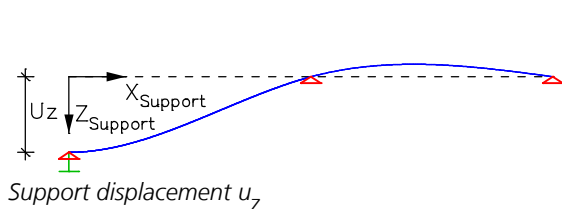
## Support Displacement

The support displacement acts on supported nodes in the direction of the bound degrees of freedom (load abbr.: *DS*).

### Input

Selection of the affected supports.

$ux, uz$  Displacements [mm].  
 $phi.y$  Rotations [rad/1000].



## Superposition of Load Cases

Through superposition (load abbreviation: *SUP*) the existing results of different load cases can be added and weighted. Other load types are not permitted in the load case in question. Because the load cases are analyzed in ascending order, the load numbers of the load cases to be superposed should be smaller than the number of the current load case.

### Input

*First load case* First load case to be superposed.  
*Last load case* Last load case to be superposed.  
*Weighting factor* Factor by which to multiply all results.

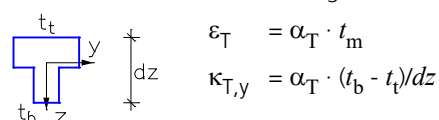
## Temperature

### Input

Selection of the stressed beams (load abbreviation: *TEMP*).

$tm$  Temperature as a difference to the installation temperature [K].  
 $tb - tt$  Temperature difference between the bottom and top side of the section [K].

For the calculation the following coherences apply:



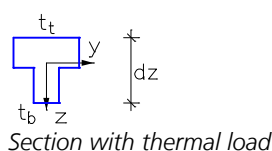
This load type is inadmissible for the section type *Beam*, because the dimension  $d_z$  is not defined. Alternatively the load type *Initial Strain* can be used.

## Temperature, Initial Strain

### Input

Selection of the stressed beams (load abbreviation: *TMPA*).

$alpha.t$  Coefficient of thermal expansion [1/K].  
 $tm$  Temperature as a difference to the installation temperature [K].  
 $(tb - tt)/d$  Temperature difference between bottom and top side of the section with respect to the section height [K/m].



## Theory

This load type (load abbreviation: *TH*) is used to specify the calculation theory for the load case in question.

### Input

<i>Geometrical theory</i>	1. / 2. / 3. order: Calculation according to geometric first-, second- or third-order theory.
<i>Number of iterations</i>	This value is not used at the moment.
<i>Error</i>	Error threshold for the relative size of the remaining residuum in [%]. Only values $\leq 1\%$ are permissible.
<i>No soil pressure &gt; 0</i>	Disregard tensile stresses in the foundation area. For a nonlinear system analysis the tension strength specified for nonlinear bedding is always used.

When using this load type, make sure to pay attention to the notes provided in the *Load group* section.

## Predeformation

This load type (load abbreviation: *VV*) is used to predeform the structure. To do so, enter the predeformation number.

## Fire Scenario

This load type (load abbreviation: *FIRE*) indicates that this load case will be calculated with respect to the selected fire scenario. It is only taken into account for nonlinear system analyses (for fire scenarios).

## Load Group

This load type (load abbreviation: *GRL*) is especially suited for preparing load cases for nonlinear calculation. It allows you to compile loads from existing load cases by retrieving the necessary load factors and to activate the desired calculation theory. The load group is used to combine the existing *Insert*, *Theory* and *Predeformation* load types into a single load type.

Load groups can also be modified via a dialog in the section *Loads* of the data base. The *Superposition* load type may not be present in the inserted load cases.

### General

<i>Theory</i>	1. / 2. / 3. order theory: Calculation according to geometric first-, second- or third-order theory.
<i>Error threshold</i>	Error threshold for the relative size of the remaining residuum in [%]. Only values $\leq 1\%$ are permissible.
<i>No soil pressure &gt; 0</i>	Disregard tensile stresses in the foundation area. For a nonlinear system analysis the tension strength specified for nonlinear bedding is always used.
<i>Add. global load factor</i>	Additional factor for all loads.
<i>Predeformation</i>	Number of the predeformation.
<i>Load cases</i>	Select load cases to add them to the selection list.
<i>Load factor</i>	The individual load cases are weighted by a factor.
<i>Apply</i>	Change the weighting factor for selected load cases.

### Concrete creeping, shrinkage

In combination with the nonlinear analysis a load case related definition of the creep and shrinkage coefficients is available for standardized concrete material with nonlinear material behavior. A description of the calculation method can be found in chapter *Nonlinear structural analysis*.

*Consider concrete creeping, shrinkage in the nonlinear analysis*

Enables concrete creeping and shrinkage for the cross-sections, which are listed in the table. For first-time activation available cross-sections are added automatically to the table. Already defined coefficients are taken over.

*Refresh*

Calculate the creep and shrinkage coefficients for every activated line.

*All sec. identical*

Takes over the input values of the 1st table entry for the further cross-sections.

<i>phi</i>	Creep coefficient $\varphi(t, t_0)$ .
<i>Calculated</i>	Yes: The creep and shrinkage coefficients are determined from the further parameters.
<i>t0, t</i>	Load start and concrete age in days.
<i>Factor</i>	Optional factor for the calculated creep coefficient. This is used for example for the determination of $\varphi_{ef}$ according to EN 1992-1-1 from the calculated coefficient.
<i>eps.cs*1e5</i>	The shrinkage coefficient $\varepsilon_{cs}(t, t_0)$ multiplied with $1 \cdot 10^5$ . A negative value must be entered for a contraction.

When using this load type, the following should be observed.

### Calculation according to the second- or third-order theory

Before carrying out a calculation according to the second- or third-order theory, make sure the system is kinematically stable according to the first-order theory. A failure in stability due to beam buckling is recognized during calculation according to the second-order theory by a singularity of the system of equations or of a beam stiffness matrix. The calculation aborts in this case and informs you about the location and the failed degree of freedom. In borderline cases it may be necessary to set the error threshold to '0' in order to determine the failure state.

### Error threshold

All nonlinear problems are solved iteratively. The length of the error vector, the residuum, is used to control the iteration. It is shown for each iteration step in the output bar. The iteration is ended when the remaining residuum has reached the specified error threshold in relation to the initial residuum. The calculation results have, on average, a relative error of the same extent. If the error threshold cannot be reached, the calculation program issues a corresponding message.

### Loss of tensile bedding

Unrealistic load combinations (for instance, a missing dead load) can lead to loss of all bedding (see also *Foundation models*). If the system becomes relocatable due to this, the calculation is aborted because of a singular system of equations. The relocatable degree of freedom is provided.

### Consideration of tension and compression beams

For structures with tension and compression beams, the iterative determination of the equilibrium state is automatically carried out with an error threshold of 1%. In the load case a smaller value can be specified. Instabilities due to beam failure lead to a singular system of equations, in which case the calculation will abort.

### Load combination

In order to combine the decisive actions for calculations according to the second-order theory or with loss of tensile bedding, transfer the corresponding load cases to the selection list and assign them load factors.

For calculations according to DIN 18800, Part 2, or EN 1993-1-1 the following different cases exist:

- Calculation according to the geometric second-order theory with linear material law according to elasticity theory and subsequent elastic-elastic or elastic-plastic check. The internal forces are to be determined according to the standard with  $\gamma_M$ -fold design values of the actions. To do so, the load cases of the selection list are to be weighted with the factor  $\gamma_F$ . Additionally,  $\gamma_M$  is to be specified as the global load factor.
- Ultimate limit state check according to the geometric second-order theory with nonlinear material law according to the plasticity theory. In this case  $\gamma_M$  is considered on the resistance side according to of the standard. The internal forces calculation is to be carried out with the design values of the actions. Therefore, only  $\gamma_F$  needs to be provided as a weighting factor.

### Prohibition of superposition

Calculations with tension and compression beams, according to the second-order theory or excluding tensile bedding are performed on the basis of nonlinear iteration. So in these cases the superposition principle is no longer valid, meaning the results may not be additively superposed with other results. Further processing using load case combinations is only permissible if these only contain mutually exclusive actions.



## Point Load

### Input

<i>Global orientation</i>	Point load in the global system (load abbreviation: <i>EGX, EGZ</i> ).
<i>Local orientation</i>	Point load in the beam system (load abbreviation: <i>EL</i> ).
<i>Load point</i>	Load point on the beam axis, which is specified depending on the load orientation. <i>Global orientation x, z:</i> Coordinate difference $\Delta x, \Delta z$ to the beginning of the beam. <i>Local orientation:</i> Distance from the beginning of the beam.
<i>Px, Pz, My</i>	Load components in the directions of the selected coordinate system [kN] or [kNm].

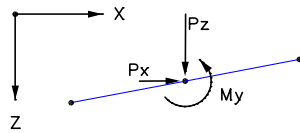
## Nodal Load

Nodal loads act in the global system (load abbreviation: *KNL*).

### Input

Selection of the stressed beam nodes.

<i>Px, Pz</i>	Forces [kN].
<i>My</i>	Moment [kNm].

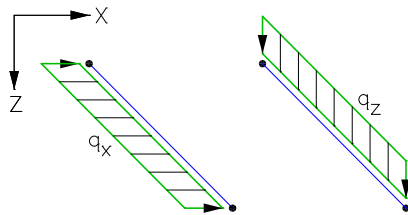


## Line Load

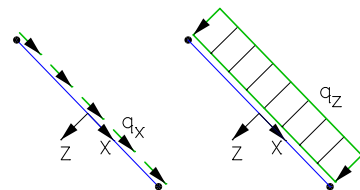
### Input

Selection of the stressed beams.

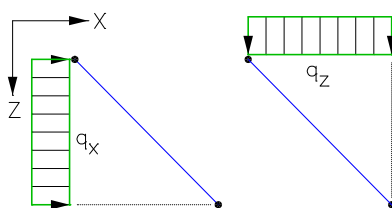
<i>Global orientation</i>	Line load in the global system (load abbreviation: <i>LG</i> ).
<i>Local orientation</i>	Line load in the beam system (load abbreviation: <i>LL</i> ).
<i>Projection orientation</i>	Projection of the trapezoidal load in the global system (load abbreviation: <i>LP</i> ).
<i>qx, qz</i>	Load components in the directions of the selected coordinate system [kN/m].



*Global line load*



*Local line load*



*Projective line load*

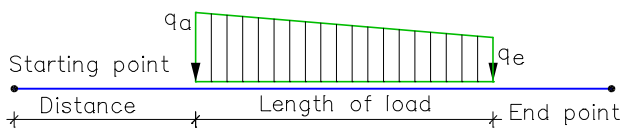
## Trapezoidal Load

### Input

Selection of the stressed beams.

<i>Starting point</i>	Load start.
<i>End point</i>	Load end.
<i>Global orientation</i>	Trapezoidal load in the global system (load abbreviation: <i>TG</i> ).
<i>Local orientation</i>	Trapezoidal load in the beam system (load abbreviation: <i>TL</i> ).
<i>Projection orientation</i>	Projection of the trapezoidal load in the global system (load abbreviation: <i>TP</i> ).
<i>Starting ordinate</i>	Load ordinate at the load start in the directions of the selected coordinate system [kN/m].
<i>End ordinate</i>	Load ordinate at the load end in the directions of the selected coordinate system [kN/m].

The load is automatically distributed over multiple beams for a trapezoidal load. The distances from the beam starts, the load lengths and the interpolated load ordinates are automatically determined for the selected beams. This allows you to describe a trapezoidal load for an entire beam series with a single input.

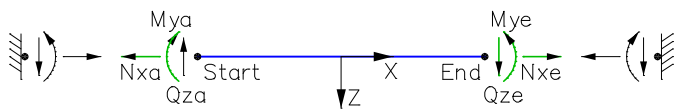


## Fixed-End Reactions

### Input

Selection of the stressed beams (load abbreviation: *S0*).

$N_a, Q_a, N_e, Q_e$	Normal and lateral force at the start and end of a beam [kN].
$M_a, M_e$	Moment at the start and end of a beam [kNm].



Fixed-end forces at the start and end of a beam.

## Analysis

The *Analysis* menu is used to make program-specific settings and to launch the analysis programs.



*Settings...*



*Batch...*

*Construction Stages (Batch calculation of all Construction Stages)*



*Statics (linear material laws)*

*Nonlinear analysis*

*Section Temperatures*

*Load Case Combination*

*BS EN 1992-1-1 Design*

*DIN 1045-1 Design*

*DIN EN 1992-1-1 Design*

*EN 1992-1-1 Design*

*OENORM B 4700 Design*

*OENORM EN 1992-1-1 Design*

*SIA 262 Design*

*SS EN 1992-1-1 Design*

*Single Design*

*DIN 18800 Steel Checks*

*EN 1993-1-1 Steel Checks*

*DIN 1052 Timber Checks*

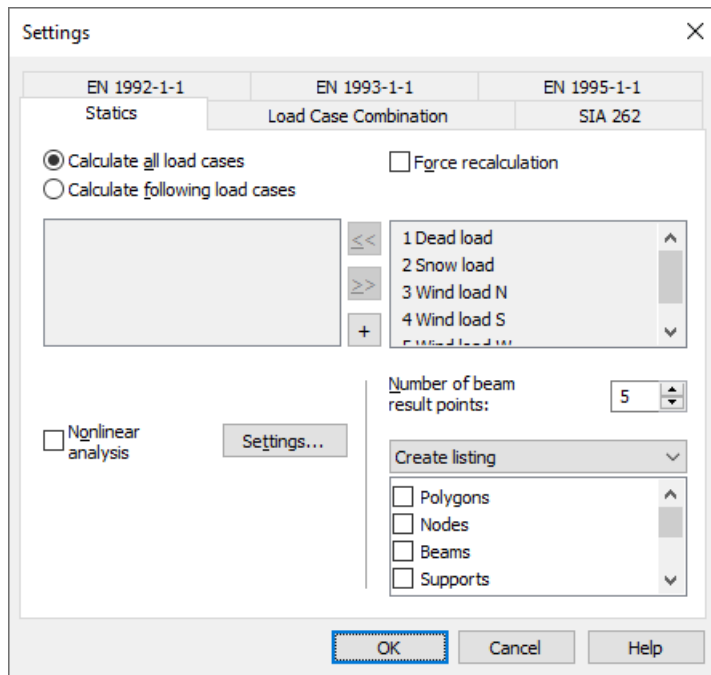
*EN 1995-1-1 Timber Checks*

*Selected Elements*

*Stop Analysis*

During the analysis the program's progress is shown in the output bar along with any possible warnings and error messages.

## Settings Statics



### Calculate all load cases

All load cases are calculated (default).

### Calculate following load cases

The load cases from the left list box are calculated.

### Force recalculation

If this switch is activated, all of the selected load cases will be recalculated. Otherwise, the calculation program checks whether a recalculation is necessary.

### Nonlinear analysis

Instead of statics with linear material laws, a nonlinear system analysis is performed. The *Settings* button opens the settings dialog for the ultimate limit state, the serviceability and the fire scenario check. Load cases selected in the setting dialog (nonlinear analysis) are calculated instead of the selection made here.

### Number of beam result locations

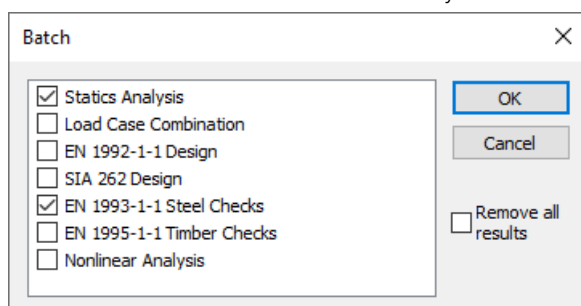
The internal forces are calculated at equidistant points in the beam. By default five result locations are used. For bedded beams the reactions are only calculated in the nodes.

### Listing

- *No listing*: No log is generated by the calculation program.
- *Create listing*: A log is generated during the calculation according to the selection list.

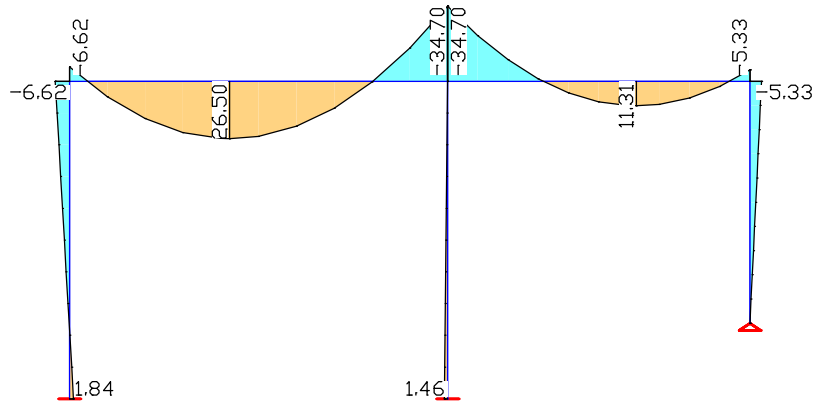
## Batch

Use this menu item to run all selected analyses as a batch job.



## Results

The following chapter describes the representation of the analysis results. Results can be shown graphically as well as in tabular form. Their selection takes place in the database or with the *Results* function of the *Structure* menu.



Internal forces representation  $M_y$  of a hall construction










### Sum of installed loads and support reactions

LC	Label	Fx [kN]	Fz [kN]
1	Dead load	0.000	227.550
	Support reactions	0.000	227.550
2	Traffic from above	0.000	30.000
	Support reactions	-0.000	30.000
3	Traffic span 1	0.000	37.500
	Support reactions	0.000	37.500
4	Traffic span 2	0.000	30.000
	Support reactions	-0.000	30.000
5	Wind from left	42.000	0.000
	Support reactions	42.000	-0.000
6	Wind from right	-32.000	0.000
	Support reactions	-32.000	0.000
7	Brunt load	10.000	0.000
	Support reactions	10.000	-0.000

Tabular output of the load sums

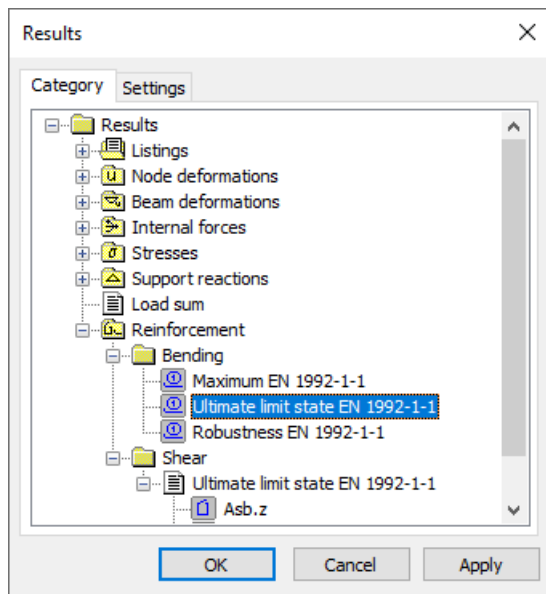
## Result Bar



The result bar includes the following buttons for representing results:

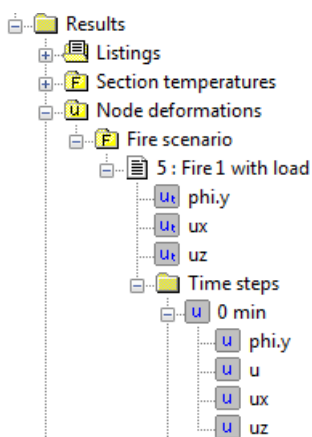
-  Start the dialog for result representation.
-  Turn off result representation.
-  Result representation of the previous load case or load case combination.
-  Result representation of the next load case or load case combination.
-  Ordinate selection for manual labeling.
-  Enable result representation for the selected beams.
-  Disable result representation for the selected beams.
-  Insert result marker for fire scenario stresses. During the analysis the section stress distribution of every time step is stored for the marked locations. The results can be displayed by double-clicking the corresponding marker.
-  Output the ordinate value at the marked point in the status bar.

## Result Categories

The following dialog shows possible result categories.



The load case and combination results as well as all presentable result components are listed in the subsection of the respective category. After making a selection, the *Settings* tab offers views described below with additional options. The symbol in front of the item identifies the type of view (e.g. : table view, : graphical view of internal forces).



Results of a fire scenario analysis for a selected time step are shown like the results of a load case. The progression of results can be represented for a selected location in a diagram. Alternatively, the results can be animated for the whole system (*colored, deformation figure, result graph*), if a superordinate component is chosen.

## Deformation Figure

### Scale

The scale can be set automatically or manually. The node displacements are multiplied by the distortion factor and added to the node coordinates. For load cases the bending curve is displayed as well.

### Display original system in dashed manner

Switch on/off the dashed non-deformed system during deformation display.

### Label

- *Auto*: Automatically add labels to the nodes listing the length of the displacement vector [mm].
- *Off*: Disable labeling.

### Limit value for labeling

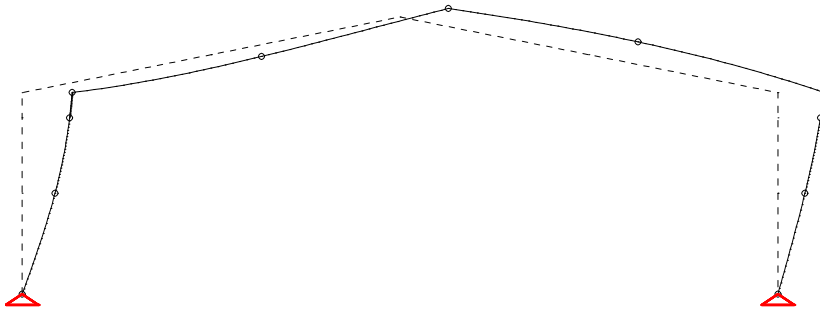
Only displacements exceeding the limit value are labeled.

**Text height [mm]**

The unit depends on the output device.

**Decimal places**

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.



*Superelevated deformation figure*

**Numeric**

This view displays the support reactions or node deformations numerically.

**Lower boundary for presentation**

Only results exceeding the limit value are output.

**Text height [mm]**

The unit depends on the output device.

**Decimal places**

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

**Result Graphs**

Beam internal forces of load cases are applied with their exact curve along the beam axis. For all other results, a linear progress of results is assumed between the result locations.

**Scale**

The scale of the ordinate display can be set automatically or manually.

**Label**

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.
- *Everywhere*: Label all ordinates.
- *Selectable*: Choose manual labeling with the help of the results bar.

**Limit value for labeling**

Only ordinates exceeding the limit value are labeled.

**Decimal places**

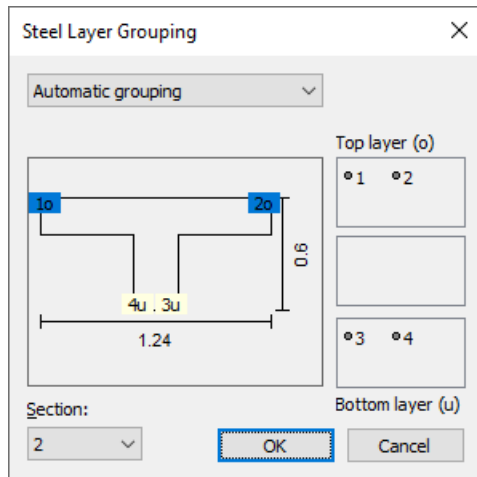
- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

**Text height [mm]**

The unit depends on the output device.

### Steel layer grouping...

Define the grouping of the steel layers with this function.



During automatic grouping, all steel layers with a z component smaller than the z component of the centroid are displayed on the upper side of the elements, all other layers on the lower side.

For manual grouping, steel layers can be moved between the list boxes using the mouse. The steel layers of the middle list box are not displayed.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

## Colored

For the representation type *Colored* the beams are colored according to their results.

### Number of colors

Set the number of colors for automatic color grading.



### Color shift

Shift of the automatic color grading.

### Grading

Enable/disable manual color grading. Color and number values can be changed.

Functions of the context menu ( right mouse button):

Save as Presetting	Saves the current grading as presetting for other projects
Load Presetting	Loads the saved presetting
Load Colors of View Database...	Sets the grading according to the current color representation Management of different gradings in the <i>lgraph.dat</i> file
Insert Row 	Adds another one above the selected color level
Delete 	Deletes the selected color level
Set All Values...	Fills the list with the help of <i>start value</i> , <i>end value</i> and <i>grading</i>

### Legend

Enable/disable the color legend.

### Dividing line

Enable/disable dividing lines at the color transitions.

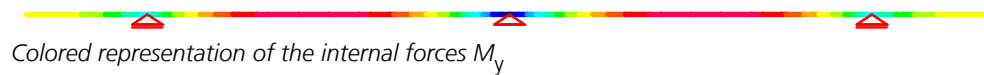
### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.



**Deformation figure**

Enable/disable the deformation figure of a load case.

**Section Stresses**

The stress curve is illustrated on a section in a dialog window. This can be accessed via the context menu of the beams when displaying result graphs for stresses or internal forces.

**Load case**

Select the calculated load cases and combinations.

**Set**

Select the internal force set for the stress determination. The absolute extreme values of the stresses from all internal force sets are displayed using *Extreme value*.

**Beam**

Number of the selected beam.

**Location**

Select the result location in the beam.

**Section**

Description of the displayed section.

**Nx, Qy, Qz, Mx, My, Mz**

Internal forces at the selected result location, from which the displayed stress curve results.

**Stress**

The stresses are generally calculated while taking the linear elastic material behavior into consideration. For results from a nonlinear system analysis, the stresses of the plastic state are displayed on polygon sections and steel sections made of construction steel (*S235 to S450 or S235-EN to S450-EN*) or *Stahl*.

- *Sigma.x*: Longitudinal stresses from bending moment and longitudinal force.
- *Sigma.v*: Comparison stresses according to the Huber-von Mises yield criterion.
- *Sigma.1,2*: Main stresses from bending moment, longitudinal force and lateral force.
- *Tau.r*: Shear stresses from lateral force parallel to the edge.
- *Tau.xy*: Shear stresses from lateral force in y direction.
- *Tau.xz*: Shear stresses from lateral force in z direction.

**Label, Text height**

Labeling of extreme values. With the right mouse button the labels for chosen ordinate values can be optionally activated or deactivated.

**Print**

Print the section stresses. Optionally, the size of the printout (*Print size: height, width*) can be defined.

**->Print list**

Insert the current view in the printing list.

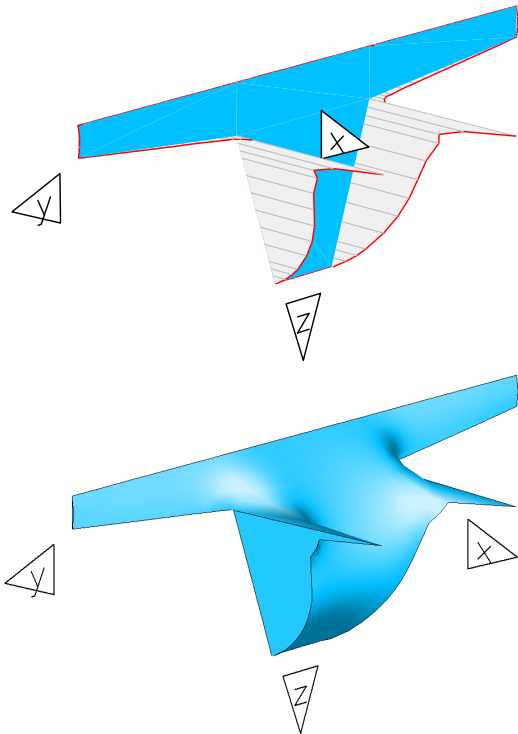
**Representation**

The stresses can be represented either as lines along the edge or as a 3D face.

The view can be rotated with the help of the mouse. The area displayed can be moved by holding down the Shift key. Use the Ctrl key to change the size of the zoom section. If you place the mouse pointer over an ordinate and wait a moment, the stress present at that location is displayed.

**Section stress Tau.xz (Elastic)**

Beam 1, result point 1, section Polygon - C20/25  
 Load case 1  
 $N_x / Q_y / Q_z = 0.00 / -0.00 / 5865.00$  kN;  $M_x / M_y / M_z = -0.00 / -58650.00 / 0.00$  kNm  
 Range of values of the stresses: -0.00157631 bis 4.18786 MN/m<sup>2</sup>.



Shear stress  $\tau_{xz}$  represented as lines along the edge and as 3D face

**Result List**

The result list for selected elements is output in the data editor. It can be accessed over the context menu for the elements.

Load case	$N_x$ [kN]	$M_y$ [kNm]
1 Dead load	-1.67	-34.70
2 Span 1	0.06	-0.48
3 Span 2	-0.29	-11.46
4 Span 3	-0.61	-5.73
5 Span 4	-4.53	1.36
6 Span 5	-13.57	3.59
7 Span 6	-0.67	0.43
8 Span 7	0.67	-0.43

The output can be limited to specific load cases using the *Load Case Selection* option of the context menu.

## Combination Information

The *Combination information* option can be used to determine the load cases involved in a combination result with their respective weightings. This function is available during the display of combination results. After the selection of the desired result location with the right mouse button, the *Combination information* function appears in the context menu.

The following functions are available in the context menu of the dialog:

- Copy (clipboard)
- Create load group...
- Create superposition load case
- Associated deformations...
- Associated support forces...

EN1992.SV.1 : 1. Permanent and temporary situation, EN 1992-1-1 Internal forces Beam 2 - Loc. 5 of 11 - My min, max				
Load case	Factor	Nx	My	Qz
1 : Dead load	1.00	-2.01	26.40	1.76
4 : Traffic span 2	1.05	-0.42	-2.74	-1.09
6 : Wind from right	1.50	-15.45	-15.59	6.99
My min		-17.89	8.07	7.66
1 : Dead load	1.35	-2.72	35.64	2.37
3 : Traffic span 1	1.50	-1.02	24.17	3.09
5 : Wind from left	0.90	-6.47	3.85	-0.88
My max		-10.21	63.66	4.59

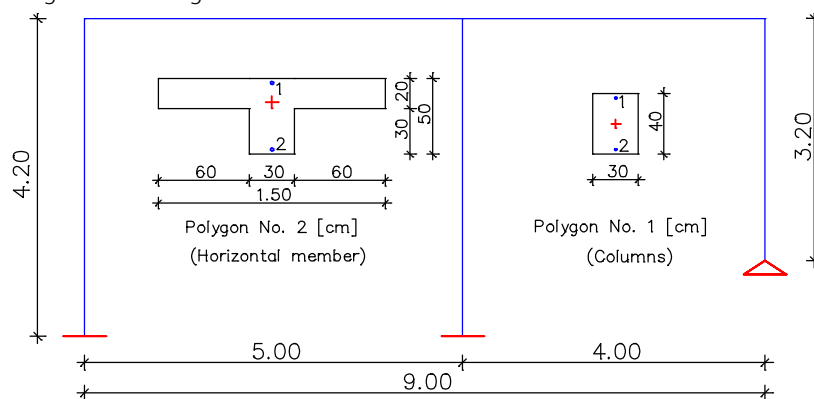
### Last Combination

Shows result values corresponding to the last *Combination information*.

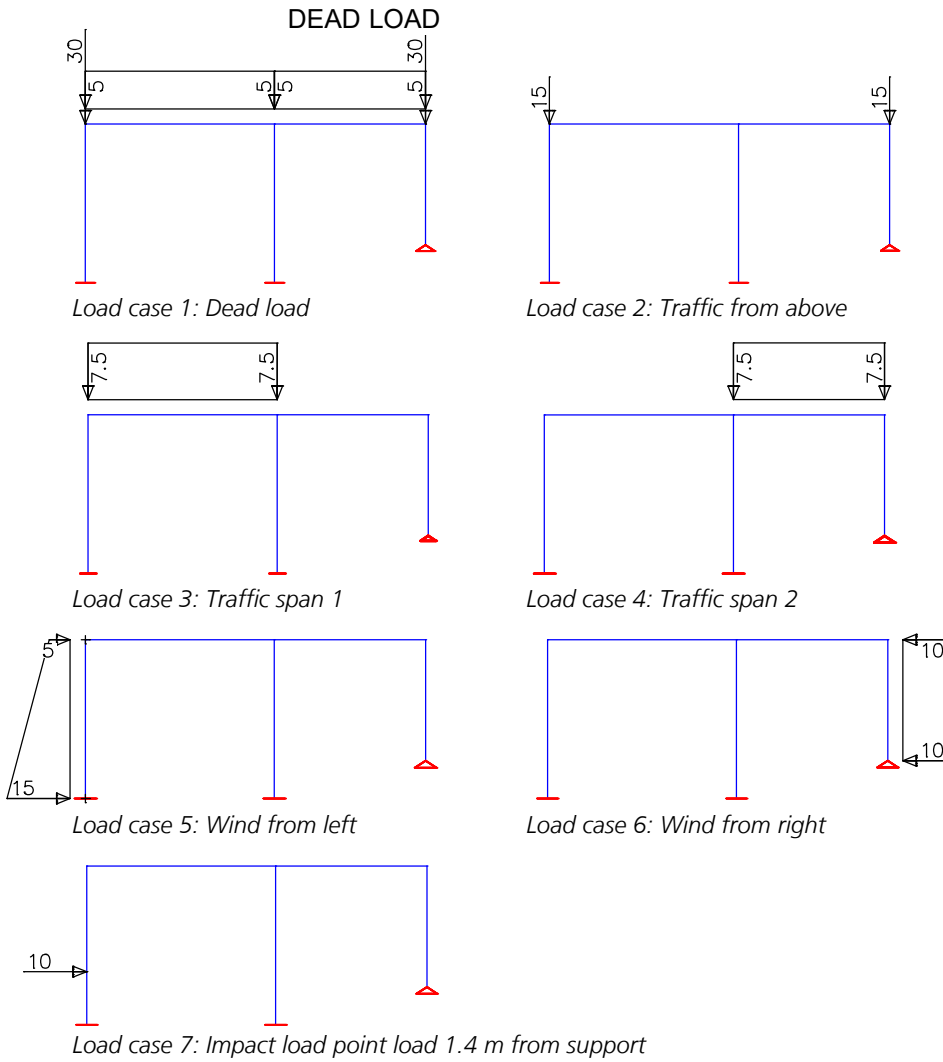
## Examples

### Reinforced Concrete Frame

The reinforced concrete frame illustrated (C20/25 - EN 1992-1-1, BSt 500/550, 3 cm overlap) is calculated and subsequently designed according to EN 1992-1-1.



**Load cases**



**EN 1992-1-1 actions**

**Standard design group**

**G - Dead load**

$\gamma_{sup} / \gamma_{inf} = 1.35 / 1$

Load cases

1 Dead load

**QN - Imposed load, traffic load**

$\gamma_{sup} / \gamma_{inf} = 1.5 / 0$

Combination coefficients for: Superstructures  
Working load - category A: Residential buildings  
 $\psi_{0.0} / \psi_{1.1} / \psi_{2.2} = 0.7 / 0.5 / 0.3$

Load cases 1. Variant, inclusive

2 Traffic from above  
3 Traffic span 1  
4 Traffic span 2

**A - Accidental action**

Load cases 1. Variant, exclusive

7 Impact load

**1. Permanent and temporary situation**

Final state

G Dead load  
QN Imposed load, traffic load  
QW Wind load

**1. Accidental situation**

Final state

G Dead load  
QN Imposed load, traffic load  
QW Wind load  
A Accidental action

**QW - Wind load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures

Wind loads

Psi.0 / Psi.1 / Psi.2 = 0.6 / 0.2 / 0

**1. Rare (characteristic) situation**

Final state

G Dead load  
 QN Imposed load, traffic load  
 QW Wind load

Load cases 1. Variant, exclusive

- 5 Wind from left
- 6 Wind from right

**Design according to EN 1992-1-1:2014**

**Design overview**

Se.	Expos. class	Prestress of component	Reinforc. M R B Q T	Fatigue B Q T P C V	Cr. wi.	De- co.	Stress C B P
1	XC4	Not prestressed	. . x x .	. . . . .	.	.	. . . .
2	XC4	Not prestressed	x . x x .	. . . . .	.	.	. . . .

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state, fatigue and stress check.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.
- (T) Torsional reinforcement at ultimate limit and fatigue state.
- (P) Prestressing steel at fatigue and stress check.
- (C) Concrete comp. stress, concrete at fatigue check under long. compression.
- (V) Concrete at fatigue check under lateral force.

**Settings for flexural and shear reinforcement**

- M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.
- fyk Quality of stirrups.
- Theta Angle of concrete truss.
- Slabs Beams are designed like slabs.
- Asl Given reinforcement according to picture 6.3, increase to maximum.
- rhov Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).
- as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).
- Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

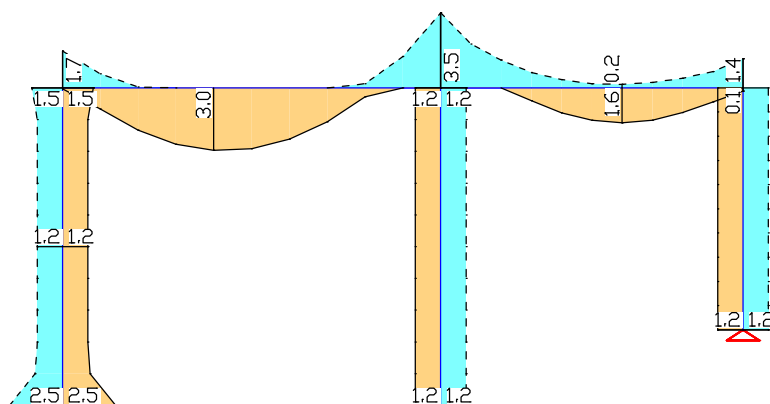
Se.	Concr.	Density [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] given max	Factor rhov as	Red. pre-str.
1	C20/25-EN	.	CM	500	1.00	.	0.00	1.00	.
2	C20/25-EN	.	ST	500	1.00	.	0.00	1.00	.

**Shear sections**

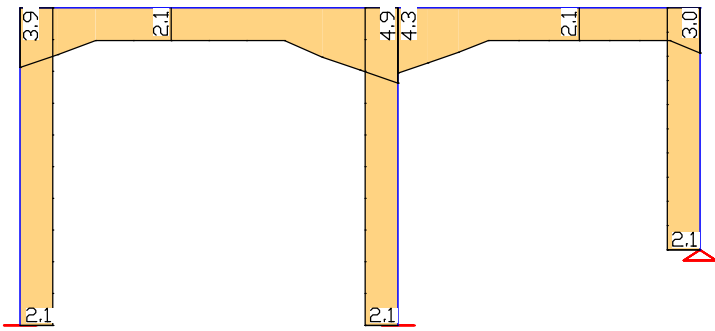
kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.

Se.	Width [m] bw	Eff. width bw.nom	bn [m]	kb	Height [m] h	h.nom	Eff.height d [m]	kd	Torsion section z1	z2	tef	B.
1	0.300	.	.	.	.	.	0.370	0.90	.	.	.	.
2	0.300	.	.	.	.	.	0.470	0.90	.	.	.	.

**Results**



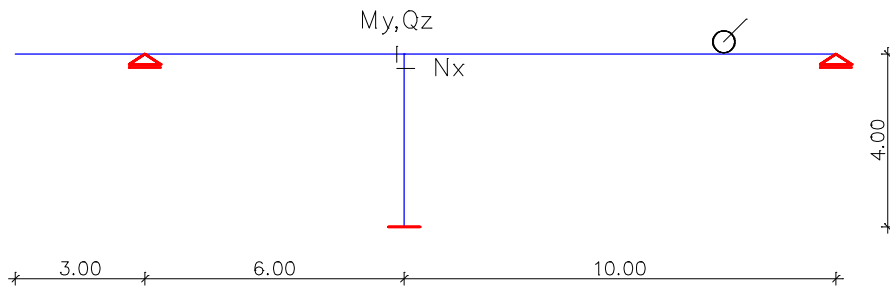
Top and bottom bending reinforcement at the ultimate limit state [cm²]



Stirrup reinforcement  $A_{sb}$  [cm<sup>2</sup>/m]

## Influence Lines

The following example illustrates the calculation of influence lines (Hirschfeld, p. 355).



Static system with the field points for  $N_x$ ,  $Q_z$ ,  $M_y$

### Material properties

No.	Mat.-type	EModule [MN/m <sup>2</sup> ]	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]	kbx [MN/m <sup>3</sup> ]	kbz [MN/m <sup>3</sup> ]	bx [m]	bz [m]
1	Frei	1	1.000e-05	25.000	0.0	0.0	1.00	1.00

### Section properties

No. 1 ST  
 $A = 1.000e+05$  [m<sup>2</sup>],  $I_y = 1.000e+00$  [m<sup>4</sup>]

### Node description

No.	x [m]	z [m]
1	0.000	0.000
2	3.000	0.000
3	9.000	0.000
4	19.000	0.000
5	9.000	4.000

### Beam elements

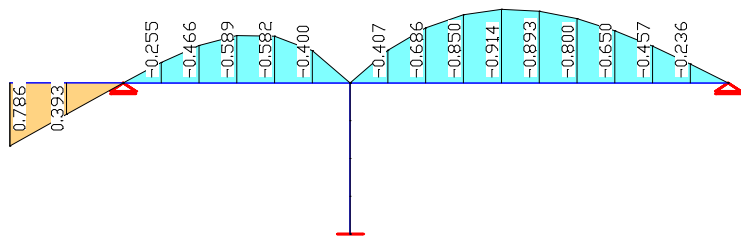
No.	Kn 1	Kn 2	CrS Mat	Joints NQM NQM	Length [m]	Layer
1	1	2	1	0 0	3.000	1
2	2	3	1	0 0	6.000	1
3	3	4	1	0 0	10.000	1
4	3	5	1	0 0	4.000	1

### Supports

Node No.	x - Direction			z - Direction			Rotation	
	F= fixed	Spring [MN/m]	Angle [grad]	F= fixed	Spring [MN/m]	Angle [grad]	F= fixed	Spring [MNm]
2	-		0.00	F		0.00	-	
4	-		0.00	F		0.00	-	
5	F		0.00	F		0.00	F	

**Load data Load case 1**

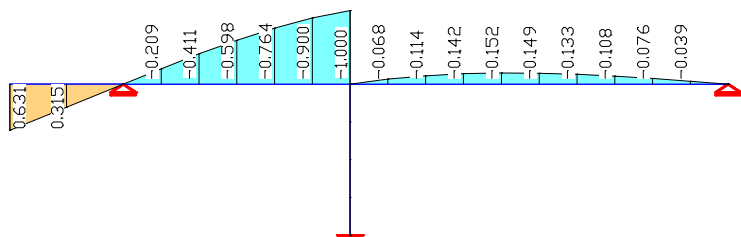
SEFL: Influence line for stress resultant  $M_y$   
 Beam 2 distance from beginning 6.00 [m]



*Influence line  $M_y$  for the load direction z global*

**Load data Load case 2**

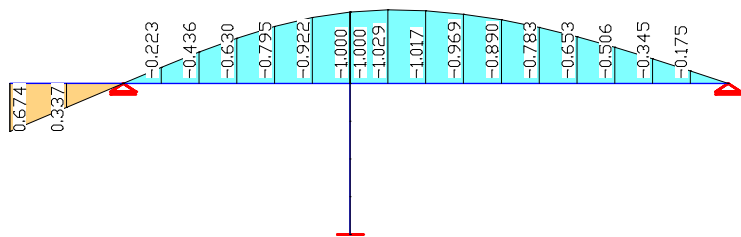
SEFL: Influence line for stress resultant  $Q_z$   
 Beam 2 distance from beginning 6.00 [m]



*Influence line  $Q_z$  for the load direction z global*

**Load data Load case 3**

SEFL: Influence line for stress resultant  $N_x$   
 Beam 4 distance from beginning 0.00 [m]

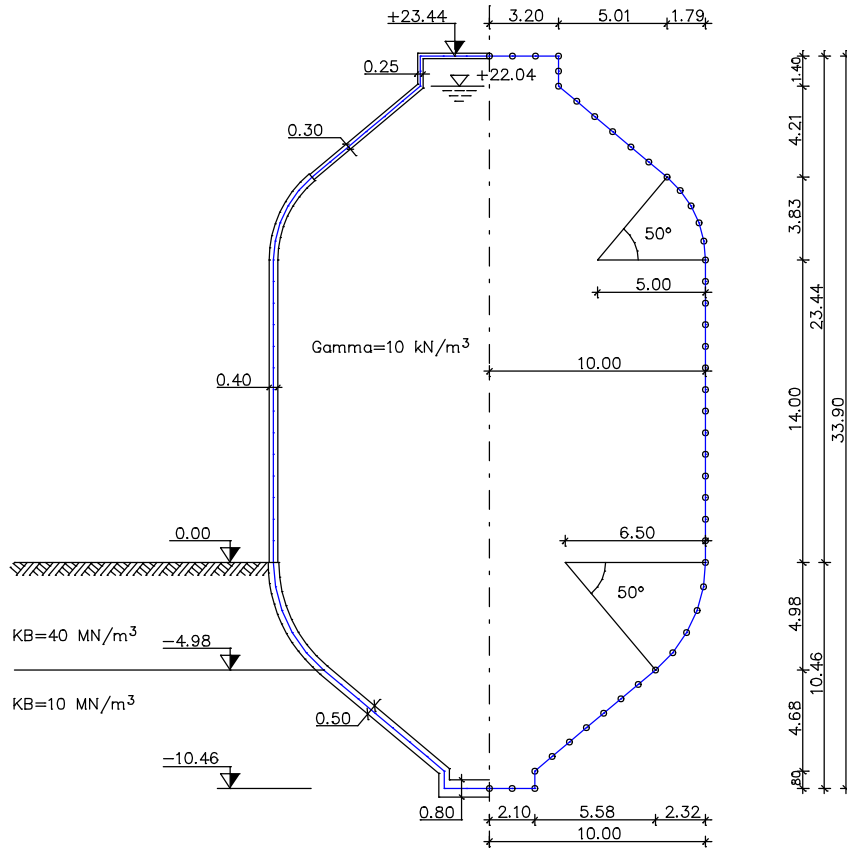


*Influence line  $N_x$  for the load direction z global*

# Axisymmetric Shell

## Basics

The *Axisymmetric Shell* program system was designed for the analysis of rotationally symmetric shell structures under rotationally symmetric load. The following example image shows the shell model of a septic tank.



Static system of a septic tank with filling

## Analysis Method

The analysis program works according to the method of finite elements and takes advantage of rotational symmetry. The structure is subdivided into elements that are connected with one another in the system nodes. Unknowns that arise for each node are the global deformations  $u_r$ ,  $u_z$  and  $\varphi$ , which can be determined by formulating the three equilibrium conditions at the node. This is done by constructing element stiffness matrices in the respective local system, transformation to the global coordinates and mapping into the global stiffness matrix of the system. After transformation of the loads to the global coordinates and introduction of the support conditions of the structure, a linear system of equations ( $K \cdot u = p$ ) is derived that can be used to determine the unknown node deformations with

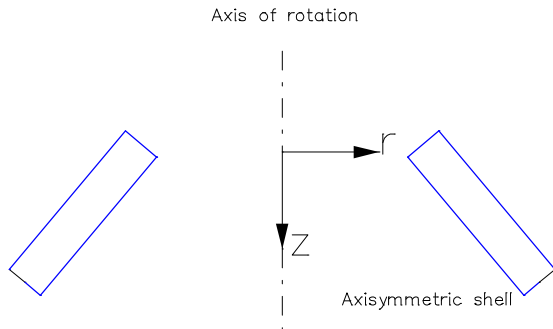
- $K$  Global stiffness matrix,
- $u$  Deformation vector of the global node deformations,
- $p$  Global load vector.



# Coordinate Systems

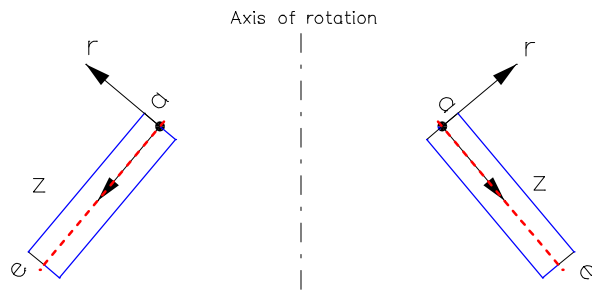
## Global coordinate system

The global coordinate system is a right-handed system located on the rotation axis.



Global coordinate system with rotation axis

## Element System

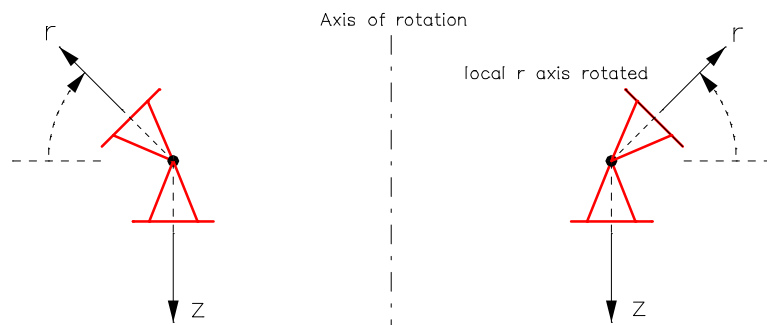


Local coordinate system of an element

The coordinate system is a right-handed system. The local element coordinates have their origin in the element's start node (a). The local z axis runs from (a) to (e), the element's end node. The r axis points toward the upper side of the elements.

## Support system

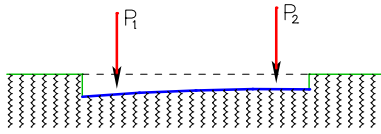
Normally, the support system corresponds to the global coordinate system. The support system can be changed by rotating the local axes.



Support system with rotated r axis

## Element Bedding

The element bedding is modeled with the help of the bedding modulus method. This is based on the assumption that the subsidence ( $s$ ) is proportional to the soil stress, meaning  $\sigma_{\theta}/s = k_b = \text{constant}$  ( $k_b$  = bedding modulus). That means a load only induces subsidences in their direction of effect.



Bedding modulus method

### Loss of tensile bedding

The structure can be calculated given the loss of tensile bedding. The  $r$  axis of the element system points toward the ground. If a loss of tensile stresses in the  $r$  direction occurs, the skin friction ( $z$  direction) is not considered.

## Singular System of Equations

Global stiffness matrices are solved in the equation solver by triangulation. If the system of equations is not explicitly solvable, meaning it is singular, the calculation terminates with an error message.

```
Load case 1
Assembling total stiffness matrix (100 %)
Installing supports
Triangulating system of equations
Line 1270
** Errors **

ROS-F001: Set of equations is singular, line 1273, node 138 (Ux).
Abort of program, 0 Warning(s), 1 Error(s).
```

The error indicates the nodal degree of freedom at which the singularity was determined. The following causes are possible:

- The system is relocatable.
- After a loss of the tensile bedding, the degrees of freedom are no longer supported.

In most cases the relocatability of the system is due to erroneous input data:

- Joint chains have been defined that make the system kinematic. When troubleshooting it can be helpful to replace all joints with stiff connections and then reactivate them one by one until the error occurs.
- Calculations excluding tensile bedding are carried out under unrealistic load assumptions, for instance without dead load, and lead to lift-off of the structure.

## Analysis Results

The sections below describe the following results:

- *Deformations*
- *Internal forces*
- *Stresses*
- *Soil pressures*
- *Support reactions*

### Deformations

The node deformations refer to the global coordinate system and are designated as follows:

$u_r, u_z$  Node displacements [m] in the direction of the global axes.

$\varphi$  Node rotation [rad].

Additionally, bending curves are calculated.

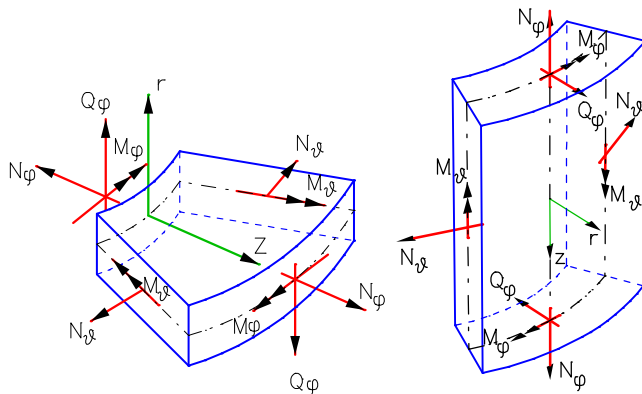
### Internal forces

The following internal forces are calculated in the element nodes:

$N_{\varphi}, Q_{\varphi}$  Meridian force and lateral force [kN/m].

$M_{\varphi}$  Meridian moment [kNm/m].

$N_\varphi$  Ring force [kN/m].  
 $M_\varphi$  Ring moment [kNm/m].



Element system with positive internal forces

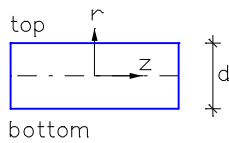
**Stresses**

The stresses [MN/m<sup>2</sup>] are determined at the upper and lower section edges.

$$\sigma_{\varphi, \text{top, bottom}} = \frac{N_\varphi}{d} \mp \frac{M_\varphi}{d^2/6} \quad \text{Normal stresses in the meridian direction.}$$

$$\sigma_{\theta, \text{top, bottom}} = \frac{N_\theta}{d} \mp \frac{M_\theta}{d^2/6} \quad \text{Normal stresses in the ring direction.}$$

The section edges are defined as follows:



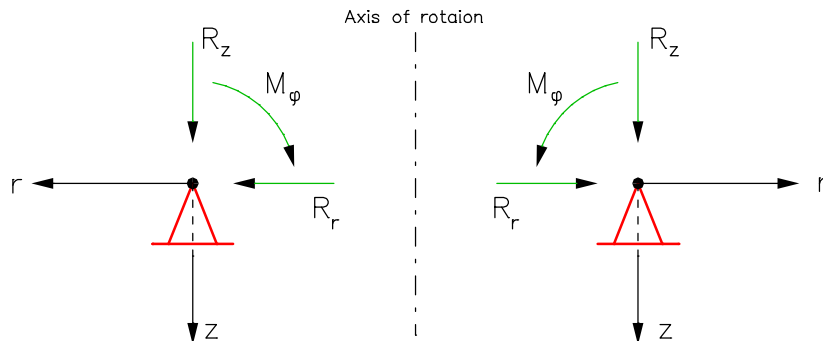
**Soil pressures**

The soil pressures  $\sigma_r$  and  $\sigma_z$  [kN/m<sup>2</sup>] are defined in the element system. Tensile stresses in the soil have a positive sign.

**Support reactions**

The support reactions are calculated in the coordinate system of the supports. They act on the corresponding support. The following support reactions are determined:

$R_r, R_z$  Support forces [kN/m].  
 $M_\varphi$  Support moment [kNm/m].



Support system with positive support reactions

## Program Capacities















Nodes	99999
Elements	99999
Load cases	9999
Load case combinations	9999

### Restrictions for the study version

Nodes	10
Elements	10
Sections	1

## Structure Description

Because of the rotational symmetry the structure model can be described with linear represented elements in the vertical section. The entire editing is done in the x, z plane, in which x corresponds to the radial direction r of the rotating body. Entries are permitted only to the right of the axis of rotation displayed at  $x = 0$ , i.e. with positive values. The elements entered this define the right half of the section curve, the left half is automatically added. The following *Structure* menu functions can be used for system input and the graphical representation of the results:

	<i>Element Input</i>
	<i>Element Properties...</i>
	<i>Element Info</i>
	<i>View...</i>
	<i>Mesh Check</i>
	<i>Element System</i>
	<i>Number Sort</i>
	<i>Supports</i>
	<i>Load</i>
	<i>Moving Nodes</i>
	<i>Condense</i>
	<i>Link Elements</i>
	<i>Print Input Data...</i>
	<i>Results...</i>

## Element Input

This menu item is used to input elements.

### Input

<i>Starting point</i>	Input an element from start to end point.
<i>Properties</i>	The element properties dialog is called up with this option.
<i>Generate</i>	The entered line is subdivided into $n$ elements of the same length. Different start and end sections can be specified. The program then interpolates the selected number of sections. This enables the modeling of a haunch.

## Element Properties

This dialog is used to edit the properties of selected elements or preset the properties for new elements yet to be created. The dialog consists of the following pages:

- *General or Joints*
- *Section*

### General

This dialog shows the general element properties.

#### Length

Display the element length [m].

#### Nodes

Display the node coordinates.

#### Joints

The joint properties can be specified separately for each degree of freedom. When selected, the corresponding degree of freedom is defined as jointed. The arrow indicates the selected end of the element.

### Section

The section describes the following properties of an element:

- Thickness
- Material
- Bedding
- Specifications for reinforced concrete design

Each section is uniquely identified by a number between 1 and 9999. In general a section is not assigned to a single element but rather a group of elements.

#### Number

Select the section. You can assign the number freely for a newly generated section. Sections can subsequently be renumbered using the context menu for the subitem *Structure Description / Section properties* of the data base.

## Height

Section thickness [m].

## Material Type

- *<Database>*: User-defined material coefficients from the system file *lgraph.dat*.
- *B25* to *B55*: Concrete qualities according to DIN 1045:1988, Table 1.
- *C12/15* to *C100/115*: Concrete qualities according to DIN 1045-1 or ENV 1992-1-1:1991 (*C55/67* or higher only for design according to DIN 1045-1).
- *LC12/13* to *LC60/66*: Concrete qualities according to DIN 1045-1.
- *C12/15-EN* to *C90/105-EN*: Concrete qualities according to EN 1992-1-1.
- *C12/15-EN-D* to *C100/115-EN-D*, *LC12/13-EN-D* to *LC80/88-EN-D*: Concrete qualities according to DIN EN 1992-1-1.
- *CX*: Concrete according to DIN 1045-1 with user-defined properties.
- *CX-EN*: Concrete according to EN 1992-1-1 with user-defined properties.
- *CX-EN-D*: Concrete according to DIN EN 1992-1-1 with user-defined properties.
- *ÖC16/20* to *ÖC50/60*: Concrete qualities according to OENORM B 4710-1.
- *ÖNB40*: Concrete quality according to OENORM B 4200-10.
- *ÖNBeton*: Concrete according to OENORM with user-defined properties.
- *SC12/15* to *SC50/60*: Concrete qualities according to SIA262 or SN EN206-1.
- *SLC12/13* to *SLC50/55*: Concrete qualities according to SIA262 or SN EN206-1.
- *SCX*: Concrete according to SIA262 or SN EN206-1 with user-defined properties.
- *S235*, *S275*, *S355*, *S450*: Construction steel according to DIN 18800 T1, Table 1.
- *S235-EN* to *S500-EN*: Construction steel according to EN 1993-1-1, Tab. 3.1 or EN 10025-2.
- *NH1*, *NH2*, *NH3*: Softwood grade I, II and III according to DIN 1052:1988.
- *BSH1*, *BSH2*: Glued laminated lumber grade I and II according to DIN 1052:1988.
- *C14* to *C50*, *D18* to *D80*: Solid timber according to EN 338.
- *GL20h* to *GL32h*, *GL20c* to *GL32c*: Glued laminated timber according to EN 14080.
- *Frei*: User-defined material.
- *Beton*: User-defined concrete.
- *Stahl*: User-defined steel.

## New

Create a new section.

## Copy

Copy the current section.

## Delete

Delete the current section.

## Label

The section can be labeled with any text.

Further dialogs include:

- *Material*
- *Bedding*
- *Reinforcing steel*
- *DIN 1045-1*
- *EN 1992-1-1*
- *BS EN 1992-1-1*
- *DIN EN 1992-1-1*
- *OENORM EN 1992-1-1*
- *SS EN 1992-1-1*
- *OENORM B 4700*
- *SIA 262*

## Material

### Properties for all Materials

#### E-Modulus

Modulus of elasticity in the meridian direction  $\varphi$  [MN/m<sup>2</sup>].

#### E-Modulus ring direction

Modulus of elasticity in the ring direction  $\upsilon$  [MN/m<sup>2</sup>].

Note: The modulus of elasticity is part of the stress-strain function according to EN 1992-1-1, Equation (3.14), and SIA 262, Equation (28). The function has a pole that can move into the decisive strain range, if the given modulus of elasticity is reduced. As a consequence, results of the reinforced concrete checks can be affected.

#### Poisson's ratio

Poisson's ratio for both directions.

#### Gamma

Specific weight of the material [kN/m<sup>3</sup>]. This is used for the *Dead load* load type.

#### Alpha.t

Coefficient of thermal expansion [1/K].

### CX (DIN 1045-1), CX-EN-D (DIN EN 1992-1-1), CX-EN (EN 1992-1-1)

For the user-defined concrete material types *CX*, *CX-EN-D* and *CX-EN* additional concrete parameters according to DIN 1045-1, Table 9 or EN 1992-1-1, Table 3.1 are defined in addition to the  $\alpha$  values.

#### fck

Characteristic value of compressive cylinder strength of concrete at 28 days [MN/m<sup>2</sup>].

#### alpha or alpha.cc

Coefficient which takes into account the long term effects on the compressive strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

#### fctm

Mean value of axial tensile strength of concrete [MN/m<sup>2</sup>].

#### alpha.ct

Coefficient which takes into account the long term effects on the tensile strength and the unfavourable effects resulting from the way the load is applied according to DIN 1045-1, Section 9.1.6 (2) or EN 1992-1-1, Section 3.1.6 (1) [-].

## Lightweight Concrete

### Apparent density

Apparent density of lightweight concrete [kg/m<sup>3</sup>].

## Concrete SIA 262

### ke for E-Mod.

Factor for calculation of the modulus of elasticity according to SIA 262, Section 3.1.2.3.3.

For the material type *SCX* the concrete properties are defined additionally according to SIA 262, Table 3.

#### fck

Characteristic cylinder compressive strength [MN/m<sup>2</sup>].

#### fck,cube

Characteristic cube compressive strength [MN/m<sup>2</sup>].

**fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

**ÖNBeton (OENorm B 4700)**

For this material type the concrete properties are defined additionally according to OENORM B 4700, Table 4.

**fcwk**

Characteristic cube compressive strength, nominal strength of the strength class [MN/m<sup>2</sup>].

**fctm**

Average concrete tensile strength [MN/m<sup>2</sup>].

**fctk**

Characteristic concrete tensile strength [MN/m<sup>2</sup>].

**taud**

Calculation value of shear stress [MN/m<sup>2</sup>].

**Stahl (General)**

**fyk**

Characteristic yield strength [MN/mm<sup>2</sup>].

**Timber EN 338, EN 14080**

**Apparent density**

Apparent density of timber [kg/m<sup>3</sup>].

**Service Class**

Service class for timber checks.

**Bedding**

**Bedding modulus r,z**

Bedding moduli for the element directions [MN/m<sup>3</sup>].



## Reinforcing Steel

### Selection of layers

Select steel layers by clicking them in the graphic.

### New layer

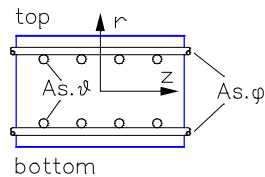
Add a new steel layer.

### Delete layer

Delete selected steel layer.

### Edge distance in meridian or ring direction

Axis distance of the steel layer from the section edge in the meridian direction  $\varphi$  or ring direction  $\upsilon$  [m]



### Base reinforcement

Minimum section of the reinforcement for the bending design. This can also be used to establish a relationship between the steel sections to be determined [cm<sup>2</sup>/m].

### Fix base reinforcement

The base reinforcement is not changed during the design.

### Quality

- 420S: Reinforcing rod with  $f_{yk} = 420$  MN/m<sup>2</sup>.
- 500S: Reinforcing rod with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- 500M: Reinforcing meshes with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- *General information:* Freely definable steel quality [MN/m<sup>2</sup>].

### Yield strength

Yield strength  $f_{yk}$  [MN/m<sup>2</sup>].

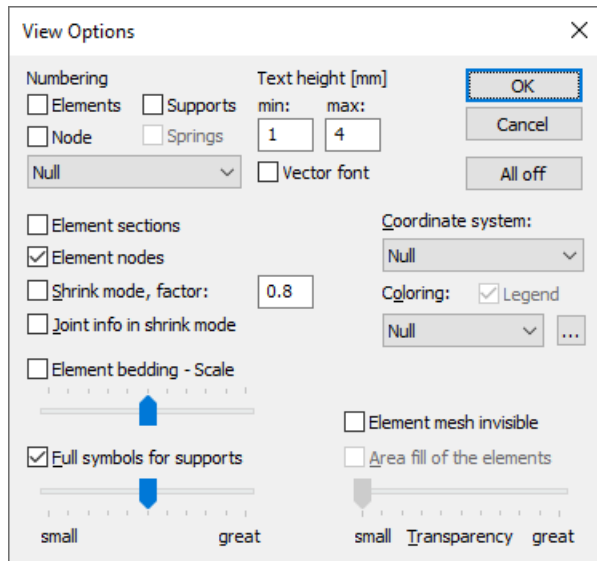
## Element Info

This function shows the following properties of an element graphically at the element and in the status bar:

- Element number
- Node numbers
- Element system
- Section
- Material
- Joints

## View

This menu item is used to display and check the structure properties.



### Numbering

You can label elements, nodes and supports with their numbers. Section numbers can also be added to elements. The display of the numbers can be limited to values  $> 1$ . Alternatively, the section name can be added to elements.

### Text height

Set minimum and maximum text heights [mm].

### Element section

The element section is drawn at the middle of the element.

### Element nodes

The element nodes are marked by a symbol.

### Shrink mode

Enable or disable shrunken element representation. Most of the input functions are disabled in this mode.

### Shrink factor

Define the shrink factor for the shrink mode.

### Joint info in shrink mode

In shrink mode the existing element end joints are labeled.

### Element bedding

The bedding moduli are plotted along the elements with the specified scale.


### Full symbols for supports

The properties of the supports are shown in detail. The scroll bar changes the size of the icons in use.

### Coordinate system

Activate the coordinate system for elements or supports.

### Coloring

The objects are drawn in different colors, depending on the selected property (e.g. *material*, *section*). If necessary, the color assignment can be modified with the  button. The possible objects that are distinguished by color include elements, design objects and model objects. The properties *asx,y 1./2*. *layer* refer to the base reinforcement of the cross-section. Color 254 is used for *Undefined*.

### Legend

Enable/disable the color legend.

**All off**

Turn off all display options.

**Mesh Check**

This menu item is used to check elements and nodes.

**Node distance**

The function checks if there are further nodes in the neighborhood of a node within the given tolerance. After confirmation, the so-called duplicate nodes are marked by red arrows, and you are asked whether you want to merge them. In this case their coordinates are averaged.

**Element distance**

Check if any elements lie on one another, for instance, due to redundant element entry. The elements that are found are marked by red arrows.

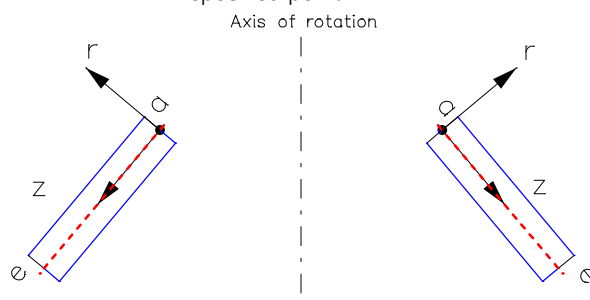
**Element System**

This menu item is used to change the coordinate system of selected elements.

**Input**

$+r, -r, +z, -z$

By exchanging the start and end nodes of the element, the positive or negative axis is directed at the specified point.

**Number Sort**

This menu item is used to change element and node numbers.

**Input**

<i>Sort</i>	Sort node or element numbers geometrically.
<i>Define</i>	Define new start values and increments for nodes and element numbers.
<i>Edit</i>	Edit node and element numbers.

**Supports**

This function is used to define supports. A support describes the support conditions of a system node. The individual degrees of freedom can be supported in a fixed or elastic manner. The direction of the degrees of freedom can deviate from the global coordinate system (described by an angle). The supported nodes are labeled with simple or detailed red icons (see *View*).

**Enter**

Start the input of supports through the selection of nodes. The subsequent editing is performed using the context menu functions:

- Support
- Orientation

**Info**

Display the support system with the bound degrees of freedom for a selected support.

Note: When the function is active only supports can be selected. This improves the handling of these objects.

## Moving Nodes

This function is used to change the location of selected nodes. In the case of a single element node, simply specify the new position. If multiple element nodes are to be moved, you should choose the *Group* or *Elements* option.

### Group

Select the element nodes to be moved.

### Elements

Select the elements, whose nodes are to be moved.

### Displacement vector

Movement through displacement. The displacement vector is defined by two points.

### Rotate

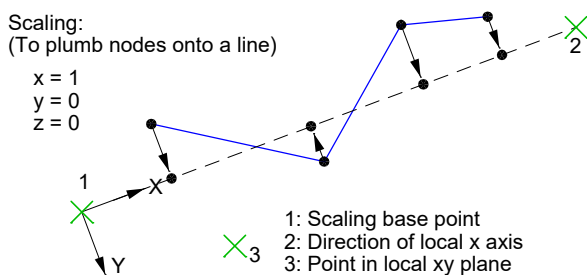
Movement through rotation.

### Mirror

Movement through mirroring.

### Scaling

Movement through scaling in the global or local reference system. You can align nodes on an axis by specifying a scaling factor of zero.



If desired, displaced nodes that coincide with non-selected nodes can be merged with these.

## Finding Nodes or Elements

If you are looking for a node with a particular number, the functions *Select Nodes* and *Move Nodes* can be of help. Enter the node number you are looking for into this function.

If you are looking for an element with a specific number, use the functions *Element Info* or *Select*. The element number can be specified in both functions. If the element is not identifiable even though it is selected, you can query further information with the *Element Properties* function. The selected objects' color and layer are displayed in the toolbar.

## Condense

This function is used to condense existing elements.

### Point

At the point specified a node is inserted on the element axis.

### Divide

Selected elements are divided. Enter the desired number of parts.

### Adapt

For the selected elements a check is performed to determine whether any nodes touch their axes. If necessary, the elements in question are divided at these nodes.

## Link Elements

Kinematic dependencies between the global degrees of freedom of two element nodes can be defined with the *Link Elements* function. This is done by expressing the nodal degrees of freedom of a dependent node through the degrees of freedom of the reference node by way of an equation of condition.

For instance,  $u_{x,dependent} = f_1 \cdot u_{x,reference} + f_2 \cdot u_{z,reference} + f_3 \cdot \varphi_{reference}$

with

$f_1, f_2$  Factors for the displacements,

$f_3$  Factor for the rotation.

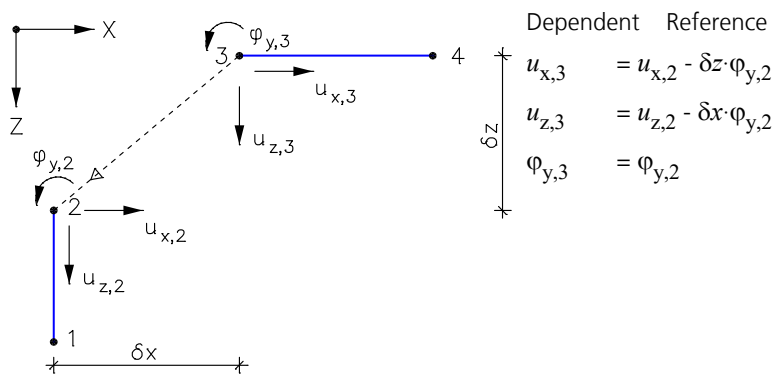
You can link any number of dependent nodes to a reference node. Nodes with supports may not be defined as dependent nodes. The link elements are displayed with dotted lines. Additionally, an arrow points to the reference node.

After selecting the dependent nodes and the reference node, you must define the link element type.

### Type

- *Stiff*: Stiff link element (abbreviation: SS).
- *Jointed*: One-sided jointed link element with a ball joint at the dependent node (abbreviation: GS).
- *ux, uz, phi.y*: Dependent degree of freedom of the user-defined equation of condition. The factors for the displacements and rotations of the equation of condition have to be defined.












### User-defined equations of condition for a stiff link element



## Load

The following chapter describes load input as well as the different load types. The specified loads are represented with icons.

A load case is made up of one or multiple loads. The following options allow you to define load cases or subsequently change them, if necessary.

-  Create a copy of the current load case.
-  Generate load cases according to the rules set out by the *Generate* function. This leads to the duplication of either all or only the selected loads.
-  Import load data.
-  Start the representation options dialog.
-  End the load representation.
-  Create a new load case.
-  Open the previous load case.
-  Open the next load case.
-  Show an existing load case.
-  Change the number and name of the current load case.
-  Start the load input.

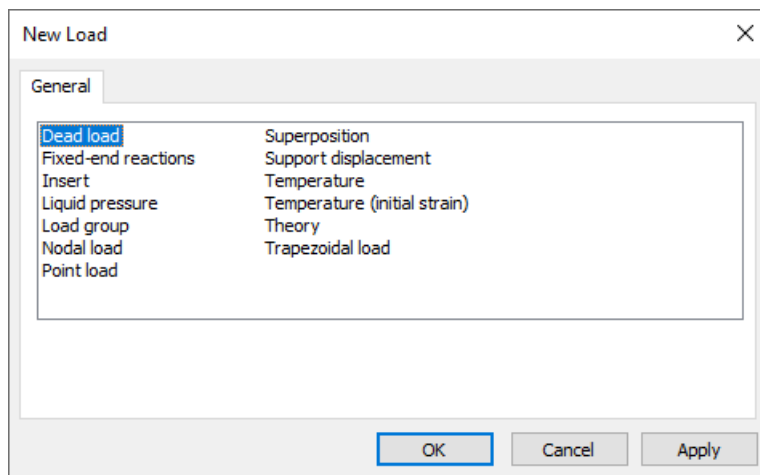
Load cases can be renumbered in groups within the database tree.

Double-clicking on an existing load object or the context function *Load* opens a dialog in which changes to the selected load objects can be made.

With the context function *Scale load* the load values of multiple selected loads can be scaled with a factor.

## Input

The dialog box is used to input load data within a load case.



The load types are explained on the following pages. For easier identification they are labeled with load abbreviations.

## Dead Load

The entire structure is stressed by its dead load in the global z direction (load abbreviation: *EG*).

## Inserting of Load Case Data

This load type (load abbreviation: *EINF*) allows you to insert load data from other load cases in the current load case in a weighted manner. The *Superposition* load type may not be present in the inserted load cases.

### Input

<i>First load case</i>	First load case that is inserted.
<i>Last load case</i>	Last load case that is inserted.
<i>Weighting factor</i>	Factor by which to multiply all load data.

## Point Load

### Input

<i>Global orientation</i>	Point load in the global system (load abbreviation: <i>EGK</i> ).
<i>Local orientation</i>	Point load in the element system (load abbreviation: <i>EL</i> ).
<i>Load point</i>	Load point on the element axis defined depending on the load direction. <i>Global orientation:</i> Global coordinates $r$ and $z$ . <i>Local orientation:</i> Distance from the element start.
<i>Pr, Pz, Mphi</i>	Load components in the directions of the selected coordinate system [kN/m] or [kNm/m].

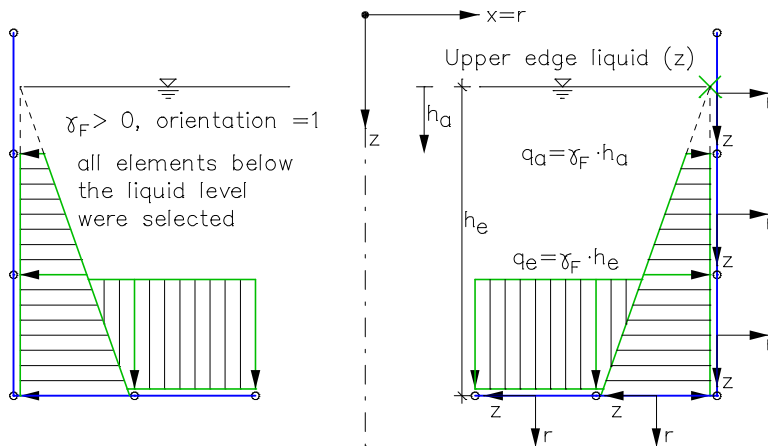
## Liquid Pressure

The liquid pressure stresses all selected elements (load abbreviation: *FD*).

### Input

Selection of the stressed elements.

<i>Upper edge liquid z</i>	Z coordinate of the liquid level [m].
<i>Specific weight</i>	Specific weight $\gamma_F$ of the liquid [kN/m <sup>3</sup> ].
<i>Orientation (-1, 1)</i>	Set orientation of the load in the $r$ direction. -1 : In direction - $r$ 1 : In direction + $r$



Elements with liquid pressure

## Nodal Load

Nodal loads act in the global direction (load abbreviation: *KNL*).

### Input

Selection of the stressed element nodes.

<i>Pr, Pz</i>	Forces [kN/m].
<i>Mphi</i>	Meridian moment [kNm/m].

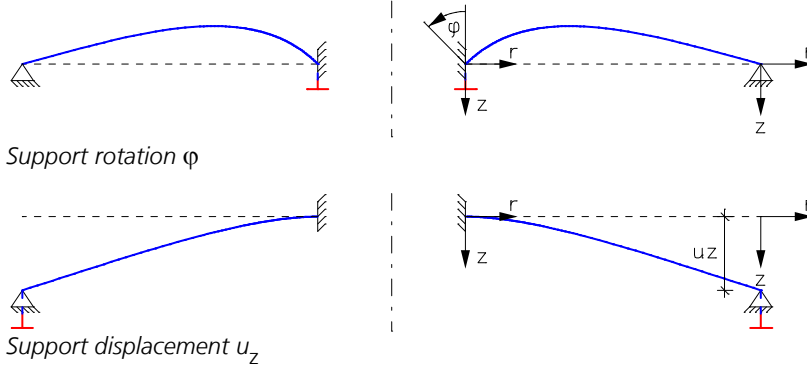
## Support Displacement

The support displacement acts on supported nodes in the direction of the bound degrees of freedom (load abbr.: *DS*).

### Input

Selection of the affected supports.

*ur, uz* Displacements [mm].  
*Phi* Rotations [rad/1000].



## Superposition of Load Cases

Through superposition (load abbreviation: *SUP*) the existing results of different load cases can be added and weighted. Other load types are not permitted in the load case in question. Because the load cases are analyzed in ascending order, the load numbers of the load cases to be superposed should be smaller than the number of the current load case.

### Input

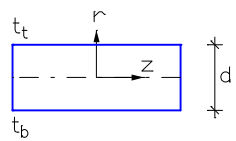
*First load case* First load case to be superposed.  
*Last load case* Last load case to be superposed.  
*Weighting factor* Factor by which to multiply all results.

## Temperature

### Input

Selection of the stressed elements (load abbreviation: *TEMP*).

*tm* Temperature as a difference to the installation temperature [K].  
*tb - tt* Temperature difference between the bottom and top side of the section [K].



Top and bottom sides of the section

## Temperature, Initial Strain

### Input

Selection of the stressed elements (load abbreviation: *TMPA*).

*alpha.t* Coefficient of thermal expansion [1/K].  
*tm* Temperature as a difference to the installation temperature [K].  
*(tb - tt)/d* Temperature difference between bottom and top side of the section with respect to the section thickness [K/m].



## Theory

This load type (load abbreviation: *TH*) is used to specify the calculation theory for the load case in question.

### Input

<i>Number of iterations</i>	This value is not used at the moment.
<i>Error</i>	Error threshold for the relative size of the remaining residuum in [%]. Only values $\leq 1\%$ are permissible.
<i>No soil pressure &gt; 0</i>	Disregard tensile stresses in the foundation area.

When using this load type, make sure to pay attention to the notes provided in the *Load group* section.

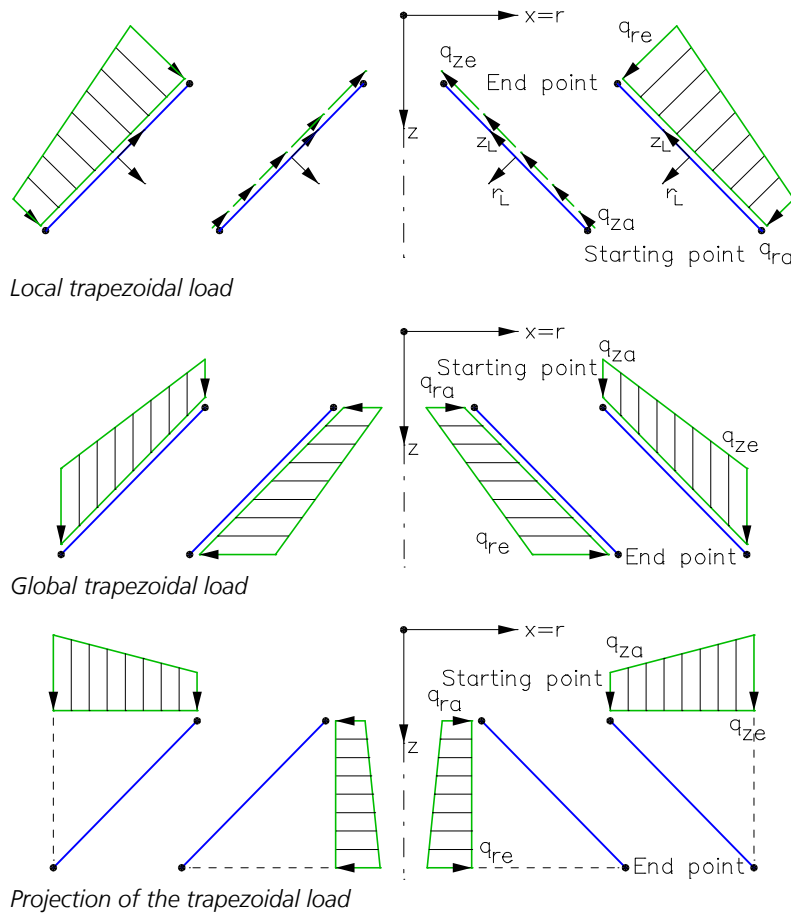
## Trapezoidal Load

### Input

Selection of the stressed elements.

<i>Starting point</i>	Load start.
<i>End point</i>	Load end.
<i>Global orientation</i>	Trapezoidal load in the global system (load abbreviation: <i>TG</i> ).
<i>Local orientation</i>	Trapezoidal load in the element system (load abbreviation: <i>TL</i> ).
<i>Projection orientation</i>	Projection of the trapezoidal load in the global system (load abbreviation: <i>TP</i> ).
<i>Starting ordinate</i>	Load ordinate at the load start in the directions of the selected coordinate system [kN/m <sup>2</sup> ].
<i>End ordinate</i>	Load ordinate at the load end in the directions of the selected coordinate system [kN/m <sup>2</sup> ].

The load is automatically distributed over multiple elements for a trapezoidal load. The interpolated load ordinates at the nodes of the selected elements are determined automatically. The trapezoidal loads always act on the entire element length.



## Load Group

This load type (load abbreviation: *GRL*) is especially suited for preparing load cases for nonlinear calculation. It allows you to compile loads from existing load cases by retrieving the necessary load factors and to activate the desired calculation theory. The *Load group* is used to combine the existing load types *Insert* and *Theory* into a single load type.

Load groups can also be modified via a dialog in the section *Loads* of the data base. The *Superposition* load type may not be present in the inserted load cases.

### Input

<i>Error threshold</i>	Error threshold for the relative size of the remaining residuum in [%]. Only values $\leq 1\%$ are permissible.
<i>No soil pressure &gt; 0</i>	Disregard tensile stresses in the foundation area.
<i>Add. global load factor</i>	Additional factor for all loads.
<i>Load cases</i>	Select load cases to add them to the selection list.
<i>Load factor</i>	The individual load cases are weighted by a factor.
<i>Apply</i>	Change the weighting factor for selected load cases.

When using this load type, the following should be observed.

### Error threshold

The nonlinear bedding problem is solved iteratively. The length of the error vector, the residuum, is used to control the iteration. It is shown for each iteration step in the output bar. The iteration is ended when the remaining residuum has reached the specified error threshold in relation to the initial residuum. The calculation results have, on average, a relative error of the same extent. If the error threshold cannot be reached, the calculation program issues a corresponding message.

### Loss of tensile bedding

Unrealistic load combinations (for instance, a missing dead load) can lead to loss of all bedding. If the system becomes relocatable due to this, the calculation is aborted because of a singular system of equations. The relocatable degree of freedom is provided.

### Load combination

In order to combine the decisive actions for calculations with loss of tensile bedding, transfer the corresponding load cases to the selection list and assign them load factors.

### Prohibition of superposition

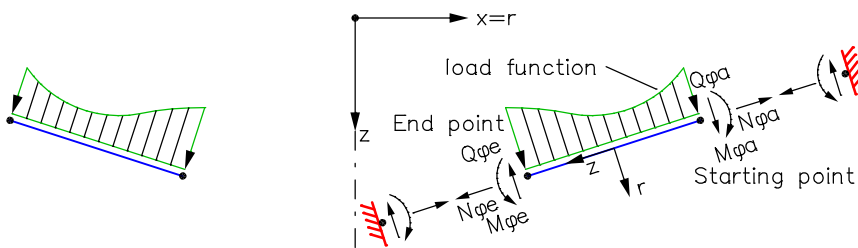
Calculations excluding the tensile bedding deliver their results after nonlinear iteration. So in these cases the superposition principle is no longer valid, meaning the results may not be additively superposed with other results. Further processing using load case combinations is only permissible if these only contain mutually exclusive actions.

## Fixed-End Reactions

### Input

Selection of the stressed elements (load abbreviation: *SO*).




<i>Nphia, Qphia</i>	Meridian force and lateral force at the start of the element [kN/m].
<i>Nphie, Qphie</i>	Meridian force and lateral force at the end of the element [kN/m].
<i>Mphia, Mphie</i>	Meridian moment at the element start or end [kNm/m].



Fixed-end reactions at the start and end of an element.

## Analysis

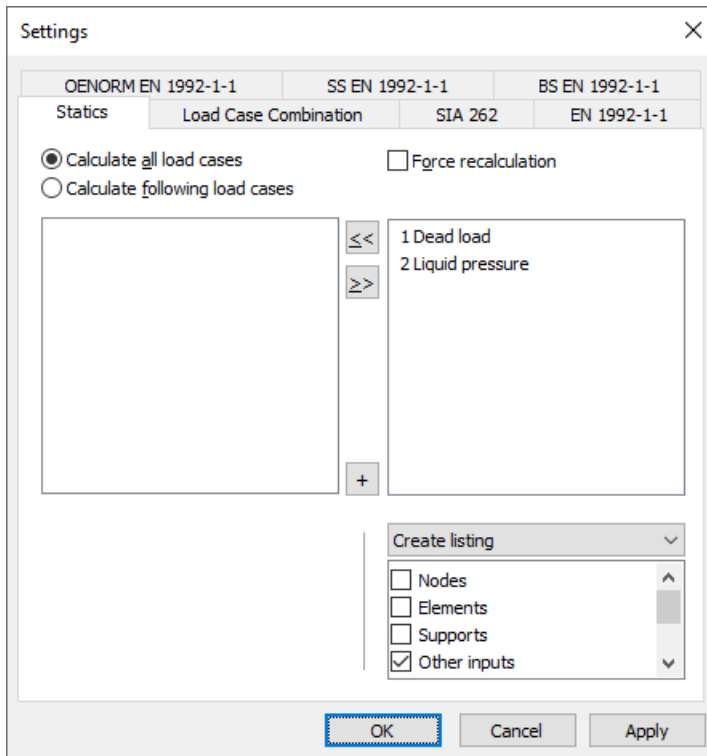
The *Analysis* menu is used to make program-specific settings and to launch the analysis programs.

-  *Settings...*
-  *Batch...*
- Construction Stages (Batch calculation of all Construction Stages)*
-  *Statics*
  - Load Case Combination*
  - BS EN 1992-1-1 Design*
  - DIN 1045-1 Design*
  - DIN EN 1992-1-1 Design*
  - EN 1992-1-1 Design*
  - OENORM B 4700 Design*
  - OENORM EN 1992-1-1 Design*
  - SIA 262 Design*
  - SS EN 1992-1-1 Design*
  - Selected Elements*
  - Stop Analysis*

During the analysis the program's progress is shown in the output bar along with any possible warnings and error messages.

## Settings Statics

This page shows the settings for the shell calculation.



### Calculate all load cases

All load cases are calculated (default).

### Calculate following load cases

The load cases from the left list box are calculated.

### Force recalculation

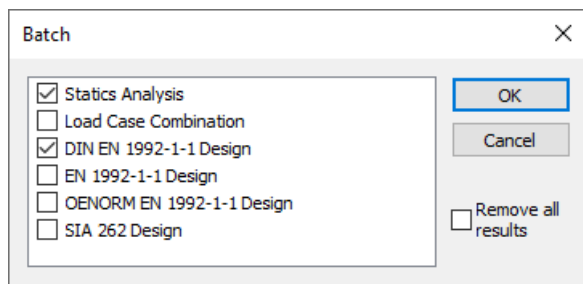
If this switch is activated, all of the selected load cases will be recalculated. Otherwise, the calculation program checks whether a recalculation is necessary.

### Listing

- *No listing*: No log is generated by the calculation program.
- *Create listing*: A log is generated during the calculation according to the selection list.

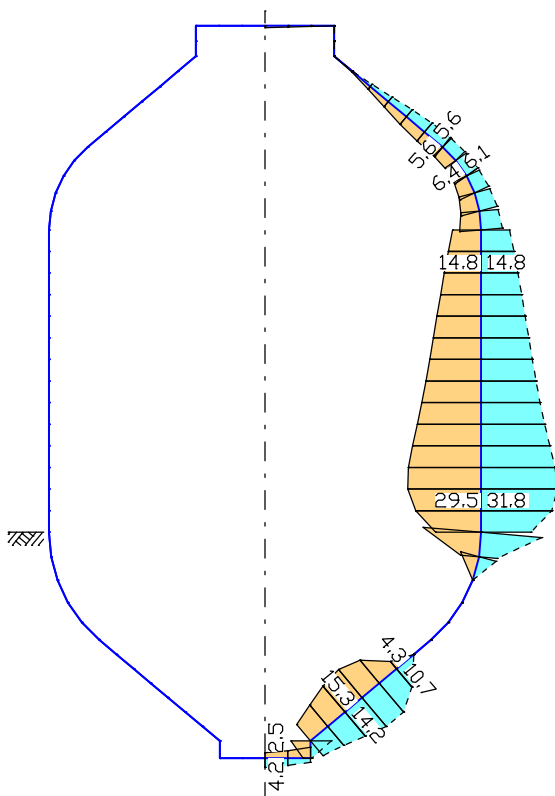
## Batch

Use this menu item to run all selected analyses as a batch job.



## Results

The following chapter describes the representation of the analysis results. Results can be shown graphically as well as in tabular form. They are selected in the database or using the *Results* function of the *Structure* menu.



Representation of the ring reinforcement of a septic tank [cm<sup>2</sup>/m]









### Soil pressures load case 1 - Dead load

	Element	Sigma.r [kN/m <sup>2</sup> ]	Sigma.z [kN/m <sup>2</sup> ]
1	31	-20.35	0.00
2	31	-16.92	0.00
3	32	-53.04	0.00
4	32	-47.65	0.00
5	33	-82.14	0.00
6	33	-76.54	0.00
7	34	-108.54	0.00
8	34	-105.54	0.00
9	35	-134.23	0.00
10	35	-136.24	0.00

Tabular output of soil pressures of the 1st load case

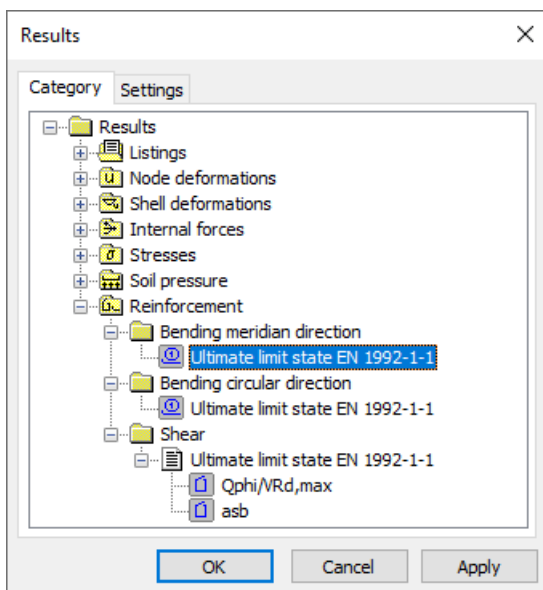
## Result Bar



The result bar includes the following buttons for representing results:

-  Start the dialog for result representation.
-  Turn off result representation.
-  Result representation of the previous load case or load case combination.
-  Result representation of the next load case or load case combination.
-  Ordinate selection for manual labeling.
-  Enable result representation for the selected elements.
-  Disable result representation for the selected elements.
-  Output the ordinate value at the marked point in the status bar.

## Result Categories

The following dialog shows possible result categories.



The load case and combination results as well as all presentable result components are listed in the subsection of the respective category. After making a selection, the *Settings* tab offers views described below with additional options. The symbol in front of the item identifies the type of view ( e.g. : table view, : graphical view of internal forces).

## Deformation Figure

### Scale

The scale can be set automatically or manually. The node displacements are multiplied by the distortion factor and added to the node coordinates. For load cases the bending curve is displayed as well.

### Display original system in dashed manner

Switch on/off the dashed non-deformed system during deformation display.

### Label

- *Auto*: Automatically add labels to the nodes listing the length of the displacement vector [mm].
- *Off*: Disable labeling.

### Limit value for labeling

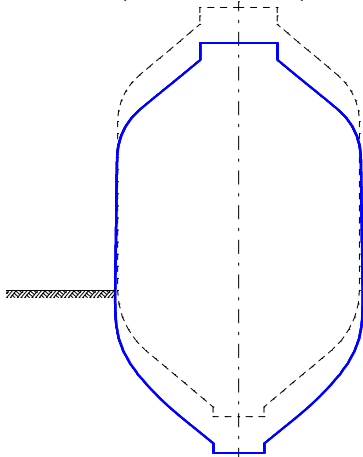
Only displacements exceeding the limit value are labeled.

### Decimal places

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

### Text height [mm]

The unit depends on the output device.



Superelevated deformation figure

## Numeric

This view displays the support reactions or node deformations numerically.

### Lower boundary for presentation

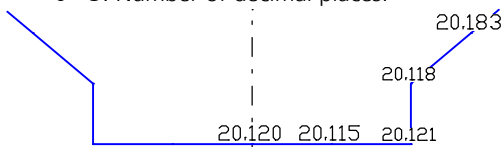
Only results exceeding the limit value are output.

### Text height [mm]

The unit depends on the output device.

### Decimal places

- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.



Numerical representation of the deformations  $u_z$  [mm]

## Result Graphs

The results are plotted along the element axis. A linear progress of results is assumed between the result locations.

### Scale

The scale of the ordinate display can be set automatically or manually.

### Label

- *Auto*: Enable automatic labeling.
- *Off*: Disable labeling.
- *Everywhere*: Label all ordinates.
- *Selectable*: Choose manual labeling with the help of the results bar.

### Limit value for labeling

Only ordinates exceeding the limit value are labeled.

### Decimal places

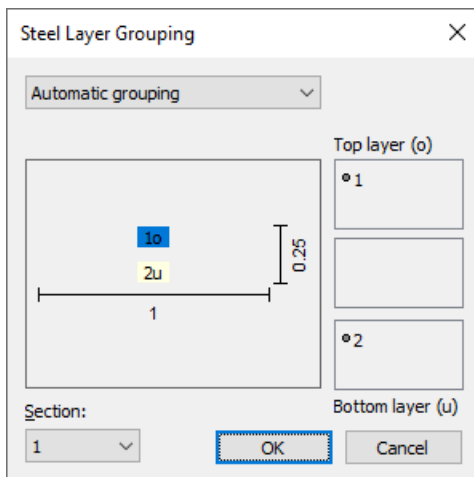
- *Standard*: Automatic setting of decimal places.
- 0 - 3: Number of decimal places.

### Text height [mm]

The unit depends on the output device.

### Steel layer grouping ...

Define the grouping of the steel layers with this function.



During automatic grouping, all steel layers whose  $r$  component is greater than the  $r$  component of the centroid are displayed on the upper side of the elements, all other layers on the lower side.

For manual grouping, steel layers can be moved between the list boxes using the mouse. The steel layers of the middle list box are not displayed.

### Subtract base reinforcement

The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

## Colored

For the representation type *Colored* the elements are colored according to their results.

### Number of colors

Set the number of colors for automatic color grading.



### Color shift

Shift of the automatic color grading.

### Grading

Enable/disable manual color grading. Color and number values can be changed.

Functions of the context menu ( right mouse button):

- Save as Presetting* Saves the current grading as presetting for other projects
- Load Presetting* Loads the saved presetting
- Load Colors of View* Sets the grading according to the current color representation
- Database...* Management of different gradings in the *lgraph.dat* file
- Insert Row*  Adds another one above the selected color level
- Delete*  Deletes the selected color level
- Set All Values...* Fills the list with the help of *start value*, *end value* and *grading*

### Legend

Enable/disable the color legend.

### Dividing line

Enable/disable dividing lines at the color transitions.

### Subtract base reinforcement

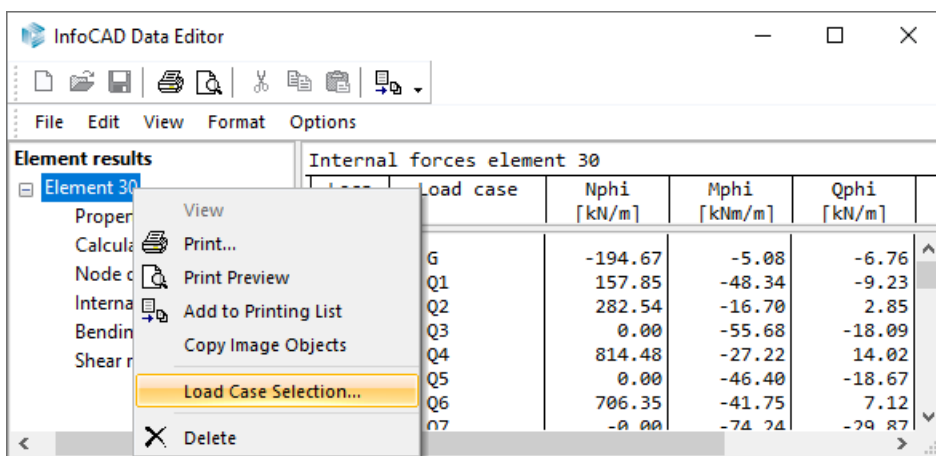
The base reinforcements defined for the sections are subtracted from the computed reinforcement and the remaining reinforcement is displayed.

### Deformation figure

Enable/disable the deformation figure of a load case.

## Result List

The result list for selected elements is output in the data editor. It can be accessed over the context menu for the elements.



The output can be limited to specific load cases using the *Load Case Selection* option of the context menu.



## Combination Information

The *Combination information* option can be used to determine the load cases involved in a combination result with their respective weightings. This function is available during the display of combination results. After the selection of the desired result location with the right mouse button, the *Combination information* function appears in the context menu.

The following functions are available in the context menu of the dialog:

- *Copy (clipboard)*
- *Create load group...*
- *Create superposition load case*
- *Associated deformations...*
- *Associated support forces...*

Combination information						
EN1992.SV.1 : 1. Permanent and temporary situation, EN 1992-1-1 Internal forces Element 29 - Loc. 2, Node 30 - Mphi min, max						
Load case	Factor	Nphi	Mphi	Qphi	Ntheta	Mtheta
1 : Dead load	1.35	-262.81	-6.86	-9.13	60.23	-1.37
2 : Liquid pressure	1.50	236.77	-72.50	-13.85	2722.09	-14.50
Mphi min		-26.04	-79.36	-22.98	2782.33	-15.87
1 : Dead load	1.00	-194.67	-5.08	-6.76	44.62	-1.02
Mphi max		-194.67	-5.08	-6.76	44.62	-1.02

Print...    ->Print list    OK

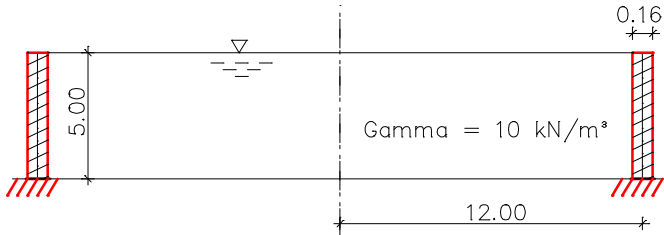
### Last Combination

Shows result values corresponding to the last *Combination information*.

# Examples

## Water Tank

The following example (Hampe, Volume 2, p. 187) illustrates the analysis of a water tank.



Water tank, system and dimensions

### Material and section properties

No.	Mat. type	EModule [MN/m <sup>2</sup> ] E.phi	Poisson-R.alpha.t E.theta	Poisson-R.alpha.t mue.phi	gamma [1/K]	thick-ness [m]	Bedding [MN/m <sup>3</sup> ] kbr	kbz
1	Frei	21000	21000	0.17	1.000e-05	25.000	0.1600	0

### Creep and shrinkage properties

Material	phi.t	rho	epsilon.s
1	0.000	0.800	0.00E-05

### Node description

No.	r [m]	z [m]
1	12.000	0.000
2	12.000	1.000
3	12.000	2.000
4	12.000	3.000
5	12.000	3.500
6	12.000	4.000
7	12.000	4.500
8	12.000	5.000

### Elements

No.	Kn 1	Kn 2	CrS Mat	Joints NQM	NQM	Length [m]	Layer
1	1	2	1	0	0	1.000	1
2	2	3	1	0	0	1.000	1
3	3	4	1	0	0	1.000	1
4	4	5	1	0	0	0.500	1
5	5	6	1	0	0	0.500	1
6	6	7	1	0	0	0.500	1
7	7	8	1	0	0	0.500	1

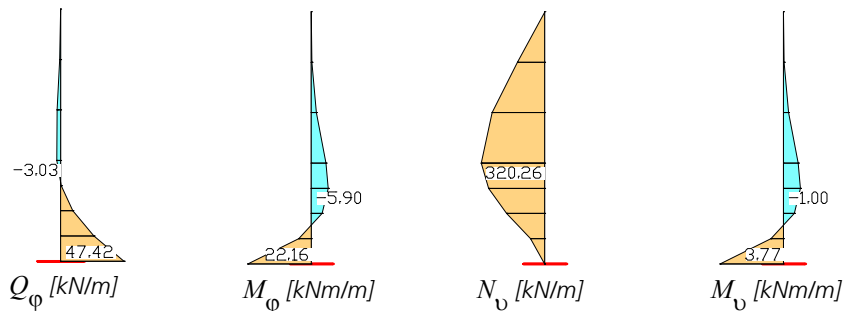
### Supports

Node No.	r - Direction			z - Direction			Rotation	
	F= fixed [MN/m]	Spring [grad]	Angle	F= fixed [MN/m]	Spring [grad]	Angle	F= fixed [MNm]	Spring
8	F		0.00	F		0.00	F	

### Load data Load case 1

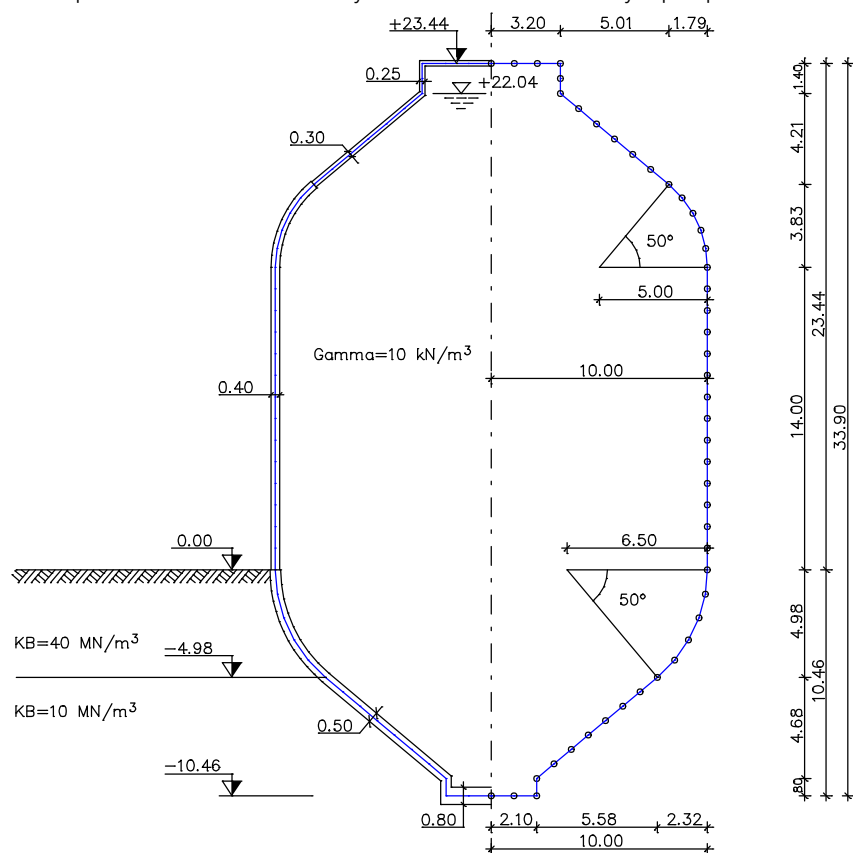
FD	Element from to	Liquid pressure liquid level z [m]	specific gravity of the liquid [kN/m <sup>3</sup> ]	sign
	1 7	0.00	10.00	1

**Internal forces load case 1**



**Septic Tank**

The septic tank shown is elastically bedded and is stressed by liquid pressure.



Septic tank with filling, C 20/25 - EN 1992-1-1, cover 5 cm

**Material and section properties**

No.	Material type	EModule [MN/m <sup>2</sup> ]		Poisson-R.alpha.t		gamma [kN/m <sup>3</sup> ]	thick-ness [m]	Bedding [MN/m <sup>3</sup> ]	
		E.phi	E.theta	mue.phi	[1/K]			kbr	kbz
1	C20/25-EN	30000	30000	0.20	1.000e-05	25.000	0.2500	0	0
2	C20/25-EN	30000	30000	0.20	1.000e-05	25.000	0.3000	0	0
3	C20/25-EN	30000	30000	0.20	1.000e-05	25.000	0.4000	0	0
4	C20/25-EN	30000	30000	0.20	1.000e-05	25.000	0.5000	40	0
5	C20/25-EN	30000	30000	0.20	1.000e-05	25.000	0.5000	10	0
6	C20/25-EN	30000	30000	0.20	1.000e-05	25.000	0.8000	10	0

**Load data Load case 1 (Dead load)**

EG : Dead load

**Load data Load case 2 (Liquid pressure)**

FD : Liquid pressure  
 Element liquid level z specific gravity of the liquid sign  
 from to [m] [kN/m<sup>3</sup>]

6	45	-22.04	10.00	1
---	----	--------	-------	---

**EN 1992-1-1 actions**

**Standard design group**

**G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

1 Dead load

**QH - Variable hydrostatic pressure**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures

Other actions

Psi.0 / Psi.1 / Psi.2 = 1 / 1 / 1

Load cases 1. Variant, exclusive

2 Liquid pressure

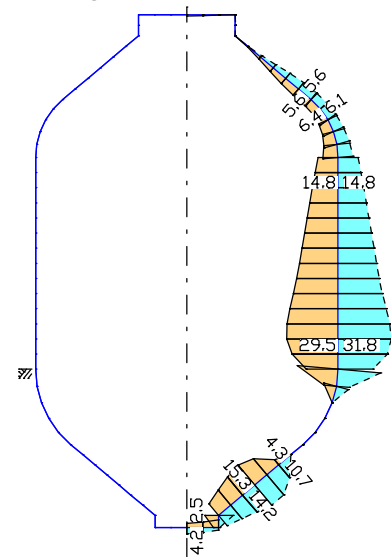
**1. Permanent and temporary situation**

Final state

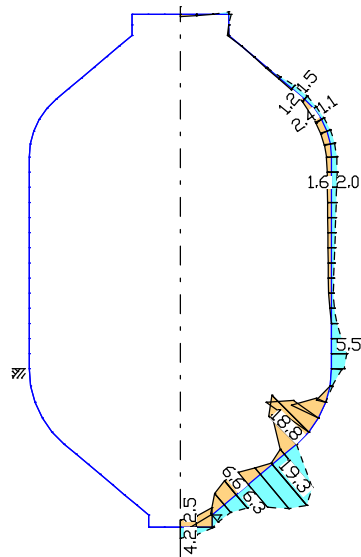
G Dead load

QH Variable hydrostatic pressure

**Bending reinforcement from EN 1992-1-1**



Ring reinforcement [cm²/m]



Meridian reinforcement [cm²/m]

# Load Case Combination

## Basics

Load case combinations are used to combine the results of a structure analysis as well as results of other load case combinations to determine minimum and maximum boundary curves.

To accomplish this, the load cases and combinations are assigned to the following action groups:

- **Permanent:** The individual load cases and combinations are always taken in consideration (dead load, for example).
- **Variable inclusive:** The individual load cases and combinations can act simultaneously (such as traffic loads on individual floor bays). They are only combined if they make a adverse contribution.
- **1. (-6.) Variable exclusive:** Only load cases and load case combinations which offer the lowest (<0) or the highest (>0) contribution are taken in consideration. The individual load cases and combinations cancel each other out (heavy truck positions, for example). You may describe up to six independent simultaneously acting action groups of this kind.
- **Exclusive selection:** The individual load cases and combinations cancel each other out. The most adverse contribution of this group is always taken into consideration.

## Input

The combination rule is entered into the database or in the analysis settings for the load case combination and saved with an associated number. The individual parts of the 'Load Case Combination' dialog are described below:

Load Case Combination 1

Label: Fundamental combination

Action: 1. Exclusive variable

Factor: 0.9

Load cases:  Multi-select

- 5 Wind from north
- 6 Wind from south
- 7 Wind from west
- 8 Wind from east
- K1 Fundamental combination

Action	Factor	Load case
Permanent	1.350	1 Dead load
Inclusive variable	1.500	2 Area 1
Inclusive variable	1.500	3 Area 2
Inclusive variable	1.500	4 Area 3

Do not recalculate contained combinations

### Label

Load case combination name.

### Load cases

List of the possible load cases or load case combinations (e.g. K1). Select items by marking them and clicking the button or drag & drop. The current action as well as the factor will be taken over. After a dynamic time step calculation (e.g. D3 for load case 3), a calculation according to the response spectrum method or the static calculation of load model 1, their results are also available for selection.

### Multi-select

Load cases and combinations can be added to the actions more than once.

### Action

Select the type of action.

### Factor

The individual load cases and combinations are weighted by a factor.

### Apply

The weighting factor or the action of selected load cases and combinations is changed.

### Do not recalculate contained combinations

This switch prevents the recalculation of the included load case combinations.

**Combinations...**

Opens a dialog that contains the first 9999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

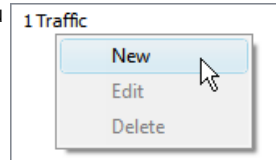
## Analysis Settings

**Calculate all combinations**

All the existing load case combinations are calculated (default).

**Calculate following selection**

Only the load case combinations present in the left list box are calculated. The context menu available in the list box allows the load case combinations to be edited.



**Create listing for ...**

Normally, only one log with the combination descriptions is created by the load case combination program. The log can also record combination results. The log is not needed in most cases.

## Results

The load case combination calculates all minimum and maximum boundary curves and all associated values for the following results.

- For example:  $\min u_x$  and associated  $u_y, u_z, \varphi_x, \varphi_y, \varphi_z$
- $\max u_x$  "  $u_y, u_z, \varphi_x, \varphi_y, \varphi_z$
- Beam internal forces:  $M_x, M_y, M_z, N_x, Q_y, Q_z$
- Shell internal forces  $m_x, m_y, m_{xy}, n_x, n_y, n_{xy}, q_x, q_y$
- Slab internal forces  $m_x, m_y, m_{xy}, q_x, q_y$
- Plain stress element internal forces  $n_x, n_y, n_{xy}$
- Axisymmetric shell internal forces  $M_\varphi, N_\varphi, M_\nu, N_\nu, Q_\varphi$
- Deformations  $u_x, u_y, u_z, \varphi_x, \varphi_y, \varphi_z$
- Accelerations  $a_x, a_y, a_z$
- Velocities  $v_x, v_y, v_z$
- Stresses on solid elements  $\sigma_x, \sigma_y, \sigma_z, \sigma_{xy}, \sigma_{xz}, \sigma_{yz}$
- Support reactions  $R_x, R_y, R_z, M_x, M_y, M_z$
- Soil reactions  $\sigma_x, \sigma_y, \sigma_z$

**Note on the load case combination:**

If the absolute value of the considered control value is smaller than the threshold value ( $10^{-3}$ , for deformations  $10^{-6}$ ), this and the associated values will be ignored during superposition. Below, results from three load cases are illustrated, which are subsequently combined inclusively.

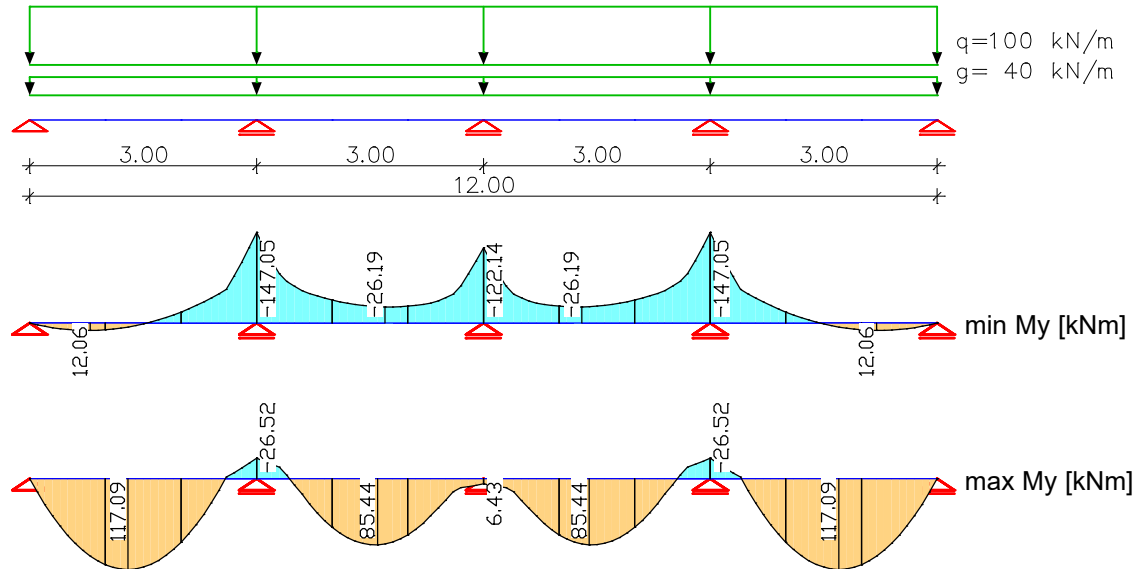
Results of load cases:				Results of load cases combination:			
Nr.	Nx [kN]	My [kN]	Mz [kNm]		Nx [kN]	My [kN]	Mz [kNm]
LF.1	0.00	40.00	-35.00	min Nx	0.00	0.00	0.00
LF.2	0.00	-10.00	20.00	max Nx	100.00	0.00	0.00
LF.3	100.00	0.00	0.00	min My	0.00	-10.00	20.00
				max My	0.00	40.00	-35.00
				min Mz	0.00	40.00	-35.00
				max Mz	0.00	-10.00	20.00

As load case 1 and 2 do not contribute to min./max. Nx, there are no associated values.

## Example

### Load Case Combination on a Multi-Span Girder

In this example a multi-span girder with the following loads is analyzed: dead load  $g$  (load case 1) and the variable inclusive traffic load  $q$  on a span-by-span basis (load cases 2 to 5). The internal forces  $\min. M_y$  and  $\max. M_y$  illustrated below result from the load case combination.



#### Load case combination log:

##### Load case combination 1 - permanent action

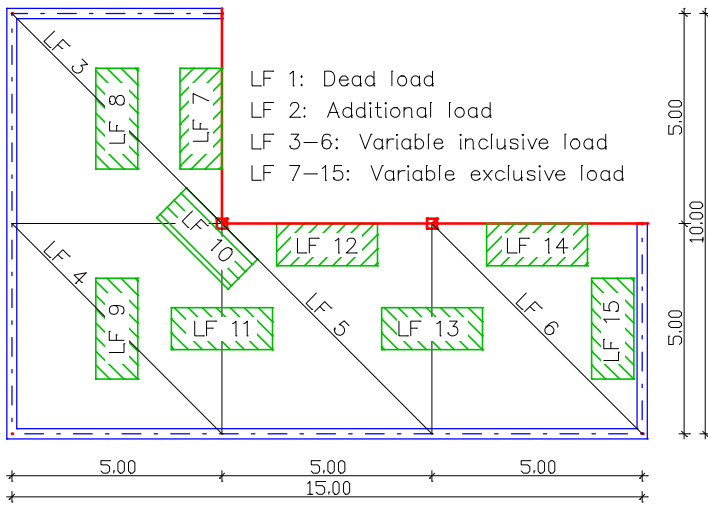
Load case	Factor
1 Dead load $g = 40 \text{ kN/m}$	1.000

##### Load case combination 1 - variable inclusive action

Load case	Factor
2 Span 1 $q = 100 \text{ kN/m}$	1.000
3 Span 2 $q = 100 \text{ kN/m}$	1.000
4 Span 3 $q = 100 \text{ kN/m}$	1.000
5 Span 4 $q = 100 \text{ kN/m}$	1.000

## Load Case Combination for a Floor Slab

The following example shows a floor slab with 15 load cases.



### Floor construction and load

The load cases 1 and 2 are constantly active. The four area loads defined on a field-by-field basis act inclusively, which means they can act simultaneously. The load cases 7-15 are considered as exclusive traffic loads, which means they cancel each other out.

The following log of the load case combination shows the assignment of the load cases to the action groups.

#### Load case combination 1 - permanent action

Load case	Factor
1 Dead load	1.000
2 Additional load	1.000

#### Load case combination 1 - variable inclusive action

Load case	Factor
3 Span-1	1.000
4 Span-2	1.000
5 Span-3	1.000
6 Span-4	1.000

#### Load case combination 1 - 1. variable exclusive action

Load case	Factor
7 GS-1	1.000
8 GS-2	1.000
9 GS-3	1.000
10 GS-4	1.000
11 GS-5	1.000
12 GS-6	1.000
13 GS-7	1.000
14 GS-8	1.000
15 GS-9	1.000



# Prestressed Concrete

## Basics

This program component was designed for prestressed concrete structures with internal prestressing, especially for the analysis of bridges, containers and floor slabs. It is based on the method of 3D tendon guides, which can be used for beam, shell and solid models. The area of application includes prestressed components without bond and with subsequent bond.

The analysis of a prestressed concrete structure is carried out in three steps:

- Input of the tendon groups, prestressing system and prestressing procedure with the *Prestressing* function of the *Structure* menu.
- Definition of a load case using the *Prestressing* load type.
- FEM analysis for load processing and internal forces calculation. The statically determined (P0) and undetermined (P1) part of the internal forces are additionally saved for beams and design objects.

If actions from creep and shrinkage are to be taken into account, perform the following additional steps:

- Input of the creep and shrinkage coefficients in the section dialog.
- Input of a creep-generating permanent load case. This load case combines the dead load, additional load and prestressing load cases with the help of the *Superposition* load type.
- Definition of a load case using the *Creep and shrinkage* load type and specification of the creep-generating continuous load case. Optionally, the internal force redistribution between concrete and prestressing steel can be taken into account, for instance for prestressing with subsequent bond.
- FEM analysis for determination of internal forces. If the relocation of internal forces is selected the resulting tendon forces are additionally saved.

The rest of the procedure depends on the standards to be used and is described in the corresponding sections:

- DIN Technical Report Bridge Checks.
- DIN 1045-1 Design.
- OENORM B 4700 Design.
- EN 1992-1-1 Design.
- EN 1992-2 Bridge Checks.
- SIA 262 Design.
- Recalculation Guideline Bridge Checks.

In general, when working with prestressed structures, the model should be designed with special care. In practice the use of mixed systems is advisable. For instance, for a multi-rib T-beam bridge it is advisable to combine prestressed beam elements for the bridge longitudinal direction (T-beam section) with slab elements (lateral direction). This provides a simple solution for the problem of load distribution. It is possible to additionally design the bridge in the lateral direction in the same step. However, due to the complexity of the problem, it is not possible to make a general statement concerning recommendable structure models.

## Composite Section

The redistributions of the internal forces between concrete and prestressing steel at the composite section as a result of creep and shrinkage are of interest for the check of prestressed concrete structures. As a simplification, these can be specified as a percentile share of the prestressing load case. Alternatively, the composite section can be treated directly. In this case, the prestressing steel layers are included in the calculation of the creep and shrinkage load case while the global stiffness matrix is being processed. This results in quasi-composite elements whose strain state is taken to determine the corresponding share of internal forces of the composite components. This approach is implemented for all element types. Creep redistribution can therefore also be determined for area and solid models. Due to this, however, the complexity of the calculation, which depends directly on the number of tendon groups, increases considerably, especially for determining the global stiffness matrix. The internal forces (normal forces, bending moments, lateral forces) given by the program always correspond to the **concrete section**. When analyzing composite elements, these alone are not in equilibrium with the external forces since the tendon group forces must be applied while taking into account their spatial orientation. As the integration of the prestressing steel stiffnesses is based on a finite element approach, **an adequate FEM mesh is especially important for beam elements**. Area and solid models, on the other hand, generally exhibit a sufficiently fine discretization.

## Creep and Shrinkage

In prestressed concrete construction, taking into account time-dependent material behavior is especially important for reasons of load-bearing safety. In general, an estimation is made for framework models using an approximation for statically determined composite structures with one tendon group layer (according to Trost or Rüschi). In the calculation model at hand, area and solid models are also used. For these, no comparable approximations can be made.

The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1+\rho \cdot \varphi} (\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,s})$$

with

$\sigma_b(t)$	Concrete stress from creep and shrinkage at time $t$ .
$E_b$	E-modulus of the concrete.
$\rho$	Relaxation coefficient according to Trost for time $t$ (normally $\rho=0.80$ ).
$\varphi$	Creep coefficient for time $t$ .
$\varepsilon_b(t)$	Concrete strain from creep and shrinkage at time $t$ .
$\varepsilon_{b,0}$	Concrete strain from creep-generating permanent load.
$\varepsilon_{b,s}$	Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, provide the internal forces and deformations of the concrete.

The creep and shrinkage coefficients are entered in the section dialog. To be taken into account in the FEM analysis, a load case must be defined that contains the *Creep and shrinkage* load type and specifies the creep-generating permanent load case.

## Actions From Prestressing

The definition of the tendon groups is carried out independent of elements. This allows for maximum flexibility for the graphical input and modification of the tendon group geometry. The allocation of the tendon groups to the elements is performed via geometric conditions when calculating the internal forces. The following types of tendon groups will be differentiated:

- **Beam:** The tendon group is allocated to a beam of the cross-section type *Polygon* or *Composite*, if it
  - penetrates both front faces,
  - penetrates one front face and starts or ends within the beam or
  - starts and ends within the beam.
 Depending on user selection, the tendon group can be located
  - within the cross-section,
  - within the exterior polygon or
  - within the bounding rectangle.
 The last two options allow a simplified tendon guide mainly for hollow sections during the design phase.
- **Area:** This type can be used for area and solid elements. The tendon group is allocated to an area element, if it
  - penetrates two lateral faces or
  - penetrates one lateral face and starts or ends within the element.
 For solid elements the tendon group can penetrate any face of the element.
- **Mixed:** This type of tendon group is used for the simultaneous prestressing of beam, area and solid elements. The main applications are the calculation and the check of composite sections. There are no limitations for the course within the element, but for beam elements the tendon group has to be located in the beam cross-section as well.

The basis for the analysis of actions from prestressing is provided by a tendon group force curve that considers strain increase, release and slippage. The influence of the tendon friction is taken into account using the typical method of cable element friction so that the prestressing procedure can generally be formulated in parameter form as follows:

$$V(p) = V(0) \cdot e^{-\mu(\alpha+l(p)\beta)}$$

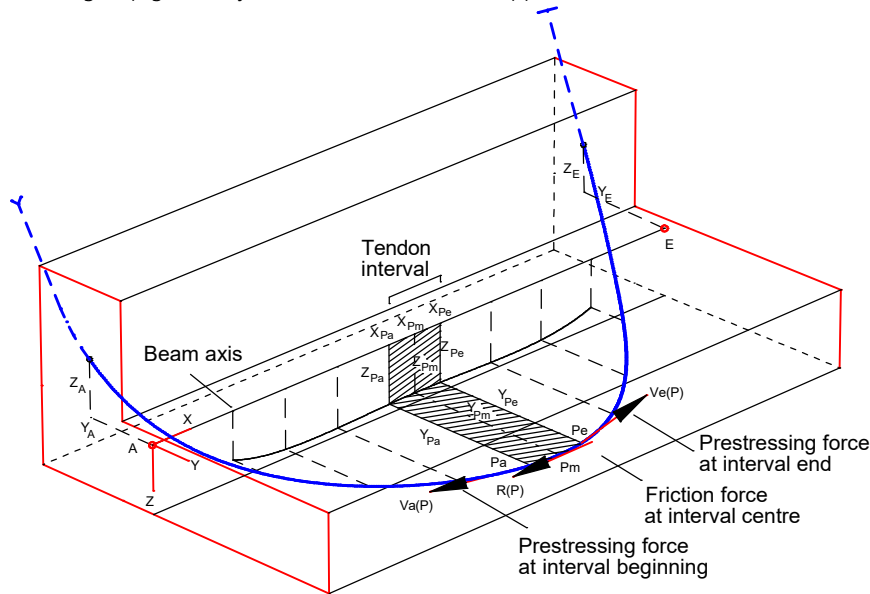
with

$p$	Curve parameter.
$V(p)$	Prestressing force at the curve parameter $p$ .
$V(0)$	Prestressing force at the prestressing anchor.
$\mu$	Friction coefficient.

- $\alpha$  Intentional deviation angle of the tendon.
- $l(p)$  Tendon length at the curve parameter  $p$ .
- $\beta$  Unintentional deviation angle.

Corresponding approaches apply to the release. Here other friction coefficients might also enter the calculation. The actions (forces and moments) from prestressing result from the corresponding tendon group and friction force for the element location being analyzed.

For the practical calculation, the tendon group is subdivided into a finite number of intervals and the eccentric friction and deviation forces are determined at the corresponding interval points. These are then applied to the corresponding element as 3D individual forces and moments. Tendon groups of the type *Area* and *Mixed* are divided equally spaced at the full tendon length, whereas tendon groups of the type *Beam* are divided equally spaced within a beam. Even with the same tendon group geometry small differences in the application of force can occur.



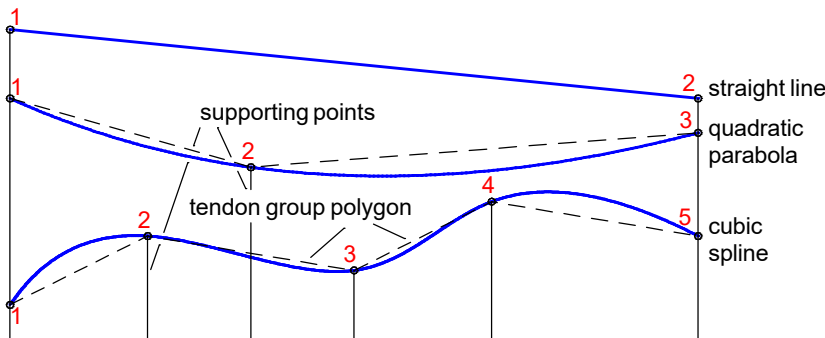
*Deviation and friction forces from prestressing*

The method described allows for the determination of the 3D internal force state of the prestressed concrete. This contains the statically determined as well as the statically undetermined internal force shares. The sum of the support reactions is always zero here as the slippage, friction and deviation forces form an equilibrium system.

To include prestressing in the FEM analysis, you need to define a load case with the *Prestressing* load type.

## Tendon Group Geometry

The tendon curve on which the calculation approach is based is represented by a 3D cubical spline function, which is defined by a so-called tendon group polygon. This makes available an ordered set of points that represent the supporting points of the spline function. The spline function is the 3D curve that runs through all spline points with the least amount of curvature. If only two supporting points are specified, a straight line results. Three supporting points result in a quadratic parabola. More than three supporting points create a cubic spline function.



Tendon group polygon and spline function

## Input

### Prestressing

The *Prestressing* menu item allows you to input or edit tendon groups. Existing tendon groups are automatically loaded.

#### Input

<i>Off</i>	Remove tendon groups from the graphical view.
<i>Save</i>	Save all tendon groups in the project file.
<i>Define</i>	Call up the <i>Tendon group properties</i> dialog to define a new tendon group. The spline points of the tendon group can be defined either graphically or using a table.
<i>Spline points</i>	Edit selected spline points.
	<i>X, Y, Z</i> Edit coordinates of the selected spline points.
	<i>Insert</i> Insert new spline point before the selected point.
	<i>Delete</i> Delete selected spline point.
<i>View...</i>	Select a tendon group view.
<i>New</i>	Enter a new tendon group view.
<i>Standard</i>	Select the <i>Standard view</i> .
<i>Zero point</i>	Set new origin for the coordinate system of the view.
<i>Base points</i>	The values selected under <i>View...</i> are depicted tabulary for the sectional and beam views at discrete supporting points.
	<i>Spline points</i> Each spline point of visible tendon groups defines a supporting point.
	<i>Variable</i> Supporting points are defined by the sectionally specification of divisions.
	<i>Equidistant</i> Supporting points result from the entered constant division.
	---x---  Input of new divisions for the options <i>Variable</i> and <i>Equidistant</i> .

## Tendon Group Properties

The tendon group properties are edited using the following dialog box.

Tendon group properties

General | Prestressing System | Prestressing Procedure

Number: 1 | Label: Unnamed | Prestressing system: 1 - SUSPA EC 140

Number of tendons: 6 | Type: Beam | Prestressing procedure: 1 - standard

In cross secti

Spline points:

	x [m]	y [m]	z [m]
1	-0.800	0.000	-0.286
2	10.000	0.000	0.480
3	20.800	0.000	-0.286
*			

Ignore in prestress load

OK Cancel

### Number

Number of the tendon group or start value for a new numbering.

### Label

Name of the tendon group.

### Prestressing System

The prestressing system to be used. Its properties are defined on the dialog page of the same name.

### Number of tendons

Number of tendons that are bundled in this tendon group.

### Type

- *Beam*: Tendon group for beam prestressing. The second combo box allows to define geometrical correlation of tendon groups and beams. Prestressed beams must have the cross-section type *Polygon* or *Composite*.
- *Area*: Tendon group for area and solid prestressing.
- *Mixed*: Tendon group for simultaneous prestressing of beam, area and solid elements, e.g. for composite sections.
- *Inactive*: Tendon group is not active.

For further explanations, see section *Actions from prestressing*.

### Ignore in prestress load

For the FEM analysis of a load case with the *Prestressing* load type, this tendon group is ignored. When creating construction stages, this option is set for all existing tendon groups.

### Prestressing Procedure

The prestressing procedure to be used. Its properties are defined on the dialog page of the same name.

### Spline points

Coordinates of the spline points of the tendon group.

## Prestressing System

The prestressing system combines typical properties that are then assigned to the tendon groups using a number.

### Number, Label

Number and name of the prestressing system. The option <Database> enables to load or to store properties by use of the file *lgraph.dat*.

### Certification

- DIN 1045-1
- DIN 4227
- EC2
- OENORM
- SIA 262

By selection of the certification, the prestressing force  $P_{m0}$  is determined according to the standard.

### Area $A_p$

Section area  $A_p$  of a tendon [mm<sup>2</sup>].

### $\beta_s, \beta_{0.2}$

Yield strength or  $\beta_{0.2}$  limit of the prestressing steel according to DIN 4227 [MN/m<sup>2</sup>].

### $f_{p0,1k}$

Characteristic value of the 0.1% strain limit of the prestressing steel per DIN 1045-1, OENORM, SIA 262 and EC2 [MN/m<sup>2</sup>].

### E-Modulus

E-modulus of the prestressing steel [MN/m<sup>2</sup>].

### $\beta_z$

Tensile strength of the prestressing steel according to DIN 4227 [MN/m<sup>2</sup>].

### $f_{pk}$

Characteristic value of the tensile strength of the prestressing steel per DIN 1045-1, OENORM, SIA 262 and EC2 [MN/m<sup>2</sup>].

### $P_{m0}$

The permissible prestressing force of a tendon [kN] that corresponds to the selected certification is displayed where the minimum of the two possible values is decisive. After releasing the input field, a different prestressing force can be defined.

Certification as per DIN 1045-1:

$$P_{m0} = A_p \cdot 0.85 f_{p0,1k} \text{ or } A_p \cdot 0.75 f_{pk} \text{ according to DIN 1045-1, Eq. (49).}$$

Certification as per DIN 4227:

$$P_{m0} = A_p \cdot 0.75 \beta_s \text{ or } A_p \cdot 0.55 \beta_z \text{ according to DIN 4227-1, Tab. 9, Row 65.}$$

Certification as per EC2:

$$P_{m0} = A_p \cdot 0.85 f_{p0,1k} \text{ or } A_p \cdot 0.75 f_{pk} \text{ according to EN 1992-1-1, Eq. (5.43).}$$

Certification as per OENORM:

$$P_{m0} = A_p \cdot 0.80 f_{p0,1k} \text{ or } A_p \cdot 0.70 f_{pk} \text{ according to OENORM B 4750, Eq. (4) and (5), and OENORM B 1992-1-1, Chapter 8.9.6.}$$

Certification as per SIA 262:

$$P_{m0} = A_p \cdot 0.7 f_{pk} \text{ according to SIA 262, Eq. (22), Chapter 4.1.5.2.2.}$$

### Duct diameter

Is used for the decompression check according to the European standard and for beam tendons to calculate the net section values [mm].

### Friction coefficients

Friction coefficients  $\mu$  for prestressing and release.

### Slippage

Slippage at the prestressing anchor [mm].

### Unintentional deviation angle $\beta'$

Unintentional deviation angle of a tendon [°/m].

## Prestressing Procedure

The prestressing procedure differentiates between the start and end of the tendon group. The size of the maximum prestressing force is determined by factors regarding the permissible prestressing. In general, this is  $P_{m0}$  (see *Prestressing system*). Using the factor specified for the release, the maximum prestressing force remaining in the tendon group is defined with respect to  $P_{m0}$ . The prestressing force that remains at the prestressing anchor is calculated from this by the program. The resulting prestressing involves immediate losses due to friction and slippage, but not due to the elastic deformations of the concrete and the short-term relaxation. Each prestressing anchor can be prestressed and released twice. The prestressing procedures are numbered.

Normalized Force	1. Tensioning	1. Release	2. Tensioning	2. Release
Start:	1	1	0	0
End:	1	1	0	0

### Number, Label

Number and name of the prestressing procedure.

### Tensioning with Pmax

Selecting this check box causes the factors for tensioning correspond to the maximum force  $P_{max}$  for tendons certified according to DIN 1045-1 or EC2 (see the following example).

**Kappa**

If tensioning with  $P_{max}$  is selected, the permissible maximum force is calculated using the allowance value  $\kappa$  to ensure there is an overstressing reserve.

**1. Tensioning**

Factor relating to  $P_{m0}$  or  $P_{max}$  for the prestressing force at the tie at the 1st instance of tensioning.

**1. Release**

Factor relating to  $P_{m0}$  for the maximum remaining prestressing force at the 1st release. '0': no release!

**2. Tensioning**

Factor relating to  $P_{m0}$  or  $P_{max}$  for the prestressing force at the tie for the 2nd tensioning. '0': no 2nd tensioning!

**2. Release**

Factor relating to  $P_{m0}$  for the maximum remaining prestressing force at the 2nd release. '0': no 2nd release!

The prestressing force curve is determined in the following sequence:

- Tensioning and release at the start,
- Tensioning and release at the end,
- Slippage at the start,
- Slippage at the end.

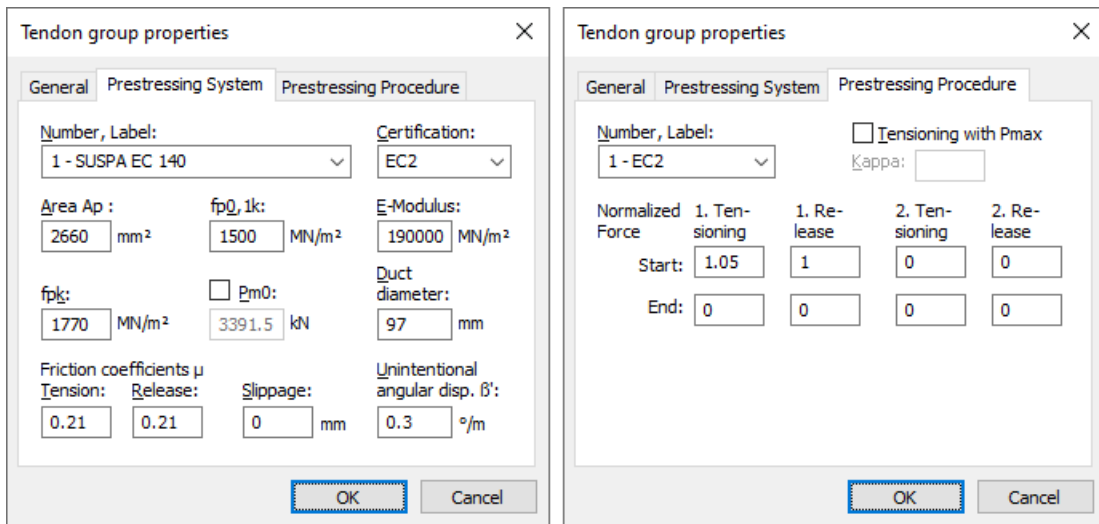
The differences between tensioning with  $P_{m0}$  and  $P_{max}$  are described in the following examples.

**The user is responsible for checking the permissibility of the maximum force during the stressing process.**

**Examples for Prestressing Procedures According to EC2**

**Tensioning with  $P_{m0}$**

The mode of action of the factors *Tensioning* and *Release* can be clarified using the example of an St 1570 / 1770 single tendon with prestressing anchor at the tendon start certified according to EC2.



The permissible prestressing forces are defined by:

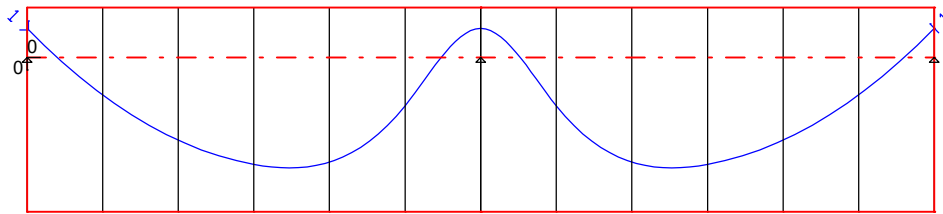
$$P_{max} = \min(A_p \cdot 0.80 f_{pk}, A_p \cdot 0.90 f_{p0.1k}) = 3591.0 \text{ kN}$$

$$P_{m0} = \min(A_p \cdot 0.75 f_{pk}, A_p \cdot 0.85 f_{p0.1k}) = 3391.5 \text{ kN}$$

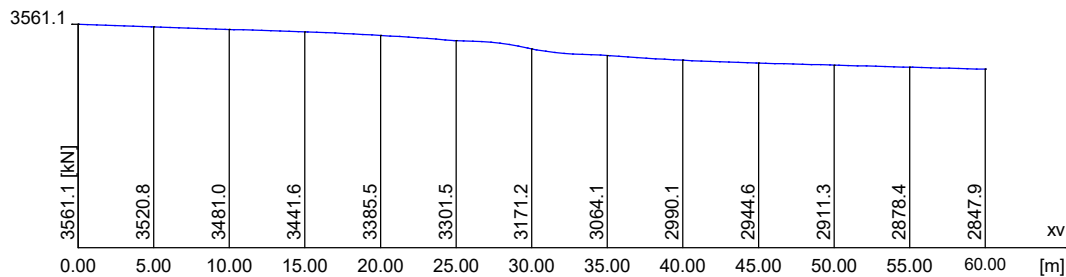
The first prestressing force curve of the following illustration results after overstressing with 5% using a factor of 1.05 relating to  $P_{m0}$ , i.e. the maximum prestressing force is 3561.1 kN <  $P_{max}$ .

The second prestressing force curve results after tensioning and release with the factors 1.05 and 1.0, i.e. the maximum prestressing force that remains in the tendon after it is fixed into place is 3389.3 kN <  $P_{m0}$ .

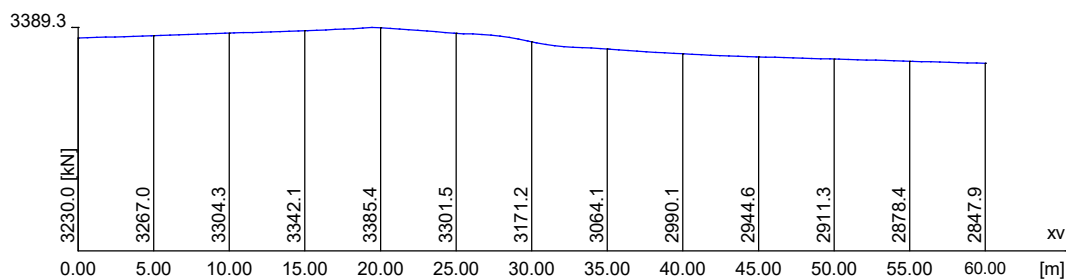




Single tendon, 10 times superelevated



Prestressing force curve after the 1st tensioning with a factor of 1.05



Prestressing force curve after the 1st release with a factor of 1.0

Potential slippage was not taken into account here to illustrate the effects described above. Slippage would result in an additional variation of the prestressing force curve. A second prestressing and release procedure would have similar effects. The same holds true for prestressing and release at the tendon end.

## Tensioning with $P_{\max}$

For tendons with certification as per DIN 1045-1 and EC2 the maximum force applied to the tendon during the stressing process is determined with the smaller of the following values:

$$P_{\max} = A_p \cdot 0.80 f_{pk} e^{-\mu\gamma(\kappa-1)} \text{ or } A_p \cdot 0.90 f_{p0.1k} e^{-\mu\gamma(\kappa-1)}$$

DIN 1045-1 rep. Book 525, Chapter 8.7.2  
 DIN TR 102, Chapter 4.2.3.5.4 (2)\*P  
 DIN EN 1992-1-1, Chapter 5.10.2.1 (NA.3)

with

$\mu$  Friction coefficient according to the general certification from the building authorities.

$\gamma$   $\Phi + k \cdot x$

$\Phi$  = sum of planned deviation angle over the length  $x$ ,

$k$  = unintentional deviation angle per unit of length ( $\beta'$  in the dialog),

$x$  = the distance between the prestressed anchor and the fixed anchor in the case of one-sided prestressing or the influence length of the respective anchor in the case of two-sided prestressing.

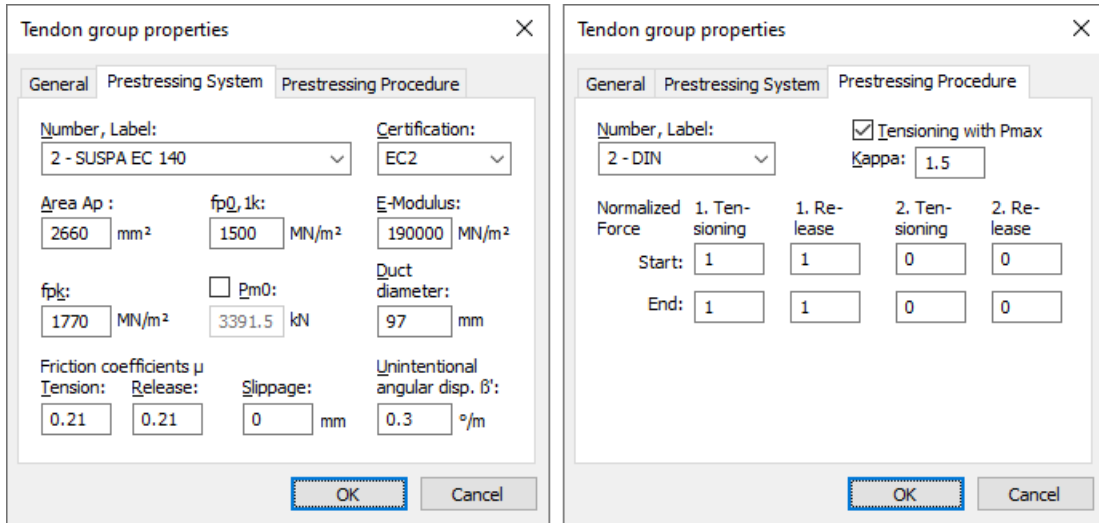
$\kappa$  Allowance value for ensuring an overstressing reserve with  $1.5 \leq \kappa \leq 2$  for tendons with supplemental bond according to the German standard and  $\kappa = 1$  for all other cases.

The program uses the specified allowance value  $\kappa$  to determine the maximum permissible value  $P_{\max}$ . The influence length  $x$  is assumed to be the tendon length for one-sided prestressing or simply half of the tendon length for two-sided prestressing.

**In this setting the overstressing factor refers to  $P_{\max}$ , which means the value 1.0 is used to select the maximum planned force according to the German standard.**

The release factor continues to refer to  $P_{m0}$ . Setting the value to 1.0 also assures that the force remaining in the tendon after it fixed into place is within the permissible range.

Using an St 1570 / 1770 single tendon prestressed on both sides with certification as per EC2, the prestressing force curve is illustrated for a value of  $\kappa = 1.5$ . Slippage is ignored for the sake of simplicity.

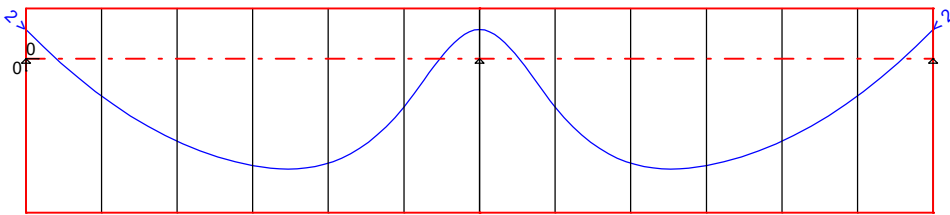


The program will determine the permissible prestressing forces as follows:

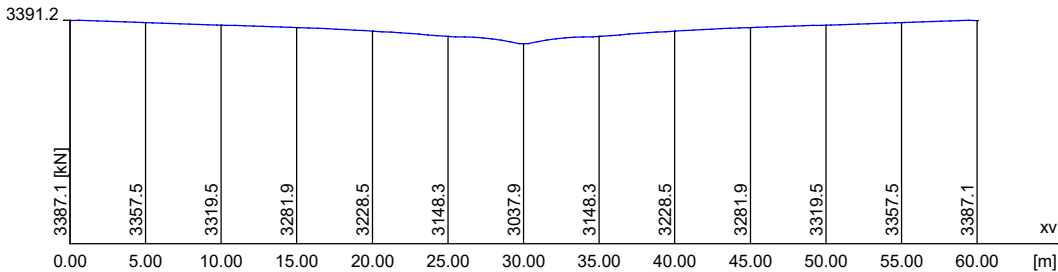
$$P_{max} = e^{-\mu\gamma(\kappa-1)} \cdot \min(A_p \cdot 0.80 f_{pk}, A_p \cdot 0.90 f_{p0.1k}) = 0.9457 \cdot 3591 = 3395.9 \text{ kN}$$

$$P_{m0} = \min(A_p \cdot 0.75 f_{pk}, A_p \cdot 0.85 f_{p0.1k}) = 3391.5 \text{ kN}$$

The maximum force  $P_{max}$  is automatically maintained with a tensioning factor of 1.0. As shown in the following force curve, 3391.2 kN remain in the tendon after it is fixed into place. Thus the limit  $P_{m0}$  is also observed.



Single tendon, 10 times superelevated

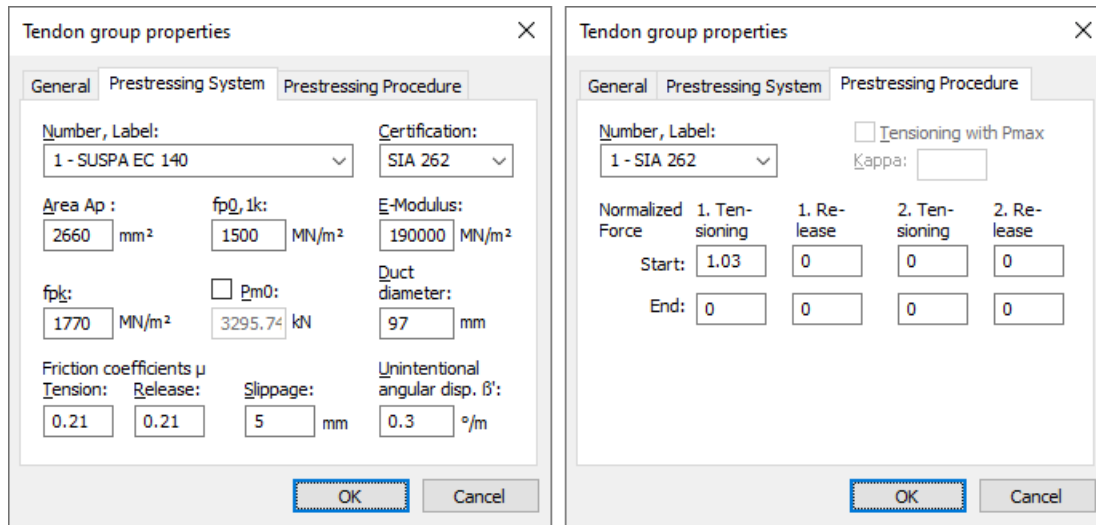


Prestressing force curve after tensioning and release

If the force calculated during prestressing is less than the value during release, then the program will make sure that the smaller value is not exceeded after the component is fixed into place.

## Examples for Prestressing Procedures According to SIA 262

The mode of action of the factors *Tensioning* and *Release* can be clarified using the example of an St 1570 / 1770 single tendon with prestressing anchor at the tendon start certified according to SIA 262.



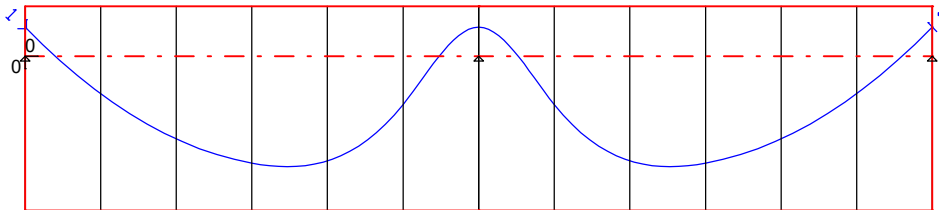
The permissible prestressing forces are defined by:

$$P_{max} = A_p \cdot 0.75 f_{pk} = 3531.2 \text{ kN}$$

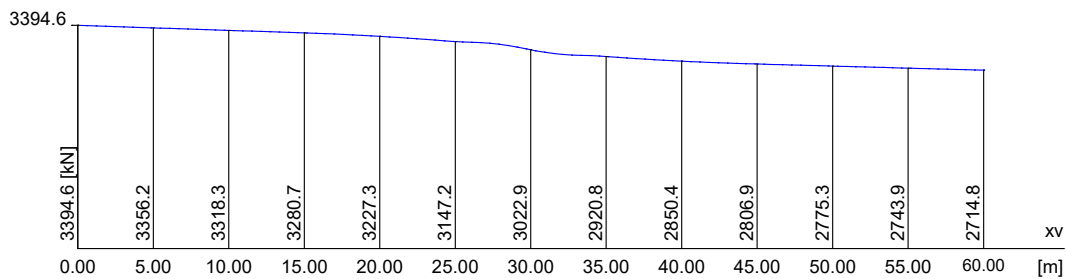
$$P_{m0} = A_p \cdot 0.70 f_{pk} = 3295.7 \text{ kN}$$

The first prestressing force curve of the following illustration results after overstressing with 3% using a factor of 1.03 relating to  $P_{m0}$ , i.e. the maximum prestressing force is  $3394.6 \text{ kN} < P_{max}$ .

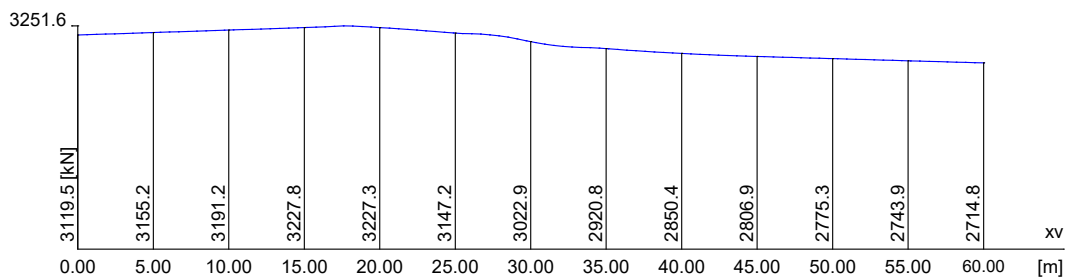
The second prestressing force curve results after setting down the press, i.e. the maximum prestressing force that remains in the tendon after it is fixed into place is  $3251.6 \text{ kN} < P_{m0}$ .



Single tendon, 10 times superelevated



Prestressing force curve after the 1st tensioning with a factor of 1.03



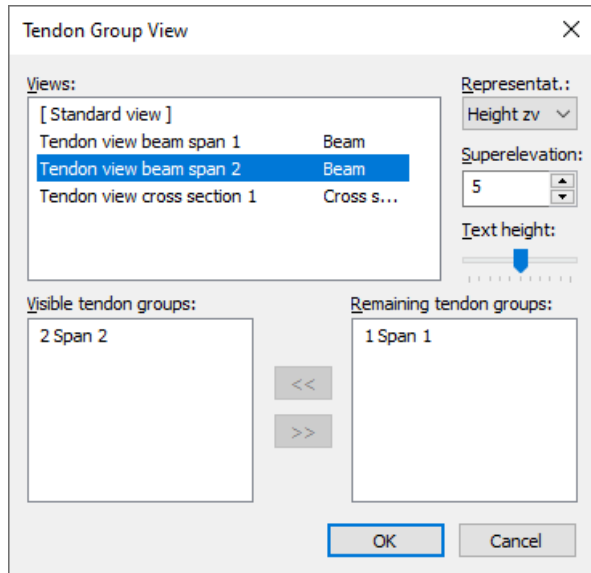
Prestressing force curve after anchor slip of 5 mm (setting down the press)

A release of the tendon was not taken into account here to illustrate the effects described above. Slippage would result in an additional variation of the prestressing force curve. A second prestressing and release procedure would have similar effects. The same holds true for prestressing and release at the tendon end.

## Tendon Group View

The following tendon group views are available:

- *Sectional view*: View of a section through area or solid elements.
- *Beam*: View of a longitudinal section through beams.
- *Cross section*: View of the beam section at any location along the beam axis.
- *Tendon*: View of a vertical section through beam or area elements along a tendon group.



### Views

List of the existing views.

### Representation

- *Nothing*: Do not display any tendon group properties.
- *Height zv*: Output the tendon group ordinates.
- *Force*: Display the prestressing force curve.
- *Radius*: Output curvature radii.

### Superelevation

Set the superelevation for the view.

### Text height

Set the text height for the labels.

### Visible, remaining tendon groups

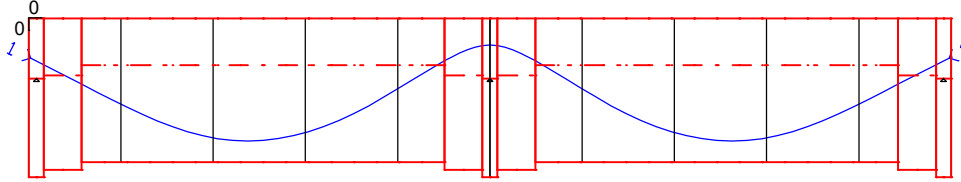
The tendon groups can be moved between both list boxes of the view.

The intersection and beam display represent the selected tendon group in a display plane xv-zv defined by three points. Also, all corresponding elements are displayed with their section thicknesses to allow the viewer to evaluate the height level of the tendon groups in the section.

In the sectional view it is optionally possible to display only the intersection points of the tendon groups with the sectional plane, similar to the crosssectional view.

The following illustration shows the superelevated display of a tendon group, its ordinates and the corresponding prestressing force curve.

Tendon groups in beam series 1, [-0.50/0.00/0.00] - [60.50/0.00/0.00] / [-0.50/0.00/1.00]  
 Superelevation = 5



Tendon group ordinates  $z_v$  [cm] at the base points

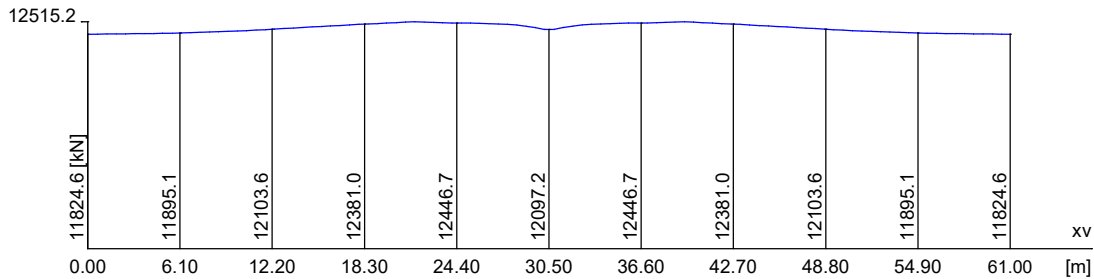
xv	0.00	6.10	12.20	18.30	24.40	30.50	36.60	42.70	48.80	54.90	61.00
1	52.0	114.4	159.0	151.9	94.0	36.0	94.0	151.9	159.0	114.4	52.0

Force function of tendon group 1 (10 tendon(s),  $l = 61.22$  m)

Prestressing system 1 - LH Holzmann. Certification according to DIN 4227.  
 $P_{m0} = 1226.6$  kN,  $A_p = 1260.0$  mm<sup>2</sup>,  $\mu_a = 0.20$ , Angle  $\beta' = 0.20$  °/m  
 $E\text{-Modulus} = 195000$  MN/m<sup>2</sup>,  $A_h = 4071.5$  mm<sup>2</sup>,  $\mu_n = 0.20$ , Slippage = 5.00 mm

Prestressing procedure 1 - Beispiel

Pre. anchor	:	Start		End	
Normal. force	:	1.080	1.050	1.080	1.050
Pre. force [kN]	:	1324.7	1252.2	1324.7	1252.2
Extension [mm]	:	301.7	-2.6	16.9	-2.6



**Explanations of the intersection and beam display**

[...]-[...] / [...]

Coordinates [m] of the three points defining the display plane. The first two points determine the  $x_v$  direction, the third point indicates the  $z_v$  direction.

*Tendon group ordinate  $z_v$*

The ordinates [cm] relate to the origin that has been specified. This is indicated in the longitudinal section (in this case on the section's upper edge).

*Prestressing System*

$P_{m0}$  Permissible prestressing force for each tendon [kN].

*E-modulus* E-modulus of the prestressing steel [MN/m<sup>2</sup>].

$A_p$  Section area of a tendon [mm<sup>2</sup>].

$A_h$  Section area of a duct [mm<sup>2</sup>].

$\mu_a, \mu_n$  Friction coefficient for prestressing and release.

*Angle  $\beta'$*  Unintentional deviation angle of a tendon [°/m].

*Slippage* Slippage at the prestressing anchor [mm].

*Prestressing Procedure* Number and name of the prestressing procedure.

*Normalized force* The factor for the tensioning and release at the start and end of a tendon.

*Prestressing force* Prestressing force at the start and end of the tendon for tensioning and release [kN].

*Extension* Calculated change in length of the tendon at the start and end of the tendon due to tensioning and release regardless of slippage and structure deformation [mm].

The maximum prestressing force in the tendon group is shown on the left side.

# DIN 1045 Design

## Basics

The design according to DIN 1045 (1988) determines the statically required bending, shear and torsion reinforcement for spatially stressed beam and area sections with the materials *B25* to *B55* and the general material type *Beton*.

The shear design is performed according to DIN 1045, Section 17.5, based on truss analogy and is therefore carried out with the equivalent rectangle for beams.

The permissible reduction in the area of immediate support and as a result of point loads near supports had to be forgone in order to keep the input task at a manageable level.

The rule to use the base values  $\tau_0$  (DIN 1045, Section 17.5.5.4), for full shear coverage (shear area 3) in the entire corresponding lateral force area of the same sign could only be considered within a beam element. Broad beams with an aspect ratio width/thickness  $> 5$  are treated like slabs (DIN 1045, Section 17.5.5.1). Here a reduction factor of  $k_1$  and the shear stress limit  $\tau_{011}$  are assumed for shear area 1.

The design is usually performed on the entire system. The necessary inputs are made as part of the section definition. The load cases and combinations to be designed, the scope of the log as well as the base values of the shear stresses to be used are controlled using the settings in the *Analysis* menu. In order to determine the decisive reinforcement, the section is designed for all internal forces. The resulting largest required reinforcement for each layer is saved.

In addition to designing the entire system, the stress range can also be checked. If the determined reinforcement is not within the desired stress range, the reinforcement is increased correspondingly. The inputs for this are also to be made in the settings of the *Analysis* menu. In the *DIN 1045 Design* folder of the database you can also perform a single design for freely defined section polygons or composite sections.

## Input

### Section Input

#### Design for M, N

Perform bending design for this section.

#### Mode

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be determined in the tensile area.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to DIN 1045, Section 25.2.2.1 (1).

#### Special concrete strength $\beta_N$

This value [MN/m<sup>2</sup>] is necessary to perform a design according to the standard if the material type *Beton* is selected, where:

- If the special concrete strength is equivalent to that of a standard concrete B15 to B55, the value of Table 12 applies for the calculation strength  $\beta_R$ . The corresponding limits of the shear stresses according to Table 13 apply, which however can be adjusted by the user.
- For all other special concrete strengths, the calculation strength is  $\beta_R = 0.6 \cdot \beta_N$ . For the limits of the shear stresses, on the safe side, the table values of the concrete class are used, whose special concrete strength is below the entered special concrete strength.

#### Design for lateral force

Perform shear design for this section.

**Quality of stirrups**

- 420S: Reinforcing rods according to DIN 488, Part 1 and DIN 1045, Table 6.
- 500S: Reinforcing rods according to DIN 488, Part 1 and DIN 1045, Table 6.
- 500M: Reinforcing meshes according to DIN 488, Part 1 and DIN 1045, Table 6.
- General: Freely definable steel quality [MN/m<sup>2</sup>].

**Effective height**

Effective static height for the shear design of area sections [m].

**kz**

Coefficient for calculating the lever arm  $z$  of the inner forces with  $z = \text{effective height} \cdot kz$ . This lever arm is used for the shear design of area sections.

**Design for torsion**

Perform torsional design for this section.

**Shear section...**

Section dimensions for the shear design of polygon sections.

**Design like slabs**

Beams are treated like slabs during shear design.

**Reduction**

Reduction factor  $k_1$  or  $k_2$  for  $\tau_{011}$  according to Section 17.5.5.2 for area sections or polygon sections that are to be designed like slabs.

**Limits of shear stress, Table 13**

Limits of the base values of the shear stress  $\tau_0$  according to Table 13, Row 1a or Row 1b.

**Full shear coverage for  $\tau_{0.0} \geq \tau_{0.11}$** 

Full shear coverage for  $\tau_0 \geq \tau_{011}$  and no check for  $\tau_0 < \tau_{011}$  according to DIN 1075, Section 9.2.3.1.

**Shear Section**

For polygon sections, additional section dimensions are required for the lateral force and torsion design according to DIN 1045. These are explained in the following. In case of sections with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.

**Width**

Section width for calculating the shear stresses resulting from  $Q_z$  and  $M_x$  [m]. This value is also used for the design rule 17.5.5.1 (1).

**Height dz**

Section height for calculating the shear stresses from  $Q_y$  and  $M_x$  [m]. This value is also used for the design rule 17.5.5.1 (1).

**Effective height**

Effective static height for the shear design for  $Q_z$  [m].

**kz**

Coefficient for calculating the lever arm  $z$  of the inner forces with  $z = \text{effective height} \cdot kz$ . This lever arm is used for the calculation of the shear stress  $\tau_{0,z}$  and of the stirrup reinforcement  $A_{sb,z}$  from  $Q_z$ .

**Effective width**

Effective static width for the shear design for  $Q_y$  [m].

**Corresp. kz**

Coefficient for calculating the lever arm  $z$  of the inner forces with  $z = \text{effective width} \cdot \text{corresp.kz}$ . This lever arm is used for the calculation of the shear stress  $\tau_{0,y}$  and of the stirrup reinforcement  $A_{sb,y}$  from  $Q_y$ .

**Core section bk, dk**

Dimensions of the core section for calculating the torsion reinforcement [m].

## Analysis Settings

**Design all combinations**

All the calculated load case combinations are designed (default).

**Design following selection**

The load cases and load case combinations in the left list box are designed.

**Listing**

- *No*: No log is generated by the design program.
- *Standard*: A log with a tabular output of design internal forces, shear stresses, shear areas and the required reinforcement is created during calculation.
- *Detailed*: This log differs from the standard log by additionally listing the internal forces of the check combination.

**Check of stress range**

For all load cases and combinations to be designed, the check of the stress range is carried out according to DIN 1045. If the determined bending reinforcement does not maintain the required stress range under service loads, the reinforcement is increased accordingly.

The check can only be carried out if bending reinforcement exists for **all** reinforcement layers. With respect to the stress range check, a minimum reinforcement should therefore be specified as the design may leave individual layers unreinforced.

**Stress range**

Stress range to be maintained by the reinforcing steel in MN/m<sup>2</sup>.

**Decisive Internal Force Combination**

Load case combination for which the stress range is to be checked. The necessary extremal steel stresses are determined taking into account the non-proportional relationships between stress and strain of the concrete and the steel. The combination rules can be taken from DIN 1075, Section 9.3.2 or DS 804. In general, it is not advisable to also include this combination in the combinations to be designed.

**Base values of shear stress...**

This button opens the dialog for modifying the base values of the shear stresses specified in DIN 1045.



## Base Values of Shear Stress

The limits can be changed if necessary and saved in the corresponding project file. The saved limits are then used for the shear design.

Row	Com- ponent	Shear range	Limits of shear stress tau.0 [MN/m <sup>2</sup> ] Name	Limits of shear stress tau.0 [MN/m <sup>2</sup> ]				
				B 15	B 25	B 35	B 45	B 55
1a	Slab	1	tau.011	0.25	0.35	0.4	0.5	0.55
1b				0.35	0.5	0.6	0.7	0.8
2		2	tau.02	1.2	1.8	2.4	2.7	3
3	Beam	1	tau.012	0.5	0.75	1	1.1	1.25
4		2	tau.02	1.2	1.8	2.4	2.7	3
5		3	tau.03	2	3	4	4.5	5

Limits of the base values of the shear stress  $\tau_0$  according to DIN 1045, Section 17.5.3, Table 13.

## Single Design

The single design function allows you to analyze individual section polygons separately from the whole system using predefined internal forces. The following data needs to be entered in the *Single Design* table, which is located in the *DIN 1045 Design* folder of the database.

<i>Section</i>	Number of the section to be designed.
<i>Br</i>	Concrete calculatory strength in [MN/m <sup>2</sup> ].
<i>Nx, My, Mz</i>	Design internal forces without safety factors.
<i>Mode BU</i>	Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
<i>Mode BS</i>	Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
<i>Mode BD</i>	For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to DIN 1045, Section 25.2.2.1 (1).
<i>Mode DZ</i>	Determine strain state for existing reinforcing steel layers.
<i>Mode IN</i>	Design disabled.
<i>Mode LF</i>	Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
<i>Mode MY</i>	Determination of the maximum bearable bending moment $M_y$ . The moment $M_y$ is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.

The calculation is carried out from the opened input table via the *Single Design* or *Page Preview* menu item.

## Punching Shear Check

When you select a check node, the key data for the checks is displayed in a dialog field.

*Standard Design* Set the specifications to the default values.  
Perform punching shear check. The Results group box will then indicate the calculated check results.

### Column

*Rectangle* Enter side lengths  $b$  and  $d$  [m] of the rectangular column.  
*Circle* Enter the diameter  $d$  [m] of the round column.  
*Location* Select the location or enter the location factor  $f_u$ .

### Slab

*Concrete* Select the concrete quality.  
*Reinforcing steel* Select reinforcing steel.  
*Eff. height* Enter the static effective height [m].  
*asx, asy* Enter the existing bending reinforcement [cm<sup>2</sup>/m].

### Punching force

*Load case* Adopt the punching force of a support node from a selected load case.  
*Combination* Adopt the punching force of a support node from a selected load case combination.  
*Direct* Enter the punching force.  
*No.* Enter load case or load case combination number.  
*Force* Punching force  $R_z$  [kN] for the check.  
*Load* Enter additional load  $q$  [kN/m<sup>2</sup>] in the perimeter of the column.  
*Factor for  $\tau_r$*  Factor  $f_t$  for taking into account not axisymmetric bending load. The calculated shear stress  $\tau_r$  is increased by this factor.

### Results

$\tau_r$  Calculated shear stress [MN/m<sup>2</sup>] in the perimeter of the column.  
 $\tau_1 = k_1 \cdot \tau_{011}$  Permissible shear stress [MN/m<sup>2</sup>] without shear reinforcement (according to DIN 1045, Section 22.5.2.).  
 $\tau_2 = k_2 \cdot \tau_{02}$  Permissible shear stress [MN/m<sup>2</sup>] with shear reinforcement (according to DIN 1045, Section 22.5.2.).  
 $\mu_g$  Existing reinforcement level:  $(a_{sx} + a_{sy}) / (2 \cdot hm)$   
 $A_{sb}$  Necessary shear reinforcement [cm<sup>2</sup>].

During the check one of the following three situations can arise:

$\tau_r < \tau_1$  No shear reinforcement necessary.  
 $\tau_1 < \tau_r < \tau_2$  Shear reinforcement necessary.  
 $\tau_2 < \tau_r$  Shear stress exceeding.

### Proposal

*asx, y* The increased bending reinforcement [cm<sup>2</sup>/m] that is recommended can reduce the necessary shear reinforcement  $A_{sb}$  in the result field or prevent shear stress exceeding.  
 *$A_{sb}$*  Additional shear reinforcement [cm<sup>2</sup>] to prevent, in conjunction with the bending reinforcement, shear stress exceeding.  
*Apply* Apply the recommended bending and shear reinforcement to the check.

# Design

## Design Internal Forces

A differentiation is made between the following design internal forces:

- from load cases
- from load case combinations
- for oblique reinforcements

The design internal forces for shell structures that are calculated for the bending design correspond to the plasticity approach according to Wolfensberger and Thürlimann. This approach takes into account how much the reinforcement deviates from the crack direction. Due to the current lack of precise data regarding the combined load of reinforced concrete shell structures from bending and normal force, the design internal forces for bending and normal force are calculated independently according to the static limit theorem of the plasticity theory and then used together as the basis for the design. This approach should always lead to results that are on the safe side.

The shear design for spatially stressed beam sections is carried out separately for the lateral forces  $Q_y$  and  $Q_z$ . The simultaneous acting of shear and torsion stress is taken into account according to DIN 1045, Section 17.5.7.

The shear design of slabs or shells is carried out for the maximum resulting lateral force of a design point. Consequently, the size of the stirrup reinforcement is independent of the internal force direction and has the dimension [cm<sup>2</sup>/m<sup>2</sup>].

## Design Internal Forces from Load Cases

### Design for m, n

Beams	$N_x, M_y, M_z$	Shells	$m_x \pm  m_{xy} , \quad n_x +  n_{xy} $
Slabs	$m_x \pm  m_{xy} $ $m_y \pm  m_{xy} $		$m_x \pm  m_{xy} , \quad n_x -  n_{xy} $ $m_y \pm  m_{xy} , \quad n_y +  n_{xy} $
Plain stress elements	$n_x \pm  n_{xy} $ $n_y \pm  n_{xy} $	Axisymmetric shells	$m_y \pm  m_{xy} , \quad n_y -  n_{xy} $ $N_\varphi, M_\varphi$ $N_u, M_u$

### Shear and Torsion Design

Beams	$Q_y, \quad Q_z, \quad M_x$	Slabs and shells	$\sqrt{q_x^2 + q_y^2}$
	$Q_z, \quad M_x$	Axisymmetric shells	$Q_\varphi$

## Design Internal Forces from Load Case Combinations

### Design for m, n

Beams	$\min N_{x'}$	<i>corresp.</i> $M_{y'}$	<i>corresp.</i> $M_z$
	$\max N_{x'}$	<i>corresp.</i> $M_{y'}$	<i>corresp.</i> $M_z$
	$\min M_{y'}$	<i>corresp.</i> $M_{z'}$	<i>corresp.</i> $N_x$
	$\max M_{y'}$	<i>corresp.</i> $M_{z'}$	<i>corresp.</i> $N_x$
	$\min M_{z'}$	<i>corresp.</i> $N_{x'}$	<i>corresp.</i> $M_y$
	$\max M_{z'}$	<i>corresp.</i> $N_{x'}$	<i>corresp.</i> $M_y$
Slabs	$\min m_x -  corresp. m_{xy} ;$	$\max m_x +  corresp. m_{xy} $	
	$\min m_y -  corresp. m_{xy} ;$	$\max m_y +  corresp. m_{xy} $	
	$corresp. m_x \pm  min m_{xy} ;$	$corresp. m_x \pm  max m_{xy} $	
	$corresp. m_y \pm  min m_{xy} ;$	$corresp. m_y \pm  max m_{xy} $	

Plain stress elements	$\min n_x$	-	$ \text{corresp. } n_{xy} $ ;	$\max n_x$	+	$ \text{corresp. } n_{xy} $
	$\min n_y$	-	$ \text{corresp. } n_{xy} $ ;	$\max n_y$	+	$ \text{corresp. } n_{xy} $
	$\text{corresp. } n_x \pm$		$ \min n_{xy} $ ;	$\text{corresp. } n_x \pm$		$ \max n_{xy} $
	$\text{corresp. } n_y \pm$		$ \min n_{xy} $ ;	$\text{corresp. } n_y \pm$		$ \max n_{xy} $
Shells	$\min m_x$	-	$ \text{corresp. } m_{xy} $ ,	$\text{corresp. } n_x \pm$		$ \text{corresp. } n_{xy} $
	$\max m_x$	+	$ \text{corresp. } m_{xy} $ ,	$\text{corresp. } n_x \pm$		$ \text{corresp. } n_{xy} $
	$\min m_y$	-	$ \text{corresp. } m_{xy} $ ,	$\text{corresp. } n_y \pm$		$ \text{corresp. } n_{xy} $
	$\max m_y$	+	$ \text{corresp. } m_{xy} $ ,	$\text{corresp. } n_y \pm$		$ \text{corresp. } n_{xy} $
	$\text{corresp. } m_x \pm$		$ \min m_{xy} $ ,	$\text{corresp. } n_x \pm$		$ \text{corresp. } n_{xy} $
	$\text{corresp. } m_x \pm$		$ \max m_{xy} $ ,	$\text{corresp. } n_x \pm$		$ \text{corresp. } n_{xy} $
	$\text{corresp. } m_y \pm$		$ \min m_{xy} $ ,	$\text{corresp. } n_y \pm$		$ \text{corresp. } n_{xy} $
	$\text{corresp. } m_y \pm$		$ \max m_{xy} $ ,	$\text{corresp. } n_y \pm$		$ \text{corresp. } n_{xy} $
	$\min n_x$	-	$ \text{corresp. } n_{xy} $ ,	$\text{corresp. } m_x \pm$		$ \text{corresp. } m_{xy} $
	$\max n_x$	+	$ \text{corresp. } n_{xy} $ ,	$\text{corresp. } m_x \pm$		$ \text{corresp. } m_{xy} $
	$\min n_y$	-	$ \text{corresp. } n_{xy} $ ,	$\text{corresp. } m_y \pm$		$ \text{corresp. } m_{xy} $
	$\max n_y$	+	$ \text{corresp. } n_{xy} $ ,	$\text{corresp. } m_y \pm$		$ \text{corresp. } m_{xy} $
	$\text{corresp. } n_x \pm$		$ \min n_{xy} $ ,	$\text{corresp. } m_x \pm$		$ \text{corresp. } m_{xy} $
	$\text{corresp. } n_x \pm$		$ \max n_{xy} $ ,	$\text{corresp. } m_x \pm$		$ \text{corresp. } m_{xy} $
	$\text{corresp. } n_y \pm$		$ \min n_{xy} $ ,	$\text{corresp. } m_y \pm$		$ \text{corresp. } m_{xy} $
	$\text{corresp. } n_y \pm$		$ \max n_{xy} $ ,	$\text{corresp. } m_y \pm$		$ \text{corresp. } m_{xy} $

Axisymmetric shells

$\min N_\varphi, \text{corresp. } M_\varphi$ ;	$\max N_\varphi, \text{corresp. } M_\varphi$
$\min M_\varphi, \text{corresp. } N_\varphi$ ;	$\max M_\varphi, \text{corresp. } N_\varphi$
$\min N_\nu, \text{corresp. } M_\nu$ ;	$\max N_\nu, \text{corresp. } M_\nu$
$\min M_\nu, \text{corresp. } N_\nu$ ;	$\max M_\nu, \text{corresp. } N_\nu$

### Shear and Torsion Design

Slabs and shells

$$\sqrt{\min q_x^2 + \text{corresp. } q_y^2}, \quad \sqrt{\max q_x^2 + \text{corresp. } q_y^2}$$

$$\sqrt{\min q_y^2 + \text{corresp. } q_x^2}, \quad \sqrt{\max q_y^2 + \text{corresp. } q_x^2}$$

Beams

$\min Q_y; \max Q_y; \min Q_z; \max Q_z; \min M_x; \max M_x$			
$\min Q_y,$	$\text{corresp. } M_x;$	$\max Q_y,$	$\text{corresp. } M_x$
$\min Q_z,$	$\text{corresp. } M_x;$	$\max Q_z,$	$\text{corresp. } M_x$
$\min M_x,$	$\text{corresp. } Q_y;$	$\max M_x,$	$\text{corresp. } Q_y$
$\min M_x,$	$\text{corresp. } Q_z;$	$\max M_x,$	$\text{corresp. } Q_z$

Axisymmetric shells

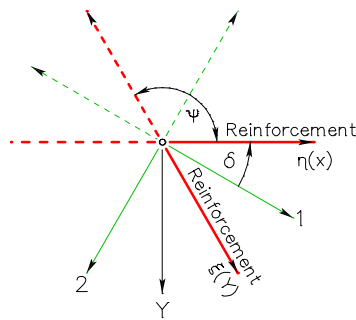
$$\min Q_\varphi, \max Q_\varphi$$

### Design Internal Forces for Oblique Reinforcements

The bending design of slabs with oblique reinforcement assemblies is carried out according to Kuyt or Rüsçh. Here the

design moments are calculated with the help of the principal moments  $m_1$ ,  $m_2$  according to the equations given in Book 166 DAFStB.

For load case combinations, the calculation is based on the extreme values of  $m_1$ ,  $m_2$ . For combined loads (bending and longitudinal force), normal design forces are additionally derived from  $n_1$ ,  $n_2$ . They are then used together as the basis for the design. This should also result in an upper limit for the load.



Coordinate Systems

Extreme values (principal bending moments):

$$m_{1,2} = \frac{1}{2}(m_x + m_y) \pm \frac{1}{2}\sqrt{(m_x - m_y)^2 + 4m_{xy}^2}$$

with  $m_1 \geq m_2$

The angle  $\delta$  assigned to  $m_1$  is:

$$\tan \delta = \frac{2 \cdot m_{xy}}{(m_x - m_y) + \sqrt{(m_x - m_y)^2 + 4 \cdot m_{xy}^2}}$$

**Design moments:**

$$m_\eta = \frac{1}{\sin^2 \psi} \left[ m_1 \sin^2(\delta + \psi) + m_2 \cos^2(\delta + \psi) \pm |m_1 \sin \delta \sin(\delta + \psi) + m_2 \cos \delta \cos(\delta + \psi)| \right]$$

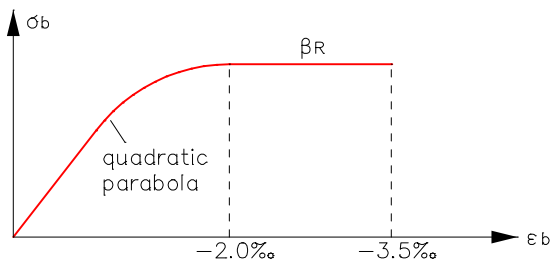
$$m_\xi = \frac{1}{\sin^2 \psi} \left[ m_1 \sin^2 \delta + m_2 \cos^2 \delta \pm |m_1 \sin \delta \sin(\delta + \psi) + m_2 \cos \delta \cos(\delta + \psi)| \right]$$

The formulas apply accordingly for the normal design forces.

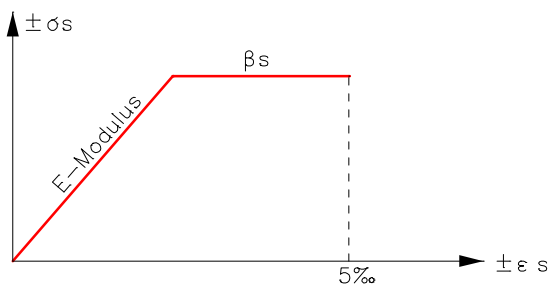
## Basics of Bending Design

### Material Properties

During the design the following stress-strain relationships are used:



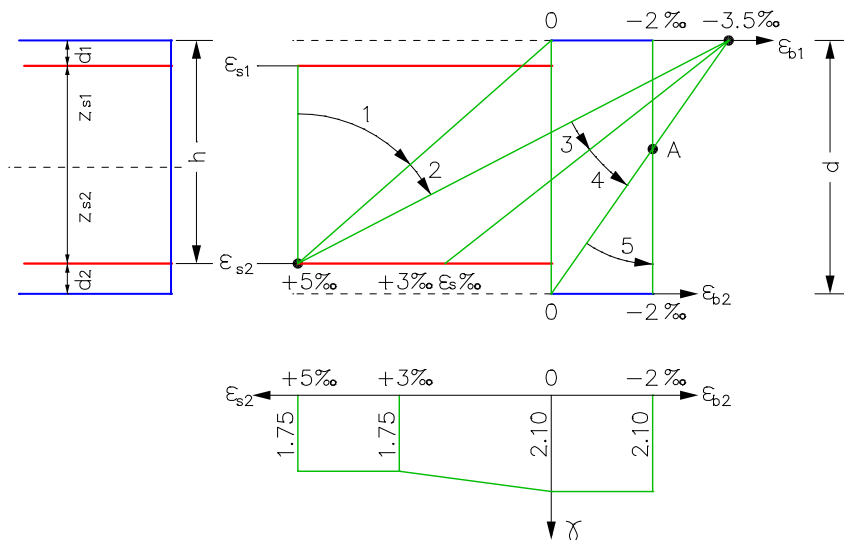
Stress-strain curve for concrete (parabola-rectangle diagram)



Stress-strain curve for reinforcing steel

### Strain areas and safety factors

The following illustration represents the strain areas that form the basis of the design with the corresponding safety factors in the calculated failure state.



Limit strain curves in the failure state and curve of the safety factor

### Equilibrium conditions

The equilibrium state at the reinforced concrete section is described by three equilibrium conditions:

$$\sum \text{forces in } x \text{ direction} = 0$$

$$\sum \text{moments about the } y \text{ axis} = 0$$

$$\sum \text{moments about the } z \text{ axis} = 0$$

The three equations form a nonlinear system of equations of the longitudinal strain, the curvature about the y axis and the

curvature about the z axis, thus completely describing the strain state of the section.

An additional unknown is the size of the necessary reinforced concrete reinforcement. Specifying minimum concrete edge strains in the compressive zone or maximum steel strains in the tensile zone according to DIN 1045 yields additional determinants, which, in conjunction with the equilibrium conditions, make it possible to solve the nonlinear optimization problem.

## Results

The calculated bending and stirrup reinforcement can be graphically represented with the *Results* function.

The log lists the design internal forces, shear stresses, shear areas and required reinforcements for each result location. If a compression reinforcement results primarily from bending, this is marked with a '\*'. The detailed log also lists the internal forces of the load case combination to be checked for each result location.

When representing the stirrup reinforcement, shear stress exceeding is labeled with the value 999. In the colored, numerical output for the area elements, the shear stresses  $\tau$  [MN/m<sup>2</sup>] are shown instead.

### Bending Reinforcement

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.
$a_{s\phi}$	Meridian reinforcement in the $\phi$ direction [cm <sup>2</sup> /m] for axisymmetric shell elements.
$a_{s\upsilon}$	Ring reinforcement in the $\upsilon$ direction [cm <sup>2</sup> /m] for axisymmetric shell elements.

### Stirrup reinforcement from lateral force

$a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area and axisymmetric shell elements.
$A_{sb,y}$	Stirrup reinforcement of beams [cm <sup>2</sup> /m] from $Q_y$ .
$A_{sb,z}$	Stirrup reinforcement of beams [cm <sup>2</sup> /m] from $Q_z$ .

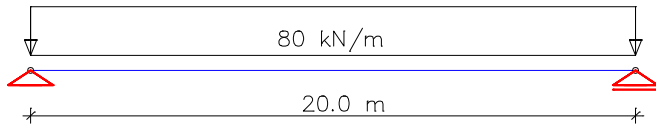
### Torsion reinforcement

$A_{sb}$	Torsional stirrup reinforcement of beams [cm <sup>2</sup> /m] from $M_x$ .
$A_{sl}$	Torsional longitudinal reinforcement of beams [cm <sup>2</sup> ] from $M_x$ .

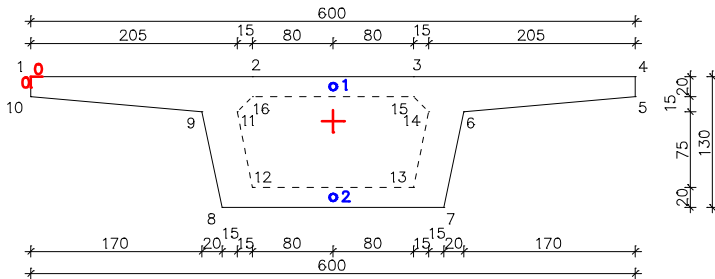
# Examples

## Beam Design

In the following example a bending and shear design is performed on a polygon section made of B25.

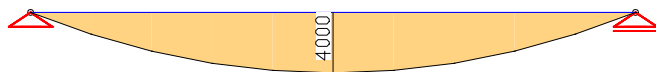


System with line load and dimensions



$Y_s = 3,00000 \text{ m}$        $Z_s = 0,44379 \text{ m}$   
 $I_y = 4,82252e-001 \text{ m}^4$        $I_z = 5,44627e+000 \text{ m}^4$   
 $I_{y'} = 4,82252e-001 \text{ m}^4$        $I_{z'} = 5,44627e+000 \text{ m}^4$   
 $\Phi = -0,00000 \text{ Grad}$        $A = 2,550000 \text{ m}^2$

Section polygon [cm] with a reinforcement edge distance of 10 cm



Bending moment  $M_y$  [kNm]

The design settings are entered as a part of the section description.

### DIN 1045 Design

Mode *Standard*  
 Steel quality of the stirrups *500S*

### DIN 1045 Shear section

Width 0.60 m  
 Height 1.30 m  
 Effective height 1.20 m  
 $k_z$  0.85

### Required beam reinforcement for load case 1

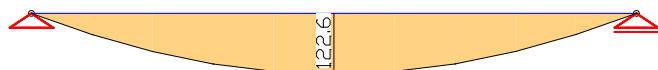
Compression reinforcement is marked in the case of predominant bending through '\*'.

```

Nx, Qy, Qz    Design forces [kN]
Mx, My, Mz    Design moments [kNm]
βWN          Concrete strength [MN/m²]
βR           Concrete strength according to table 12 and/or 0,6 * βWN
As           Longitudinal reinforcement from Nx, My, Mz [cm²]
βs.Bü       Steel strength of the stirrups and torsion reinforcement [MN/m²]
Asb.y       Two-legged stirrups from Qy [cm²/m]
Asb.z       Two-legged stirrups from Qz [cm²/m]
Asb         Torsion links from Mx [cm²/m] (one set)
Asl         Longitudinal reinforcement from Mx [cm²]
tau.0       Shear stress from shear force [MN/m²]
tau.t       Shear stress from torsion [MN/m²]
SB          Shear range according to table 13, DIN 1045
k1,k2       Reduction coefficient for tau.011
            (DIN 1045, chapter 17.5.5.2)
tau.011     Base values of the shear stresses according to line 1a and/or line 1b
            (DIN 1045, Table 13)
Shear coverage
full        Full shear coverage, no design for tau.0 < tau.011
            (DIN 1075, chapter 9.2.3)
Fatigue range
aug. As     Augmented longitudinal reinforcement after design of fatigue range [cm²]
    
```



Element	section	βWN	reinforcement	design mode		
1	No.	1 25.0	No.	1 Standard		
	loc	layer	Nx	My	Mz	As
1	1	1	.	.	.	0.00
1	2	2	.	.	.	0.00
2	1	1	.	.	.	0.00
2	2	2	-0.00	1440.00	0.00	43.21
3	1	1	.	.	.	0.00
3	2	2	-0.00	2560.00	0.00	77.61
4	1	1	.	.	.	0.00
4	2	2	-0.00	3360.00	0.00	102.49
5	1	1	.	.	.	0.00
5	2	2	-0.00	3840.00	0.00	117.54
6	1	1	.	.	.	0.00
6	2	2	-0.00	4000.00	0.00	122.57
7	1	1	.	.	.	0.00
7	2	2	-0.00	3840.00	0.00	117.54
8	1	1	.	.	.	0.00
8	2	2	-0.00	3360.00	0.00	102.49
9	1	1	.	.	.	0.00
9	2	2	-0.00	2560.00	0.00	77.61
10	1	1	.	.	.	0.00
10	2	2	-0.00	1440.00	0.00	43.21
11	1	1	.	.	.	0.00
11	2	2	.	.	.	0.00



Bending reinforcement [cm<sup>2</sup>]

shear section	βWN	βs.Bü	k1	tau.011	shear coverage			
No.	1 25.0	500						
loc	Qy	Asb.y	tau.0	SB	Qz	Asb.z	tau.0	SB
1	0.00	0.00	0.00	1	800.00	19.94	1.31	2
2	0.00	0.00	0.00	1	640.00	12.76	1.05	2
3	0.00	0.00	0.00	1	480.00	7.18	0.78	2
4	0.00	0.00	0.00	1	320.00	4.39	0.52	1
5	0.00	0.00	0.00	1	160.00	2.20	0.26	1
6	0.00	0.00	0.00	1	-0.00	0.00	0.00	1
7	0.00	0.00	0.00	1	-160.00	2.20	0.26	1
8	0.00	0.00	0.00	1	-320.00	4.39	0.52	1
9	0.00	0.00	0.00	1	-480.00	7.18	0.78	2
10	0.00	0.00	0.00	1	-640.00	12.76	1.05	2
11	0.00	0.00	0.00	1	-800.00	19.94	1.31	2

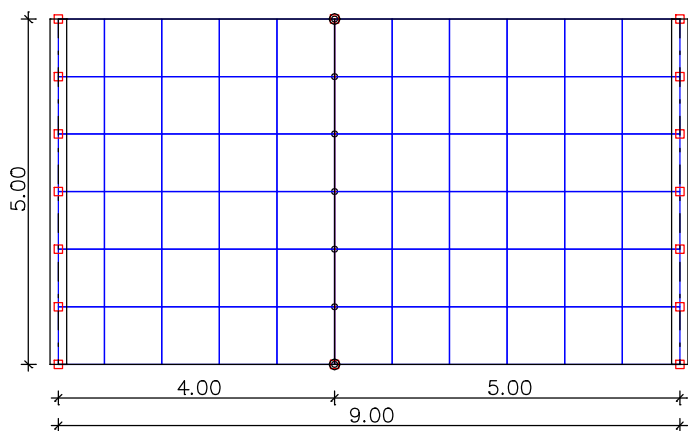


Stirrup reinforcement [cm<sup>2</sup>/m]

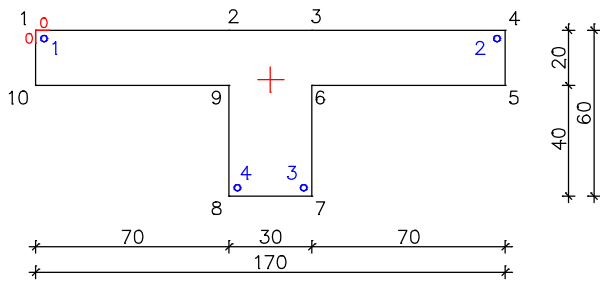
### Slab With Downstand Beam

In this example a rectangular slab (d =20 cm, B25, BSt 500 S) with a downstand beam is analyzed. This joint-supported slab will be subjected to a uniformly distributed load of 15 kN/m<sup>2</sup>.

The necessary design specifications and the stirrup reinforcements calculated for the slab and the downstand beam are shown.



The following image shows the dimensions of the downstand beam. The axis distance of the reinforcing steel from the section edge is 3 cm.



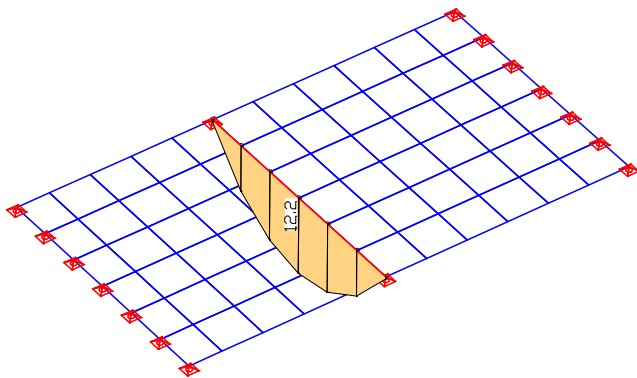
Design specifications and reinforcing steel description of the slab (section 1):

- Edge distance of the reinforcing steel for the x and y direction of the upper (1st) and lower (2nd) layer: 0.03 m
- Bending design mode: Standard
- Steel quality: 500S
- Effective height: 0.17 m
- Coefficient  $k_z$  for calculating the lever arm  $z$  of the inner forces: 0.85
- Reduction factor for  $\tau_{011}$ :  $k_1$
- Limits of the base values of the shear stress  $\tau_0$ , Table 13: Row 1a

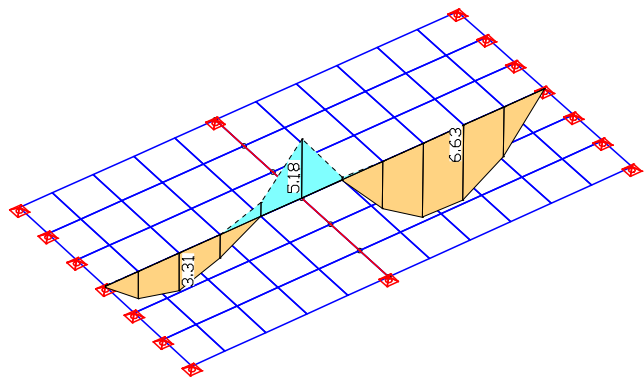
Design specifications of the torsion-flexible downstand beam (section 2):

- Bending design mode: *Standard*
- Steel quality of the stirrups: 500S
- Shear section:  
Width: 0.30 m  
Height  $d_z$ : 0.60 m  
Effective height: 0.57 m  
 $k_z$ : 0.85

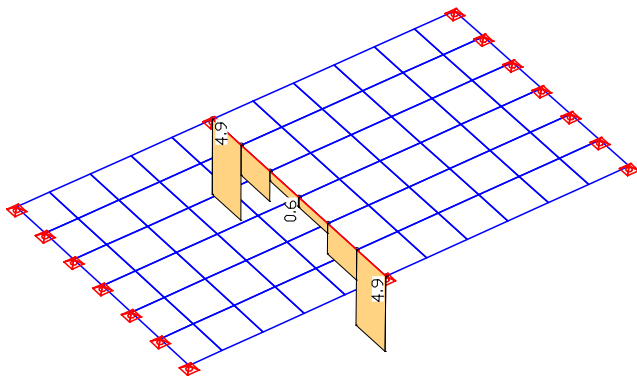
The calculated reinforcements are shown in the illustrations below.



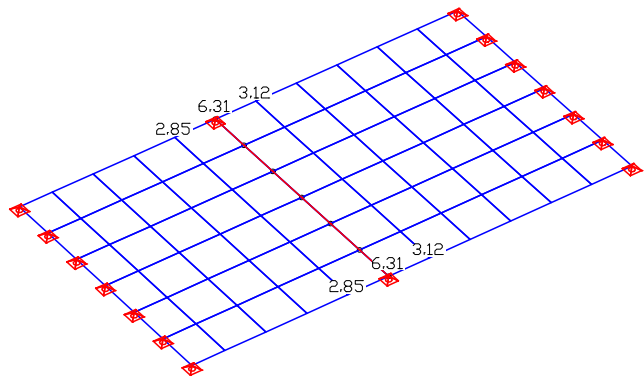
Bending reinforcement of the beams [cm²]



Slab reinforcement in intersection direction [cm²/m]



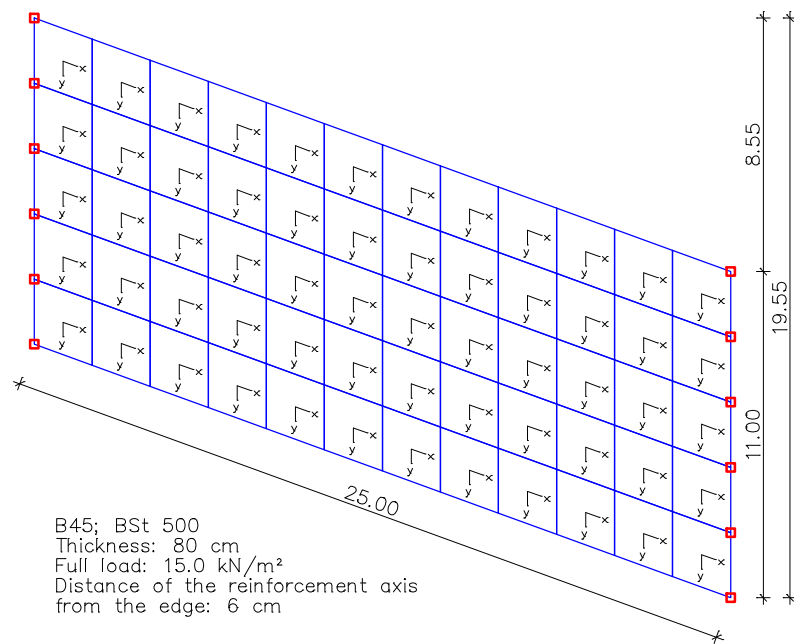
Stirrup reinforcement of the beams [cm²/m]



Averaged stirrup reinforcement of the slab at the nodes [cm²/m²]

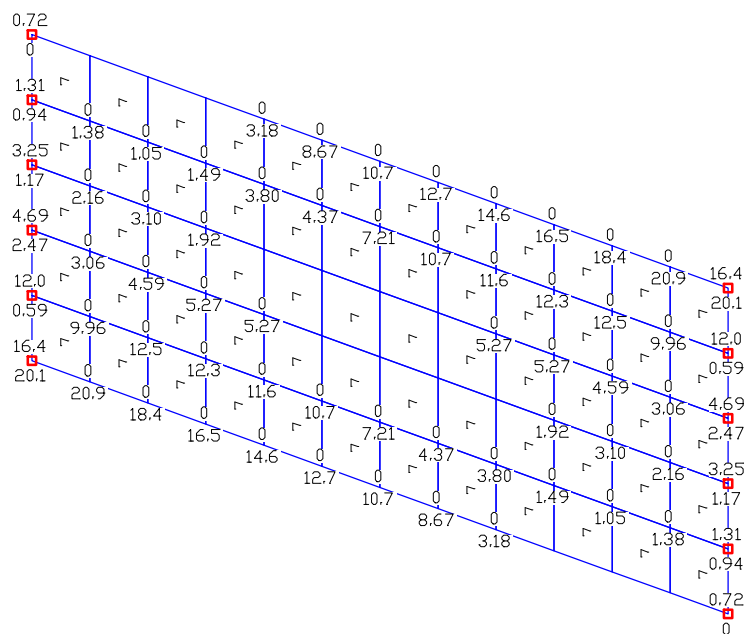
## Slab With Oblique Reinforcement

In this example the procedure for designing a flexibly supported slab with oblique reinforcement is illustrated.

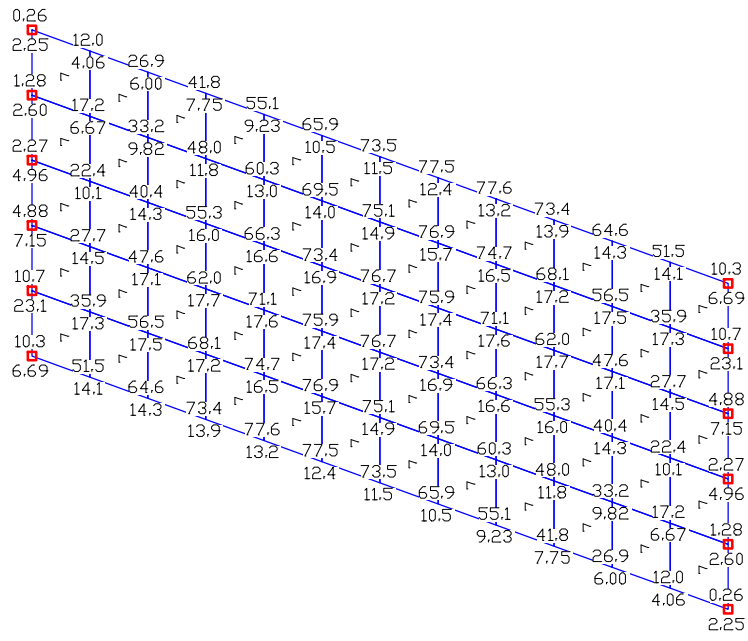


To align the reinforcement directions, first the x axis of the local internal force system is rotated according to the oblique slab edge, then the reinforcement assembly  $a_{sy}$  is oriented parallel to the support line. This results in the reinforcement systems shown above.

If oblique reinforcement systems exist, the bending design is automatically performed according to Rüsçh.



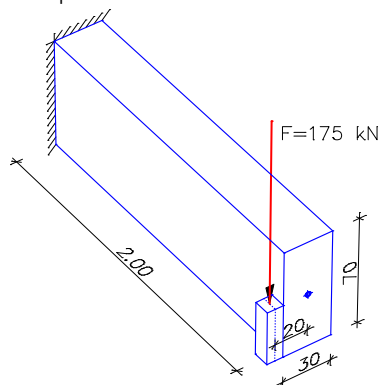
Upper reinforcement  $a_{sx}/a_{sy}$  [cm<sup>2</sup>/m]



Lower reinforcement  $a_{sx}/a_{sy}$  [cm<sup>2</sup>/m]

## Torsional Beam

The depicted cantilever is subjected to an eccentrically acting load  $F = 175 \text{ kN}$ . The required shear, torsion longitudinal and stirrup reinforcements are listed in the following log.



System drawing

### Concrete Design Per DIN 1045

#### Cross section geometry and beam reinforcement

Sec. point	concrete	reinforcement						
		y [m]	z [m]	Es	βs [MN/m <sup>2</sup> ]	y [m]	z [m]	As [cm <sup>2</sup> ]
1	1	0.000	0.000	210000.0	500.0	0.055	0.055	0.00
	2	0.300	0.000	210000.0	500.0	0.245	0.055	0.00
	3	0.300	0.700	210000.0	500.0	0.245	0.645	0.00
	4	0.000	0.700	210000.0	500.0	0.055	0.645	0.00

#### Shear cross sections

No.	width	effect. width	cooresp. kz	height	effect. height	corresp. kz	core section width	core section height
	[m]	[m]		[m]	[m]		[m]	[m]
1	0.30	0.25	0.850	0.70	0.65	0.850	0.19	0.59

#### Required beam reinforcement for load case 1

Compression reinforcement is marked in the case of predominant bending through ''.

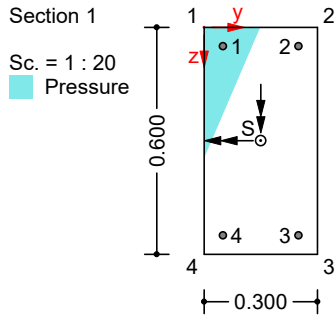
Nx, Qy, Qz Design forces [kN]  
 Mx, My, Mz Design moments [kNm]  
 βWN Concrete strength [MN/m<sup>2</sup>]  
 βR Concrete strength according to table 12 and/or 0,6 \* βWN  
 As Longitudinal reinforcement from Nx, My, Mz [cm<sup>2</sup>]  
 βs.Bü Steel strength of the stirrups and torsion reinforcement [MN/m<sup>2</sup>]  
 Asb.y Two-legged stirrups from Qy [cm<sup>2</sup>/m]  
 Asb.z Two-legged stirrups from Qz [cm<sup>2</sup>/m]  
 Asb Torsion links from Mx [cm<sup>2</sup>/m] (one set)  
 Asl Longitudinal reinforcement from Mx [cm<sup>2</sup>]  
 tau.0 Shear stress from shear force [MN/m<sup>2</sup>]  
 tau.t Shear stress from torsion [MN/m<sup>2</sup>]  
 SB Shear range according to table 13, DIN 1045  
 k1,k2 Reduction coefficient for tau.011 (DIN 1045, chapter 17.5.5.2)  
 tau.011 Base values of the shear stresses according to line 1a and/or line 1b (DIN 1045, Table 13)  
 Shear coverage  
 full Full shear coverage, no design for tau.0 < tau.011 (DIN 1075, chapter 9.2.3)  
 Fatigue range Fatigue range after reinforcement increment [MN/m<sup>2</sup>]  
 aug. As Augmented longitudinal reinforcement after design of fatigue range [cm<sup>2</sup>]

Element	shear	section	βWN	βs.Bü	k1	tau.011	shear coverage
1	No.		1 35.0	500			
	loc	Qy	Asb.y	tau.0	SB	Qz	Asb.z tau.0 SB
	1	0.00	0.00	0.00	1	175.00	4.95 1.06 2
	2	0.00	0.00	0.00	1	175.00	4.95 1.06 2
	loc	Mx	Qy	Qz	tau.t	Asb	Asl
	1	35.00	0.00	175.00	2.15	5.46	8.52
	2	35.00	0.00	175.00	2.15	5.46	8.52

# Single Design

A single rectangular section is designed under bending and normal force. Internal forces without safety factors are applied for the design.

## Pos. 1 - Reinforced concrete design per DIN 1045



Action  $N = 10.00 \text{ kN}; M_y = 50.00; M_z = 20.00 \text{ kNm}$   
 Resistance  $N = 10.00 \text{ kN}; M_y = 50.00; M_z = 20.00 \text{ kNm}$   
 Force system  $y_s / z_s = 0.150 / 0.300 \text{ m}$   
 Strength  $\beta_R = 17.50 \text{ MPa}; \gamma = 1.75$   
 Design mode Standard  
 Reinforcement  $5.68 \text{ cm}^2; 0.32 \%; \text{Concrete area} = 1800.00 \text{ cm}^2$

Concrete section Point	y [m]	z [m]	eps[%]	sigma[MPa]	Inner Forces	y [m]	z [m]	F [kN]
1	0.000	0.000	-2.19	-17.50	Compr.	0.040	0.093	-135.62
	0.148	0.000	0.00	0.00	Tension	0.185	0.450	145.63
2	0.300	0.000	2.25	0.00	Lev. arm	0.145	0.358	
3	0.300	0.600	6.06	0.00				
4	0.000	0.600	1.62	0.00				
	0.000	0.345	0.00	0.00				

Reinforcement Point	y [m]	z [m]	d1 [m]	Es, $\beta_s$ [MPa]	Zv0 [kN]	eps[%]	sigma[MPa]	As [cm <sup>2</sup> ]
1	0.050	0.050	0.050	210000 500	0.0	-1.13	-238.29	0.00
2	0.250	0.050	0.050	210000 500	0.0	1.82	383.07	1.33
3	0.250	0.550	0.050	210000 500	0.0	5.00	500.00	2.42
4	0.050	0.550	0.050	210000 500	0.0	2.04	428.64	1.94

## References

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# DIN 1045-1 Design

## Basics

The reinforced concrete and prestressed concrete design according to DIN 1045-1 can be used for all engineering structures that need not be checked according to the guidelines of the DIN Technical Report 102. In the calculation settings you can select which version of the standard will be used:

- DIN 1045-1:2001-07 with revision A2:2005-06
- DIN 1045-1:2008-08

Permitted structure models include beam, area and solid structures. Prestressed structures can only be checked in the FEM module.

Differing components can be combined in a structure model:

- Non-prestressed components
- Prestressed components with subsequent bond
- Prestressed components without bond
- Components with external prestressing
- Mixed-construction components

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with DIN 1055-100. The program will take into account the preset safety factors and combination coefficients for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements.

The actions and check selection dialogs can be opened from the analysis settings. Detailed check specifications and reinforcement data must be entered during section definition.

The checks are limited to elements with materials *C12/15* to *C100/115*, *LC12/13* to *LC60/66*. and the user-defined material *CX*. For strength classes *C55/67* and *LC55/60* or higher, guideline 5.3.3 (9) of the standard applies.

For beams and design objects, all checks are carried out at the polygon section. For general notes on using design objects, refer to the relevant chapter of the manual.

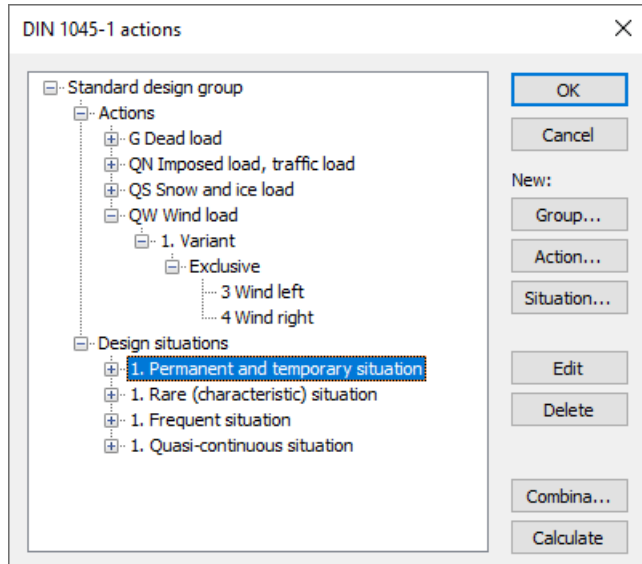
In the *DIN 1045-1 Design* folder of the database you can also perform a single design for user-defined polygon sections or composite sections.

# Input

## Actions and Design Situations

The load design values are calculated based on the internal forces of individual load cases and load case combinations. To do so, the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Prestressing (P)
- Creep and shrinkage, relaxation (CSR1, CSR2)  
These actions are only available if a P action has been defined. In the combinations they are treated, along with P, as a single action.
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)  
The assigned load cases should contain a design-relevant set of loads with partial safety factors and combination coefficients such as for example a load group to take into account nonlinear effects. The selected load cases are combined exclusively.
- Cyclic fatigue actions (Qfat)

### Group...

Open the dialog for entering a new design group. According to e.g. standard DIN 1055-3, Chapter 6.3 (4), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

### Situation...

Open the dialog for entering new design situations. Situations must be classified as either a construction stage or a final state in order to control the checking process. For prestressed concrete structures with subsequent bond, you can specify that the tendons are still ungrouted.

### Edit

Open the Edit dialog for the selected action or situation.

### Delete

Delete the selected action or situation.



### Combinations...

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

### Calculate

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to execute the checking module. Each time you execute the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

The following table demonstrates how the situations are used in the various checks. The numbers refer to the DIN 1045-1 chapters.

Situation	Ultimate limit state	Chapter	Serviceability limit state	Chapter
Permanent and temp.	Longitudinal reinf.	10.2		
Accidental	Lateral reinf.	10.3		
Earthquake	Torsional reinf.	10.4		
Characteristic (rare)	Robustness reinf. (following DIN TR 102, 4.3.1.3)	5.3.2	Concrete compr. stress Reinforcing steel stress Prestressing steel stress Decompression Class A Crack reinforcement Crack width Class B	11.1.2 11.1.3 11.1.4 11.2.1 11.2.2 11.2.4
Frequent	Fatigue, simplified	10.8.4	Decompression Class B Crack width Class C, D	11.2.1 11.2.4
Quasi-continuous			Concrete compr. stress Prestressing steel stress Decompression Class C Crack width Class E, F Deformations	11.1.2 11.1.4 11.2.1 11.2.4 11.3
Fatigue	Fatigue reinf. steel Fatigue prestr. steel Fatigue concrete	10.8.3 10.8.3 10.8.3		

## Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.


**Label**

User-defined label for the action.


**Gamma.sup, Gamma.inf**

Partial safety factors  $\gamma_{\text{sup}}$  and  $\gamma_{\text{inf}}$ .

**Combination coefficients psi for:**

Input fields for selecting the combination coefficients for variable actions. The  button allows you to view and change the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .



**Load cases**

List of possible load cases or load case combinations. You can choose an item from the list by selecting it and clicking the  button or by using drag & drop.

**Multi-select**

Load cases and combinations can be added to the actions more than once.

**Exclusive variants**

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the  or  buttons.

**Inclusive load cases**

Selected load cases and combinations that can act simultaneously.

**Exclusive load cases**

Selected load cases and combinations that exclude each other.

**Prestressing loss from relaxation of prestressing steel**

The prestressing loss is defined as a constant percentage reduction of prestress.

**CS as constant reduction of prestress**

As an alternative to defining load cases, you can allow for the effect of creep and shrinkage (CS) by defining a constant percentage reduction of prestress.

**Internal prestressing**

Selected load cases that describe internal prestressing. The reactions of the individual load cases are added up.

**External prestressing**

Selected load cases that describe external prestressing. The reactions of the individual load cases are added up.

## Partial Safety Factors

The partial safety factors of the construction materials are preset with the values specified by DIN 1045-1, Table 1, and can be modified if necessary. In design situations resulting from earthquakes, the factors of the permanent and temporary design situation apply as specified in DIN 4149:2005-04, Chapter 8.1.3 (3). In version 6.12 or higher, the partial safety factors for actions are determined by the definition of actions in accordance with Table 2 of the standard. These factors appear in the Partial Safety Factors dialog for compatibility reasons only and therefore cannot be modified.

## Section Input

The section inputs contain all of the specific settings made for checks in the ultimate limit and serviceability states. In addition to these specifications, the selected material properties and the properties of the reinforcing steel are also relevant for the design. An overview of the design specifications can be accessed in the *DIN 1045-1 Design* section of the database.

## Checks

The following dialog is used to define which ultimate limit state and serviceability checks are available for the section. The analysis settings allow to override this selection for the entire structure.

**Properties for element 12 - DIN 1045-1 - Checks**

Section: Form, Shear stresses, Material, Bedding, DIN 1045-1 (Checks, Base values, Shear section, Concrete stress, Crack width, Fatigue, Variation coefficients), DIN TR 102, Recalculation Guideline, Thermal analysis, General, Eccentricity

Number: 1 - Roc, Section Type: Polygon, Material Type: C45/55

Label: Roof girder

Prestress of component: Subsequent bond, Requir. class: D

Ult. limit state

- Bend and longitudinal force
- Lateral force
- Fatigue for concrete
- Robustness
- Torsion
- Fatigue reinforcing, prestressed steel

Serviceab. limit state

- Concrete compressive stress
- Steel tensile stress
- Minimum crack reinforcement
- Decompression
- Calculation of the crack width

Buttons: OK, Cancel, Help

### Prestress of component

The type of prestressing can be selected for each section separately:

- *not prestressed*
- *subsequent bond*
- *without bond*
- *external*
- *mixed construction*

### Requirement class

The check conditions for the decompression and crack width check are defined in DIN 1045-1, Chapter 11.2.1, Table 18, based on the requirement classes A through F. The minimum requirement class is derived from Table 19 depending on the exposure class of the structure and the prestress type of the component.

### Robustness

This check determines the minimum reinforcement for ensuring ductile component behavior according to DIN 1045-1, Chapter 5.3.2 (also referred to as robustness reinforcement in Book 525).

## Base Values

The base values apply for all checks in the ultimate limit and serviceability states.

### Design mode

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined ratios between the reinforcement layers will not be affected.
- *Compression member*: For compression members, a symmetrical design is carried out with allowance for the minimum reinforcement according to DIN 1045-1, Chapter 13.5.2.

### Alternative concrete

This value is necessary to perform a design according to the standard if the material type *Beton* is selected.

### Reduction factor of prestr. for robustness

In the program the regulations of the DIN Technical Report 102, Section 4.3.1.3 (108) are decisive for the arrangement of the robustness reinforcement. Thus for the determination of the tensile zone the statically determined effect of prestressing is not taken into account. Because this cannot be determined for area elements the prestress can alternatively be reduced by a reduction factor. The specification of an appropriate value is subject to the discretion of the user.

### Effective height

Effective static height for the shear design of area elements [m].

### cot Theta, Method

$\cot \Theta$  defines the concrete strut angle according to DIN 1045-1, Chapter 10.3.4 (3). The program will suggest a value of 1 (45° strut angle). You can choose to ignore the suggestion and pick any value between 0.58 and 3 (normal concrete) or 2 (lightweight concrete). Entering a higher number will normally result in a lower necessary lateral force reinforcement  $A_{sw}$ , a lower absorbable lateral force  $V_{Rd,max}$  and a larger displacement  $a_1$  according to Eq. (147). Three calculation methods can be chosen for the check:

- *Standard*: The input value is limited to the range permitted in accordance with DIN 1045-1, Eq. (73) for lateral force, torsion and combined loads (method with load-dependent strut angle).
- *Constant*: The check is carried out using the chosen value for  $\cot \Theta$  without further limitations (cf. interpretation No. 24 of NABau).
- *Std./45°*: For lateral force  $\cot \Theta$  is limited according to Eq. (73), for torsion a constant strut angle of 45° is assumed for simplification according to Chapter 10.4.2 (2).

The actual effective concrete strut angle is logged for each check location.

## Bending reinforcement Asl according to picture 32

The bending reinforcement to be added according to DIN 1045-1, Chapter 10.3.3, Figure 32 [cm<sup>2</sup>].

## Automatic extension of Asl to

You can optionally specify a maximum value for area elements and the program will automatically increase the above input value until that maximum value is reached in order to avoid stirrup reinforcement [cm<sup>2</sup>].

## Quality of stirrups

- 420S: Reinforcing rod with  $f_{yk} = 420$  MN/m<sup>2</sup>.
- 500A: Reinforcing rod with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- 500M: Reinforcing meshes with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- General: Freely definable steel quality [MN/m<sup>2</sup>].

## Factor for min rhow

The minimum reinforcement level  $\min \rho_w$  complies with DIN 1045-1, Chapter 13.2.3 (5), and is defined using a factor related to the base values  $\rho$  according to Tab. 29. The program will suggest a factor of 1 for beams and design objects and a factor of 0.6 for area elements as per 13.3.3 (2). The factor can be any number between 0 and 1.6, which is the nominal value for structured sections with prestressed tension chord.

## Design like slabs

Beams or design objects are treated like slabs, which means that a minimum lateral force reinforcement will not be determined as per 13.3.3 (2) if no lateral force reinforcement is required for computation.

## Laying measure $c_{v,1}$

In DIN 1045-1:2008-08, Chapter 10.3.4 (2), and NABau No. 24, the internal lever arm  $z$  is limited to the maximum value derived from  $z = d - c_{v,1} - 30$  mm and  $z = d - 2c_{v,1}$ . Note that  $c_{v,1}$  is the laying measure of the longitudinal reinforcement in the concrete compressive zone. For  $c_{v,1}$  the program will suggest the shortest axis distance of the longitudinal reinforcement from the section edge  $d_1$ .

## Separate check for x and y direction

For two-axes stressed slabs, the lateral force check can be performed separately in the x and y stress directions as described in DIN 1045-1:2008-08, Chapter 10.3.1 (5), and NABau No. 76. The user is responsible for properly aligning the reinforcement directions.

## Shear Section

For polygon sections, additional section dimensions are required for the lateral force and torsion design according to DIN 1045-1. These dimensions are explained in the following. In case of sections with internal prestressing or with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.

Properties for element 12 - DIN 1045-1 - Shear section

- [-] Section
  - Form
  - Shear stresses
  - [+] Material
  - Bedding
  - [-] DIN 1045-1
    - Checks
    - Base values
    - Shear section
    - Concrete stress
    - Crack width
    - Fatigue
    - Variation coefficients
  - [+] DIN TR 102
  - [+] Recalculation Guideline
    - Thermal analysis
  - General
  - Eccentricity

Number: 1 - Roc    Section Type: Polygon    Material Type: C45/55   

Label: Roof girder    Properties concern further elements.

Height [m]    Nom. height

2.3    2.22

Eff. width [m]

0.45

Width [m]    Nom. width [m]

0.5    0.38

teff [m]

0.1     Box section

Core section Ak= z1 \* z2

z1 [m]    z2 [m]

2.2    0.4

**Width**

Section width for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

**Height**

Section height for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

**Effective height**

Effective static height for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

**Effective width**

Effective static width for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

**Nom. width, nom. height**

The nominal width or height of internally prestressed components as per DIN 1045-1, Chapter 10.3.4 (8) for including the duct diameter in the calculation of the design value of the lateral load-bearing capacity  $V_{Rd,max}$ .

**Core section  $A_k = z_1 * z_2$** 

Dimensions of the core section for calculating the torsion reinforcement [m].

**teff**

The effective wall thickness of the torsion section according to DIN 1045-1, Figure 36 [m].

**Box section**

Data for determining the factor  $\alpha_{c,red}$  in DIN 1045-1, Eq. (93) and the torsion section utilization according to Eq. (94) or (95).

**Concrete Stress**
**Concrete compressive stress**

The concrete compressive stress  $\sigma_c$  must be limited to  $0.60 f_{ck}$  under the rare (characteristic) combination in the construction stages and final states according to DIN 1045-1, Chapter 11.1.2 (1). This condition is normally checked by the program. If serviceability is significantly influenced by creep, the limit  $0.45 f_{ck}$  should be maintained under the quasi-continuous combination according to 11.1.2 (2).

**Decompression**

Decisive stress for the decompression check for area elements ( $\sigma_1, \sigma_x, \sigma_y$ ).

## Crack Width

These specifications apply to the minimum crack reinforcement calculation and the crack width check.

Properties for element 12 - DIN 1045-1 - Crack width

Section: Form, Shear stresses, Material, Bedding, DIN 1045-1 (Checks, Base values, Shear section, Concrete stress, Crack width, Fatigue, Variation coefficients), DIN TR 102, Recalculation Guideline, Thermal analysis, General, Eccentricity

Number: 1 - Roc, Section Type: Polygon, Material Type: C45/55, New, Copy, Delete

Label: Roof girder, Properties concern further elements.

Crack width  $w_{k,per}$  [mm]: 0.2, Reinf. steel max.  $d_s$  [mm]: 20, Prestr. steel coeff.  $\xi_1$ : 0.27

Minimum reinforcement: Determ. of the tensile zone: Comb. acc. to class, Concr. age: 3 - 5 d, Coeff.  $k$ : 1, Thick comp. as per 11.2.2(8) [unchecked]

Crack width limitation: Check method: Direct calculation, Concr. age: 3 - 5 d, Limit for  $s_{r,max}$  [mm] [checked], max.  $s$  [mm]:

OK, Cancel, Help

### $w_{k,per}$

Calculation value of the crack width according to DIN 1045-1, Chapter 11.2.1, Table 18 [mm]. The program will suggest a tabular value based on the selected requirement class and the prestressing of the component. This value can be modified after the input field is enabled.

### max. $d_s$

Largest existing bar diameter of the reinforcing steel reinforcement according to 11.2.2 (6), Eq. (131) [mm].

### Coefficient $\xi_1$

The bond coefficient  $\xi_1$  according to DIN 1045-1, Eq. (130) defines the extend to which prestressing steel as per 11.2.2 (7) can be taken into account for the minimum crack reinforcement. It is also used in the calculation of the effective reinforcement level according to Eq. (133) and thus the direct calculation of the crack width. Data input is blocked for area elements since prestressing steel is normally not taken into account here.

### Determ. of the tensile zone

You can specify the tensile zone where the minimum crack reinforcement described in Chapter 11.2.2 will be placed by selecting either an action combination (AC) or a restraint (bending, central tension).

### Thick component

Based on Chapter 11.2.2 (8) of edition 2008 the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined to Equation (130a). Therewith a reduction compared to the calculation with Equation (127) can be achieved.

### Coefficient $k$

Coefficient used to take into account nonlinearly distributed concrete tensile stresses in the section according to 11.2.2 (5).

### Concrete age

The age of the concrete is used to determine the effective concrete tensile strength  $f_{ct,eff}$  as per 11.2.2 (5). This is done separately for the minimum reinforcement calculation and the crack width calculation.

### Check method

The crack width check can be performed either through direct calculation of the standard as described in Chapter 11.2.4 or by simply limiting the bar distances according to the information provided in Table 21. According to *Zilch and Rogge* (2002, p. 277), the simplified method only yields definitive results for single-layer tensile reinforcement with  $d_1 = 4$  cm. The user is responsible for the evaluation of these requirements.

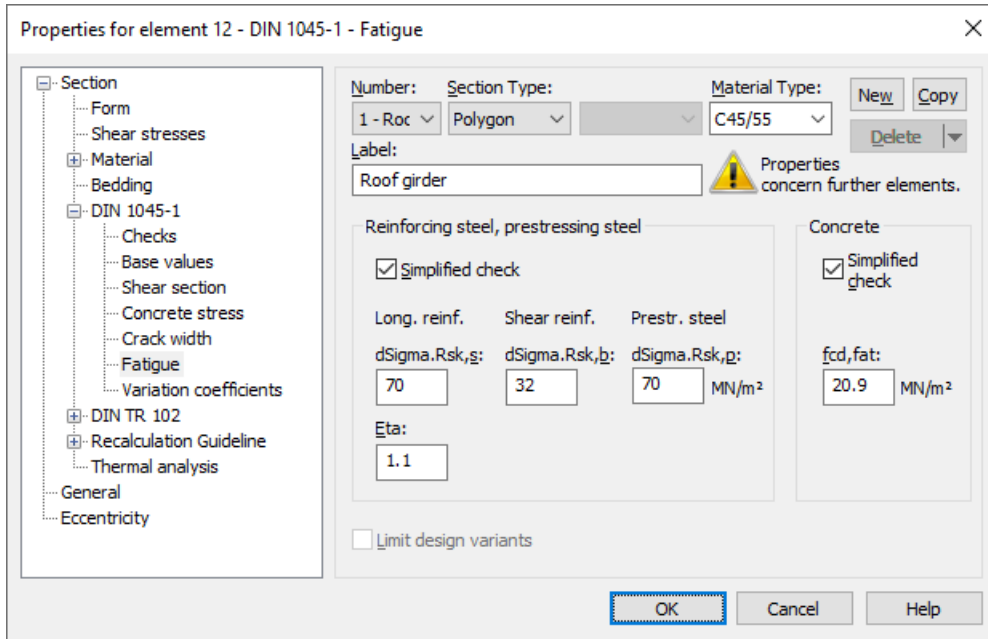
**sr,max**

When calculating the crack width, by default the crack distance is determined using Equation (137) of the standard. Alternatively, you can preset an upper limit for  $s_{r,max}$  [mm] so that, for example, the special conditions of Equation (138) or Paragraph (8) of Chapter 11.2.4 are taken into account.

**max. s**

Largest existing bar distance of the reinforcing steel reinforcement for the simplified crack width check [mm].

**Fatigue**



**dSigma.Rsk,s, dSigma.Rsk,b**

The permissible characteristic stress range  $\Delta\sigma_{Rsk}(N^*)$  of the longitudinal reinforcement and shear reinforcement at  $N^*$  load cycles according to the Wöhler curves specified in Chapter 10.8.3 [MN/m²]. The value found in Table 16, Row 1 (beam sections) resp. Row 2 (area sections, edition 2008-08), is suggested in the dialog. For the shear reinforcement, the mandrel diameter is taken to be  $d_{br} = 4 d_s$ .

**dSigma.Rsk,p**

The permissible characteristic stress range  $\Delta\sigma_{Rsk}(N^*)$  of the prestressing steel at  $N^*$  load cycles according to the Wöhler curves specified in Chapter 10.8.3 [MN/m²]. The value found in Table 17, Row 4, is suggested in the dialog.

**Eta**

Increase factor  $\eta$  for the reinforcing steel stress of the longitudinal reinforcement. This factor is used to take into account the varying bonding behavior of concrete and prestressing steel as per Chapter 10.8.2 (3), Eq. (118).

**fcd,fat**

Concrete compressive strength before onset of cyclic load according to DIN 1045-1, Chapter 10.8.4, Eq. (124) [MN/m²]. In general, the following applies:

$$f_{cd,fat} = \beta_{cc}(t_0) \cdot f_{cd} \cdot \left(1 - \frac{f_{ck}}{250}\right) \tag{124}$$

with

$$\beta_{cc}(t_0) = e^{0.2(1 - \sqrt{28/t_0})}$$

$t_0$  Time of the initial stressing of the concrete.

$f_{cd,fat}$  for  $t_0 = 28$  and  $f_{cd} = 0.85 \cdot f_{ck} / \gamma_{c,fat}$  is suggested in the dialog.



### Simplified check

The simplified check according to Chapter 10.8.4 bases on the frequent action combination including the traffic loads used for the serviceability checks. The method for concrete is defined in Chapter 10.8.4 (4), the permissible stress ranges for steel are suggested according to Chapter 10.8.4 (2) in the dialog. For shear reinforcement this value is reduced analogous to Table 16.

### Limit design variants

For area elements, the variants for determining the stress range can be limited to the corresponding sets of design internal forces. For more information see chapter '*Check Against Fatigue > Special Characteristic of Shell Structures*'.

## Variation Coefficients

Properties for element 12 - DIN 1045-1 - Variation coefficients

Section: Form, Shear stresses, Material, Bedding, DIN 1045-1 (Checks, Base values, Shear section, Concrete stress, Crack width, Fatigue, Variation coefficients), DIN TR 102, Recalculation Guideline, Thermal analysis, General, Eccentricity

Number: 1 - Roc, Section Type: Polygon, Material Type: C45/55

Label: Roof girder

Variation coefficients for the internal prestressing in the proofs of decompression, crack reinforcement and crack width.

	r.sup	r.inf
Construction stage:	1	1
Final state:	1.1	0.9

Properties concern further elements.

OK, Cancel, Help

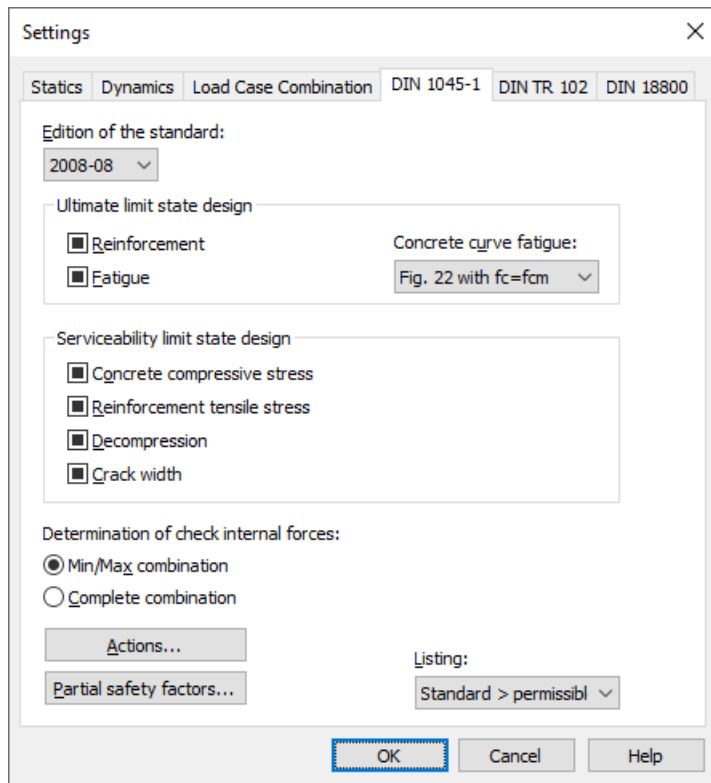
The coefficients used to take into account the variation of prestressing force are defined in DIN 1045-1 depending on the prestressing type. In the dialog, values are suggested according to Chapter 8.7.4 (2) for subsequent bond. Lower variation levels can be specified in the construction stage as shown in Book 525. In the actions, design situations are declared as construction stage or final state. The defined variation coefficients are taken into account for the effects from internal prestressing in the following checks:

- Decompression check
- Minimum reinforcement for crack width limitation
- Crack width check

Regarding the effects from external prestressing, the variation coefficients correspond to  $r_{sup} = r_{inf} = 1$  on the basis of DIN Technical Report 102, Chapter 2.5.4.2 (4).

## Analysis Settings

The *DIN 1045-1* dialog page can be opened using the *Settings* function of the *Analysis* menu.



### Edition of the standard

The edition you select will be used for all subsequent calculations.

### Check selection

When selecting checks, the following cases are to be distinguished:

- or  The check is performed according to the settings in the section dialog (see *Section inputs*).
- or  The check is performed for all sections of the structure.
- or  The check is performed for no sections of the structure.

Corresponding section settings are bundled as follows:

Reinforcement    Bend and longitudinal force, Lateral force, Torsion, Robustness  
Crack width      Minimum crack reinforcement, Calculation of the crack width

An overview of the checks can be accessed using the *Design Settings* function in the *DIN 1045-1 Design* folder of the database.

### Concrete curve fatigue

For the fatigue checks, the curve to determine the concrete compressive stresses can be selected.

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *EN 1992-1-1 Design > Check internal forces*.

### Actions...

Open the dialog for describing actions.

**Partial safety factors...**

Open the dialog for modifying partial safety factors.

**Listing**

- *No*: No log is generated by the checking program.
- *Standard*: Log with tabular output of results.
- *Detailed*: Additional output of the decisive combination internal forces at the check locations.
- *Standard > permissible*: Standard log limited to check locations where the permissible limit values are exceeded.
- *Detailed > permissible*: Detailed log limited to check locations where the permissible limit values are exceeded.

## Single Design

The single design function allows you to analyze individual sections independently of the global system using predefined internal forces. The calculation is carried out from the opened input table via the *Single Design* item in the *Analysis* menu or the *Print Preview* function.

The following data needs to be entered in the *Single Design* table, which is located in the *DIN 1045-1 Design* folder of the database.

**Section**

Number of the section to be designed. Both polygon and composite sections can be designed.

**Concrete**

Concrete class *C12/15*, ... *C100/115* or *LC12/13*, ... *LC60/66*

**Apparent density**

Apparent density of the lightweight concrete [kg/m<sup>3</sup>].

**Combination**

Design situation according to DIN 1045-1, Table 2:

- *0*: Permanent and temporary design situation.
- *1*: Accidental design situation.

**Nsd, Mysd, Mzsd**

Internal forces being designed. The internal forces refer to the centroid in polygon sections or the section zero point in composite sections.

**Mode**

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members a symmetrical design is carried out with allowance for the minimum reinforcement according to DIN 1045-1, Chapter 13.5.2.
- *Strains*: Determine strain state for existing reinforcing steel layers.
- *Strains SLS*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A linear strain-stress curve of the concrete is used in the compression zone to determine the strain state.
- *Strains SLS2*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A nonlinear strain-stress curve of the concrete is used as shown in Figure 22. Note that a horizontal progression is assumed for strains exceeding  $\epsilon_{c1}$ .
- *Load bearing capacity*: Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
- *Maximum bending moment My*: Determination of the maximum bearable bending moment  $M_y$ . The moment  $M_y$  is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.
- *Inactive*: Design disabled.

## Punching Shear Check

When you select a check node, the key data for the checks is displayed in a dialog field. This dialog is divided into three pages.

### 1a. Input data, column

The column forms *Rectangle* and *Round*, with the locations *Intern*, *Edge parallel to x*, *Edge parallel to y* and *Corner* are available. When you enter a new column, the program will suggest the dimensions of existing columns. The edge distances  $a_x$  and  $a_y$  are used to calculate the perimeters  $u_i$  of the check sections. Alternatively the check locations *Wall end* and *Wall corner* can be chosen.

### 1b. Input data, slab

This section shows the material properties, the existing reinforcement as well as additional coefficients for calculating punching shear resistances.

### 1c. Input data, action

The action  $V_{Ed}$  can either be added as a support force from a previous design according to DIN 1045-1 or defined directly. All medium soil pressures  $\sigma_0$  lower the design value of the lateral force by  $0.5 \cdot \sigma_0 \cdot A_{crit}$ . The medium longitudinal forces  $N_{Ed}$  are used to calculate the normal concrete stress.

### 1d. Book 525

The option allows the reduction of the load rising factor  $\beta$  according to Eq. (H.10-8).

## 2. Aperture

This dialog page is used to define the geometry and location of an opening.

## 3. Results

This dialog page shows the calculated punching shear resistances, the necessary punching shear reinforcement (if applicable) and the minimum bending reinforcement. You can call up an improved bending reinforcement by clicking the *Proposal* button.

## Example

**Input data**

Column: Rectangle, inside

Width  $b_x$ : 0.45 m

Height  $b_y$ : 0.45 m

Concrete: C35/45

Steel: BSt 500

Concrete parameters:  $a_{sx}$ : 31.42,  $a_{sy}$ : 31.42,  $\beta$ : 1.05

Steel parameters: Eff. h.  $d_x$ : 0.19, Eff. h.  $d_y$ : 0.19,  $sw$ : 0.142

Other parameters: Slab height: 0.24, Truss angle: 90

Action:  VEd from calculation,  Enter VEd

Results:

**Results of the fundamental combination**

Available long. reinf.  $a_{sx} = 31.42 \text{ cm}^2/\text{m}$ ,  $a_{sy} = 31.42 \text{ cm}^2/\text{m}$

$V_{Ed} = 236.6 \text{ kN/m}$ ,  $V_{Rd,ct} = 205.8 \text{ kN/m}$ ,  $V_{Ed}/V_{Rd,ct} = 1.15$

$\rho_1 = 0.0165$

$A_{sw1} = 11.70 \text{ cm}^2$ ,  $V_{Rd,sy1} = 354.4 \text{ kN/m}$ ,  $V_{Ed1}/V_{Rd,sy1} = 1.00$

$A_{sw2} = 4.76 \text{ cm}^2$ ,  $V_{Rd,sy2} = 258.3 \text{ kN/m}$ ,  $V_{Ed2}/V_{Rd,sy2} = 1.00$

$V_{Ed,a} = 167.2 \text{ kN/m}$ ,  $V_{Rd,ct,a} = 184.5 \text{ kN/m}$ ,  $V_{Ed,a}/V_{Rd,ct,a} = 0.91$

**The check is OK!**

**Proposal 1**

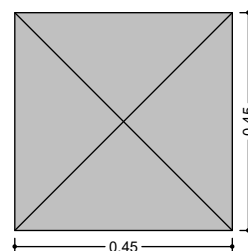
$a_{sx} = 14.15 \text{ cm}^2/\text{m}$ ,  $a_{sy} = 14.15 \text{ cm}^2/\text{m}$ ,  $\Sigma A_{sw} = 45.66 \text{ cm}^2$

### Punching shear check node 4312

The check is performed according to DIN 1045-1:2008-08.

#### 1. Measurements, situation and material

Rectangular column with width  $b_x = 0.45 \text{ m}$  and height  $b_y = 0.45 \text{ m}$   
 Situation: Inside;  $\beta = 1.05$



Critical perimeter  $u = 3.59$  m (distance = 0.29 m);  $A_{crit} = 0.97$  m<sup>2</sup>

Slab height  $h = 0.240$  m

Effective height of the slab  $d_x = 0.190$  m;  $d_y = 0.190$  m

Available longitudinal reinforcement  $a_{sx} = 31.42$  cm<sup>2</sup>/m;  $a_{sy} = 31.42$  cm<sup>2</sup>/m

Truss angle  $\alpha = 90.0^\circ$

Concrete: C35/45       $f_{ck} = 35.00$  N/mm<sup>2</sup>       $\alpha = 0.85$   
 $\gamma_c = 1.50$        $f_{cd} = \alpha \cdot f_{ck} / \gamma_c = 19.83$  N/mm<sup>2</sup>

Reinforce.: BSt 500       $f_{ck} = 500.00$  N/mm<sup>2</sup>       $\gamma_c = 1.15$   
 $f_{yd} = f_{yk} / \gamma_s = 434.78$  N/mm<sup>2</sup>

## 2. Action from fundamental combination

$V_{Ed} = 809.00$  kN       $N_{Ed} = 0.00$  kN/m       $\sigma_0 = 0.00$  kN/m<sup>2</sup>

$v_{Ed} = \beta \cdot V_{Ed} / u = 236.57$  kN/m

## 3. Punching resistance without punching reinforcement

$$v_{Rd,ct} = \left( (0,21/\gamma_c) \cdot \eta_1 \cdot \kappa \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} - 0,12\sigma_{cd} \right) \cdot d$$

with  $\eta_1 = 1.00$        $\kappa = 2.00$   
 $\rho_l = 0.0165$        $f_{ck} = 35.00$  N/mm<sup>2</sup>  
 $\sigma_{cd} = 0.00$  N/mm<sup>2</sup>       $d = 0.19$  m

$v_{Rd,ct} = 205.79$  kN/m

$v_{Ed} / v_{Rd,ct} = 1.15 > 1$       Punching reinforcement is required!

## 4. Punching reinforcement (normal)

$$A_{s,w1} = \frac{(v_{Ed} - v_{Rd,c}) \cdot u_1}{\kappa_s \cdot f_{yd}} \quad A_{s,w i} = \frac{(v_{Ed} - v_{Rd,c}) \cdot u_i \cdot s_w}{\kappa_s \cdot f_{yd} \cdot d}$$

$A_{s,w,min} = \min \rho_w \cdot s_w \cdot u_i$

with  $v_{Rd,c} = 205.79$  kN/m       $f_{yd} = 434.78$  N/mm<sup>2</sup>  
 $\kappa_s = 0.70$        $s_w = 0.142$  m  
 $\min \rho_w = 0.102$  %       $d = 0.19$  m

Row 1: Distance = 0.10 m;  $u_1 = 2.40$  m;       $v_{Ed,1} = 354.39$  kN/m;

$A_{s,w,1} = 11.70$  cm<sup>2</sup>       $> A_{s,w,1,min} = 3.47$  cm<sup>2</sup>

Row 2: Distance = 0.24 m;  $u_2 = 3.29$  m;       $v_{Ed,2} = 258.26$  kN/m;

$A_{s,w,2} = 4.24$  cm<sup>2</sup>       $< A_{s,w,2,min} = 4.76$  cm<sup>2</sup>

External perimeter: Distance = 0.52 m;  $u_a = 5.08$  m;  $v_{Ed,a} = 167.22$  kN/m;

$\kappa_a = 0.897$ ;  $v_{Rd,ct,a} = 184.52$  kN/m

$v_{Ed,a} \leq v_{Rd,ct,a}$       The check is OK!

## 5. Minimum momenta and corresponding longitudinal reinforcement according to section 10.5.6

$m_{Ed,x} = \eta_x \cdot V_{Ed} = 0.125 \cdot 809.00 = 101.12$  kNm/m

corresponding longitudinal reinforcement  $a_{sx,min} = 12.93$  cm<sup>2</sup>/m  $\leq a_{sx} = 31.42$  cm<sup>2</sup>/m

$m_{Ed,y} = \eta_y \cdot V_{Ed} = 0.125 \cdot 809.00 = 101.12$  kNm/m

corresponding longitudinal reinforcement  $a_{sy,min} = 12.93$  cm<sup>2</sup>/m  $\leq a_{sy} = 31.42$  cm<sup>2</sup>/m

## Prestressed Structures

### Internal Prestressing

For internal prestressing, the tendon groups as well as the prestressing system and procedures are entered using the *Prestressing* function of the *Structure* menu. To include them in the FEM calculation, you then need to define a load case of the *Prestressing* load type. For more information, refer to the *Prestressed Concrete* section of the manual.

Prestressing with bond and prestressing without bond are differentiated in the section inputs and the specifications for the *Creep and shrinkage* load case. For prestressed components with subsequent bond the tendons can be set ungrouted for the respective design situation in the action dialog.

### External Prestressing, Mixed Construction

External prestressing can be taken into account by entering the external forces directly in the program. For mixed construction, the additional tendons with bond must be entered as described above.

### Variation of Prestressing

For checks in the ultimate limit state, the following applies for the design value of the prestressing force according to DIN 1045-1, Chapter 8.7.5 (1):

$$P_d = \gamma_p \cdot P_{m,t}$$

with

$P_{m,t}$  Mean value of prestressing force at time  $t$  including prestressing losses from friction, slippage, creep, shrinkage and relaxation.

$\gamma_p$  Partial safety factor of prestressing force,  $\gamma_p = 1$  as specified in Chapter 8.7.5 (1).

In the serviceability limit state, two characteristic values for the prestressing force are defined in Chapter 8.7.4 (1):

$P_{k,sup} = r_{sup} \cdot P_{m,t}$  Upper characteristic value according to Eq. (52).

$P_{k,inf} = r_{inf} \cdot P_{m,t}$  Lower characteristic value according to Eq. (53).

The variation coefficients for internal prestressing are defined separately for construction stages and final states. They are used in the following checks:

- Minimum reinforcement for crack width limitation.
- Crack width check.
- Decompression check.

Regarding the effects from external prestressing, the variation coefficients are set to  $r_{sup} = r_{inf} = 1$  on the basis of DIN Technical Report 102, Chapter 2.5.4.2 (4).

## Creep and Shrinkage

Similar to prestressing, creep and shrinkage are taken into account by specifying a corresponding load case (*Creep and shrinkage* load type) in the FEM calculation. Besides the creep-generating permanent load case, you also need to specify whether the internal forces relocation between concrete and prestressing steel is to be taken into account. This option is only useful in the case of tendons with bond.

The decisive creep and shrinkage coefficients for calculating the *Creep and shrinkage* load case are entered in the section dialog. Alternatively, you can also use this dialog to calculate the coefficients according to Book 525, Section 9.1.4.

The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1+\rho \cdot \varphi} (\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,s})$$

In this case:

- $\sigma_b(t)$  Concrete stress from creep and shrinkage at time  $t$ .
- $E_b$  E-modulus of the concrete.
- $\rho$  Relaxation coefficient according to Trost for time  $t$  (normally  $\rho = 0.80$ ).
- $\varphi$  Creep coefficient for time  $t$ .
- $\varepsilon_b(t)$  Concrete strain from creep and shrinkage at time  $t$ .
- $\varepsilon_{b,0}$  Concrete strain from creep-generating continuous load.
- $\varepsilon_{b,s}$  Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, yield the internal forces and deformations of the concrete. The resulting stress changes in the prestressing steel are also determined provided they are selected in the load case. Any influence from the relaxation of the prestressing steel will be ignored in this case. According to Zilch/Rogge (2002, p. 256), this influence can be calculated separately (see following section) and combined with the changes from creep and shrinkage for all time-dependent prestressing losses:

$$\Delta\sigma_{p,csr} = \Delta\sigma_{pr} + E_p \cdot \Delta\varepsilon_{cpt}$$

with

$\Delta\sigma_{pr}$  Prestressing loss from relaxation of the prestressing steel.

$\Delta\varepsilon_{cpt}$  Concrete strain change from creep and shrinkage.

$E_p$  E-modulus of the prestressing steel.

## Relaxation of Prestressing Steel

According to DIN 1045-1, Chapter 8.7.3, the stress change  $\Delta\sigma_{pr}$  in the tendons at position  $x$  caused by relaxation must be taken into account in addition to the stress loss from concrete creep and shrinkage. The relaxation can be defined by the ratio of initial stress to characteristic tensile stress ( $\sigma_{p0}/f_{pk}$ ) from the building inspection certification with an initial stress of

$$\sigma_{p0} = \sigma_{pg0} - 0.3 \Delta\sigma_{p,csr} \tag{51}$$

with

$\Delta\sigma_{p,csr}$  Stress change in the tendons due to creep, shrinkage and relaxation at position  $x$  at time  $t$ .

$\sigma_{pg0}$  Initial stress in the tendons from prestressing and permanent actions.

Since the entire stress loss cannot be known in advance, the input value  $\Delta\sigma_{p,csr}$  for Eq. (51) must be estimated and then iteratively corrected if necessary (cf. König et al. 2003, p. 38). Alternatively, you can set  $\sigma_{p0} = \sigma_{pg0}$  and for conventional buildings  $\sigma_{p0} = 0.95 \cdot \sigma_{pg0}$  according to DIN 1045-1 for the sake of simplicity and to be on the safe side. The following table shows an example of stress loss due to relaxation.

Characteristic values of the stress losses $\Delta\sigma_{pr}$ in % of the initial tension $\sigma_{p0}$ for prestressing steel strand St 1570 / 1770 with very low relaxation							
$\sigma_{p0}/f_{pk}$	Time interval after prestressing in hours						
	1	10	200	1000	5000	$5 \cdot 10^5$	$10^6$
0.45							
0.50							
0.55						1.0	1.2
0.60					1.2	2.5	2.8
0.65				1.3	2.0	4.5	5.0
0.70			1.0	2.0	3.0	6.5	7.0
0.75		1.2	2.5	3.0	4.5	9.0	10.0
0.80	1.0	2.0	4.0	5.0	6.5	13.0	14.0

For tendons with DIN 4227 certification, the example of  $t = \infty$  with a permissible utilization of 0.55 according to DIN 4227-1, Tab. 9, Row 65, results in a stress loss of around 1%, which generally can be ignored.

Tendons with new certification may be utilized by up to 0.65 according to DIN 1045-1, Chapter 11.1.4. This results in significantly higher stress losses that must be accounted for.

You can define the stress losses in the CSR actions of the *DIN 1045-1 Actions* dialog.



## Checks in the Ultimate Limit States

The following checks are available:

- Bending with or without longitudinal force or longitudinal force only (DIN 1045-1, Chapter 10.2).
- Ensuring ductile component behavior (Chapter 5.3.2).
- Lateral force (Chapter 10.3).
- Torsion and combined stressing (Chapter 10.4).
- Punching shear (Chapter 10.5).
- Fatigue check (Chapter 10.8).

## Design Combinations

The following combinations in accordance with DIN 1055-100, Chapter 9.4, are taken into account in the ultimate limit states:

- Combination for permanent and temporary design situations

$$E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_P \cdot P_k \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \right\} \quad (14)$$

- Combination for accidental design situations

$$E \left\{ \sum_{j \geq 1} \gamma_{GA,j} \cdot G_{k,j} \oplus \gamma_{PA} \cdot P_k \oplus A_d \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (15)$$

- Combination for design situations resulting from earthquakes

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \gamma_1 \cdot A_{Ed} \oplus \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (16)$$

The weighting factor for the earthquake action is assumed as  $\gamma_1 = 1$  according to DIN 4149, Eq. (37).

For the check against fatigue two alternative action combinations can be used:

- Frequent combination for simplified checks according to DIN 1055-100, Chapter 10.4, Equation (23), in conjunction with DIN 1045-1, Chapter 10.8.4.

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (23)$$

- Fatigue combination according to DIN EN 1992-1-1, Chapter 6.8.3, Equation (6.69), for checks with damage equivalent stress ranges based on DIN 1045-1, Chapter 10.8.3.

$$E \left\{ \left( \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right) \oplus Q_{fat} \right\} \quad (6.69)$$

In this equation  $Q_{k,1}$  and  $Q_{k,i}$  are non-cyclic, non-permanent actions, whereas  $Q_{fat}$  defines the action of the relevant fatigue load.

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

## Stress-Strain-Curves

The following characteristics are used for section design:

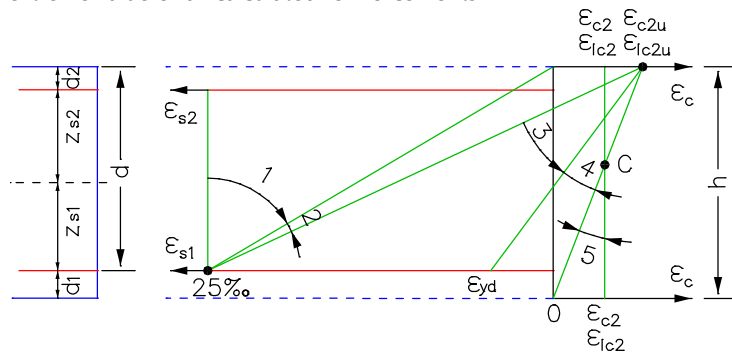
- Concrete: parabola-rectangle diagram according to DIN 1045-1, Figure 23.
- Reinforcing steel: stress-strain curve according to Figure 27, with rising upper branch.
- Prestressing steel: stress-strain curve according to Figure 29, with horizontal upper branch according to Chapter 9.3.3 (2).

For the fatigue checks, the user defines one of the following curves to determine the concrete compressive stresses:

- Stress-strain curve according to Figure 22, with  $f_c = f_{cm}$ .
- Parabola-rectangle diagram according to Figure 23.
- Linear curve with the gradient  $\arctan E_{cm}$ .

## Design for Bending With and Without Longitudinal Force or Longitudinal Force Only

The design for longitudinal force and bending moment is performed according to DIN 1045-1, Chapter 10.2. The design internal forces are calculated in the same way as for the design according to EN 1992-1-1. The reinforcement required for each internal force combination at the reinforced concrete section is determined iteratively based on the formulation of equilibrium conditions as well as the limit strain curve depicted in the illustration below. The final result is derived from the extreme value of all calculated reinforcements.



Strain areas for the design

You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes.

### Mode Standard

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tension section to the greatest degree possible. Given ratios between certain reinforcement layers in the tension or compression zone are maintained as far as possible. Due to reasons of economy and to limit the compression zone height according to Chapter 8.2 (3), the compression reinforcement in strain area 3 will be determined in such a way that the following applies for steel strain  $\epsilon_{s1}$ :

$$\epsilon_{s1} \geq \max \left( \epsilon_{yd}, \frac{\epsilon_{c2u}}{x/d} - \epsilon_{c2u} \right) \quad [‰]$$

with

$\epsilon_{yd}$  The steel strain associated with the design value for strength  $f_{yd}$ .

$x/d$  Referenced compressive zone height.

≤ 0.45 for concrete qualities up to C50/60.

≤ 0.35 for concrete qualities C55/67 or higher and lightweight concrete.

The procedure in strain areas 4 and 5 is the same as with symmetrical design. The requirements for the minimum reinforcement for the less stressed direction of slabs and walls according to Chapter 13.3.2 (3) resp. 13.7.1 (5) are not taken into account during bending design.

### Mode Symmetrical

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected.

**Mode Compression member**

The design is performed symmetrically. In addition, the minimum reinforcement required by Chapter 13.5.2 is calculated:

$$A_{s,\min} = 0.15 |N_{Ed}| / f_{yd} \quad (155)$$

with

$N_{Ed}$  Design value of the longitudinal force to be absorbed.

$f_{yd}$  Design value of the reinforcing steel strength at the yield strength.

Concrete compression according to Chapter 10.2 (6) cannot be checked.

**Inclusion of tendons with bond**

When designing beams and design objects, the internal forces of the concrete section is reduced by the statically determined portions which result from prestressing minus the losses from creep, shrinkage and prestressing steel relaxation (CSR). Situations prior to the grouting of the tendons are excluded. So only the restraint portions from 'P+CSR' and the external loads are contained in the remaining internal forces for the composite section. If necessary, the reinforcing steel positioned by the user will be increased until the composite internal forces can be absorbed. In the design mode *compression member* the prestressing steel area is taken into account when determining the minimum reinforcement according to Chapter 13.5.2.

The position of the tendon groups in the section, the prestressing losses from CSR, the statically determined portions and the internal forces of the concrete section and the composite section are written to the detailed log.

As a separation into statically determined and undetermined shares of the internal forces from prestressing is not possible for shell structures, the prestressing is taken into account fully on the action side when designing the longitudinal reinforcement. As a result, on the resistance side only mild steel and concrete are considered whereas the strain reserves of the tendons with bond are not used.

**Minimum Reinforcement for Ensuring Ductile Component Behavior**

According to DIN 1045-1, Chapter 5.3.2, component failures that occur without warning during initial crack formation must be prevented (ductility criterion). This requirement is fulfilled for reinforced concrete and prestressed concrete components as long as a minimum reinforcement is included as described in Chapter 13.1.1. This minimum reinforcement, which is also referred to as the *Robustness reinforcement* in Book 525 and Technical Report 102, must be calculated for the crack moment (given prestressing without taking into account the prestressing force) using the average tensile strength of concrete  $f_{ctm}$  and the steel stress  $\sigma_s = f_{yk}$ :

$$A_s = M_{cr} / (f_{yk} \cdot z_s)$$

with

$M_{cr}$  Crack moment by which a tensile stress of  $f_{ctm}$  occurs without prestressing effect at the section edge.

$z_s$  Lever arm of internal forces.

The reinforcement must be distributed throughout the tensile zone based on the constructive guidelines set forth in Chapter 13.1.1 (3). DIN 1045-1 provides no information on the action combination that is used to determine the tensile zone. The corresponding rule specified in DIN Technical Report 102, Chapter 4.3.1.3, is therefore used in the program. Based on that rule, the minimum reinforcement should be placed in areas where tensile stresses in the concrete occur under the infrequent action combination. According to Technical Report 102, Chapter 4.3.1.3 (107), the statically undetermined prestressing effect should be taken into account in this combination rather than the statically determined prestressing effect. Since the infrequent combination is not defined in DIN 1045-1, to be on the safe side it is replaced by the rare (characteristic) combination for the check. It is the responsibility of the user to observe the remaining constructive guidelines of Chapter 13.1.1 (3).

The program determines all stresses at the gross section. The statically determined prestressing effect can only be subtracted for beams and design objects. For area elements the prestress is alternatively reduced by a user-defined reduction factor. The crack moment results in  $M_{cr} = W_c \cdot f_{ctm}$ , the lever arm  $z_s$  of the internal forces is assumed to be  $0.9 \cdot d$  for the sake of simplicity. The calculated reinforcement is evenly distributed to the reinforcement layers in the tensile zone. In the design mode *symmetrical* reinforcement is also applied to the remaining layers. This will not affect the predefined relationships between the individual reinforcement layers. For sections with mode *compression member* the robustness reinforcement is not checked because minimum reinforcement is already determined during the design for bending with longitudinal force.

The option to take tendons into account as per Chapter 13.1.1 (2) remains unused.

## Minimum Surface Reinforcement for Prestressed Members

A minimum surface reinforcement must always be specified for prestressed components in accordance with DIN 1045-1, Chapter 13.1.2, regardless of the guidelines on robustness or crack reinforcement. The reinforcement determined according to Table 29 and 30 can be entered into the program by specifying a base reinforcement in the reinforcing steel description.

## Design for Lateral Force

Lateral force design involves determining the lateral force tensile reinforcement and includes a concrete strut check as defined by DIN 1045-1, Chapter 10.3. The following special conditions apply:

- The angle of the lateral force reinforcement is assumed to be 90°.
- In the calculation, the value for  $\cot \Theta$  is limited to the range permitted in accordance with Eq. (73) (method with load-dependent strut angle), unless the check with a constant value is selected in the section dialog. The actual effective concrete strut angle is logged for each check location. Edition 2008, Chapter 10.3.4 (3): For perpendicular stirrups or longitudinal tensile load,  $\cot \Theta$  should not fall below the limit value of 1.0. This is guaranteed by the program provided the user does not specify a smaller value.
- The minimum reinforcement is maintained in the calculated stirrup reinforcement as described in Chapter 13.2.3 of the standard while the reinforcement level  $\rho$  specified in Table 29 is weighted with a user-defined factor. For areas, the minimum reinforcement will only be determined if a lateral force reinforcement is required for computation (cf. Building and Civil Engineering Standards Committee (NABau) No. 131).
- For beams and design objects, the shear design is performed separately for the  $Q_y$  and  $Q_z$  lateral forces.
- Slab and shell elements are designed for lateral force  $q_T = \sqrt{(q_x^2 + q_y^2)}$ . Depending on which has a negative effect, either the principal compressive force or principal tensile force is used for the associated longitudinal force. If selected, the check will be carried out separately for the reinforcement directions x and y in accordance with Chapter 10.3.1 (5). In this case, the normal force in reinforcement direction is used for the associated longitudinal force. If lateral force reinforcement is necessary, it must be added from both directions.
- There is **no** reduction of the action from loads near supports, as specified in Chapter 10.3.2, Section (1) or (2).
- For beams and design objects, the decisive values of the equivalent rectangle are determined by the user independently of the normal section geometry.
- As described in Chapter 10.3.4 (2), the internal lever arm is assumed as  $z = 0.9 d$  and is limited to the maximum value derived from  $z = d - c_{v,1} - 30$  mm and  $z = d - 2c_{v,1}$  (cf. NABau No. 24). Note that  $c_{v,1}$  is the laying measure of the longitudinal reinforcement in the concrete compressive zone. If  $c_{v,1}$  is not specified, the program will use the shortest axis distance of the longitudinal reinforcement from the section edge  $d_1$  in its place.
- For beam sections with internal prestressing, the design value of lateral load-bearing capacity  $V_{Rd,max}$  according to Chapter 10.3.4 (8) is determined using the nominal value  $b_{w,nom}$  of the section width.
- Edition 2008: The lateral load-bearing capacity  $V_{Rd,max}$  is only checked for lateral forces  $V_{Ed} > V_{Rd,ct}$  as explained in Chapter 10.3.1 (4).
- The necessity of a lateral force reinforcement is analyzed according to Chapter 10.3.3 of the standard. As in the previous case, **no** reduction of the action from loads near supports occurs.

The formulas of DIN 1045-1 that are used are listed below.

### Components without computationally necessary lateral force reinforcement

The design value  $V_{Rd,ct}$  of the absorbable lateral force in a component without lateral force reinforcement results from Equation (70).

$$V_{Rd,ct} = \left[ 0.10 \kappa \cdot \eta_1 \cdot (100 \rho_1 \cdot f_{ck})^{1/3} - 0.12 \cdot \sigma_{cd} \right] \cdot b_w \cdot d \quad (70:2001)$$

$$V_{Rd,ct} = \left[ \frac{0.15}{\gamma_c} \cdot \kappa \cdot \eta_1 \cdot (100 \rho_1 \cdot f_{ck})^{1/3} - 0.12 \cdot \sigma_{cd} \right] \cdot b_w \cdot d \quad (70:2008)$$

Edition 2008: In this case you may use a minimum value for the lateral load-bearing capacity  $V_{Rd,ct,min}$  based on Equation (70a):

$$V_{Rd,ct,min} = [\eta_1 \cdot v_{min} - 0.12 \sigma_{cd}] \cdot b_w \cdot d \quad (70a)$$

with

$$v_{min} = \frac{\kappa_1}{\gamma_c} \cdot \sqrt{\kappa^3 \cdot f_{ck}}$$

where

$$\kappa \quad \text{is a scale factor with} \quad \kappa = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (71)$$

$\gamma_c$  is the partial safety factor for reinforced concrete as per 5.3.3 (6), Table 2.

$\eta_1$  = 1.0 for normal concrete; for lightweight concrete Table 10 applies.

$\rho_1$  is the longitudinal reinforcement level with

$$\rho_1 = \frac{A_{sl}}{b_w \cdot d} \leq 0.02$$

$A_{sl}$  is the area of the tensile reinforcement that extends beyond the section being viewed by at least measure  $d$  and is effectively fixed at that position (see Figure 32). For prestressing with immediate bond, the entire prestressing steel area can be taken into account for  $A_{sl}$ .

$b_w$  is the smallest section width in the tensile zone of the section in mm.

$d$  is the effective static height of the bending reinforcement in the viewed section in mm.

$f_{ck}$  is the characteristic value of the concrete compressive strength in N/mm<sup>2</sup>.

$\sigma_{cd}$  is the design value of the concrete longitudinal stress at the level of the section's centroid with  
 $\sigma_{cd} = N_{Ed} / A_c$  in N/mm<sup>2</sup>.

$N_{Ed}$  is the design value of the longitudinal force in the section as a result of external actions or prestressing ( $N_{Ed} < 0$  as longitudinal compressive force).

$\kappa_1$  = 0.0525 for  $d \leq 600$  mm  
 = 0.0375 for  $d \geq 800$  mm  
 For  $600 \text{ mm} < d < 800$  mm,  $\kappa_1$  can be interpolated linearly.

#### Components with computationally necessary lateral force reinforcement

The design value of the absorbable lateral force that is limited by the load-bearing capacity of the lateral force reinforcement is determined according to Equation (75).

$$V_{Rd,sv} = \frac{A_{sw}}{s_w} \cdot f_{yd} \cdot z \cdot \cot \Theta \quad (75)$$

where

$A_{sw}$  is the section area of the lateral force reinforcement.

$s_w$  is the distance of the reinforcement perpendicular to the component axis measured in the direction of the component axis.

$z$  =  $0.9 \cdot d < \max(d - 2c_{v,1}, d - c_{v,1} - 30 \text{ mm})$  with  $c_{v,1}$  the laying measure of the longitudinal reinforcement according to Chapter 10.3.4 (2).

The strut angle  $\Theta$  of the truss is limited to the following value:

$$0.58 \leq \cot \Theta \leq \frac{1.2 - 1.4 \sigma_{cd} / f_{cd}}{1 - V_{Rd,c} / V_{Ed}} \leq 3.0 \text{ for normal concrete} \quad (73)$$

$$\leq 2.0 \text{ for lightweight concrete}$$

Edition 2008:  $\cot \Theta < 1$  should only be used as an exception. In the case of longitudinal tensile stress this lower limit applies basically (ref. also Book 525, Corr. 1:2005-05). The program takes the limit into account as long as the user does not enter a smaller value.

where

$$V_{Rd,c} = \beta_{ct} \cdot 0.10 \cdot \eta_1 \cdot f_{ck}^{1/3} \left(1 + 1.2 \frac{\sigma_{cd}}{f_{cd}}\right) \cdot b_w \cdot z \quad (74:2001)$$

$$V_{Rd,c} = c_j \cdot 0.48 \cdot \eta_1 \cdot f_{ck}^{1/3} \left(1 + 1.2 \frac{\sigma_{cd}}{f_{cd}}\right) \cdot b_w \cdot z \quad (74:2008)$$

$$\beta_{ct} = 2.4$$

$$c_j = 0.50$$

$\eta_1$  = 1.0 for normal concrete; for lightweight concrete Table 10 applies.

$\sigma_{cd}$  is the design value of the concrete longitudinal stress at the level of the section's centroid with  
 $\sigma_{cd} = N_{Ed} / A_c$  in N/mm<sup>2</sup>

$N_{Ed}$  is the design value of the longitudinal force in the section as a result of external actions or prestressing ( $N_{Ed} < 0$  as longitudinal compressive force).

$V_{Ed}$  is the design value of the acting lateral force.

The design value of the maximum lateral load-bearing capacity  $V_{Rd,max}$  is determined according to Equation (76).

$$V_{Rd,max} = \frac{b_w \cdot z \cdot \alpha_c \cdot f_{cd}}{\cot \Theta + \tan \Theta} \quad (76)$$

where

$\alpha_c$  is the reduction factor for the strut strength.

$\alpha_c = 0.75 \cdot \eta_1$  with  $\eta_1 = 1.0$  for normal concrete; for lightweight concrete Table 10 applies.

### Lateral force reinforcement

$$\rho_w = \frac{A_{sw}}{s_w \cdot b_w \cdot \sin \alpha} \quad (151)$$

where

$\rho_w$  is the reinforcement level of the lateral force reinforcement.

$\alpha$  is the angle between the lateral force reinforcement and the beam axis.

$\min \rho_w$  is the minimum value of  $\rho_w$  according to 13.2.3 (5)

In general:  $\rho_w = 1.0 \rho$

Slabs:  $\rho_w = 0.6 \rho$

Structured sections with prestressed tension chord:  $\rho_w = 1.6 \rho$

$\rho$  is the basis value for the determination of the minimum reinforcement according to Table 29 of the standard.

## Design for Torsion and Combined Loads

The design for torsion is carried out according to DIN 1045-1, Chapter 10.4. This design involves determining the diagonal tensile reinforcement and the longitudinal reinforcement and includes a concrete strut check under maximum torsion stress and a concrete strut check under simultaneously acting lateral force. The strut angle is determined according to Equation (73) with the lateral force according to Equation (90). Alternatively a strut angle of  $45^\circ$  for torsion according to Chapter 10.4.2 (2) or a constant value  $\cot \Theta$  for lateral force and torsion (cf. interpretation No. 24 of NABau) can be chosen in the section dialog.

The equivalent section on which this design is based is defined by the user independently of the normal section geometry.

Formulas used from the standard:

$$T_{Ed} \leq \frac{V_{Ed} \cdot b_w}{4.5} \quad (87)$$

$$V_{Ed} \left[ 1 + \frac{4.5 T_{Ed}}{V_{Ed} \cdot b_w} \right] \leq V_{Rd,ct} \quad (88)$$

$$V_{Ed,T} = \frac{T_{Ed} \cdot z}{2A_k} \quad (89)$$

where

$V_{Ed,T}$  is the shear force in a wall of the check section as a result of a torsion moment.

$A_k$  is the area enclosed by the center lines of the walls.

$z$  is the height of the wall, which is defined by the distance of the intersection points of the wall center line to the center lines of the adjacent walls.

$$V_{Ed,T+V} = V_{Ed,T} + \frac{V_{Ed} \cdot t_{eff}}{b_w} \quad (90)$$

where

$V_{Ed}$  is the design value of the acting lateral force according to 10.3.2.

$t_{eff}$  is the effective thickness of a wall;  $t_{eff}$  is twice the distance from the center line to the exterior but not greater than the thickness of the existing wall (see Figure 36).

$$T_{Rd,sy} = \frac{A_{sw}}{s_w} \cdot f_{yd} \cdot 2A_k \cdot \cot \Theta \quad (91)$$

$$T_{Rd,sy} = \frac{A_{sl}}{u_k} \cdot f_{yd} \cdot 2A_k \cdot \tan \Theta \quad (92)$$

where

$T_{Rd,sy}$  is the design value of the absorbable torsion moment of the section.

$A_{sw}$  is the section area of the torsion reinforcement perpendicular to the component axis.

$s_w$  is the distance of the torsion reinforcement measured in the direction of the component axis.

$A_{sl}$  is the section area of the torsion longitudinal reinforcement.

$u_k$  is the perimeter of area  $A_k$ .

$\Theta$  is the strut angle of the truss.

$$T_{Rd,max} = \frac{\alpha_{c,red} \cdot f_{cd} \cdot 2A_k \cdot t_{eff}}{\cot \Theta + \tan \Theta} \quad (93)$$

where

$T_{Rd,max}$  is the design value of the maximum absorbable torsion moment of the section.

$\alpha_{c,red} = 0.7\alpha_c$  in general (with  $\alpha_c$  according to 10.3.4 (6)).

$\alpha_{c,red} = \alpha_c$  for box sections with reinforcement at the inner and outer sides of the walls.

$$\left[ \frac{T_{Ed}}{T_{Rd,max}} \right]^2 + \left[ \frac{V_{Ed}}{V_{Rd,max}} \right]^2 \leq 1 \quad \text{for compact sections} \quad (94)$$

$$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed}}{V_{Rd,max}} \leq 1 \quad \text{for box sections} \quad (95)$$

where

$V_{Rd,max}$  is the design value of the absorbable lateral force according to Equation (76).

## Punching Shear

The load-bearing safety check against punching shear is carried out according to DIN 1045-1, Chapter 10.5. This check is used to determine the necessary punching reinforcement. The following special conditions apply:

- The average static height  $d$  is determined based on the input parameters  $d_x$  and  $d_y$  at  $d = (d_x + d_y) / 2$ . They are selected as shown in Figure 37, 42, 43 or 45.
- The action can be entered directly or taken from the analyzed design situation at the ultimate limit state. In this case,  $V_{Ed}$  is set to the maximum support force  $R_z$  for each corresponding action combination.

The check is considered fulfilled if:

1. For slabs w i t h o u t punching reinforcement

$$v_{Ed} \leq V_{Rd,ct} \quad (101)$$

2. For slabs w i t h punching reinforcement

$$v_{Ed} \leq V_{Rd,max} \quad (102)$$

$$v_{Ed} \leq V_{Rd,sy} \quad (103)$$

$$v_{Ed} \leq V_{Rd,ct,a} \quad (104)$$

3. The minimum longitudinal reinforcement is maintained.  
with

$$v_{Ed} = \frac{\beta \cdot V_{Ed}}{u} \tag{100}$$

- $v_{Ed}$  Lateral force to be absorbed in the check section under consideration for each unit of length.
- $V_{Ed}$  Design value of the entire lateral force to be absorbed.
- $\beta$  Coefficient for taking into account the non-rotationally symmetric distribution of lateral force in the perimeter of the edge and corner columns and for internal columns in irregular systems. For edge and corner columns in conventional buildings, this value may be reduced when performing the ultimate limit state check outside the punching reinforcement ( $v_{Ed} \leq v_{Rd,ct,a}$ ) according to Book 525, Eq. (H.10-8).

$$\beta_{red} = \frac{\beta}{1 + 0.1 \cdot l_w / d} \geq 1.1 \tag{H.10-8}$$

- $l_w$  Width of the area with punching reinforcement outside of the load discharge area (see Figure 45).
- $d$  Average effective height in mm.  
 $d = (d_x + d_y) / 2$
- $d_x, d_y$  Effective height of the slab in the x or y direction in the perimeter under consideration.
- $u$  Circumference of the perimeter under consideration according to Figure 45.
- $v_{Rd,ct}$  Design value of the lateral force bearing capacity along the critical perimeter of a slab without punching reinforcement.
- $v_{Rd,ct,a}$  Design value of the lateral force bearing capacity along the external perimeter outside the punching reinforced area. This design value describes the transfer of the punching resistance without lateral force reinforcement  $v_{Rd,ct}$  to the lateral force resistance according to 10.3.3 in relation to the width  $l_w$  of the punching reinforced area (see Figure 45).
- $v_{Rd,sy}$  Design value of the lateral force bearing capacity with punching reinforcement along the internal check sections.
- $v_{Rd,max}$  Maximum lateral force bearing capacity for slabs with punching reinforcement in the critical perimeter.

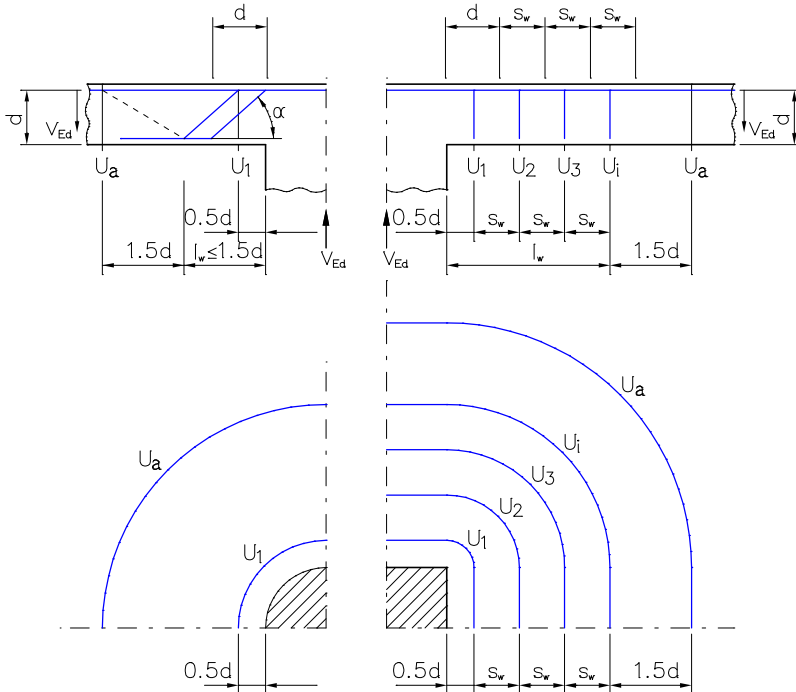


Figure 45 – Check sections of the punching reinforcement

The load discharge areas and check sections as per Chapter 10.5.2, Section (1) to (9), are taken into consideration. The user-specified opening dimensions are used to calculate the check sections.



Punching resistance without punching reinforcement is calculated as

$$v_{Rd,ct} = \left[ 0.14 \eta_1 \kappa \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} - 0.12 \sigma_{cd} \right] \cdot d \quad (105:2001)$$

$$v_{Rd,ct} = \left[ (0.21/\gamma_c) \cdot \eta_1 \cdot \kappa \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} - 0.12 \cdot \sigma_{cd} \right] \cdot d \quad (105:2008)$$

where

$$\kappa = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (106)$$

$\eta_1$  = 1.0 for normal concrete; for lightweight concrete Table 10 applies.

$\rho_1$  is the longitudinal reinforcement level with

$$\rho_1 = \sqrt{\rho_{1x} \cdot \rho_{1y}} \begin{cases} \leq 0.40 f_{cd}/f_{yd} \\ \leq 0.02 \end{cases} \quad (\text{Edition 2001-07})$$

$$\rho_1 = \sqrt{\rho_{1x} \cdot \rho_{1y}} \begin{cases} \leq 0.50 f_{cd}/f_{yd} \\ \leq 0.02 \end{cases} \quad (\text{Edition 2008})$$

$\rho_{1x}, \rho_{1y}$  is the reinforcement level based on the tensile reinforcement in the x or y direction which is located inside the perimeter under consideration and fixed in place outside the perimeter under consideration. For corner and edge columns, see 10.5.2 (9).

$\sigma_{cd}$  is the design value of the normal concrete stress within the perimeter under consideration with

$$\sigma_{cd} = N_{Ed} / \text{slab thickness}$$

$N_{Ed}$  is the design value of the average longitudinal force ( $N_{Ed} < 0$  as longitudinal compressive force).

Punching resistances with punching reinforcement are calculated as

$$1) v_{Rd,max} = 1.5 v_{Rd,ct} \quad (107)$$

2a) For the first reinforcement row with a distance of  $0.5 d$  from the column edge, the following applies:

$$v_{Rd,sy} = v_{Rd,c} + \frac{\kappa_s \cdot A_{sw} \cdot f_{yd}}{u} \quad (108)$$

2b) For the other reinforcement rows with a distance of  $s_w \leq 0.75 d$  from each other, the following applies:

$$v_{Rd,sy} = v_{Rd,c} + \frac{\kappa_s \cdot A_{sw} \cdot f_{yd} \cdot d}{u \cdot s_w} \quad (109)$$

where

$v_{Rd,c}$  is the concrete bearing portion;  $v_{Rd,c} = v_{Rd,ct}$  from Equation (105) can be assumed.

$\kappa_s$  is the coefficient for taking into account how the component height influences the efficiency of the reinforcement with

$$\kappa_s = 0.7 + 0.3 \frac{d - 400}{400} \begin{cases} \geq 0.7 \\ \leq 1.0 \end{cases} \text{ with } d \text{ in mm} \quad (110)$$

3) For diagonal bars ( $45^\circ \leq \alpha \leq 60^\circ$ ) as punching reinforcement, the following applies:

$$v_{Rd,sy} = v_{Rd,c} + \frac{1.3 A_s \cdot \sin \alpha \cdot f_{yd}}{u} \quad (111)$$

4) For the external perimeter with a distance of  $1.5 d$  from the last reinforcement row, the following applies:

$$v_{Rd,ct,a} = \kappa_a \cdot v_{Rd,ct} \quad (112)$$

with

$\kappa_a$  The coefficient for taking into account the transition to the slab area with the load-bearing capacity as per 10.3.3 with

$$\kappa_a = 1 - \frac{0.29 l_w}{3.5 d} \geq 0.71 \quad (113)$$

5) For the minimum required punching reinforcement of the internal check sections, the following applies:

$$\rho_w = \frac{A_{sw}}{s_w \cdot u} \geq \min \rho_w \quad (114)$$

or

$$\rho_w = \frac{A_s \cdot \sin \alpha}{d \cdot u} \geq \min \rho_w \text{ with } \min \rho_w \text{ according to 13.2.3 (5).}$$

In accordance with 13.3.3 (7), the following also applies:

If only one reinforcement row is computationally necessary with respect to stirrups as the punching reinforcement, a second row with the minimum reinforcement according to equation (114) must always be provided. In this case use  $s_w = 0.75 d$ .

The minimum longitudinal reinforcement is found based on the design of the minimum moments:

$$m_{Ed,x} = \eta_x \cdot V_{Ed} \text{ and } m_{Ed,y} = \eta_y \cdot V_{Ed} \quad (115)$$

where

$\eta_x, \eta_y$  are the moment coefficients as per Table 14 for the x or y direction.

## Check Against Fatigue

The user can select two alternative methods for design:

- Simplified check for the frequent action combination according to DIN 1045-1, Chapter 10.8.4, taking the relevant traffic loads at serviceability limit state into account.
- Check with damage equivalent stress ranges for the fatigue combination according to DIN 1045-1, Chapter 10.8.3, considering the specific fatigue load  $Q_{fat}$  specified in EN 1992-1-1, Chapter 6.8.3.

The curve to determine the concrete compressive stresses in state II is selected in the settings dialog.

## Fatigue of Longitudinal Reinforcement, Shear Reinforcement and Prestressing Steel

The fatigue check is carried out according to DIN 1045-1, Chapter 10.8. The steel stresses are calculated for longitudinal reinforcement from bending and longitudinal force as well as for prestressing steel in beams and design objects under the assumption of a cracked concrete section. For shear and longitudinal reinforcement from lateral force and torsion, the stresses are calculated according to 10.8.2 (4) based on a truss model with the strut angle  $\tan \Theta_{fat} = \sqrt{\tan \Theta}$  acc. to 10.8.2 (5). Where  $\Theta$  is the angle between the concrete compression struts and the beam axis used in the corresponding ultimate limit state design. The prestressing steel stresses in area elements are determined at the uncracked concrete section. Tendons without bond and external tendons are not checked.

### Simplified check

According to Chapter 10.8.4 (2), adequate fatigue resistance may be assumed if the stress range under the frequent action combination does not exceed 70 MN/m<sup>2</sup> for unwelded reinforcing bars. The condition described in Chapter 10.8.4 (3) for couplings in prestressed components is not examined by the program.

### Check with damage equivalent stress ranges

The check is considered completed if the following applies for reinforcing steel and prestressing steel:

$$\gamma_{F,fat} \cdot \gamma_{Ed,fat} \cdot \Delta\sigma_{s,equ} \leq \Delta\sigma_{Rsk}(N^*) / \gamma_{s,fat} \quad (119)$$

with

$$\gamma_{F,fat} = \gamma_{Ed,fat} = 1 \text{ as specified in Chapter 5.3.3 (2).}$$

$$\gamma_{s,fat} = 1.15 \text{ for reinforcing steel and prestressing steel with new certification.}$$

$$= 1.25 \text{ for reinforcing steel with DIN 4227-1 certification in accordance with ARS 11/03 (13).}$$

$\Delta\sigma_{Rsk}(N^*)$	Permitted characteristic stress range at $N^*$ load cycles based on the Wöhler curves specified in Tab.17 for prestressing steel or Tab.16 for reinforcing steel.
$\Delta\sigma_{s,equ}$	Damage equivalent stress range with $\Delta\sigma_{s,equ} = \max \Delta\sigma_s$ according to 10.8.3 (5).
$\max \Delta\sigma_s$	Calculated maximum stress range for longitudinal reinforcement from bending and longitudinal force including increase factor $\eta$ as specified in Chapter 10.8.2 (3) to account for the varying bond behavior of reinforcing and prestressing steel.

The values for  $\Delta\sigma_{Rsk}(N^*)$  and  $\eta$  are specified by the user in the Section dialog.

### Calculation method

The maximum from the robustness, crack and bending reinforcement is taken as the existing bending reinforcement. If as a result the load from the fatigue combination in state II cannot be absorbed, the design will be repeated using the existing reinforcement and the check internal forces.

The maximum stress range per steel layer that results from the strain state in state II or the truss model is determined separately for each check situation. Multiplying the coefficient  $\eta$  yields the damage equivalent stress range  $\Delta\sigma_{s,equ}$ . If for longitudinal and shear reinforcement this range exceeds the permitted stress range according to Eq. (119), the necessary reinforcement will be iteratively increased until the check succeeds for all situations. In the *Symmetrical* and *Compression member* design modes the longitudinal reinforcement is applied at all predefined locations. This will not affect the predefined relationships between the individual reinforcement layers.

The decisive reinforcement used for the check, which may have been increased, is recorded in the check log and saved for graphical representation.

## Fatigue of Concrete Under Compressive Stress

The fatigue check for concrete that is subject to compressive stress is performed for bending and longitudinal force at the cracked section. This check takes into account the final longitudinal reinforcement and may include an increase applied during the fatigue check for reinforcing steel. The struts of components subject to lateral force stress are not analyzed.

### Simplified check

The check according to Chapter 10.8.4 (4) is considered successfully if Eq. (123) is fulfilled.

$$\frac{\max|\sigma_{cd}|}{f_{cd,fat}} \leq 0.5 + 0.45 \cdot \frac{\min|\sigma_{cd}|}{f_{cd,fat}} \leq 0.9 \text{ for concrete up to C50/60 or LC50/55} \quad (123)$$

$$\leq 0.8 \text{ for concrete of at least C55/67 or LC55/60}$$

with

$\max|\sigma_{cd}|, \min|\sigma_{cd}|$  Design values of the maximum and minimum concrete compressive stress. In the case of tensile stresses,  $\min|\sigma_{cd}|$  is assumed to be zero.

$f_{cd,fat}$  Design value of the concrete compressive strength before cyclic load is applied. You can specify this value in the Section dialog.

### Check with damage equivalent concrete compressive stresses

The check according to Chapter 10.8.3 (6) is proved, if Eq. (120) is fulfilled:

$$E_{cd,max,equ} + 0.43\sqrt{1 - R_{equ}} \leq 1 \quad (120)$$

with

$$R_{equ} = \min|\sigma_{cd,equ}| / \max|\sigma_{cd,equ}| \quad (121)$$

$$E_{cd,max,equ} = \max|\sigma_{cd,equ}| / f_{cd,fat} \quad (122)$$

In that  $\max|\sigma_{cd,equ}|$  and  $\min|\sigma_{cd,equ}|$  are the upper and lower compressive stress of the damage equivalent stress range for  $N = 10^6$  cycles.

## Special Characteristic of Shell Structures

In shell structures the strain state at the cracked concrete section under general stress cannot be determined unambiguously. The design is therefore carried out separately for the reinforcement directions x and y with the design internal forces from Wolfensberger/Thürlimann or Rüsç as described above. The reinforcement calculated in this manner yields a reliable load-bearing capacity.

When calculating the stress range for reinforcing steel and concrete, this method can lead to unrealistic results in the case of torsional or shear stresses as shown in the following example:

Assume two identical sets of slab internal forces:

Set	$m_x$ [kNm/m]	$m_y$ [kNm/m]	$m_{xy}$ [kNm/m]
1	300	200	100
2	300	200	100

According to Wolfensberger/Thürlimann, this results in design variants for the x direction:

Set	Variant	$m$ [kNm/m]
1	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$
2	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$

The torsional moments generate a variation of the design moments and thus a calculatory stress range. This may lead to a necessary reinforcement increase in the fatigue check due to apparent overstressing. For normal design forces, this applies correspondingly to the shear forces.

Selecting **Limit design variants** in the Section dialog allows you to avoid the described effect. In this case only the corresponding variants are compared when determining the stress range, i.e. only the first and second variants of both sets in this example. Assuming constant stress, the stress range is thus correctly determined to be zero.

**This alternative, however, does not ensure that all conceivable stress fluctuations are analyzed. You should therefore be particularly careful when assessing the results. For this purpose the detailed log indicates the main variants and design internal forces used for the check.**

When determining the design internal forces according to Rüsç for inclined reinforcement, the described relationships apply accordingly.

## Checks in the Serviceability Limit States

The following checks are performed:

- Limiting the concrete compressive stresses (DIN 1045-1, Chapter 11.1.2).
- Limiting the reinforcing steel stresses (Chapter 11.1.3).
- Limiting the prestressing steel stresses (Chapter 11.1.4).
- Check of decompression (Chapter 11.2.1).
- Minimum reinforcement for crack width limitation (Chapter 11.2.2).
- Calculation of the crack width (Chapter 11.2.3 and 11.2.4).
- Limiting deformations (Chapter 11.3).

## Design Combinations

In accordance with DIN 1055-100, Chapter 10.4, the following combinations are taken into account in the serviceability limit states:

- Combination for rare (characteristic) situations

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus Q_{k,1} \oplus \sum_{i > 1} \psi_{0,i} \cdot Q_{k,i} \right\} \quad (22)$$

- Combination for frequent situations

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (23)$$

- Combination for quasi-continuous situations

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (24)$$

For each combination you can define different design situations for the construction stages and final states. If necessary, the combination required by the check will automatically be determined from the section specifications. Each check is carried out for all the situations of a combination.

## Stress Determination

For uncracked concrete sections, the program assumes that concrete and steel under tensile and compressive stress behave elastically. With respect to cracked concrete sections, the concrete compressive stresses are determined by the strain-stress curve shown in Figure 22 with  $f_c = f_{cm}$ . Note that a horizontal course is assumed for strains exceeding  $\epsilon_{c1}$  (cf. Reg. No. 098 in the Knowledge Base of the Building and Civil Engineering Standards Committee (NABau)).

### Area elements

For area elements the concrete stresses are calculated at the gross section. The steel stress check is carried out for reinforcing steel by determining the strain state at the cracked concrete section and for the prestressing steel at the uncracked concrete section.

### Beams and design objects

The action combination stresses that can be determined without checks are always calculated at the gross section.

Conversely, in the checks the stresses are determined as follows and are graphically displayed or logged:

- When checking the crack reinforcement and crack width, the concrete stress is calculated at the gross section
- When checking the decompression and concrete compressive stresses, the concrete stress is calculated
  - without internal tendons at the gross section
  - with internal tendons without bond at the net section
  - with internal tendons with bond for situations before being grouted at the net section or otherwise at the ideal section
- The reinforcing and prestressing steel stresses are checked by determining the strain state at the cracked concrete section

## Limiting the Concrete Compressive Stresses

The concrete compressive stress check is carried out according to DIN 1045-1, Chapter 11.1.2. Based on DIN Technical Report 102, Chapter 4.4.1.1 (5), the cracked state is assumed if the tensile stress calculated in the uncracked state under the rare action combination exceeds the value  $f_{ctm}$ .

The calculation in the cracked state is performed by determining the strain state with the final longitudinal reinforcement (maximum from robustness, crack and bending reinforcement including a possible increase from the fatigue check). For beams and design objects, the tendons with bond are taken into account on the resistance side provided that they are grouted in the check situation. For area elements the compressive stress for both reinforcement directions is determined separately and the extreme value is checked since the general strain state cannot be determined unambiguously.

In the construction stages and final states, the concrete compressive stress  $\sigma_c$  as defined in Chapter 11.1.2 (1) is to be limited to  $0.60f_{ck}$  under the rare combination. If serviceability is significantly impacted by the effect of creep, the limit  $0.45f_{ck}$  should be maintained under the quasi-continuous combination according to 11.1.2 (2). Both options are considered based on the user's specifications.

## Limiting the Reinforcing and Prestressing Steel Stresses

### Reinforcing steel

For reinforcing steel, the limitation of steel stress under the rare combination to  $0.80f_{yk}$  is checked in accordance with 11.1.3. In this check the reinforcement corresponds to the maximum value from the robustness, crack and bending reinforcement, including a possible increase as a result of the fatigue check. The determination of the strain state is performed at the cracked concrete section. If for beams and design objects tendons with bond are grouted in the check situation, they will be taken into account on the resistance side.

### Prestressing steel

For tendons with bond, the limitation of steel stress is checked at the cracked concrete section for beams and design objects and at the uncracked concrete section for area elements. In such cases the following limits apply:

Tendons with DIN 1045-1 and EC2 certification

- $0.65f_{pk}$  as per Chapter 11.1.4 (1) under the quasi-continuous combination
- $0.90f_{p0.1k}$  or  $0.80f_{pk}$  as per Chapter 11.1.4 (2) under the rare combination

Tendons with DIN 4227 certification

- $0.75\beta_s$  or  $0.55\beta_z$  according to DIN 4227-1, Tab. 9, Row 65, under the quasi-continuous combination and rare combination

For situations prior to grouting and for tendons without bond, the stress  $\sigma_{pm0}$  is checked in accordance with DIN 1045-1, Eq. (49) or DIN 4227-1, Tab. 9, Row 65. External tendons are not checked.

## Check of Decompression

This check is carried out for prestressed components of requirement classes A-C with the combinations specified in DIN 1045-1, Table 18. For area sections, the principal tensile stress  $\sigma_1$  or one of the longitudinal stresses  $\sigma_x$  or  $\sigma_y$  are checked based on the user's selection. The latter can be used to limit the check to the direction of the prestressing if the internal force systems are appropriately aligned (cf. Reg. No. 069 of the Knowledge Base of the Building and Civil Engineering Standards Committee (NABau) on DIN Technical Report 102). In all other cases, the rules for stress analysis listed above apply. The permissible stress limits are defined in Chapter 11.2.1 (9) as follows:

### Construction stage

In the construction stage, the section 'at the edge of the precompressed tensile zone as a result of prestressing' (i.e., at the section edge facing the tendon) must be subjected to compressive stresses. The program determines the respective edge as follows:

- Beams and design objects: If the edge point next to the tendon is above the centroid, the stress on the upper side of the section will be checked. If not, the lower side of the section will be checked.
- Area elements: The check will be carried out for the upper or lower section edge if the tendon next to the check point is located above or below the centroid level of the element in question. Tendons outside of the element are taken into account at a distance of up to five times the section height.

If the tendon guide is ambiguous, the check will be carried out for both sides.

**Final state**

In the final state the section must be completely subjected to compressive stresses.

**Minimum Reinforcement for Crack Width Limitation**

The minimum reinforcement for crack width limitation is defined in DIN 1045-1, Chapter 11.2.2. According to 11.2.2 (5), minimum reinforcement is to be applied in areas where tensile forces are expected. Areas under tension can be defined in the section dialog by choosing either an action combination or a restraint (bending, central tension). Reinforcing steel layers that are not under tension are also provided with reinforcement in the *symmetrical* and *compression member* design modes. This will not affect the predefined relationships between the individual reinforcement layers.

For profiled sections, each subsection (web or flange) should be checked individually in accordance with Section (4). This cannot be done if any polygonal section geometries are taken into consideration. For this reason, the program always determines the minimum reinforcement based on the entire section. For full rectangular sections, Equation (128) is used. In all other cases, Equation (128a) applies.

**Determining the minimum reinforcement**

The minimum reinforcement  $A_s$  is determined using Equation (127) of the standard:

$$A_s = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} / \sigma_s \quad (127)$$

In this formula

$k_c$  is the coefficient for consideration of stress distribution prior to crack formation.

For rectangular sections and webs of T-beams and box girders:

$$k_c = 0.4 (1 + s_c / (k_1 \cdot f_{ct,eff})) \leq 1 \quad (128)$$

For tension flanges of T-beams and box girders:

$$k_c = 0.9 \cdot F_{cr} / A_{ct} / f_{ct,eff} \geq 0.5 \quad (128a)$$

with the tensile force  $F_{cr}$  in the tension chord in state I directly before crack formation with the edge stress  $f_{ct,eff}$ . The tensile force is calculated by integrating the tensile stresses over the area  $A_{ct}$ .

$\sigma_c$  is the concrete stress at the level of the centroidal axis of the section or subsection, which, in an uncracked state, is subject to the action combination that leads to the initial crack formation on the entire section. ( $\sigma_c < 0$  for compressive stress).

$k_1 = 1.5 h/h'$  for compressive normal force

$= 2/3$  for tensile normal force

$h$  is the height of the section or subsection.

$h'$  =  $\min(h; 1 \text{ m})$ .

$k$  is the coefficient for taking into account nonlinearly distributed tensile stresses entered by the user.

$A_{ct}$  is the area of the concrete tensile zone at initial crack formation in state I. Here the program scales the bending moments caused by the action combination until the maximum edge stress in state I reaches the value  $f_{ct,eff}$ .

$f_{ct,eff}$  is the effective concrete tensile strength depending on the age of the concrete according to 11.2.2 (5):

$f_{ct,eff} = 0.5 f_{ctm}$  at an age of 3-5 days,

$f_{ct,eff} = f_{ctm}$  at an age of 6-28 days,

$f_{ct,eff} = f_{ctm}$  but not less than 3 MN/m<sup>2</sup>, if older than 28 days.

$\sigma_s$  is the maximum permitted stress in the reinforcing steel reinforcement in relation to the limiting diameter of the reinforcing steel.

The largest available bar diameter  $d_s$  is specified in the section dialog. Equation (129) provides a modified limiting diameter  $d_s^*$  to be used as an input value for Table 20:

$$d_s = d_s^* \cdot k_c \cdot k \cdot h_t / (4(h-d)) \cdot f_{ct,eff} / f_{ct0} \geq d_s^* \cdot f_{ct,eff} / f_{ct0} \quad (129)$$

where

$d_s^*$  is the effective area of the reinforcement according to Table 20.

$h$	is the component height.
$d$	is the effective static height.
$h_t$	is the height of the tensile zone in the section or subsection before initial crack formation.
$f_{ct0}$	is the tensile strength of the concrete from which the values in Table 20 are derived ( $f_{ct0} = 3.0 \text{ MN/m}^2$ ).

According to *Zilch/Rogge* (2002, p. 277), the expression  $k_c \cdot k \cdot h_t / (4(h-d))$  is generalized to  $0.6 \cdot k_c \cdot k \cdot A_{ct} / A_{c,eff}$  with the effective tensile zone  $A_{c,eff}$  as shown in Figure 53. Using the modified limiting diameter  $d_s^*$  and the allowed crack width  $w_k$ , the permissible reinforcing steel stress  $\sigma_s$  for equation (127) can be determined from Table 20.

If the crack width check is to be carried out at the same time, the program will determine whether the specified crack width according to Chapter 11.2.4 is maintained by inserting the calculated minimum reinforcement. If necessary, the minimum reinforcement can be increased iteratively until the check limit is reached. The increased reinforcement is indicated by an exclamation mark "!" in the log.

Guideline 11.2.1 (13) for the reinforcing mesh joint areas is not considered by the program.

Edition 2008: Based on Chapter 11.2.2 (8), the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined to Equation (130a), but the value may not fall below the value in Equation (130b). It is not necessary to insert more reinforcing steel as results from Equation (127). The rules specified before will be used, if the option is selected by the user, whereas the possibility of lower reinforcement for slowly hardening concrete according to Section (9) will not be used.

### Special characteristic of prestressed concrete structures

According to Chapter 11.2.2 (7), for a 300 mm square section around a tendon with immediate or subsequent bond, the minimum reinforcement required for this region may be reduced by  $\xi_1 \cdot A_p$ .

Where

$A_p$	is the section area of the prestressing steel in the tendon.
$\xi_1$	is the ratio of the prestressing and reinforcing steel bond strengths.

For beams and design objects, the tendons with bond can be added using the  $\xi_1$  value specified in the section dialog as long as they are grouted in the check situation. Note that prestressed steel cannot be taken into account for area elements.

According to Section (3) of Chapter 11.2.2, the minimum reinforcement for prestressed components with bond is not necessary in areas in which compressive concrete stresses larger than  $1 \text{ MN/m}^2$  occur at the section edge under the rare (characteristic) action combination and the characteristic prestress values. This condition is automatically checked by the program.

To delimit the areas where no minimum reinforcement is required, the concrete compressive stresses in state I are calculated at the gross cross-section with the mean characteristic prestress. The affected structural areas can be evaluated in the graphical stress representation for the characteristic combination.

## Calculation of the Crack Width

The crack width check is performed by means of direct calculation as per DIN 1045-1, Chapter 11.2.4, with the action combination that is based on the requirement class specified in Tab. 18. Enter the limit diameter and the age of the concrete in the Section dialog to determine the effective tensile strength.

Depending on concrete edge stress  $\sigma_c$  in state I, the following crack states must be differentiated (cf. Book 525, p. 191):

$\sigma_c \leq f_{ct,eff}$	Stage of single crack formation
$\sigma_c > f_{ct,eff}$	Stage of completed crack formation
with	
$f_{ct,eff}$	Effective concrete tensile strength depending on the age of the concrete according to 11.2.2 (5). Edition 2008: In this case a minimum concrete tensile strength is not included.

By limiting the maximum crack distance and the difference of the strains, the formulas in Section 11.2.4 of the standard as specified in Book 525, p. 104, can be used for the both the single crack formation and the completed crack formation



stages. This is why the program checks the crack width for all cases where  $\sigma_c > 0$ .

The program performs the check according to the following steps:

- Determine strain state II under the check combination defined by the requirement class with the stress-strain curve shown in Figure 22. For beams and design objects, all tendons in a bond are considered on the resistance side.
- Define effective tensile zone  $A_{c,eff}$  (see next section), determine reinforcing steel and prestressing steel layers within  $A_{c,eff}$
- Calculate reinforcement level:

$$eff \rho = (A_s + \xi_1^2 \cdot A_p) / A_{c,eff} \quad (133)$$

$$\rho_{tot} = (A_s + A_p) / A_{c,eff} \quad (134)$$

with

$\xi_1$  Bond coefficient according to user specification.

- Determine individually for each reinforcing steel layer:

#### Difference of the average strain for concrete and reinforcing steel

$$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - 0.4 f_{ct,eff} / eff \rho (1 + \alpha_E \cdot eff \rho)] / E_s \geq 0.6 \sigma_s / E_s \quad (136)$$

with

$$\alpha_E = E_s / E_{cm}$$

$$\sigma_s = \sigma_2 + 0.4 f_{ct,eff} (1/eff \rho - 1/\rho_{tot}) \quad (132)$$

$\sigma_s$  is limited to  $f_{yk}$  in the program

$\sigma_2$  Reinforcing steel stress from strain state II.

$f_{ct,eff}$  Effective concrete tensile strength at the considered time according to 11.2.2 (5).

Edition 2008: In this case a minimum concrete tensile strength is not taken into account.

#### Maximum crack spacing

$$s_{r,max} = d_s / (3.6 eff \rho) \leq \sigma_s \cdot d_s / (3.6 f_{ct,eff}) \quad (137)$$

If an upper limit for the crack distance based on Equation (137) was specified in the section dialog, then the special conditions of Equation (138) and Paragraph (8) of Chapter 11.2.4 can be taken into account.

#### Calculated crack width

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (135)$$

The layer with the largest calculated crack width is shown in the log.

- For sections under tension, the check is performed separately for each of the two effective tensile zones. The maximum value is shown in the log.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the crack width. For that purpose a design is carried out using the decisive check combination for calculating the crack width. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The crack width is checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check) and saved for graphical representation together with the decisive reinforcing steel stress.

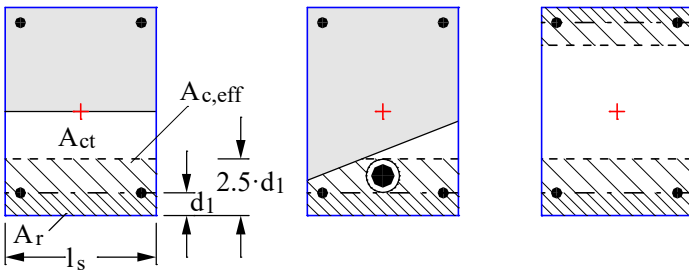
## Determining the Effective Area $A_{c,eff}$

According to DIN 1045-1, Figure 53, the effective tensile zone  $A_{c,eff}$  defines the area of a rectangular, uniaxially stressed concrete section in which the model assumptions specified in Book 466 are applicable (cf. Book 525, Figure 53). Although the program can apply this model to any section and stress situation, the user has the responsibility and discretion to do so.

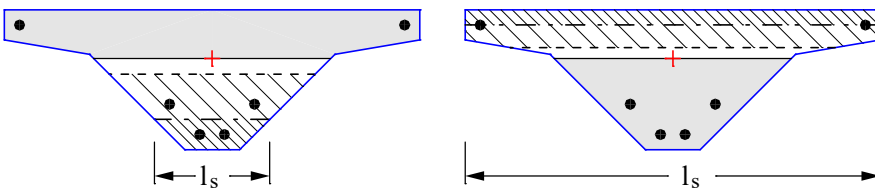
When determining  $A_{c,eff}$ , the program performs the following steps:

- Determine tensile zone  $A_{ct}$  in state I: when calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the requirement class. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Define the centroid line of the reinforcement as a regression line through the reinforcing steel layers in the tensile zone. In 2D frameworks and for area elements, a horizontal line through the centroid of the reinforcement layers under tension is assumed.
- Determine the truncated residual area  $A_r$  to the edge and the sum of section lengths  $l_s$ . The average edge distance is then assumed as  $d_1 = A_r / l_s$ , yet not less than the smallest edge distance of the reinforcing steel layers in the tensile zone.
- Shift the centroid line in parallel by  $1.5 \cdot d_1$ . For area elements,  $2.5 \cdot d_1 \leq (h-x) / 2$  is maintained ( $x$  = compression zone height in state I).
- The resulting polygon is intersected with the tensile zone and then defines the effective tensile zone  $A_{c,eff}$ .
- If all the reinforcing steel layers of the section are under tension, then two zones will be determined; one for the layers above the centroid and the other for layers below the centroid. The area of each zone is limited to  $A_c / 2$ .
- Edition 2008: If the minimum reinforcement for thicker components under central restraint is selected in the section dialog, the height of  $A_{c,eff}$  is  $h_{eff} \geq 2.5 d_1$  according to Figure 53 d).

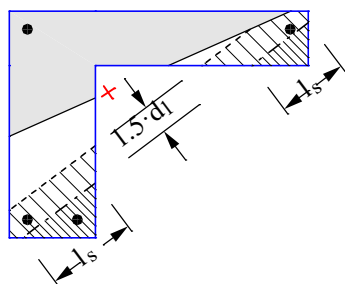
The following illustrations show the effective tensile zones determined by the program in typical situations. The last case (edge beam) deviates from the model assumptions in Book 466 to such a degree that it is questionable as to whether it should be used.



Effective tensile zones at a rectangular section under uniaxial bending, normal force with double bending and central tension



Effective tensile zones at a bridge section under uniaxial bending



Effective tensile zone at an edge beam under uniaxial bending

## Crack Width Check by Limitation of the Bar Distances

As an alternative to direct calculation of the crack width according to DIN 1045-1, Chapter 11.2.4, the simplified check as specified in 11.2.3 (2) for limiting the bar distances as specified in Table 21 can be selected in the section dialog.

The program performs the check according to the following steps:

- Determine the strain state II under the check combination specified by the requirement class with the stress-strain curve shown in Figure 22. For beams and design objects, all tendons with bond are considered on the resistance side.
- Calculate the reinforcing steel stress  $\sigma_s$  for every reinforcement layer using Equation (132).
- Compare the value given in the dialog (**max. s**) with the table value (**perm. s**), which results from the calculated steel stress  $\sigma_s$  and the permissible crack width  $w_k$ . The position with the largest (**max. s / perm. s**) quotient is indicated in the protocol.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the permissible bar distances. For that purpose a design calculation is carried out using the action combination relevant for the check. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The bar distance check is then carried out for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check).

### Note:

According to Chapter 11.2.3 (2), the simplified check can only be applied in the case of crack formation resulting from mainly direct actions (loads). Further, according to *Zilch and Rogge* (2002, p. 277) this method only provides safe results with a single layer of tensile reinforcement with  $d_1 = 4$  cm. The user is responsible for the evaluation of these requirements.

## Limiting Deformations

According to DIN 1045-1, Chapter 11.3.1, the deformations of a component or structure may not impair its proper functioning or appearance. Consequently, a beam, slab or cantilever under the quasi-continuous action combination should not sag more than 1/250th of the span as specified in Section (8). To avoid damaging adjacent components, the deformation should be limited to 1/500th of the span.

The standard does not include a method for direct calculation of deformations. Book 525, Section 11.3.2, only makes references to various bibliographic sources.

The InfoCAD program system allows you to perform a realistic check as part of a nonlinear system analysis for beam and shell structures that takes geometric and physical nonlinearities into account. The resistance of the tendons with bond is currently not included in the calculation.

Editing is performed in the following steps:

- Define the check situation with the *Load group* function in the Load dialog through grouping the decisive individual load cases. The variable loads must first be weighted with the combination coefficients  $\psi_2$  for the quasi-continuous combination.
- Select the check load cases in the *Nonlinear Analysis / Serviceability* dialog of the analysis settings for the FEM or framework analysis.
- Set the reinforcement determined in the ultimate limit state in the *Start reinforcement* selection field (maximum from bending, robustness, crack check and fatigue).
- Perform the FEM or framework analysis to determine the deformations in state II.
- Check the system deformations displayed graphically or in tabular form.

For a detailed description of the nonlinear system analysis, refer to the relevant chapter of the manual.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The resulting bending, robustness and crack reinforcement, the decisive maximum value and the stirrup and torsion reinforcement are provided for the graphical representation as well.

The log shows the design internal forces and necessary reinforcements, checked stresses or crack widths at each result location. If the permissible limit values are exceeded, they are reported as warnings and indicated at the check location. The detailed log also lists the decisive combination internal forces of all design situations.

### Stresses for beams and design objects

$\sigma_x$	Longitudinal stresses in the decompression and concrete compressive stress checks [MN/m <sup>2</sup> ].
$\sigma_s, \Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd}, \Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete under longitudinal compression [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,y}, \Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from $Q_y$ and $Q_z$ [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,T}, \Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and for longitudinal torsion reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{perm}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

### Stresses for area elements

$\sigma_x, \sigma_y, \sigma_1$	Longitudinal stress in x or y direction or principal tensile stresses in the decompression check (depending on user specification) [MN/m <sup>2</sup> ].
$\sigma_2$	Principal compressive stresses [MN/m <sup>2</sup> ].
$\sigma_{sx}, \Delta\sigma_{sx}$	Stresses and stress ranges for reinforcing steel in the x direction [MN/m <sup>2</sup> ].
$\sigma_{sy}, \Delta\sigma_{sy}$	Stresses and stress ranges for reinforcing steel in the y direction [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd,x}, \Delta\sigma_{cd,x}$	Stresses and stress ranges in the concrete fatigue check under longitudinal compression in the x-
$\sigma_{cd,y}, \Delta\sigma_{cd,y}$	and y-direction [MN/m <sup>2</sup> ].

$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{\text{perm}}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{\text{perm}}$	Stress range utilization.

### Bending reinforcement

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams and design objects.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in x and y direction.
$a_{s\phi}$	Meridian reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.
$a_{s\psi}$	Ring reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.

### Reinforcement from lateral force

$a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area and axisymmetric shell elements from $q_r$ .
$a_{sbx}, a_{sby}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area elements from $q_x$ and $q_y$ .
$A_{sb,y}, A_{sb,z}$	Stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $Q_y$ and $Q_z$ .
$A_{sl}$ for $a_{sb}=0$	Longitudinal reinforcement [cm <sup>2</sup> ] of area elements.

### Torsion reinforcement

$A_{sb,T}$	Torsional stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $M_x$ .
$A_{sl,T}$	Torsional longitudinal reinforcement [cm <sup>2</sup> ] of beams and design objects from $M_x$ .

### Design values

$V_{Rd,ct}, v_{Rd,ct}$	Absorbable design lateral force without shear reinforcement [kN or kN/m].
$v_{Rd,max}$	Absorbable design lateral force of concrete struts for area elements [kN/m].
$V_{Rd,max}$	Absorbable design lateral force of concrete struts for beams and design objects [kN].
$T_{Rd,max}$	Design value of the maximum absorbable torsion moment [kNm].
$Q/V_{Rd} + M_x/T_{Rd}$	For compact sections: $(Q/V_{Rd,max})^2 + (M_x/T_{Rd,max})^2$ For box sections: $Q/V_{Rd,max} + M_x/T_{Rd,max}$

### Crack width

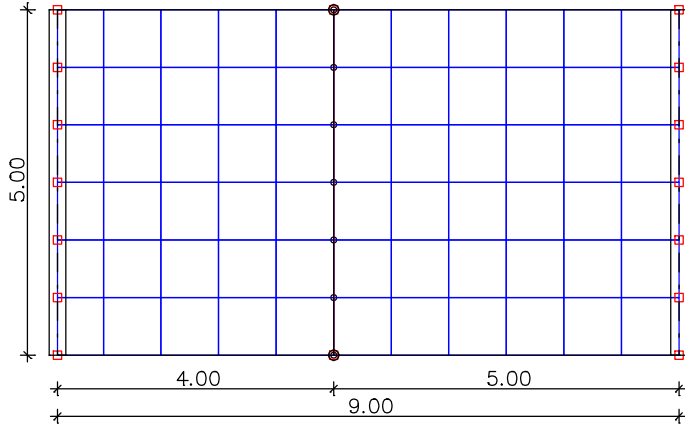
$w_{k,top}, w_{k,bottom}$	Computed crack width at the top and bottom of the cross-section separately for the x and y reinforcement direction for area elements.
$w_k / w_{\text{per}}$	Crack width utilization.

## Examples

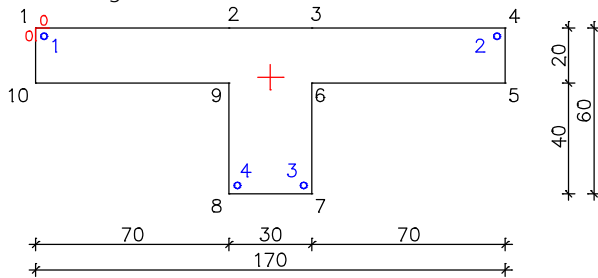
### Slab With Downstand Beam

In this example a rectangular slab ( $d = 20 \text{ cm}$ , C30/37, BSt 500 S, requirement class E) with a downstand beam will be analyzed. This slab supported with joints will be subjected to its dead load and a traffic load of  $10 \text{ kN/m}^2$ .

The checks will be carried out for all possible combinations of load cases. This method is selected in the calculation settings and can take a very long time to complete if there is a large number of load cases.



The following image shows the dimensions of the downstand beam. The axis distance of the reinforcing steel from the section edge is  $3 \text{ cm}$ . The dead load of the downstand beam is reduced by the portion attributed to the slab.



Design specifications and reinforcing steel description of the slab (section 1):

- Edge distance of the reinforcing steel for the x and y direction of the upper (1st) and lower (2nd) layer:  $0.03 \text{ m}$
- Bending design mode: *Standard*
- Steel quality: *500S*
- Effective height:  $0.17 \text{ m}$
- Strut angle  $\cot \Theta$ :  $3.0$ .  
The default value is limited to the range specified in DIN 1045-1, Eq. (73) when the design is carried out.
- Bending tensile reinforcement  $A_{s1}$  for the lateral force design:  $1.88 \text{ cm}^2$

Design specifications of the torsion-flexible downstand beam (section 2):

- Bending design mode: *Standard*
- Steel quality of the stirrups: *500S*
- Shear section:  
Width:  $0.30 \text{ m}$   
Effective height:  $0.57 \text{ m}$
- Strut angle  $\cot \Theta$ :  $3.0$ .  
The default value is limited to the range specified in DIN 1045-1, Eq. (73) when the design is carried out.
- Bending tensile reinforcement  $A_{s1}$  for the lateral force design:  $2.90 \text{ cm}^2$

**DIN 1045-1 actions****Standard design group****G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

1 Dead load

**QN - Imposed load, traffic load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
 Working load - category A - living rooms and lounges  
 Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

Load cases 1. Variant, inclusive

2 Traffic span 1  
 3 Traffic span 2

**1. Permanent and temporary situation**

Final state

G Dead load  
 QN Imposed load, traffic load

**1. Rare (characteristic) situation**

Final state

G Dead load  
 QN Imposed load, traffic load

**1. Quasi-continuous situation**

Final state

G Dead load  
 QN Imposed load, traffic load

**Design according to DIN 1045-1 (2008-08)****Design overview**

Se.	Class, Prestress of component	Reinforc. M R B Q T	Fatig. S P C	Crack- width	De- comp.	Stress checks Comp. S P
1	E Not prestressed	x x x x . . . .	. . . .	x	.	x x .
2	E Not prestressed	x x x x . . . .	. . . .	x	.	x x .

(M) Nominal reinforcement to guarantee robustness (ductility).  
 (R) Nominal reinforcement for crack width limitation.  
 (B) Flexural reinforcement at ultimate limit state.  
 (Q) (Nominal-)lateral force reinforcement at ultimate limit state.  
 (T) Torsional reinforcement at ultimate limit state.  
 (S) Reinforcing steel at stress and fatigue check.  
 (P) Prestressing steel at stress and fatigue check.  
 (C) Concrete at fatigue check.

**Settings for flexural and shear reinforcement**

M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 fyk Quality of stirrups.  
 Theta Angle of concrete truss. Program-sided, the given value of cot Theta is limited to the value range according to Eq. (73).  
 Slabs Beams are designed like slabs.  
 Asl Given reinforcement according to picture 32, increase to maximum.  
 rhom Minimum reinf. min rhom = Factor \* rho with rho according to table 29.  
 x,y Separate lateral force design for reinforcement directions x and y.  
 cvl Laying measure of the long. reinforcement to limit the lever arm z.  
 Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den- sity [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] given	Fac. rhom	Dsn. cvl x,y [mm]	Red. pre- str.
1	C30/37	.	ST	500	3.00	.	0.00	0.00	0.60	30 .
2	C30/37	.	ST	500	3.00	.	0.00	1.00	30 .	.

**Shear sections**

bw,nom Nominal width of the prestressed section acc. to 10.3.4 (8).  
 h,nom Nominal height of the prestressed section acc. to 10.3.4 (8).  
 z1, z2 Height and width of the core section for torsion.  
 teff Thickness of the torsion box.  
 B. Box section.

Se.	Width [m]		Eff. width		Height [m]		Eff.height	Torsion section [m]			
	bw	bw,nom	bn	[m]	h	h,nom		d	z1	z2	teff B.
1	1.000	.	.	.	0.200	.	0.170	.	.	.	.
2	0.300	.	0.270	.	0.600	.	0.570	0.540	0.240	0.060	.

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel.  
 max.s Maximal given bar spacing of the reinforcing steel.  
 Xil Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 sr,max Upper limit for the crack spacing from equ. (137).  
 Method Direct calculation of the crack width as per chapt. 11.2.4 or check by limiting the bar spacing according to table 21.  
 TM Thick member according to chapt 11.2.2(8) to determine As,min.

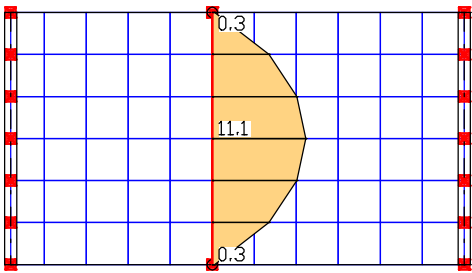
Se.	wk,per	ds	max.s	Coeff.	sr,max	Concr.	age	Method for	Tensile zone	TM
	[mm]	[mm]	[mm]	Xil	k	As,min	wk	crack w.	for As,min	
1	0.30	12	.	1.00	.	3- 5d	> 28d	Calcul.	Cmb. per class	.
2	0.30	12	.	1.00	.	3- 5d	> 28d	Calcul.	Cmb. per class	.

**Settings for the check of concrete stresses**

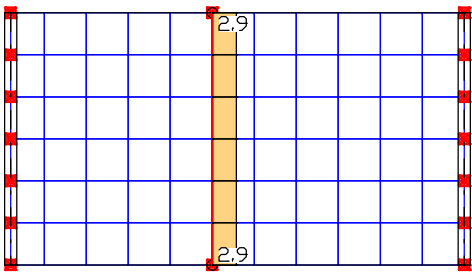
(CC) Characteristic combination (QC) Quasi-continuous combination

Se.	per.sigma.c	per.sigma.c	Dekompression
	(CC)	(QC)	Stress
1	0.60 fck	.	.
2	0.60 fck	.	.

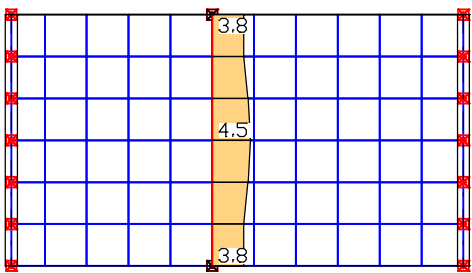
The calculated reinforcements are shown in the illustrations below.



Longitudinal reinforcement of the beams in the ultimate limit state [cm²]

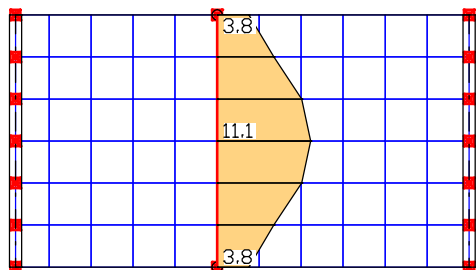


Longitudinal reinforcement of the beams to ensure robustness (ductility) [cm²]

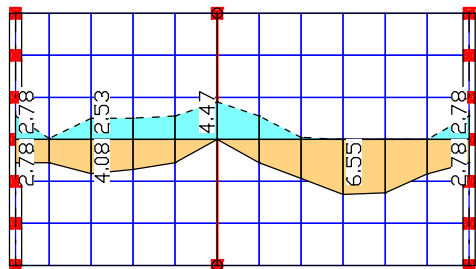


Longitudinal reinforcement of the beams to limit the crack width [cm²]

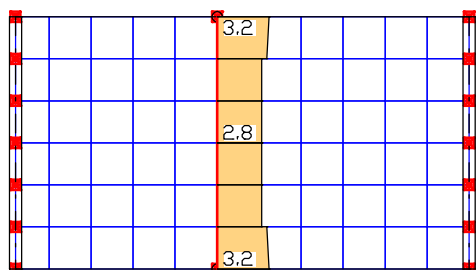




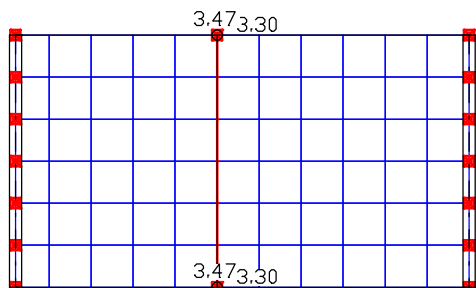
Maximum longitudinal reinforcement of the beams [cm<sup>2</sup>]



Maximum slab reinforcement in the intersection direction based on the robustness (ductility), crack width and design checks in the ultimate limit state [cm<sup>2</sup>/m]



Lateral force reinforcement of the beams [cm<sup>2</sup>/m]



Averaged lateral force reinforcement of the slab at the nodes [cm<sup>2</sup>/m<sup>2</sup>]

An excerpt of the detailed log for the midspan of the downstand beam is provided below.

#### Design of longitudinal reinforcement

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. for robustness as per 13.1.1 (Charact. C.)  
 fctm Average centric concrete tensile strength [MN/m<sup>2</sup>]  
 zs,t/b Lever arm of inner strengths top/bottom with zs=0,9\*d [m]  
 fyk,t/b Strength of longitudinal reinforcement top/bottom [MN/m<sup>2</sup>]  
 max Sc Maximal concrete edge stress from Charact. C. [MN/m<sup>2</sup>]
- (R) Nominal/requ. reinforcement as per 11.2.2 for crack width limitation  
 Increase of reinforcement due to crack width check is marked by "!".  
 wk,per Permissible crack width as per specification [mm]  
 ds Maximal given steel diameter [mm]  
 k Coefficient for consideration of non-linear distributed tensile stress  
 fct,eff Concrete strength at date of cracking [MN/m<sup>2</sup>]  
 kc Coefficient to consider stress distribution in tensile zone  
 as per Eq. (128a), for rectangles per Eq. (128)  
 max Sx Maximal concrete edge stress from action combination [MN/m<sup>2</sup>]
- (B) Design of reinforcement at ultimate limit state  
 In case of dominant bending, compression reinforcement is marked with "\*\*".  
 fck Concrete strength for design of reinforcement [MN/m<sup>2</sup>]

**Location 1**

Beam 70, x = 0.00 m (Beam length 0.83 m)  
 Cross-section 2: Polygon - C30/37  
 Steel 2; Design mode: Standard  
 (M) fctm=2.9; zs,t/b=0.513/0.513; fyk,t/b=500/500  
 (R) wk,per=0.3; ds=12; k=1; fct,eff=1.45  
 (B) fck=30

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

**1. Characteristic (rare) combination (CC.1): G+QN, Final state**

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
1 :	0.00	69.95	0.00
2 :	0.00	196.53	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1 :	L1
2 :	L1+L2+L3

**1. Quasi-continuous combination (QC.1): G+QN, Final state**

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
2 :	0.00	107.92	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2 :	L1+0.30*L2+0.30*L3

**1. Permanent and temporary comb. (PC.1): G+QN, Final state**

Relevant concrete internal forces from 8 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
2 :	0.00	284.31	0.00
5 :	0.00	69.95	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2 :	1.35*L1+1.50*L2+1.50*L3
5 :	L1

**Design of longitudinal reinforcement**

Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	max Sc [MN/m <sup>2</sup> ]	kc	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	Situation
1	M	0.00	69.95	0.00	.	.	.	0.00	CC.1,1
	R	0.00	107.92	0.00	4.25	.	.	0.00	QC.1,2
	B	0.00	69.95	0.00	.	.	.	0.00	PC.1,5
2	M	0.00	69.95	0.00	.	.	.	0.00	CC.1,1
	R	0.00	107.92	0.00	4.25	.	.	0.00	QC.1,2
	B	0.00	69.95	0.00	.	.	.	0.00	PC.1,5
3	M	0.00	196.53	0.00	7.73	.	.	1.44	CC.1,2
	R	0.00	107.92	0.00	.	.	.	2.27!	QC.1,2
	B	0.00	284.31	0.00	.	.	.	5.57	PC.1,2
4	M	0.00	196.53	0.00	7.73	.	.	1.44	CC.1,2
	R	0.00	107.92	0.00	.	.	.	2.27!	QC.1,2
	B	0.00	284.31	0.00	.	.	.	5.57	PC.1,2

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to 13.2.3 (5) is considered.

Ac	Section area for calculating the concrete stress from long. force [m <sup>2</sup> ]
bw	Effective width for calculation of shear stresses from Qz and Mx [m]
bn	Statically effective width for shear design using Qy [m]
kb	Factor to calculate the inner lever arm from bn
h	Effective height for calculation of shear stresses from Qy and Mx [m]
d	Statically effective height for shear design using Qz [m]
kd	Factor to calculate the inner lever arm from d
Angle	Angle cot Theta between the compressive strut and the beam axis
Asl giv.	Chargeable longitudinal reinf. acc. to Pic. 32 [cm <sup>2</sup> ]
rho <sub>w,min</sub>	Minimal percentage of lateral reinforcement acc. to 13.2.3 (5)
Qy, Qz	Lateral forces for design in y- and z-direction [kN]
VRdct	Absorbable lat. force without lat. reinf. per 10.3.3 (1) [kN]
VRdmax	Absorbable lateral force of comp. struts per 10.3.4 (7) [kN]
z	Inner lever arm z=kb*bn resp. z=kd*d [m]; z<=max(d-2cvl,d-cvl-30mm)
cvl	Laying measure of the long. reinforcement to limit the lever arm z [m]
Asb,y,z	Req. stirrup reinforcement from Qy resp. Qz [cm <sup>2</sup> /m]
Asl	Req. longitudinal reinf. acc. to Pic. 32 [cm <sup>2</sup> ] for req.Asb

**Location 1**

Beam 70, x = 0.00 m (Beam length 0.83 m)  
 Cross-section 2: Polygon - C30/37  
 bw/bn/kb=0.3/0.27/0.9; h/d/kd=0.6/0.57/0.9  
 cvl=0.03; fyk=500; Asl giv./max=0/0; min rho<sub>w</sub>=1\*rho

**1. Permanent and temporary comb. (PC.1): G+QN, Final state**

Relevant concrete internal forces from 8 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
2 :	0.00	284.31	0.00	0.00	0.00	-30.79

Load case combinations for the relevant sets of internal forces

```
Set      Combination
2       : 1.35*L1+1.50*L2+1.50*L3
```

#### Check of the shear reinforcement and the compressive struts

Action	z	Q/	Asb.y	Asb.z	Asb.T	Asl.T	Asl	Situation
max	[m]	Angle VRdct	[cm <sup>2</sup> /m]	[cm <sup>2</sup> /m]	[cm <sup>2</sup> /m]	[cm <sup>2</sup> ]	[cm <sup>2</sup> ]	
Qy	: 0.21	3.00	0.00	0.00	.	.	0.00	-, -
Qz	: 0.51	3.00	0.47	.	2.79	.	0.00	PC.1,2

Action	z	Qy/	Qz/	Mx/	Q/VRd+	Situation
max	[m]	Angle VRdmax	VRdmax	TRdmax	Mx/TRd	
Qy	: 0.21	3.00	0.00	.	.	-, -
Qz	: 0.51	3.00	.	0.05	.	PC.1,2

#### Check of crack widths

The check is led by direct calculation of the crack width.  
The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

(CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

wk,per Permissible crack width as per specification [mm]  
ds Maximal given steel diameter [mm]  
fct,eff Concrete strength at date of cracking [MN/m<sup>2</sup>]  
Sigma.c Maximal concrete edge stress in state I [MN/m<sup>2</sup>]  
wk Calculated value of crack width as per 11.2.4 [mm]  
sr,max Calculated / given maximal crack spacing as per 11.2.4 (4) [mm]  
Ac,eff Effective region of reinf. [m<sup>2</sup>] acc. to Pic. 53  
As,eff Reinforcing steel within Ac,eff [cm<sup>2</sup>]  
Ap,eff Prestressing steel with bond within Ac,eff [cm<sup>2</sup>]  
Sigma.s Reinf. steel stress in state II [MN/m<sup>2</sup>]

#### Location 1

Beam 70, x = 0.00 m (Beam length 0.83 m)  
Cross-section 2: Polygon - C30/37  
wk,per=0.3; ds=l2; fct,eff=2.9

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross	: 0.460	0.850	0.178	0.0107	0.0828	0.0000

#### 1. Quasi-continuous combination (QC.1): G+QN, Final state

Relevant concrete internal forces from 4 sets of internal forces

```
Set      Nx[kN]  My[kNm]  Mz[kNm]
2       : 0.00   107.92   0.00
```

Load case combinations for the relevant sets of internal forces

```
Set      Combination
2       : L1+0.30*L2+0.30*L3
```

#### Check of crack width for reinf. layer 4 (bottom)

Nx	: 0.00 kN	As,eff	: 11.15 cm <sup>2</sup>
My	: 107.92 kNm	Ap,eff	: . cm <sup>2</sup>
Mz	: 0.00 kNm	Ac,eff	: 0.023 m <sup>2</sup>
Sigma.c	: 4.25 MN/m <sup>2</sup>	Sigma.s	: 176.55 MN/m <sup>2</sup>
Situation	: QC.1,2	sr,max	: 67.37 mm
		wk	: 0.05 per. 0.30 mm

#### Check of concrete compressive stress

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not treatable on cracked section, (I\*) is marked.

fck Characteristic compressive concrete strength [MN/m<sup>2</sup>]  
Sigma.x,min Total maximal longitudinal compressive stress [MN/m<sup>2</sup>]  
Sigma.x,zul = 0.60\*fck resp. 0.66\*fck for Charact. C. (CC) as per 11.1.2 (1)  
top, bottom Position of the edge point: above, below of centre

#### Location 1

Beam 70, x = 0.00 m (Beam length 0.83 m)  
Cross-section 2: Polygon - C30/37  
0.6\*fck=18

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross	: 0.460	0.850	0.178	0.0107	0.0828	0.0000

#### 1. Characteristic (rare) combination (CC.1): G+QN, Final state

Relevant concrete internal forces from 4 sets of internal forces

```
Set      Nx[kN]  My[kNm]  Mz[kNm]
1       : 0.00   69.95   0.00
2       : 0.00  196.53   0.00
```

Load case combinations for the relevant sets of internal forces

```
Set      Combination
1       : L1
2       : L1+L2+L3
```

Check of compressive stress in concrete for the Characteristic (rare) combination

Side	Se.- Pnt.	Sigma.x,min [MN/m <sup>2</sup> ]	Sigma.x,per [MN/m <sup>2</sup> ]	Period	Situation
top	4 (II)	-6.37	-18.00	Final	CC.1,2
bottom	7 (II)	0.00	-18.00	Final	CC.1,1

Check of steel stress

For the check, a cracked concrete section is assumed.

Type S Long. reinf. from N and M, layer number, Charact. C. (CC)  
 fck Concrete strength to determine the strain state [MN/m<sup>2</sup>]  
 Sigma.s,per = 0.80 \* fyk resp. 1.0 \* fyk (CK) as per 11.1.3

Location 1

Beam 70, x = 0.00 m (Beam length 0.83 m)  
 Cross-section 2: Polygon - C30/37  
 fck=30; Steel 2

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

1. Characteristic (rare) combination (CC.1): G+QN, Final state

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
1 :	0.00	69.95	0.00
2 :	0.00	196.53	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1 :	L1
2 :	L1+L2+L3

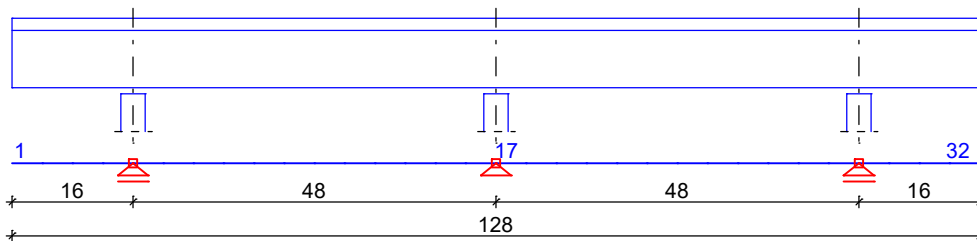
Check of steel stress

Steel Type No.	Nx [kN]	My [kNm]	Mz [kNm]	As [cm <sup>2</sup> ]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]	Situation
S 1	0.00	69.95	0.00	0.00	.	400.00	CC.1,1
S 2	0.00	69.95	0.00	0.00	.	400.00	CC.1,1
S 3	0.00	196.53	0.00	5.57	321.62	400.00	CC.1,2
S 4	0.00	196.53	0.00	5.57	321.62	400.00	CC.1,2

## Prestressed Roof Construction

This example involves the wide-spanned roof construction of an entrance hall that is represented as a continuous girder over two spans with a double-sided cantilever. A T-beam is selected as the section. The figure below shows the system in longitudinal and lateral section view.

Limited prestressing with subsequent bond is applied to the roof construction in the longitudinal direction. Prestressing in the lateral direction is not applied for reasons of economy. The construction is designed to meet requirement class D. According to Table 18 of the DIN 1045-1, a decompression check is not necessary for this class.

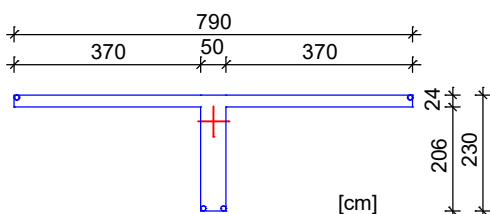


Static system and dimensions [m]

Material

Concrete C45/55  
 Reinforcing steel BSt 500, axis distance from edge 5 cm

Section

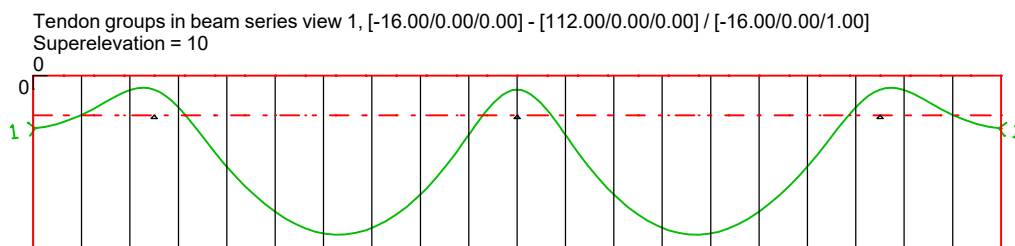


**Prestressing steel and prestressing system**

Prestressing steel quality	St 1500/1770
Certification of the prestressing system	DIN 1045-1, Cona 1206
Number of tendons in the bundle	4
Section surface $A_p$	1800 mm <sup>2</sup>
E-modulus of the prestressing steel	195000 MN/m <sup>2</sup>
0.1% strain limit (yield strength) of the prestressing steel $f_{p0.1k}$	1500 MN/m <sup>2</sup>
Tensile strength of the prestressing steel $f_{pk}$	1770 MN/m <sup>2</sup>
Permissible prestressing force of a tendon $P_{m0}$	2295 kN
Prestressing loss from relaxation of prestressed steel	4.5 %
Friction coefficients when prestressing and releasing $\mu$	0.2
Unintentional deviation angle of a tendon $\beta'$	0.3 °/m
Slippage at prestressing anchor	6 mm
Duct diameter $d_h$	82 mm
Allowance value for ensuring an overstressing reserve $\kappa$	1.5

Variation coefficients of the internal prestressing as per DIN 1045-1, Eq. (52), (53)	
Construction stage according to Book 525 ( $r_{sup} / r_{inf}$ )	1.0 / 1.0
Final state ( $r_{sup} / r_{inf}$ )	1.1 / 0.9

The tendon guide is shown in the next figure. 4 bundled tendons are arranged such that they stretch across the entire girder length and are prestressed at both girder ends. The prestressing system, prestressing procedure and prestressing curve for a tendon group are also shown.



Tendon group ordinates  $z_v$  [cm] at the base points

xv	0.00	6.40	12.80	19.20	25.60	32.00	38.40	44.80	51.20	57.60	64.00	70.40	76.80	83.20	89.60	96.00	102.40	108.80	115.20	121.60	128.00
1	70.0	52.0	19.2	42.0	120.2	180.1	208.8	201.5	157.4	78.3	18.5	78.3	157.4	201.5	208.8	180.1	120.2	42.0	19.2	52.0	70.0

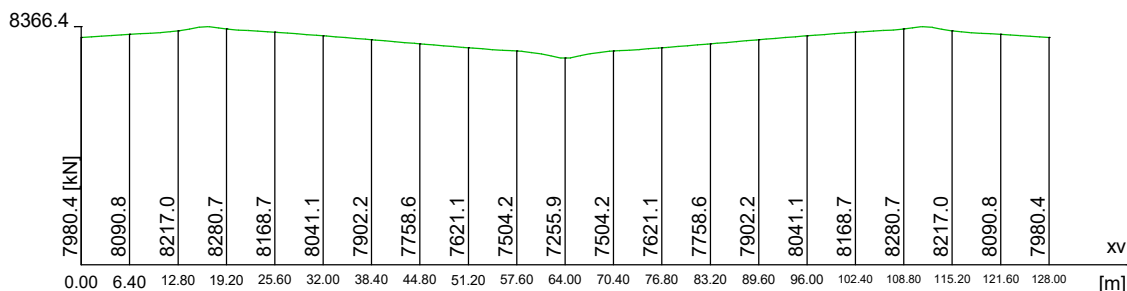
Force function of tendon group 1 (4 tendon(s),  $l = 128.41$  m)

Prestressing system 1 - CONA 1206. Certification according to DIN 1045-1.  
 $P_{m0} = 2295.0$  kN,  $A_p = 1800.0$  mm<sup>2</sup>,  $\mu_a = 0.20$ , Angle  $\beta' = 0.30$  °/m  
 E-Modulus = 195000 MN/m<sup>2</sup>,  $A_h = 5281.0$  mm<sup>2</sup>,  $\mu_n = 0.20$ , Slippage = 6.00 mm

Prestressing procedure 1 - Example

Tensioning with  $P_{max}$  (DIN Report, DIN 1045-1, DIN EN 1992-1-1).  $\kappa = 1.5$ .

Pre. anchor	:	Start	End
Normal. force	:	1.000	1.000
Pre. force [kN]	:	2203.9	2203.9
Extension [mm]	:	667.0	70.0



Tendon guide and prestressing curve in the longitudinal section (4 tendons).

**Loads**

Load case 1	Dead load
Load case 2	Additional dead load: $q=11.06$ kN/m
Load case 3	Snow load: $q=7.90$ kN/m
Load case 10	Prestressing
Load case 15	Creep-generating permanent load: Dead load, additional dead load and prestressing
Load case 20	Creep and shrinkage Coefficients: $\varphi_{t_{\infty}} = 2.55$ ; $\rho = 0.8$ ; $\epsilon_{t_{\infty}} = -24.8 \cdot 10^{-5}$ Creep-generating permanent load case: 15 The redistribution of internal forces between concrete and prestressing steel are taken into account.

**DIN 1045-1 actions****Standard design group****G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

1 Dead load

**G - Additional dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

2 Additional dead load

**P - Prestressing**

Gamma.sup / gamma.inf = 1 / 1

Load cases internal prestressing

10 Prestressing

**CSR1 - Creep, shrinkage, relaxation**

Prestressing loss from relaxation of prestressed steel: 4.5 %.

Load cases

20 Creep, shrinkage

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures

Snow and ice load - places to NN + 1000 m

Psi.0 / Psi.1 / Psi.2 = 0.5 / 0.2 / 0

Load cases 1. Variant, inclusive

3 Snow load

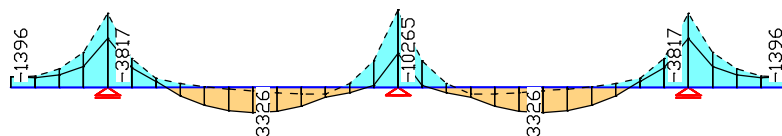
In this example all possible combinations of load cases are generated and designed. This method is selected in the calculation settings and can be very slow when applied for a large number of load cases.

Below you will find an example of the curve of bending moment  $M_y$  for design situations in the ultimate limit states.

**1. Permanent and temporary situation - Structural cond.**

Construction stage - UngROUTED

G Dead load  
P Prestressing

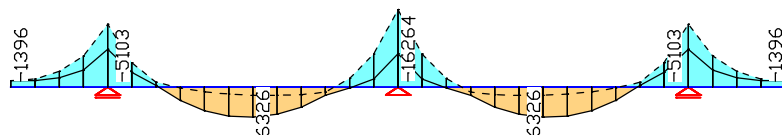


Bending moment  $M_y$  [kNm]

**2. Permanent and temporary situation - t0**

Final state

- G Dead load
- G Additional dead load
- P Prestressing
- QS Snow and ice load

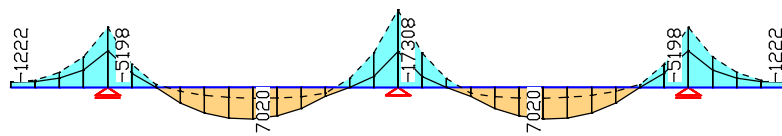


Bending moment  $M_y$  [kNm]

**3. Permanent and temporary situation - t00**

Final state

- G Dead load
- G Additional dead load
- P Prestressing
- CSR1 Creep, shrinkage, relaxation
- QS Snow and ice load



Bending moment  $M_y$  [kNm]

**Design according to DIN 1045-1 (2008-08)**

**Design overview**

Se.	Class, Prestress of component	Reinforc. M R B Q T	Fatig. S P C	Crack- width	De- comp.	Stress checks Comp. S P
1	D supplm. bond	x x x x .	. . . .	x	.	x x x

- (M) Nominal reinforcement to guarantee robustness (ductility).
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state.
- (T) Torsional reinforcement at ultimate limit state.
- (S) Reinforcing steel at stress and fatigue check.
- (P) Prestressing steel at stress and fatigue check.
- (C) Concrete at fatigue check.

**Dispersion of prestressing**

The dispersion of prestressing is considered at the following checks:

- Check of decompression
- Nominal reinforcement for crack width limitation
- Check of crack width

All other checks are made using the mean value  $P_{m,t}$  of prestressing.

Se.	Prestressing of component	Const.period r.sup	Final state r.inf	r.sup	r.inf
1	supplm. bond	1.00	1.00	1.10	0.90

**Settings for flexural and shear reinforcement**

- M,N Design mode for bend and longitudinal force: (ST) Standard, (SY) Symmetrical, (CM) Compression member.
- fyk Quality of stirrups.
- Theta Angle of concrete truss. Program-sided, the given value of cot Theta is limited to the value range according to Eq. (73).
- Slabs Beams are designed like slabs.
- Asl Given reinforcement according to picture 32, increase to maximum.
- rhov Minimum reinf. min rhov = Factor \* rho with rho according to table 29.
- x,y Separate lateral force design for reinforcement directions x and y.
- cvl Laying measure of the long. reinforcement to limit the lever arm z.
- Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den- sity [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] Pic. 32 given max	Fac. rhov	Dsn. x,y [mm]	cvl 50	Red. pre- str.
1	C45/55	.	ST	500	3.00	.	0.00	.	1.60	.	.

**Shear sections**

bw.nom Nominal width of the prestressed section acc. to 10.3.4 (8).  
 h.nom Nominal height of the prestressed section acc. to 10.3.4 (8).  
 z1, z2 Height and width of the core section for torsion.  
 teff Thickness of the torsion box.  
 B. Box section.

Se.	Width [m]	Eff. width bw	Eff. width bn [m]	Height [m] h	h.nom	Eff.height d [m]	Torsion section [m] z1	z2	teff	B.
1	0.500	0.380	0.450	2.300	2.220	2.250	2.200	0.400	0.100	.

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel.  
 max.s Maximal given bar spacing of the reinforcing steel.  
 Xi1 Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 sr,max Upper limit for the crack spacing from equ. (137).  
 Method Direct calculation of the crack width as per chapt. 11.2.4 or  
 check by limiting the bar spacing according to table 21.  
 TM Thick member according to chapt 11.2.2(8) to determine As,min.

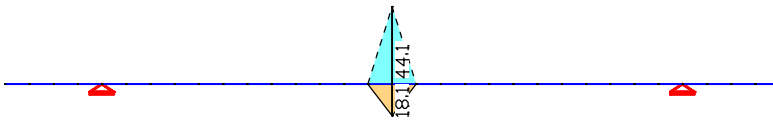
Se.	wk [mm]	per ds [mm]	max.s [mm]	Coeff.	Xi1	k	sr,max [mm]	Concr. age	Method for crack w.	Tensile zone	TM
1	0.20	20	.	0.27	1.00	.	3- 5d	3- 5d	Calcul.	Cmb. per class	.

**Settings for the check of concrete stresses**

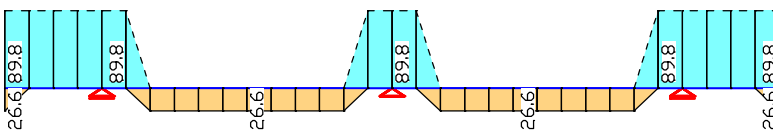
(CC) Characteristic combination (QC) Quasi-continuous combination

Se.	per.sigma.c (CC)	per.sigma.c (QC)	Dekompression Stress Sigma.x
1	0.60 fck	0.45 fck	Sigma.x

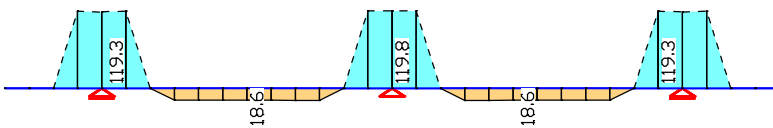
The following illustrations show the curve of the required bending and shear reinforcement.



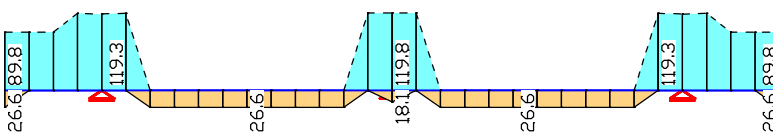
Longitudinal reinforcement  $A_s$  from the design in the ultimate limit states [cm²]  
 (upper reinforcement with dashed lines).



Minimum reinforcement  $A_s$  for ensuring robustness (ductility) [cm²]  
 (upper reinforcement with dashed lines).

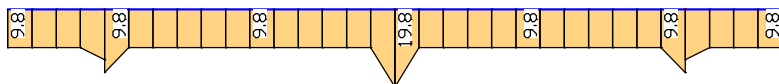


Reinforcement  $A_s$  for limiting the crack width [cm²]  
 (upper reinforcement with dashed lines).



Enclosing reinforcement  $A_s$  from the checks [cm²]  
 (upper reinforcement with dashed lines).





(Minimum) lateral force reinforcement  $A_{sb,z}$  in the ultimate limit states [cm<sup>2</sup>/m].

The following pages contain excerpts from the detailed check log for beam 16 at location 2 (middle column).

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. for robustness as per 13.1.1 (Charact. C.)
  - fctm Average centric concrete tensile strength [MN/m<sup>2</sup>]
  - zs,t/b Lever arm of inner strengths top/bottom with zs=0,9\*d [m]
  - fyk,t/b Strength of longitudinal reinforcement top/bottom [MN/m<sup>2</sup>]
  - max Sc Maximum concrete edge stress from Charact. C. [MN/m<sup>2</sup>]  
without the statically determined part of prestressing
- (R) Nominal/requ. reinforcement as per 11.2.2 for crack width limitation  
Increase of reinforcement due to crack width check is marked by "!".
  - wk,per Permissible crack width as per specification [mm]
  - ds Maximal given steel diameter [mm]
  - k Coefficient for consideration of non-linear distributed tensile stress
  - fct,eff Concrete strength at date of cracking [MN/m<sup>2</sup>]
  - kc Coefficient to consider stress distribution in tensile zone  
acc. to Eq. (128) resp. Eq. (128a)
  - Ap' Part of prestr. steel area  $X_{l1} \cdot A_p$  which was used to reduce req.As
  - Xl1 Bond coefficient for prestressing steel as per Eq. (130)
  - max Sx Maximal concrete edge stress from action combination [MN/m<sup>2</sup>]
- (B) Design of reinforcement at ultimate limit state  
In case of dominant bending, compression reinforcement is marked with "\*\*".
  - fck Concrete strength for design of reinforcement [MN/m<sup>2</sup>]
  - N0, M0 Statically determined forces of tendons with bond [kN, kNm]
  - fp0.1k Charact. value of the 0.1% strain limit of the prestr. steel [MN/m<sup>2</sup>]
  - fpk Charact. value of the tensile strength of the prestr. steel [MN/m<sup>2</sup>]

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - C45/55, 1 tendon group with bond  
 Steel 1; Design mode: Standard  
 (M) fctm=3.8; zs,t/b=2.025/2.025; fyk,t/b=500/500  
 (R) wk,per=0.2; ds=20; k=1; fct,eff=1.9; Xl1=0.27  
 r.sup/inf(Constr.)=1/1; r.sup/inf(Final)=1.1/0.9  
 (B) fck=45

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.962	3.950	0.521	1.2601	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7255.93	0.00

**1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted**

Relevant concrete internal forces from 1 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
1	-7255.87	-4530.46	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L10

**2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.63	-.	-.	-.	-.	-.	-.

Stat. determ. part (P+CSR): Nx0=-6557.10 kN; My0=2229.41; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces

Set	Concrete section			Bond section		
	Nx [kN]	My [kNm]	Mz [kNm]	Nx [kN]	My [kNm]	Mz [kNm]
2	-6430.65	-9821.11	0.00	126.45	-12050.52	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	L1+L2+0.96*L10+L20+L3

**1. Frequent combination (TC.1): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**2. Frequent combination (TC.2): G.1+G.2+P+CSR1+QS, Final state grouted**

Relevant concrete internal forces from 4 sets of internal forces  
 Set      Nx[kN]      My[kNm]      Mz[kNm]  
 2      : -5787.59   -9486.58      0.00 r.inf

Load case combinations for the relevant sets of internal forces  
 Set      Combination  
 2      : L1+L2+0.96\*L10+L20+0.20\*L3

**1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted**

No set of internal forces in this situation was relevant.

**2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups  
 No.   CSR[%]   No.   CSR[%]   No.   CSR[%]   No.   CSR[%]  
 1    9.63       -.-       -.-       -.-       -.-

Stat. determ. part (P+CSR): Nx0=-6557.10 kN; My0=2229.41; Mz0=0.00 kNm

Relevant values from 8 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6430.65	-17307.97	0.00	126.45	-19537.39	0.00

Load case combinations for the relevant sets of internal forces  
 Set      Combination  
 2      : 1.35\*L1+1.35\*L2+0.96\*L10+L20+1.50\*L3

**Design of longitudinal reinforcement**

Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	max Sc [MN/m <sup>2</sup> ]	kc	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	Situation
1	M	126.45	-12050.52	0.00	5.08	.	44.91	CC.3,2	
	R	-5787.59	-9486.58	0.00	1.99	0.50	59.91	TC.2,2	
	B	-6430.65	-17307.97	0.00	.	.	22.06	PC.3,2	
2	M	126.45	-12050.52	0.00	5.08	.	44.91	CC.3,2	
	R	-5787.59	-9486.58	0.00	1.99	0.50	59.91	TC.2,2	
	B	-6430.65	-17307.97	0.00	.	.	22.06	PC.3,2	
3	M	0.06	-6997.47	0.00	.	.	0.00	CC.1,1	
	R	-5787.59	-9486.58	0.00	1.99	.	0.00	TC.2,2	
	B	-6430.65	-17307.97	0.00	.	.	9.03*	PC.3,2	
4	M	0.06	-6997.47	0.00	.	.	0.00	CC.1,1	
	R	-5787.59	-9486.58	0.00	1.99	.	0.00	TC.2,2	
	B	-6430.65	-17307.97	0.00	.	.	9.03*	PC.3,2	

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to 13.2.3 (5) is considered.

- Ac      Section area for calculating the concrete stress from long. force [m<sup>2</sup>]
- bw      Effective width for calculation of shear stresses from Qz and Mx [m]
- bw.nom   Nominal value of the width when deducting the duct diameter [m]
- bn      Statically effective width for shear design using Qy [m]
- kb      Factor to calculate the inner lever arm from bn
- h      Effective height for calculation of shear stresses from Qy and Mx [m]
- h.nom   Nominal value of the height when deducting the duct diameter [m]
- d      Statically effective height for shear design using Qz [m]
- kd      Factor to calculate the inner lever arm from d
- Angle   Angle cot Theta between the compressive strut and the beam axis
- Asl.giv.   Chargeable longitudinal reinf. acc. to Pic. 32 [cm<sup>2</sup>]
- rho.min   Minimal percentage of lateral reinforcement acc. to 13.2.3 (5)
- Qy, Qz   Lateral forces for design in y- and z-direction [kN]
- VRdct   Absorbable lat. force without lat. reinf. per 10.3.3 (1) [kN]
- VRdmax   Absorbable lateral force of comp. struts per 10.3.4 (7) [kN]
- z      Inner lever arm z=kb\*bn resp. z=kd\*d [m]; z<=max(d-2c<sub>vl</sub>,d-c<sub>vl</sub>-30mm)
- c<sub>vl</sub>   Laying measure of the long. reinforcement to limit the lever arm z [m]
- Asb.y,z   Req. stirrup reinforcement from Qy resp. Qz [cm<sup>2</sup>/m]
- Asl      Req. longitudinal reinf. acc. to Pic. 32 [cm<sup>2</sup>] for req.Asb

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - C45/55, 1 tendon group with bond  
 bw/bw.nom/bn/kb=0.5/0.38/0.45/0.9; h/h.nom/d/kd=2.3/2.22/2.25/0.9  
 c<sub>vl</sub>=0.05; f<sub>yk</sub>=500; Asl.giv./max=0/0; min rho=1.6\*rho

**1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted**

No set of internal forces in this situation was relevant.

**2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Relevant concrete internal forces from 8 sets of internal forces  
 Set      Nx[kN]      My[kNm]      Mz[kNm]      Mx[kNm]      Qy[kN]      Qz[kN]  
 2      : -6430.65   -17307.97      0.00      0.00      0.00      -3080.23

Load case combinations for the relevant sets of internal forces  
 Set      Combination  
 2      : 1.35\*L1+1.35\*L2+0.96\*L10+L20+1.50\*L3

## Check of the shear reinforcement and the compressive struts

Action	z	Q/	Asb.y	Asb.z	Asb.T	Asl.T	Asl	Situation
max	[m]	Angle VRdct	[cm <sup>2</sup> /m]	[cm <sup>2</sup> /m]	[cm <sup>2</sup> /m]	[cm <sup>2</sup> ]	[cm <sup>2</sup> ]	
Qy	: 0.37	3.00	0.00	0.00	.	.	0.00	-, -
Qz	: 2.02	1.76	5.35	.	19.83	.	0.00	PC.3,2

Action	z	Qy/	Qz/	Mx/	Q/VRd+	Situation
max	[m]	Angle VRdmax	VRdmax	TRdmax	Mx/TRd	
Qy	: 0.37	3.00	0.00	.	.	-, -
Qz	: 2.02	1.76	.	0.49	.	PC.3,2

## Check of crack widths

The check is led by direct calculation of the crack width.  
The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

(CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

wk,per	Permissible crack width as per specification [mm]
ds	Maximal given steel diameter [mm]
fct,eff	Concrete strength at date of cracking [MN/m <sup>2</sup> ]
Sigma.c	Maximal concrete edge stress in state I [MN/m <sup>2</sup> ]
wk	Calculated value of crack width as per 11.2.4 [mm]
sr,max	Calculated / given maximal crack spacing as per 11.2.4 (4) [mm]
Ac,eff	Effective region of reinf. [m <sup>2</sup> ] acc. to Pic. 53
As,eff	Reinforcing steel within Ac,eff [cm <sup>2</sup> ]
Ap,eff	Prestress steel with bond within Ac,eff [cm <sup>2</sup> ]
Sigma.s	Reinf. steel stress in state II acc. to Equ. (132) [MN/m <sup>2</sup> ]
Xil	Bond coefficient for prestressing steel as per Eq. (130)

## Location 2

Beam 16, x = 4.00 m (Beam length 4.00 m)  
Cross-section 1: Polygon - C45/55, 1 tendon group with bond  
wk,per=0.2; ds=20; fct,eff=1.9; Xil=0.27  
r.sup/inf(Constr.)=1/1; r.sup/inf(Final)=1.1/0.9

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross	: 2.926	3.950	0.525	1.2560	9.8822	0.0000
net	: 2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.962	3.950	0.521	1.2601	9.8822	0.0000

## Tendon groups with bond

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,lk [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7255.93	0.00

## 1. Frequent combination (TC.1): G.1+G.2+P+QS, Final state grouted

No set of internal forces in this situation was relevant.

## 2. Frequent combination (TC.2): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.63	-.	-.	-.	-.	-.	-.

Stat.determ.part (P+CSR)\*r.inf: Nx0=-5901.39 kN; My0=2006.47; Mz0=0.00 kNm

Relevant values from 4 sets of internal forces

Set	Concrete section			Bond section			
	Nx [kN]	My [kNm]	Mz [kNm]	Nx [kN]	My [kNm]	Mz [kNm]	
2	: -5787.59	-9486.58	0.00	113.80	-11493.05	0.00	r.inf

Load case combinations for the relevant sets of internal forces

Set	Combination
2	: L1+L2+0.96*L10+L20+0.20*L3

## Check of crack width for reinf. layer 1 (top)

Nx	: -5787.59 kN	As,eff	: 119.83 cm <sup>2</sup>
My	: -9486.58 kNm	Ap,eff	: 0.00 cm <sup>2</sup>
Mz	: 0.00 kNm	Ac,eff	: 0.987 m <sup>2</sup>
Sigma.c	: 1.99 MN/m <sup>2</sup>	Sigma.s	: 63.13 MN/m <sup>2</sup>
Situation	: TC.2,2	sr,max	: 184.60 mm
		wk	: 0.03 per. 0.20 mm

## Check of concrete compressive stress

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not treatable on cracked section, (I\*) is marked.

fck	Characteristic compressive concrete strength [MN/m <sup>2</sup> ]
Sigma.x,min	Total maximal longitudinal compressive stress [MN/m <sup>2</sup> ]
Sigma.x,zul	= 0.60*fck resp. 0.66*fck for Charact. C. (CC) as per 11.1.2 (1) = 0.45*fck for Q.-cont. C. (QC) as per 11.1.2 (2)
top, bottom	Position of the edge point: above, below of centre

## Location 2

Beam 16, x = 4.00 m (Beam length 4.00 m)  
Cross-section 1: Polygon - C45/55, 1 tendon group with bond  
0.45\*fck=20.25; 0.6\*fck=27

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross	: 2.926	3.950	0.525	1.2560	9.8822	0.0000
net	: 2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.962	3.950	0.521	1.2601	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7255.93	0.00

**1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted**

Relevant concrete internal forces from 1 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
1	-7255.87	-4530.46	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L10

**2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.63	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR): Nx0=-6557.10 kN; My0=2229.41; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6430.65	-9821.11	0.00	126.45	-12050.52	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	L1+L2+0.96*L10+L20+L3

**1. Quasi-continuous combination (QC.1): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.63	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR): Nx0=-6557.10 kN; My0=2229.41; Mz0=0.00 kNm

Relevant values from 1 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
1	-6430.65	-8051.51	0.00	126.45	-10280.92	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L2+0.96*L10+L20

**Check of compressive stress in concrete for the Characteristic (rare) combination**

Side	Se.- Pnt.	min [MN/m <sup>2</sup> ]	Sigma.x per. [MN/m <sup>2</sup> ]	Sigma.x Period	Situation
top	1 (I)	-0.59	-27.00	Constr.	CC.1,1
bottom	7 (I)	-16.04	-27.00	Final	CC.3,2

**Check of compressive stress in concrete for the Quasi-continuous combination**

Side	Se.- Pnt.	min [MN/m <sup>2</sup> ]	Sigma.x per. [MN/m <sup>2</sup> ]	Sigma.x Period	Situation
top	1 (I)	1.16	-20.25	Final	QC.1,1
bottom	7 (I)	-13.54	-20.25	Final	QC.1,1

**Check of steel stress**

For the check, a cracked concrete section is assumed.  
For tendon groups without bond and/or for situations before grouting,  
the prestressing steel stress is checked acc. to Eq. (49).

Type S Long. reinf. from N and M, layer number, Charact. C. (CC)

Type P Prestressing steel, Tendon number, Q.-cont. C. (QC)

and Charact. C. (CC)

N0, M0 Statically determined forces of tendons with bond [kN, kNm]

fck Concrete strength to determine the strain state [MN/m<sup>2</sup>]

Sigma.s,per = 0.80 \* fyk resp. 1.0 \* fyk (CK) as per 11.1.3

Sigma.p,zul = 0.65 \* fpk (QK) resp. min(0.9 \* fp0.1k; 0.8 \* fpk) (CK) per 11.1.4

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)

Cross-section 1: Polygon - C45/55, 1 tendon group with bond  
fck=45; Steel 1

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.962	3.950	0.521	1.2601	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7255.93	0.00

**1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted**

Relevant concrete internal forces from 1 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
1	-7255.87	-4530.46	0.00

Load case combinations for the relevant sets of internal forces

Set      Combination  
1      : L1+L10

### 2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted

No set of internal forces in this situation was relevant.

### 3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.63	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR): Nx0=-6557.10 kN; My0=2229.41; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6430.65	-9821.11	0.00	126.45	-12050.52	0.00

Load case combinations for the relevant sets of internal forces

Set      Combination  
2      : L1+L2+0.96\*L10+L20+L3

### 1. Quasi-continuous combination (QC.1): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.63	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR): Nx0=-6557.10 kN; My0=2229.41; Mz0=0.00 kNm

Relevant values from 1 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
1	-6430.65	-8051.51	0.00	126.45	-10280.92	0.00

Load case combinations for the relevant sets of internal forces

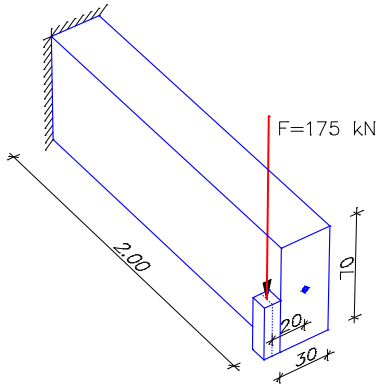
Set      Combination  
1      : L1+L2+0.96\*L10+L20

### Check of steel stress

Steel Type	No.	Nx [kN]	My [kNm]	Mz [kNm]	As [cm <sup>2</sup> ]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]	Situation
S	1	-6430.65	-9821.11	0.00	59.91	54.05	400.00	CC.3,2
S	2	-6430.65	-9821.11	0.00	59.91	54.05	400.00	CC.3,2
S	3	-7255.87	-4530.46	0.00	9.03	-48.90	400.00	CC.1,1
S	4	-7255.87	-4530.46	0.00	9.03	-48.90	400.00	CC.1,1
P	1	-6430.65	-8051.51	0.00	72.00	923.60	1150.50	QC.1,1
P	1	.	.	.	72.00	1007.77	1275.00	CC.1,--

## Torsional Beam

The depicted cantilever is subjected to an eccentrically acting load  $F = 175 \text{ kN}$ . The required shear, torsion longitudinal and stirrup reinforcements are listed in the following log.



System drawing

### Design according to DIN 1045-1 (2008-08)

#### Settings for flexural and shear reinforcement

M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 fyk Quality of stirrups.  
 Theta Angle of concrete truss. Program-sided, the given value of cot Theta is limited to the value range according to Eq. (73).  
 Slabs Beams are designed like slabs.  
 Asl Given reinforcement according to picture 32, increase to maximum.  
 rhov Minimum reinf. min rhov = Factor \* rho with rho according to table 29.  
 x,y Separate lateral force design for reinforcement directions x and y.  
 cvl Laying measure of the long. reinforcement to limit the lever arm z.  
 Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den-sity [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] Pic. 32 given max	Fac. rhov	Dsn. x,y [mm]	cvl	Red. pre-str.
1	C35/45	.	.	500	1.00	.	1.00	1.00	.	55	.

#### Shear sections

bw.nom Nominal width of the prestressed section acc. to 10.3.4 (8).  
 h.nom Nominal height of the prestressed section acc. to 10.3.4 (8).  
 z1, z2 Height and width of the core section for torsion.  
 teff Thickness of the torsion box.  
 B. Box section.

Se.	Width [m]	Eff. width bw.nom	Eff. width bn [m]	Height [m] h	Eff.height h.nom	Torsion section [m] d	z1	z2	teff	B.
1	0.300	.	0.245	0.700	.	0.645	0.590	0.190	0.110	.

#### Design of shear reinforcement

The percentage of nominal reinforcement acc. to 13.2.3 (5) is considered.

Ac Section area for calculating the concrete stress from long. force [m²]  
 bw Effective width for calculation of shear stresses from Qz and Mx [m]  
 bn Statically effective width for shear design using Qy [m]  
 kb Factor to calculate the inner lever arm from bn  
 h Effective height for calculation of shear stresses from Qy and Mx [m]  
 d Statically effective height for shear design using Qz [m]  
 kd Factor to calculate the inner lever arm from d  
 z1, z2 Height and width of the core section Ak for torsion [m]  
 teff Wall thickness of the torsion section [m]  
 Angle Angle cot Theta between the compressive strut and the beam axis  
 Asl.giv. Chargeable longitudinal reinf. acc. to Pic. 32 [cm²]  
 rhov,min Minimal percentage of lateral reinforcement acc. to 13.2.3 (5)  
 Qy, Qz Lateral forces for design in y- and z-direction [kN]  
 VRdct Absorbable lat. force without lat. reinf. per 10.3.3 (1) [kN]  
 VRdmax Absorbable lateral force of comp. struts per 10.3.4 (7) [kN]  
 z Inner lever arm  $z = kb \cdot bn$  resp.  $z = kd \cdot d$  [m];  $z \leq \max(d - 2cvl, d - cvl - 30\text{mm})$   
 cvl Laying measure of the long. reinforcement to limit the lever arm z [m]  
 Asb.y,z Req. stirrup reinforcement from Qy resp. Qz [cm²/m]  
 Asl Req. longitudinal reinf. acc. to Pic. 32 [cm²] for req.Asb  
 Mx Torsional moment for design [kNm]  
 Q/VRd+ Block section:  $(Q/VRdmax)^2 + (Mx/TRdmax)^2$   
 Mx/TRd Box section:  $Q/VRdmax + Mx/TRdmax$   
 TRdmax Maximum absorbable torsional moment as per 10.4.2 (4) [kNm]  
 Asb.T Req. stirrup reinforcement from torsion [cm²/m]  
 Asl.T Req. longitudinal reinforcement from torsion [cm²]

**Beam 1****Location 1**

Beam 1, x = 0.00 m (Beam length 2.00 m)  
 Cross-section 1: Polygon - C35/45  
 bw/bn/kb=0.3/0.245/0.9; h/d/kd=0.7/0.645/0.9  
 cv1=0.055; fyk=500; As1 giv./max=1/0; min rhov=1\*rhov  
 Block section z1/z2=0.59/0.19; teff=0.11

**1. Permanent and temporary comb. (PC.1): G, Final state**

Check of the shear reinforcement and the compressive struts

Action	z	Q/ Angle VRdct	Asb.y [cm <sup>2</sup> /m]	Asb.z [cm <sup>2</sup> /m]	Asb.T [cm <sup>2</sup> /m]	Asl.T [cm <sup>2</sup> ]	Asl [cm <sup>2</sup> ]	Situation
max, cor.	[m]	1.00 0.00	.	.	.	.	.	-, -
Qy, Mx	: 0.16	1.00 0.00	.	.	.	.	.	-, -
Mx, Qy	: 0.16	1.00 0.00	0.00	.	4.85	7.56	1.00	PC.1, Qz+
Qz, Mx	: 0.56	1.00 3.24	.	9.70	4.85	7.56	1.00	PC.1, Qz+
Mx, Qz	: 0.56	1.00 3.24	.	9.70	4.85	7.56	1.00	PC.1, Qz+

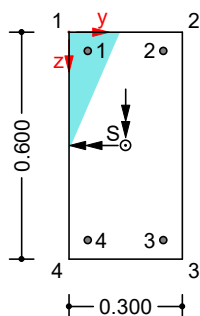
Action	z	Qy/ Angle VRdmax	Qz/ VRdmax	Mx/ TRdmax	Q/VRd+ Mx/TRd	Situation
max	[m]	1.00 0.00	.	.	.	-, -
Qy	: 0.16	1.00 0.00	.	.	.	-, -
Qz	: 0.56	1.00 .	0.19	.	.	PC.1, Qz+
Mx	: .	1.00 .	.	0.37	.	PC.1, Qz+
Qy "+" Mx	: 0.16	1.00 0.00	.	0.37	0.14	PC.1, Qz+
Qz "+" Mx	: 0.56	1.00 .	0.19	0.37	0.17	PC.1, Qz+

**Single Design Reinforced Concrete**

A single rectangular section is designed under bending and normal force.

**Pos. 1 - Reinforced concrete design per DIN 1045-1**

Section 1

Sc. = 1 : 20  
Pressure

Action N = 10.00 kN; My = 67.50; Mz = 27.00 kNm  
 Resistance N = 10.00 kN; My = 67.50; Mz = 27.00 kNm  
 Force system ys / zs = 0.150 / 0.300 m  
 Strength C20/25; gamma.c = 1.50; gamma.s = 1.15  
 Design mode Standard  
 Reinforcement 3.87 cm<sup>2</sup>; 0.21 %; Concrete area = 1800.00 cm<sup>2</sup>

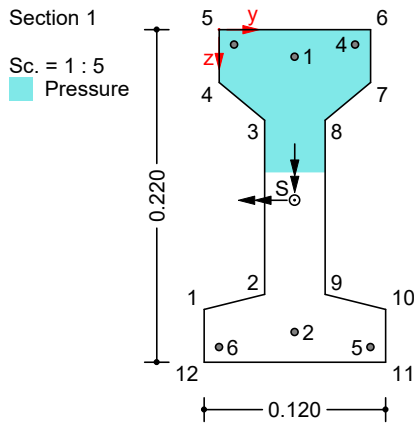
Concrete section	Point	y [m]	z [m]	eps[%]	sigma[MPa]	Inner Forces	y [m]	z [m]	F [kN]
1	0.000	0.000	-3.50	-11.33	Compr.	0.038	0.088	-160.15	
	0.135	0.000	0.00	0.00	Tension	0.203	0.498	170.15	
2	0.300	0.000	4.29	0.00	Lev. arm	0.165	0.409		
3	0.300	0.600	11.04	0.00					
4	0.000	0.600	3.24	0.00					
	0.000	0.312	0.00	0.00					

Reinforcement	Point	y [m]	z [m]	d1 [m]	Es, βs [MPa]	Zv0 [kN]	eps[%]	sigma[MPa]	As [cm <sup>2</sup> ]
1	0.050	0.050	0.050	200000	500	0.0	-1.64	-327.84	0.00
2	0.250	0.050	0.050	200000	500	0.0	3.56	436.10	0.41
3	0.250	0.550	0.050	200000	500	0.0	9.17	441.45	2.55
4	0.050	0.550	0.050	200000	500	0.0	3.98	436.50	0.91

# Single Design Prestressed Concrete

In the following example a failure safety check is performed on a prestressed concrete section. In this case tendon groups are defined as prestressed concrete steel layers 1 and 2.

## Pos. 1 - Reinforced concrete design per DIN 1045-1



Action N = 0.00 kN; My = 40.00; Mz = 0.00 kNm  
 Resistance N = -0.00 kN; My = 40.00; Mz = 0.00 kNm  
 Force system ys / zs = 0.050 / 0.113 m  
 Strength C45/55; gamma.c = 1.50; gamma.s = 1.15  
 Design mode Standard  
 Reinforcement 4.90 cm<sup>2</sup>; 3.30 %; Concrete area = 148.50 cm<sup>2</sup>  
 Prestress P0 N0 = -129.60 kN; My0 = -9.10; Mz0 = -0.00 kNm  
 Resistance + P0 N+N0 = -129.60 kN; My+My0 = 30.90; Mz+Mz0 = 0.00 kNm

Concrete section Point	y [m]	z [m]	eps[‰]	sigma[MPa]	Inner Forces	y [m]	z [m]	F [kN]
1	-0.010	0.185	3.35	0.00	Compr.	0.050	0.025	-224.39
2	0.030	0.175	2.98	0.00	Tension	0.050	0.203	224.39
3	0.030	0.095	0.00	0.00	Lev. arm	0.000	0.178	
4	0.030	0.060	-1.28	-22.17				
5	0.000	0.035	-2.20	-25.50				
6	0.000	0.000	-3.50	-25.50				
7	0.100	0.000	-3.50	-25.50				
8	0.100	0.035	-2.20	-25.50				
9	0.070	0.060	-1.28	-22.17				
10	0.070	0.094	0.00	0.00				
11	0.070	0.175	2.98	0.00				
12	0.110	0.185	3.35	0.00				
12	0.110	0.220	4.65	0.00				
12	-0.010	0.220	4.65	0.00				

Reinforcement Point	y [m]	z [m]	d1 [m]	Es,fyk [MPa]	Zv0 [kN]	eps[‰]	sigma[MPa]	As [cm <sup>2</sup> ]
1	0.050	0.018	0.018	205000 1420	12.0	-2.83	-280.83	0.40
2	0.050	0.200	0.020	205000 1420	117.6	3.91	1234.78	1.20
3	0.010	0.010	0.010	200000 500	0.0	-3.13	-435.69	0.78
4	0.090	0.010	0.010	200000 500	0.0	-3.13	-435.69	0.78
5	0.100	0.210	0.010	200000 500	0.0	4.28	436.79	0.87
6	0.000	0.210	0.010	200000 500	0.0	4.28	436.79	0.87

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# EN 1992-1-1 Design

## Basics

The reinforced concrete and prestressed concrete design specified in EN 1992-1-1 (Eurocode 2) can be used for buildings and engineering constructions under observance of the following standards:

- EN 1992-1-1:2004/A1:2014 as the base document
- DIN EN 1992-1-1:2015 with the National Annex Germany 2015-12
- OENORM EN 1992-1-1:2015 with the National Annex Austria B 1992-1-1:2018-01
- SS EN 1992-1-1:2014 with the National Annex Sweden 2019-01 (EKS 11)
- BS EN 1992-1-1:2014 with the National Annex Great Britain 2015-07

The desired rule is selected in the *Design Codes* dialog in the *Options* menu. The relevant entry, calculation and results dialogs appear depending on which rule is selected. When selecting the material the following alternatives are available:

- *C12/15-EN-D* to *C100/115-EN-D*, *LC12/13-EN-D* to *LC80/88-EN-D* and the user-defined material *CX-EN-D* for design in accordance with DIN EN 1992-1-1
- *C12/15-EN* to *C90/105-EN*, *LC12/13-EN* to *LC80/88-EN* and the user-defined material *CX-EN* for design in accordance with the other standards

Permitted structure models include beam, area and solid structures. Prestressed structures can only be checked in the FEM module.

Differing components can be combined in a structure model:

- Non-prestressed components
- Prestressed components with subsequent bond
- Prestressed components without bond
- Components with external prestressing
- Mixed-construction components

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with EN 1991:2002 (Eurocode 1), Part 1. The program will take into account the preset safety factors and combination coefficients defined in EN 1990:2021 (Eurocode 0) for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements.

The actions and check selection dialogs can be opened from the analysis settings. Detailed check specifications and reinforcement data must be entered during section definition.

For beams and design objects, all checks are carried out at the polygon section. In addition, composite sections can be verified in the ultimate limit state. For general notes on using design objects, refer to the relevant chapter in the manual.

In the *EN 1992-1-1 Design* folder of the database and the national variants folders, a single design can also be performed for the user-defined polygon and composite sections.

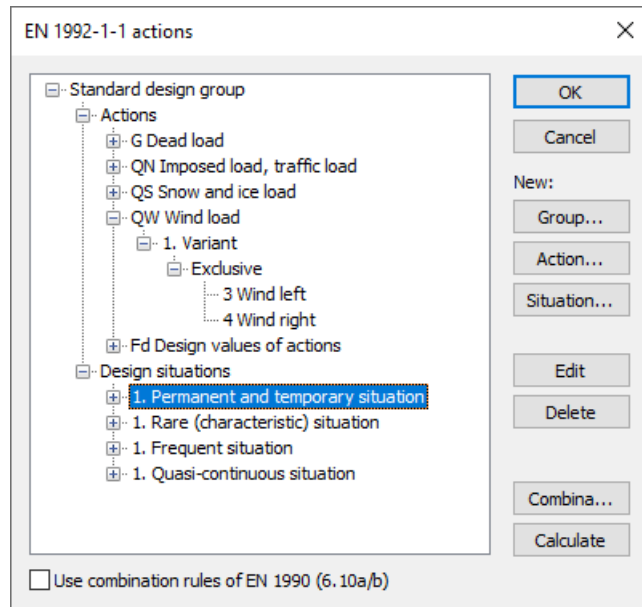
The EN 1992-1-1 guidelines are primarily cited for the following explanations. Reference to the relevant national settings is only made if they contain different or complementary rules. The passages in question are marked by a vertical line left of the text.

# Input

## Actions and Design Situations

The design values of the load are calculated based on the internal forces of individual load cases and load case combinations. For this the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



Action dialog for EN 1992-1-1 (national variants corresponding)

### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Prestressing (P)
- Creep and shrinkage, relaxation (CSR1, CSR2). These actions are only available if a P action has been defined. In the combinations they are treated, along with P, as a single action.
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)
 

The assigned load cases should contain a design-relevant set of loads with partial safety factors and combination coefficients such as for example a load group to take into account nonlinear effects. The selected load cases are combined exclusively.
- Cyclic fatigue actions (Qfat)

### Group...

Open the dialog for entering a new design group. According to e.g. standard EN 1991-1-1, Chapter 6.2.2 (2), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

### Situation...

Open the dialog for entering new design situations. Situations must be classified as either a construction stage or a final state in order to control the checking process. For prestressed concrete structures with subsequent bond, you can specify that the tendons are still ungrouted.

### Edit

Open the Edit dialog for the selected action or situation.

### Delete

Delete the selected action or situation.

**Combinations...**

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

The following example shows the total variants of the *permanent and temporary situation* according to Eq. (6.10) to be examined with the load cases (L1...L6) involved and their weighting factors.

Actions	Load cases	$\gamma_{sup}$	$\gamma_{inf}$	$\Psi_0$
Dead load	1	1.35	1.0	-
Imposed load, traffic load	2, 3 (inclusive)	1.5	0	0.7
Wind load	4	1.5	0	0.6
$F_d$ Design values of actions	5, 6	1.0	1.0	-

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
1	1.35					
2	1.35	1.50	1.50	0.90		
3	1.35	1.50	1.50			
4	1.35	1.50		0.90		
5	1.35	1.50				
6	1.35		1.50	0.90		
7	1.35		1.50			
8	1.35	1.05	1.05	1.50		
9	1.35	1.05		1.50		
10	1.35		1.05	1.50		
11	1.35			1.50		
12	1.00					

Buttons: OK, Cancel, Load group...

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
13	1.00	1.50	1.50	0.90		
14	1.00	1.50	1.50			
15	1.00	1.50		0.90		
16	1.00	1.50				
17	1.00		1.50	0.90		
18	1.00		1.50			
19	1.00	1.05	1.05	1.50		
20	1.00	1.05		1.50		
21	1.00		1.05	1.50		
22	1.00			1.50		
23					1.00	
24						1.00

Buttons: OK, Cancel, Load group...

**Calculate**

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to execute the checking module. Each time you execute the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

**Use combination rules of EN 1990 (6.10a/b)**

Optionally the Eq. (6.10a/b) are used for the combination of the permanent and temporary situation, otherwise Eq. (6.10).

The following table demonstrates how the situations are used in the various checks. The numbers refer to the Chapters of the EN 1992-1-1 standard.

Situation	Ultimate limit state	Chapter	Serviceability limit state	Chapter
Perm. and temp. Accidental Earthquake	Longitudinal reinf.	6.1		
	Lateral reinf.	6.2		
	Torsional reinf.	6.3		
Characteristic (rare)	Robustness reinf. (EN 1992-2, 6.1 (110))	9.2.1.1	Concrete compr. stress	7.2 (2)
			Reinforcing steel stress	7.2 (5)
			Prestressing steel stress	7.2 (5)
			Crack width, prestr. with immed. bond	7.3.1DE
Frequent	Fatigue, simplified	6.8.6 (2)	Decompr. Class XD1-XS3	7.3.1
			Crack width, prestr. with bond	7.3.1
Quasi-continuous			Concrete compr. stress	7.2 (2)
			Prestressing steel stress	7.2 (5)DE
			Decompr. Class XC2-XC4	7.3.1
			Crack w., reinf.concr. & prestr. w/o b.	7.3.1
			Deformations	7.4
Fatigue	Fatigue reinf. steel	6.8.4		
	Fatigue prestr. steel	6.8.4		
	Fatigue concrete	6.8.7 (1)		

## Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.

### Name

User-defined label for the action.

### Gamma.sup, Gamma.inf

Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$ . The nationally valid values are suggested based on EN 1990, Table A.1.2 (B). For action P, the country-specific coefficients according to EN 1992-1-1, Chapter 2.4.2.2 (1), and for the other actions the nationally valid values according to EN 1990, Table A.1.2 (B), are proposed.

DIN EN 1992-1-1:

In accordance with 2.3.1.3 (4) a partial safety factor for settlements  $\gamma_{G,Set} = 1.0$  can be assumed for the linear-elastic determination of internal forces with stiffnesses of uncracked sections.

SS EN 1990:

The program suggests the partial safety factors as they result in accordance with Section A, Article 11, for safety class 3 from  $\gamma_d \cdot \gamma_{sup}$  with the reduction factor  $\gamma_d = 1.0$  as per Article 14. If required, lower safety classes can be taken into account entering lower values.

### Combination coefficients psi for:

Input fields for selecting the combination coefficients for variable actions according to EN 1990. The default number values are based on the national specifications in Table A.1.1 of the standard. Click the button to view and edit the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .

### Load cases

List of the possible load cases or load case combinations. Select items by highlighting them and clicking the button or use drag & drop.

### Multi-select

Load cases and combinations can be added to the actions more than once.

### Exclusive variants

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the or buttons.

### Inclusive load cases

Selected load cases and combinations that can have a simultaneous effect.

### **Exclusive load cases**

Selected load cases and combinations that are mutually exclusive.

### **Prestressing loss from relaxation of prestressing steel**

The prestressing loss is defined as a constant percentage reduction of prestress.

### **CS as constant reduction of prestress**

As an alternative to defining CS load cases, you can allow for the effect of creep and shrinkage by defining a constant percentage reduction of prestress.

### **Internal prestressing**

Selected load cases that describe internal prestressing. The reactions of the individual load cases are added together.

### **External prestressing**

Selected load cases that describe external prestressing. The reactions of the individual load cases are added together.

## **Partial Safety Factors**

The partial safety factors of the construction materials are preset with the nationally applicable values as specified in EN 1992-1-1, Table 2.1. In the design situations due to earthquakes, the safety factors of the accidental design situation may be assumed in accordance with EN 1998-1, Chapter 5.2.4 (3), if the strength loss is taken into account when determining the material properties. Otherwise, the factors of the permanent and temporary design situation must be applied in accordance with Chapter 5.2.4 (2).

The partial safety factors for the actions are specified in the definition of the actions based on EN 1990, Table A.1.2(B).

OENORM B 1998-1:

In design situations resulting from earthquakes, the factors for construction materials according to OENORM B 1998-1, Chapter 5.2.4 (3), apply.

DIN EN 1998-1:

In the design situations due to earthquakes, according to the NDP to 5.2.4 (1) and (3), the safety factors of the permanent and temporary design situation generally apply.

## Section Inputs

The section inputs contain all of the specific settings made for checks in the ultimate limit and serviceability states. In addition to these specifications, the selected material properties and the properties of the reinforcing steel are also relevant for the design. An overview of the design specifications can be accessed in the *EN 1992-1-1 Design* folder of the database and in the folders of the national variants.

## Checks

The following dialog is used to define which checks are available for the cross-section in the ultimate, fatigue and serviceability limit states. For composite sections, the selection is limited to the load-bearing capacity checks. The analysis settings allow to override this selection for the entire structure.

**Properties for element 6 - EN 1992-1-1 - Checks**

Number: 1 - Roc    Section Type: Polygon    Material Type: C45/55-EN    New Copy  
 Label: Roof girder    ⚠ Properties concern further elements.

Prestress of component: Subsequent bond    Exposure class: XC4

Ult. limit state

<input checked="" type="checkbox"/> Bend, long. force	<input type="checkbox"/> Reinf., prestr. steel	<input type="checkbox"/> Concr. long. stress
<input checked="" type="checkbox"/> Lateral force	<input type="checkbox"/> Lateral force	<input type="checkbox"/> Concr. comp. strut
<input type="checkbox"/> Torsion	<input type="checkbox"/> Torsion	
<input checked="" type="checkbox"/> Robustness		
<input type="checkbox"/> Shear joint		

Serviceab. limit state

<input checked="" type="checkbox"/> Concrete comp. stress	<input checked="" type="checkbox"/> Reinf. tensile stress
<input checked="" type="checkbox"/> Decompression	<input checked="" type="checkbox"/> Crack width
<input checked="" type="checkbox"/> Minimum crack reinf.	

OK    Cancel    Help

Check selection for EN 1992-1-1 (national variants corresponding)

### Prestressing of the component

The type of prestressing can be selected for each section separately:

- *Not prestressed*
- *Subsequent bond*
- *Without bond*
- *External*
- *Mixed construction*

### Exposure class

The check conditions for the decompression and crack width check are grouped by exposure class in EN 1992-1-1, Chapter 7.3, Table 7.1N. A component can be assigned to an exposure class based on the information provided in Table 4.1 of the standard.

SS EN 1992-1-1:

In addition, the service life class as per Article 10 can be selected to determine the crack width according to Table D-2 and the crack safety factor according to Table D-3.

### Robustness

This check determines the minimum reinforcement against failure without notice (robustness reinforcement) based on EN 1992-1-1, Chapter 5.10.1 (5)P with the method specified for prestressed concrete bridges in EN 1992-2, Chapter 6.1 (109), Equation (6.101a). It thus offers an alternative to minimum reinforcement as per EN 1992-1-1, Chapter 9.2.1.1 (1), Equation (9.1N). The latter can be taken into account when necessary by specifying a base reinforcement in the reinforcing steel description.

DIN EN 1992-1-1:

According to Chapter 9.2.1.1 (1), the ductile component behavior must always be ensured for components with or without prestressing by applying robustness reinforcement.

## Steel tensile stresses

For components with internal prestressing, both the prestressing steel stresses and the stresses of the longitudinal reinforcement are checked.

## Minimum crack reinforcement, crack width

The crack width check is carried out according to Chapter 7.3.4. In this check the final longitudinal reinforcement is set as the maximum value from the bending reinforcement, robustness reinforcement and minimum crack reinforcement as per 7.3.2. The latter will be increased automatically if necessary to maintain the crack width.

## Base Values

Unless otherwise specified, the base values apply for all checks in the ultimate, fatigue and serviceability limit states.

### Design mode

- *Standard*: Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary.
- *Compression member*: For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to Section 9.5.2 (2).

### Factor for $a_s$ in secondary direction

According to EN 1992-1-1, Section 9.3.1.1 (2), secondary longitudinal reinforcement of one-way slabs should not be less than 20% of the principal reinforcement. The examination is carried out on the program side with the results of the bending design separately for the upper and lower side of the cross-section. The direction with the largest amount of reinforcement per cross-sectional side defines each principal reinforcement direction. The assignment of the factorized reinforcement in secondary direction then takes place via corresponding reinforcement layers.

DIN EN 1992-1-1:

In the case of two-way slabs, the less stressed direction should be reinforced with at least 20% of the higher stressed direction.

### Reduction factor of prestr. for robustness

In the program the regulations of the EN 1992-2, Chapter 6.1 (110) are decisive for the arrangement of the robustness reinforcement. Thus for the determination of the tensile zone the statically determined effect of prestressing is not taken into account. Because this cannot be determined for area elements alternatively the prestress can be reduced by a reduction factor. The specification of an appropriate value is subject to the discretion of the user.

### Design without considering given reinforcement ratios

If selected, the reinforcement increase required in the design is performed without taking into account the reinforcement ratios specified by the basic reinforcement.



**Effective height**

Effective static height for the shear design of area elements [m].

**Angle cot Theta**

$\cot \Theta$  defines the concrete strut angle according to Chapter 6.2.3 (2), Equation (6.7N). The program will suggest a value of 1 (45° strut angle). You can choose to ignore the suggestion and pick any value within the permissible national limits. Entering a higher number will normally result in a lower necessary lateral force reinforcement  $A_{sw}$ , a lower absorbable lateral force  $V_{Rd,max}$  and a larger displacement  $a_1$  according to Chapter 9.2.1.3, Equation (9.2).

DIN EN 1992-1-1:

Three calculation methods can be chosen for the check:

- *Standard*: The input value is limited to the range permitted in accordance with Eq. (6.7aDE) for lateral force, torsion and combined loads (method with load-dependent strut angle).
- *Constant*: The check is carried out using the chosen value for  $\cot \Theta$  without further limitations (cf. interpretation No. 24 of NABau for DIN 1045-1).
- *Std./45°*: For lateral force  $\cot \Theta$  is limited according to Eq. (6.7aDE), for torsion a constant strut angle of 45° is assumed for simplification according to Chapter 6.3.2 (2).

The actual effective angle of the concrete struts is logged for each check location.

OENORM B 1992-1-1:

The concrete strut angle is defined by  $\tan \Theta$  and should be limited according to equations (3AT) and (4AT).

SS EN 1992-1-1:

According to Article 15 and differing from Equation (6.7N), for prestressed components the condition  $1.0 \leq \cot \Theta \leq 3.0$  applies.

**Asl acc. to Fig. 6.3**

The bending tensile reinforcement to be taken into account according to Chapter 6.2.2, Figure 6.3 [cm<sup>2</sup>].

**Asl extension to**

You can optionally specify a maximum value for areas and the program will automatically increase the above input value until that maximum value is reached in order to avoid stirrup reinforcement [cm<sup>2</sup>].

**Quality of the stirrups**

- *420S*: Reinforcing rod with  $f_{yk} = 420$  MN/m<sup>2</sup>.
- *500A*: Reinforcing rod with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- *500M*: Reinforcing meshes with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- *General information*: Freely definable steel quality [MN/m<sup>2</sup>].

**Design like slabs**

Beams or design objects are treated like slabs, which means that a minimum lateral force reinforcement will not be determined as per Chapter 6.2.1 (4), if no lateral force reinforcement is required for computation.

**Factor for rho.w,min**

The minimum reinforcement level  $\rho_{w,min}$  is defined using a factor related to the standard value for beams according to EN 1992-1-1, Chapter 9.2.2 (5).

DIN EN 1992-1-1, OENORM B 1992-1-1:

For slabs with  $V_{Ed} > V_{Rd,c}$  at least the 0.6-fold value of the minimum shear reinforcement of beams is necessary.

DIN EN 1992-1-1:

For structured sections with prestressed tension chord the 1.6-fold value is to be applied according to Equation (9.5bDE).

SS EN 1992-1-1:

If the fire safety class is 1 or 2 and no shear reinforcement is required,  $\rho_{w,min}$  can be set to zero as per Article 26.

### Design as circular cross-section

For circular and annular cross-sections, the lateral force design according to Bender et al. (2010) can be selected as an alternative for the resulting shear force  $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$ . The corresponding inputs are made on the *Shear Section* dialog page.

### Laying measure $c_{v,1}$

DIN EN 1992-1-1:

In Chapter 6.2.3 (1) the inner lever arm  $z$  is limited to the maximum value derived from  $z = d - c_{v,1} - 30$  mm and  $z = d - 2c_{v,1}$ . Note that  $c_{v,1}$  is the laying measure of the longitudinal reinforcement in the concrete compressive zone. For  $c_{v,1}$  the program will suggest the smallest axis distance of the longitudinal reinforcement to the section edge  $d_1$ .

### Separate check for x and y direction

DIN EN 1992-1-1:

For two-axes stressed slabs, the lateral force check can be performed separately in the x and y stress directions as described in Chapter 6.2.1 (10). The user is responsible for properly aligning the reinforcement directions.

## Shear Section

For polygon and composite sections, additional section dimensions are required for the lateral force and torsion design. These are explained in the following. In case of sections with internal prestressing or with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.

Properties for element 6 - EN 1992-1-1 - Shear section

Section: Form, Shear stresses, Material, Bedding, EN 1992-1-1 (Checks, Base values, Shear section, Shear joint, Stresses, Crack width, Fatigue, Variation coefficients), EN 1992-2 (Thermal analysis), General, Isolated column, Eccentricity

Number: 1 - Roc, Section Type: Polygon, Material Type: C45/55-EN, New, Copy, Delete

Label: Roof girder, Properties concern further elements.

Diagram: T-section with width 7.9, height 2.3, effective width 0.45.

Height [m]: 2.3, Nom. height: 2.3, Eff. width [m]: 0.45, Factor kb: 0.9, Qy

Width [m]: 0.5, Nom. width [m]: 0.5, tef [m]: 0.1,  Box section, Core section Ak = z1 \* z2, z1 [m]: 2.2, z2 [m]: 0.4, Qz, Mx

Eff. height [m]: 2.25, Factor kd: 0.9, OK, Cancel, Help

### Width

Section width for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Height

Section height for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Effective height

Effective static height for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Effective width

Effective static width for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Nominal width, nominal height

The nominal width or height of internally prestressed components as per EN 1992-1-1, Chapter 6.2.3 (6), for including the duct diameter in the calculation of the design value of the lateral load-bearing capacity  $V_{Rd,max}$ .

### Factor kb, Factor kd

Factor for calculating the inner lever arm  $z$  from the effective width  $b_w$  or effective height  $d$  in the lateral loadbearing capacity check for  $Q_y$  or  $Q_z$ .

### Core section $A_k = z_1 \cdot z_2$

Dimensions of the core section for calculating the torsion reinforcement [m].

### tef

The effective wall thickness of the torsion section according to Figure 6.11 [m].

### Box section

Selection of the rules applicable for box sections for the check of the maximum load-bearing capacity according to Chapter 6.3.2 (4) and for the required reinforcement according to Chapter 6.3.2 (5) in case of combined stress from lateral force and torsion.

### Circular and annular cross-section

If the circular design according to Bender et al. (2010) was selected for the resulting lateral force  $Q_r$  on the *Base values* dialog page, the equivalent cross sections for the shear design must be defined in the following dialog.

### Width $b_w$

Effective section width for calculation of the lateral force bearing capacity for  $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$ . According to the recommendation of the German Committee for Standardization in Civil Engineering (NABau), the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces should be selected for the effective width  $b_w$ . For circular cross-sections, the program suggests the dimension of the square inscribed in the circle ( $R \cdot \sqrt{2}$ ) for  $b_w$ , and twice the wall thickness for annular cross-sections.

### Effective height $d$

Statically effective height for calculation of the lateral force bearing capacity for  $Q_r$ . The program suggests  $d = h - d_1$ , where the height is set to  $h = R \cdot \sqrt{2}$  and  $d_1$  indicates the edge distance of the outer reinforcement layer.

### Factor kd

Factor for calculating the inner lever arm  $z$  from the effective height  $d$  in the verification for  $Q_r$ .

### Efficacy factor

According to Bender et al. (2010), p. 422, the efficacy factor  $\alpha_k$  is stress-dependent ( $0.715 \leq \alpha_k \leq 0.785$ ) and can be

assumed with the mean value  $\alpha_k = 0.75$ .

### Helix inclination

Angle between shear force reinforcement and component axis. When entering an inclination of  $90^\circ$ , annular single stirrups are assumed.

### $z_1, z_2, t_{ef}$

The dimensions  $z_1, z_2$  of the square core cross-section and the effective wall thickness  $t_{ef}$  of the torsion box are defined according to EN 1992-1-1, Figure 6.11. The design for torsion is carried out according to the standard for vertical stirrups.

## Shear Joint

The shear joint check is available for polygon and composite sections. The input values proposed by the program must be checked by the user and adjusted if necessary.

Properties for element 6 - EN 1992-1-1 - Shear joint

Section: 1 - Roc, Polygon, Material Type: C45/55-EN, Label: Roof girder

Joint location:  Between slab and web (automatic),  Distance from top edge dz [m]: 0.24

Joint roughness: Smooth, Factor c: 0.2

Joint width bi [m]: 0.5, Stress perpendicular to joint (comp. neg.) [N/mm<sup>2</sup>]: 0

Dynamic or fatigue load acc. to 6.2.5(5)

Buttons: OK, Cancel, Help

### Joint location

The program can automatically determine the location of the joint at the transition between the slab and the web. Alternatively, the user can define the distance of the joint from the top edge of the cross-section  $dz$  [m].

### Joint roughness

The roughness of the joint (very smooth, smooth, rough, indented).

### Factor c

Factor for determining the shear resistance in the joint, which is specified depending on the joint roughness according to EN 1992-1-1, Chapter 6.2.5 (2) and can only be adjusted by the user if the joint is very smooth.

### Joint width $b_i$

Width of the joint over which shear forces are transferred between existing and new concrete [m].

### Stress perpendicular to joint (comp. neg.)

Stress  $\sigma_n$  caused by the minimum normal force perpendicular to the joint which can act simultaneously with the lateral force [N/mm<sup>2</sup>]. Compressive stresses must be entered with a negative sign and are limited in the check according to 6.2.5 (1).

### Dynamic or fatigue stress according to 6.2.5(5)

If this option is selected, a dynamic or fatigue stress on the cross-section is assumed and the factor  $c$  is adjusted according to 6.2.5 (5).

## Stresses

Properties for element 6 - EN 1992-1-1 - Stresses

Number: 1 - Roc Section Type: Polygon Material Type: C45/55-EN

Label: Roof girder

Check of the concrete compressive stresses

Charact. comb.	Quasi-continuous c.	At the time t of prestressing	
perm. sigma.c:	perm. sigma.g:	perm. sigma.c(t):	Concr. strength fck(t) [MN/m <sup>2</sup> ]:
<input checked="" type="radio"/> 0.60 fck	<input type="checkbox"/> 0.45 fck	<input checked="" type="radio"/> 0.45 fck(t)	45
		<input type="radio"/> 0.60 fck(t)	

Reinforcing steel stresses

Characteristic comb. perm. sigma.g:

0.80 fyk  
 1.00 fyk

Decompression

Check combination: Comb. acc. to class

OK Cancel Help

### perm. sigma.c

The concrete compressive stress  $\sigma_c$  must be limited to  $0.60 \cdot f_{ck}$  under the characteristic action combination in the construction stages and final states according to EN 1992-1-1, Chapter 7.2 (2). If stress in the concrete under quasi-continuous combination does not exceed the limit  $0.45 \cdot f_{ck}$ , linear creep can be assumed according to 7.2 (3). If this is not the case, non-linear creep must be taken into account.

### perm. sigma.c(t)

Permissible concrete stress  $\sigma_{c(t)}$  at time  $t$  when prestressing is introduced. If the compressive stress exceeds the value  $0.45 \cdot f_{ck(t)}$ , the nonlinearity of the creep should be taken into account according to the standard. The program assumes that prestressing is introduced in design situation 'G+P'.

### fck(t)

Concrete compressive strength at time  $t$  when prestressing is introduced according to Chapter 5.10.2.2 (5) of the standard [MN/m<sup>2</sup>].

### Reinforcing steel stresses

According to Chapter 7.2 (5) the tensile stresses in the reinforcement may not exceed the value  $0.8 \cdot f_{yk}$  under the characteristic action combination. For stresses resulting from indirect action, the limits can be assumed as  $1.0 \cdot f_{yk}$ .

SS EN 1992-1-1:

According to Article 19, the limit  $1.0 \cdot f_{yk}$  can be generally assumed.

### Decompression, check combination

The action combination (AC) for the decompression check normally results from the selected exposition class. Alternatively, a deviating combination can be chosen.

## Crack Width

These specifications apply to the minimum crack reinforcement calculation and the crack width check.

### Section edge

The following properties can be defined differently for the section edges and the reinforcement directions:

- $w_{max}$  limit for the calculated crack [mm].
- $s_{r,max}$  largest permissible crack spacing [mm].
- $k_c$  calculation method for coefficient  $k_c$ .
- $max. d_s$  largest existing bar diameter [mm].
- $max. s$  largest existing bar spacing [mm].

The following options are available for editing:

- Standard* The standard properties are used for the unspecified edges and directions.
- Top, bottem, x, y* Definition for the top or bottom edge in the x or y reinforcement direction.
- <Add>* Starts the dialog for adding a section edge.
- <Delete>* Deletes the displayed section edge.

### wmax

Limit for the calculated crack width according to EN 1992-1-1, Chapter 7.3.1, Table 7.1N [mm]. The program will suggest a tabular value according to the national requirements based on the selected exposure class and the prestressing of the component. This value can be modified after the input field is enabled.

SS EN 1992-1-1:

In addition, the service life class is taken into account to determine the suggested value according to Article 20, Table D-2. For prestressed components the tabular values for higher corrosion are taken, for reinforced concrete the values for slight corrosion apply.

### sr,max

When calculating the crack width, the crack spacing  $s_{r,max}$  is determined by default using Equation (7.11) of the standard. Alternatively, the user can specify an upper limit to take into account any special conditions of Equation (7.14) or Sections (4) and (5) of Chapter 7.3.4, for example.

### Coefficient kc

The following methods are available for calculating the coefficient  $k_c$ :

- auto* For rectangular solid sections,  $k_c$  is calculated according to Eq. (7.2), in all other cases according to Eq. (7.3).
- web*  $k_c$  is calculated according to Eq. (7.2).
- chord*  $k_c$  is calculated according to Eq. (7.3).

**max. ds**

Largest existing bar diameter of the reinforcing steel reinforcement for evaluating Equations (7.6N), (7.7N) and (7.11) in Chapter 7.3 of the standard [mm].

**max. s**

Largest existing bar spacing of the reinforcement for the simplified crack width check as per Chapter 7.3.3 (2) [mm].

**Determ. of the tensile zone**

You can specify the tensile section where the minimum crack reinforcement as per Chapter 7.3.2 will be placed by selecting either an action combination or a restraint (bending, central tension).

**Thick component**

DIN EN 1992-1-1:

Based on DIN EN 1992-1-1, Chapter 7.3.2 (5), the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined according to Equation (NA 7.5.1). Therewith a reduction compared to the calculation with Equation (7.1) can be achieved.

**Minimum reinforcement according to Eq. (16AT)**

OENORM B 1992-1-1:

The minimum reinforcement for the crack width limitation under central restraint can be determined according to Equation (16AT). Therewith a reduction compared to the calculation with Equation (7.1) can be achieved.

**Coefficient k**

Coefficient for taking into account nonlinear distributed concrete tensile stresses in the section in Chapter 7.3.2, Equation (7.1). Depending on the flange width or the web height  $h$  the value  $k$  can be assumed between 0.65 ( $h \geq 800$  mm) and 1.0 ( $h \leq 300$  mm).

DIN EN 1992-1-1:

In case of restraint within the component,  $k$  can be multiplied by 0.8 whereby the minimum of the height and the width of the section or section part shall be used for  $h$ . For tensile stresses due to restraint generated outside of the component,  $k = 1.0$  applies.

SS EN 1992-1-1:

Depending of the section dimension  $h$  (flange thickness resp. web height), the factor  $k$  can be assumed between 0.50 ( $h \geq 680$  mm) and 0.90 ( $h \leq 200$  mm) according to Article 4a.

**Factor for  $f_{ctm}$** 

This factor is used to specify the effective concrete tensile strength  $f_{ct,eff}$  based on the average value of tensile strength  $f_{ctm}$ . This is done separately for the minimum reinforcement calculation according to Equation (7.1) and the crack width calculation according to Equation (7.9) of the standard. The tensile strength, which depends on the age of the concrete, is defined in Equation (3.4) of Chapter 3.1.2.

DIN EN 1992-1-1:

If it is not certain whether crack formation will occur within the first 28 days, a tensile strength of at least 3.0 MN/m<sup>2</sup> for normal concrete and 2.5 MN/m<sup>2</sup> for lightweight concrete should be assumed for Eq. (7.1). The program meets this requirement if 1.0 is entered for the reduction factor.

 **$A_{c,eff}$  ring-shaped**

For circular solid and hollow sections, the effective area of the reinforcement  $A_{c,eff}$  for the check of the minimum reinforcement and the crack width can be determined ring-shaped according to Wiese et al. (2004).

**Coefficient  $\xi_1$** 

The bond coefficient  $\xi_1$  according to Chapter 7.3.2, Equation (7.5), defines the extent to which prestressing steel as per 7.3.2 (3) can be taken into account for the minimum crack reinforcement. It is also used in calculating the effective reinforcement level according to Chapter 7.3.4, Equation (7.10), and thus enters into the direct calculation of the crack width. Data input is blocked for area elements since prestressing steel is normally not taken into account here.

OENORM B 1992-1-1:

The bond coefficient  $\xi_1$  is used to take into account the different bonding behavior of concrete and prestressing steel for the stress checks according to Chapter 7.2 of the standard.

**Check combination**

The action combination (AC) for the crack width check normally results from the selected exposition class. Alternatively, a deviating combination can be chosen.

**Check method**

The crack width can be verified either by direct calculation according to Chapter 7.3.4 or simplified by limiting the bar spacing using Table 7.3N. Table 7.3N should only be used for single-layer tensile reinforcement with  $d_1 = 4$  cm under loading (cf. Zilch, Rogge (2002), p. 277; Fingerloos et al. (2012), p. 109; Book 600 of the DAfStb (2012), p. 127). For both methods, a constant average steel strain within  $A_{c,eff}$  can optionally be chosen as the basis for calculation.

OENORM B 1992-1-1:

Die The method is applicable to single-layer reinforcement with a bar spacing according to Table 10AT or 11AT. These are valid for concrete covers  $25 \text{ mm} \leq c_{nom} \leq 40 \text{ mm}$  with bar diameters  $8 \text{ mm} \leq d_s \leq 20 \text{ mm}$ .

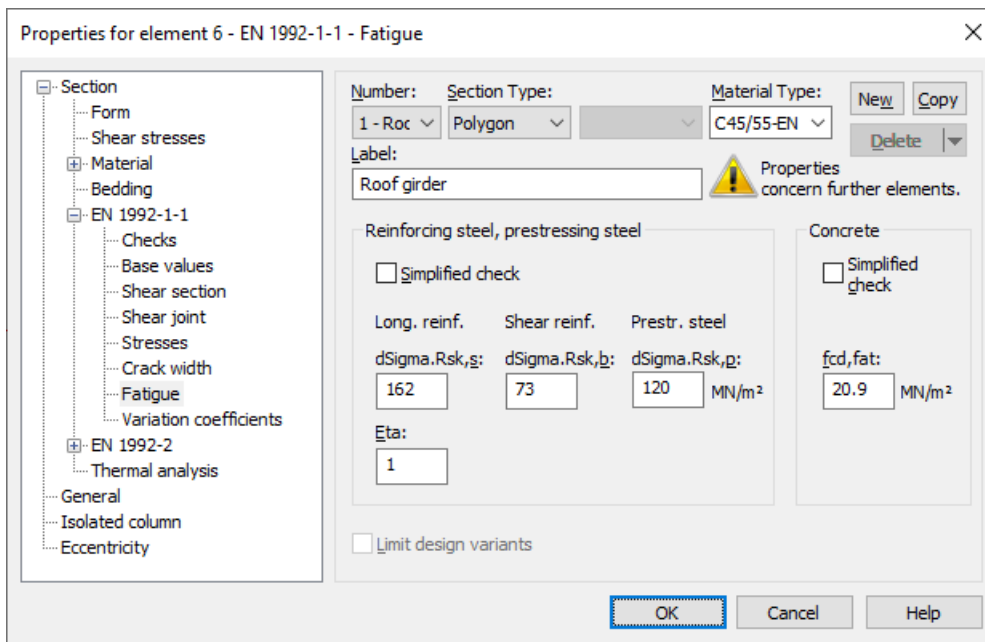
**Load duration; kt**

This selection defines the factor  $k_t$  in Equation (7.9) for crack width calculation.

**Note for waterproof concrete structures**

For components that are to be designed according to national guidelines for waterproof concrete structures, the permitted crack widths given there can be entered after activating the dialog control  $w_{max}$ . If required, the check-relevant action combinations can also be defined differently from the requirements of EN 1992-1-1.

**Fatigue**



**dSigma.Rsk,s, dSigma.Rsk,b**

The permissible characteristic stress range  $\Delta\sigma_{Rsk}$  ( $N^*$ ) of the longitudinal reinforcement and shear reinforcement at  $N^*$  load cycles according to the S-N curves specified in EN 1992-1-1, Chapter 6.8.4 [MN/m<sup>2</sup>]. The national decisive value found in Table 6.3N, Row 1 (beam sections) resp. Row 2 (area sections), is suggested in the dialog. For the shear reinforcement, the mandrel diameter is assumed to be four bar diameters.

OENORM B 1992-1-1:

In the dialog, the value according to Table 5, line 1 (beam sections) or line 4 (area sections) is suggested for the longitudinal reinforcement. For the shear reinforcement, the value according to line 1 is suggested.

**dSigma.Rsk,p**

The permissible characteristic stress range  $\Delta\sigma_{Rsk}$  ( $N^*$ ) of the prestressing steel at  $N^*$  load cycles according to the S-N curves specified in Chapter 6.8.4 [MN/m<sup>2</sup>]. The value found in Table 6.4, Row 4, is suggested in the dialog.



DIN EN 1992-1-1, OENORM B 1992-1-1:

The value for prestressing steel of class 1 is suggested.

### Eta

Increase factor  $\eta$  for the reinforcing steel stress of the longitudinal reinforcement. This factor is used to take into account the varying bonding behavior of concrete and prestressing steel as per Chapter 6.8.2 (2)P, Eq. (6.64).

### $f_{cd,fat}$

Concrete compressive strength before onset of cyclic load according to Chapter 6.8.7 (1), Eq. (6.76) [MN/m<sup>2</sup>]. In general, the following applies:

$$f_{cd,fat} = k_1 \cdot \beta_{cc}(t_0) \cdot f_{cd} \cdot \left(1 - \frac{f_{ck}}{250}\right) \quad (6.76)$$

with

$$\beta_{cc}(t_0) = e^{s(1-\sqrt{28/t_0})}$$

$s$  Coefficient depending on the cement type.

$t_0$  Time of the initial stressing of the concrete.

$k_1 = 0.85$

$f_{cd,fat}$  for  $s = 0.2$ ,  $t_0 = 28$  and  $f_{cd}$  according to Eq. (3.15) is suggested in the dialog.

DIN EN 1992-1-1, OENORM B 1992-1-1, SS EN 1992-1-1:

$k_1 = 1.0$

BS EN 1992-1-1:

For the proposed value of  $f_{cd,fat}$ ,  $f_{cd}$  is determined with  $\alpha_{cc} = 1.0$  in Eq. (3.15).

### Simplified check

The simplified check according to Chapter 6.8.6 (2) bases on the frequent action combination including the traffic loads at serviceability limit state. The method for concrete is defined in Chapter 6.8.7 (2), the permissible stress ranges for steel are suggested according to Chapter 6.8.6 (1) in the dialog. For shear reinforcement this value is reduced analogous to Table 6.3N.

### Limit design variants

For area elements, the variants for determining the stress range can be limited to the corresponding sets of design internal forces. For more information see chapter '*Check Against Fatigue > Special Characteristic of Shell Structures*'.

## Variation Coefficients

Properties for element 6 - EN 1992-1-1 - Variation coefficients

Section: 1 - Roc, Section Type: Polygon, Material Type: C45/55-EN

Label: Roof girder

Variation coefficients for the internal prestressing in the serviceability checks.

	r.sup	r.inf
Construction stage:	1.1	0.9
Final state:	1.1	0.9

OK Cancel Help

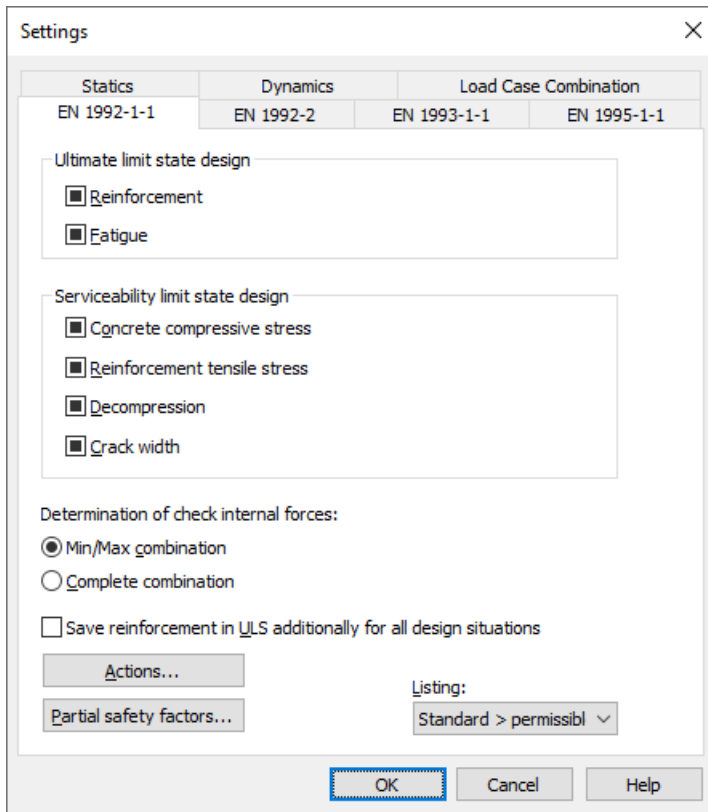
The coefficients used to take into account the variation of prestressing force are defined in EN 1992-1-1 depending on the prestressing type. In the dialog, values are suggested according to Chapter 5.10.9 (1)P for subsequent bond. In the actions, design situations are declared as construction stage or final state. The defined variation coefficients are taken into account for the effects from internal prestressing in the following checks:

- Decompression and concrete compressive stress check.
- Minimum reinforcement for crack width limitation.
- Crack width check.

Regarding the effects from external prestressing, the variation coefficients correspond to  $r_{sup} = r_{inf} = 1$ .

## Analysis Settings

The *EN 1992-1-1* dialog page can be opened using the *Settings* function in the *Analysis* menu.



### Check selection

When selecting checks, the following cases are to be distinguished:

- or  The check is performed according to the settings in the section dialog (see Section inputs).
- or  The check is performed for all sections of the structure.
- or  The check is performed for no sections of the structure.

Corresponding section settings are bundled as follows:

Reinforcement	Bend and longitudinal force Lateral force Torsion Robustness Shear joint
Crack width	Minimum crack reinforcement Calculation of the crack width

An overview of the checks can be accessed using the *Design Settings* function in the *EN 1992-1-1 Design* folder of the database.

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *Check internal forces*.

### Save reinforcement in ULS additionally for all design situations

In addition to the maximum required ultimate limit state reinforcement, the reinforcement is saved separately for each design situation in the ultimate limit state.

**Actions...**

Open the dialog for describing actions.

**Partial safety factors...**

Open the dialog for modifying partial safety factors.

**Listing**

- *No*: No log is generated by the checking program.
- *Standard*: Log with tabular output of results.
- *Detailed*: Additional output of the decisive combination internal forces at the check locations.
- *Standard > permissible*: Standard log limited to check locations where the permissible limit values are exceeded.
- *Detailed > permissible*: Detailed log limited to check locations where the permissible limit values are exceeded.

## Single Design

The single design function allows you to analyze individual sections independently of the global system using predefined internal forces. The calculation is carried out from the opened input table via the *Single Design* item in the *Analysis* menu or the *Print Preview* function.

Enter the information listed below in the *Single Design* table in the *EN 1992-1-1 Design* folder of the database or the folders of the national variants.

### Section

Number of the section to be designed. Both polygon and composite sections can be designed.

### Combination

Design situation according to EN 1992-1-1, Table 2.1.

- 0: Permanent and temporary design situation
- 1: Accidental design situation

### Nsd, Mysd, Mzsd

Internal forces being designed. The internal forces refer to the centroid in polygon sections or the section zero point in composite sections.

### Mode

- *Standard*: Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members a symmetrical design is carried out taking into account the minimum reinforcement according to Chapter 9.5.2 (2).
- *Strains*: Determine strain state for existing reinforcing steel layers.
- *Strains SLS*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. In the compression zone, a linear strain-stress curve of the concrete with the gradient  $\tan \alpha = E_{cm}$  is used.
- *Strains SLS2*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A nonlinear strain-stress curve of the concrete is used as shown in Figure 3.2. Note that a horizontal progression is assumed for strains exceeding  $\varepsilon_{c1}$ .
- *Load bearing capacity*: Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
- *Maximum bending moment My*: Determination of the maximum bearable bending moment  $M_y$ . The moment  $M_y$  is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.
- *Inactive*: Design disabled.

OENORM B 1992-1-1:

In the modes *SLS* and *SLS2* the stress increase of the prestressing steel layers is determined according to Eq. (13AT) with the bond coefficient  $\xi_1$  specified for the section to be checked.

## Punching Shear Check

When you select a check node, the key data for the checks is displayed in a dialog field. This dialog is divided into three pages.

### 1a. Input data, column

The column forms *rectangle* and *round* with the locations *internal*, *edge parallel to x*, *edge parallel to y* and *corner* are available. When you enter a new column, the program will suggest the dimensions of existing columns. The edge distances  $a_x$  and  $a_y$  are used to calculate the perimeters  $u_i$  of the check sections for columns near to an edge or a corner.

#### DIN EN 1992-1-1:

In addition, the check locations *Wall end*, *Wall corner Fig. NA.6.12.1 (top)* or *Wall corner Fig. NA.6.12.1 (bottom)* can be selected.

In the case of *Wall end*, the check sections are determined according to the middle part of Figure NA.6.12.1 and, in the case of *Wall corner Fig. NA.6.12.1 (top)*, according to the upper part of Figure NA.6.12.1. The larger of the two entered dimensions  $b_x$ ,  $b_y$  is taken as value  $a$ , the smaller as value  $b$  and from this the dimensions  $a_1$ ,  $b_1$  are calculated as shown in the figure.

In the case of *Wall corner Fig. NA.6.12.1 (bottom)*, the check sections are determined according to the bottom part of Figure NA.6.12.1. Accordingly, the dimensions  $b_x$ ,  $b_y$  are not entered by the user in the dialog, but are calculated with  $1.5 \cdot d$  and used for the check section.

#### OENORM B 1992-1-1:

In addition, the check locations *Wall end* or *Wall corner* can be selected. The check sections are determined according to Figure 3AT. The larger of the two entered dimensions  $b_x$ ,  $b_y$  is taken as value  $a$ , the smaller as value  $b$  and from this the dimensions  $a_1$ ,  $b_1$  are calculated as shown in the figure.

### 1b. Input data, slab

This section shows the material properties, the existing reinforcement ( $a_{sx}$ ,  $a_{sy}$ ) as well as additional coefficients for calculating punching shear resistances:

- $\beta$  load increase factor for taking into account eccentric load introduction
- $s_r$  radial distance of the punching reinforcement rows [m]
- $d$  average value of the effective heights  $d_x$  and  $d_y$  in orthogonal directions [m]
- $d^*$  factor for the distance of the critical perimeter related to the effective height  $d$

### 1c. Input data, action

The action  $V_{Ed}$  can either be added as a support force from a previous design according to EN 1992-1-1 or defined directly. All medium soil pressures  $\sigma_0$  lower the design value of the lateral force within the area of the decisive perimeter. The medium longitudinal forces  $N_{Ed}$  are used to calculate the normal concrete stress.

## 2. Aperture

This dialog page is used to define the geometry and location of an opening.

## 3. Results

This dialog page shows the calculated punching shear resistances, the necessary punching shear reinforcement (if applicable) and the minimum bending reinforcement (if nationally relevant). You can call up an improved bending reinforcement by clicking the *Proposal* button.

## Example

EN 1992-1-1 Punishing Shear Check Node 4312

Input data | Aperture | Results

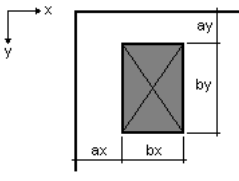
Column: Rectangle, corner

Width  $b_x$ : 0.4 m

Height  $b_y$ : 0.5 m

Edge spacing  $a_x$ : 0.3 m

Edge spacing  $a_y$ : 0.2 m



Concrete: C30/37-EN

Steel: BSt 500

Consider aperture

$a_{sx}$ : [cm<sup>2</sup>/m] 8

$a_{sy}$ : [cm<sup>2</sup>/m] 8

$\beta$  1.5

Eff. h.  $d_x$  [m] 0.17

Eff. h.  $d_y$  [m] 0.17

$s_r$  [m] 0.127

Slab height [m] 0.2

Truss angle [°] 90

Distance of the critical perimeter:  $d^*$  2

Action:

$V_{Ed}$  from calculation

Enter  $V_{Ed}$

	$V_{Ed}$ [kN]	$N_{Ed}$ [kN/m]	$\sigma_0$ [kN/m <sup>2</sup> ]
Fundamental combination	135	0	0
Accidental combination	0	0	0

OK Cancel Help

EN 1992-1-1 Punishing Shear Check Node 4312

Input data | Aperture | Results

**Results of the fundamental combination**

Available long. reinf.  $a_{sx} = 8.00$  cm<sup>2</sup>/m  $a_{sy} = 8.00$  cm<sup>2</sup>/m

$V_{Ed} = 0.62$  MN/m<sup>2</sup>  $V_{Rd,c} = 0.58$  MN/m<sup>2</sup>  $V_{Ed}/V_{Rd,c} = 1.06$

$\rho_1 = 0.0047$

$A_{sw1} = 1.14$  cm<sup>2</sup>  $V_{Rd,cs1} = 0.64$  MN/m<sup>2</sup>  $V_{Ed}/V_{Rd,cs1} = 0.96$

$A_{sw2} = 1.29$  cm<sup>2</sup>  $V_{Rd,cs2} = 0.66$  MN/m<sup>2</sup>  $V_{Ed}/V_{Rd,cs2} = 0.93$

$V_{Ed,out} = 0.58$  MN/m<sup>2</sup>  $V_{Rd,c} = 0.58$  MN/m<sup>2</sup>  $V_{Ed,out}/V_{Rd,c} = 1.00$

**The check is OK!**

**Proposal 1**

$a_{sx} = 9.58$  cm<sup>2</sup>/m  $a_{sy} = 9.58$  cm<sup>2</sup>/m  $\Sigma A_{sw} = 0.00$  cm<sup>2</sup>

**Proposal 2**

$a_{sx} = 0.01$  cm<sup>2</sup>/m  $a_{sy} = 0.01$  cm<sup>2</sup>/m  $\Sigma A_{sw} = 3.89$  cm<sup>2</sup>

Proposal Undertake proposal 1 Undertake proposal 2

OK Cancel Help

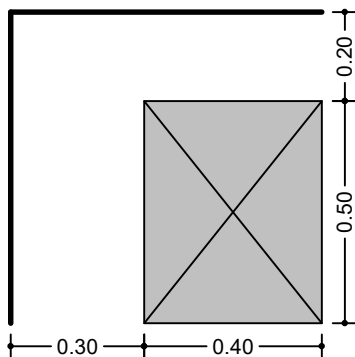
## Punching shear check node 4312

The check is performed according to EN 1992-1-1:2004/A1:2014.

## 1. Measurements, situation and material

Rectangular column with width  $b_x = 0.40$  m and height  $b_y = 0.50$  m

Situation: Corner column; Edge spacing  $a_x = 0.30$  m; Edge spacing  $a_y = 0.20$  m;  $\beta = 1.50$



Critical perimeter  $u_1 = 1.93$  m (Distance = 0.34 m);  $A_1 = 1.06$  m<sup>2</sup>

Slab height  $h = 0.200$  m

Effective height of the slab  $d_x = 0.170$  m;  $d_y = 0.170$  m;  $d = (d_x + d_y) / 2 = 0.170$  m

Available longitudinal reinforcement  $a_{sx} = 8.00$  cm<sup>2</sup>/m;  $a_{sy} = 8.00$  cm<sup>2</sup>/m

Truss angle  $\alpha = 90.0^\circ$

Concrete: C35/45-EN  $f_{ck} = 30.00$  MN/m<sup>2</sup>  $\alpha_{cc} = 1.00$

$\gamma_c = 1.50$

$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 20.00$  MN/m<sup>2</sup>

Reinforce.: BSt 500  $f_{ck} = 500.00$  MN/m<sup>2</sup>

$\gamma_s = 1.15$

$f_{yd} = f_{yk} / \gamma_s = 434.78$  MN/m<sup>2</sup>

## 2. Action from fundamental combination

$$V_{Ed} = 135.00 \text{ kN} \quad N_{Ed} = 0.00 \text{ kN/m} \quad \sigma_0 = 0.00 \text{ kN/m}^2$$

$$v_{Ed} = \beta \cdot V_{Ed} / (u_i \cdot d) \quad (6.38)$$

$$\text{with } u_i = u_1$$

$$v_{Ed} = 0.62 \text{ MN/m}^2$$

## 3. Punching resistance without punching reinforcement

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \quad (6.47)$$

$$C_{Rd,c} = 0.18 / \gamma_c$$

$$\text{with } C_{Rd,c} = 0.12$$

$$k = 2.00$$

$$\rho_l = 0.0047$$

$$f_{ck} = 30.00 \text{ MN/m}^2$$

$$k_1 = 0.10$$

$$\sigma_{cp} = -N_{Ed} / h = 0.00 \text{ MN/m}^2$$

$$v_{min} = 0.54 \text{ MN/m}^2$$

$$v_{Rd,c} = 0.58 \text{ MN/m}^2$$

$$v_{Ed} / v_{Rd,c} = 1.06 > 1 \quad \text{Punching reinforcement is required!}$$

$$v_{Ed,0} = \beta \cdot V_{Ed} / (u_0 \cdot d) = 2.34 < v_{Rd,max} = 4.22 \text{ MN/m}^2 \quad (6.53)$$

$$\text{with } u_0 = 0.51 \text{ m}$$

## 4. Punching reinforcement (normal)

$$A_{sw} = \frac{(v_{Ed} - 0.75 \cdot v_{Rd,c})}{1.5 \cdot (d / s_r) \cdot f_{ywd,ef} \cdot (1 / (u_1 \cdot d))} \quad (6.52)$$

$$A_{sw,i,min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk} \cdot \frac{s_r \cdot u_{cont,i}}{1.5} \quad (9.11)$$

$$\text{with } v_{Ed} = 0.62 \text{ MN/m}^2$$

$$v_{Rd,c} = 0.58 \text{ MN/m}^2$$

$$s_r = 0.12 \text{ m}$$

$$f_{ywd,ef} = 292.50 \text{ MN/m}^2$$

$$f_{ck} = 30.00 \text{ MN/m}^2$$

$$f_{yk} = 500.00 \text{ MN/m}^2$$

$$\text{Row 1: Distance} = 0.09 \text{ m}; \quad u_{cont,1} = 1.53 \text{ m}; \quad A_{sw,1} = 1.08 \text{ cm}^2 > A_{sw,1,min} = \mathbf{1.08 \text{ cm}^2}$$

$$\text{Row 2: Distance} = 0.21 \text{ m}; \quad u_{cont,2} = 1.72 \text{ m}; \quad A_{sw,2} = 0.96 \text{ cm}^2 > A_{sw,2,min} = \mathbf{1.21 \text{ cm}^2}$$

External perimeter according to Equ. (6.54) and Fig. 6.22 A

$$u_{out} = \beta \cdot V_{Ed} / (v_{Rd,c} \cdot d) = 2.05 \text{ m}$$

$$\text{Distance} = 0.42 \text{ m}$$

The outermost reinf. row is placed at a spacing of  $0.21 \text{ m} \leq 1.5 \cdot d = 0.26 \text{ m}$ . The check is OK!

Maximal load bearing capacity with punching reinforcement acc. to Eq. (6.52)

$$v_{Ed} = 0.62 \leq k_{max} \cdot v_{Rd,c} = 1.50 \cdot 0.58 = 0.87. \quad \text{The check is OK!}$$



# Prestressed Structures

## Internal Prestressing

For internal prestressing, the tendon groups as well as the prestressing system and procedures are entered using the *Prestressing* function of the *Structure* menu. To include them in the FEM calculation, you then need to define a load case of the *Prestressing* load type. For more information, refer to the *Prestressed Concrete* section of the manual.

Prestressing with bond and prestressing without bond are differentiated in the section inputs and the specifications for the *Creep and shrinkage* load case. For prestressed components with subsequent bond the tendons can be set ungrouted for the respective design situation in the action dialog.

## External Prestressing, Mixed Construction

External prestressing can be taken into account by entering the external forces directly in the program. For mixed construction, the additional tendons in a bond must be entered as described above.

## Variation of Prestressing

For checks in the ultimate limit state, the following applies for the design value of the prestressing force according to EN 1992-1-1, Chapter 5.10.8 (1):

$$P_{d,t}(x) = \gamma_P \cdot P_{m,t}(x)$$

with

$P_{m,t}(x)$  Mean value of prestressing force at time  $t$  and location  $x$  including prestressing losses from friction, slippage, creep, shrinkage and relaxation.

$\gamma_P$  Partial safety factor of prestressing force,  $\gamma_P = 1$  as specified in Chapter 2.4.2.2 (1).

In the serviceability limit state, two characteristic values for the prestressing force are defined in Chapter 5.10.9 (1):

$$P_{k,sup} = r_{sup} \cdot P_{m,t}(x) \quad \text{Upper characteristic value according to Equation (5.47).}$$

$$P_{k,inf} = r_{inf} \cdot P_{m,t}(x) \quad \text{Lower characteristic value according to Equation (5.48).}$$

The variation coefficients for internal prestressing are defined separately for construction stages and final states. They are used in the following checks:

- Decompression and concrete compressive stress check.
- Minimum reinforcement for crack width limitation.
- Crack width check.

Regarding the effects from external prestressing, the variation coefficients correspond to  $r_{sup} = r_{inf} = 1$ .

For internal prestressing, the recommended country-specific values are:

- For tendons with immediate bond or without bond:

$$r_{sup} = 1.05 \text{ and } r_{inf} = 0.95.$$

- For tendons with subsequent bond:

$$r_{sup} = 1.10 \text{ and } r_{inf} = 0.90.$$

OENORM B 1992-1-1:

- For tendons with immediate bond or without bond:

$$r_{sup} = r_{inf} = 1.0.$$

- For tendons with subsequent bond:

$$r_{sup} = 1.05 \text{ and } r_{inf} = 0.95.$$

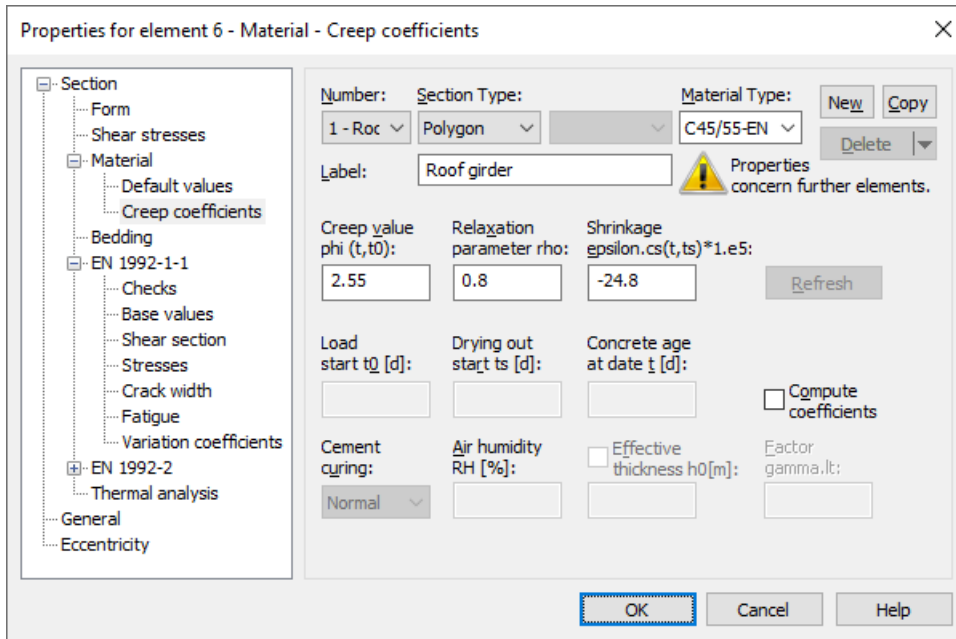
BS EN 1992-1-1:

$r_{sup} = r_{inf} = 1.0$  generally applies.

## Creep and Shrinkage

Similar to prestressing, creep and shrinkage are taken into account by specifying a corresponding load case (*Creep and shrinkage* load type) in the FEM calculation. Besides the creep-generating continuous load case, you also need to specify whether the internal forces relocation between concrete and prestressing steel is to be taken into account. This option is only useful in the case of tendons with bond.

The decisive creep and shrinkage coefficients for calculating the *Creep and shrinkage* load case are entered in the section dialog. Alternatively, you can also use this dialog to calculate the coefficients according to Chapter 3.1.4 with Annex B.



The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1 + \rho \cdot \varphi} (\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,s})$$

Explanation of the individual terms:

- $\sigma_b(t)$  Concrete stress from creep and shrinkage at time  $t$ .
- $E_b$  E-modulus of the concrete.
- $\rho$  Relaxation coefficient according to Trost for time  $t$  (normally  $\rho = 0.80$ ).
- $\varphi$  Creep coefficient for time  $t$ .
- $\varepsilon_b(t)$  Concrete strain from creep and shrinkage at time  $t$ .
- $\varepsilon_{b,0}$  Concrete strain from creep-generating continuous load.
- $\varepsilon_{b,s}$  Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, yield the internal forces and deformations of the concrete. The resulting stress changes in the prestressing steel are also determined provided they are selected in the load case. Any influence from the relaxation of the prestressing steel will be ignored in this case. According to Zilch/Rogge (2002, p. 256), this influence can be calculated separately (see following section) and combined with the changes from creep and shrinkage for all time-dependent prestressing losses:

$$\Delta\sigma_{p,csr} = \Delta\sigma_{pr} + E_p \cdot \Delta\varepsilon_{cpt}$$

with

$\Delta\sigma_{pr}$  Prestressing loss from relaxation of the prestressing steel.

$\Delta\varepsilon_{cpt}$  Concrete strain change from creep and shrinkage.

$E_p$  E-modulus of the prestressing steel.

## Relaxation of Prestressing Steel

According to EN 1992-1-1, Chapter 5.10.6, the stress change  $\Delta\sigma_{pr}$  in the tendons at position  $x$  and time  $t$  due to relaxation must be taken into account in addition to the stress loss from concrete creep and shrinkage. The relaxation of the steel depends on the deformation of the concrete caused by creep and shrinkage. According to 5.10.6 (1) (b), this interaction can be taken into account in a general and approximate manner by specifying a reduction coefficient of 0.8.

The stress change  $\Delta\sigma_{pr}$  can be determined for the initial stress in the tendons as a result of prestressing and quasi-continuous actions according to 5.10.6 (2). More details are provided in Chapter 3.3.2 of the standard.

The stress losses are defined in the CSR actions of the *EN 1992-1-1 actions* dialog.

DIN EN 1992-1-1:

The stress change  $\Delta\sigma_{pr}$  can be determined using the specifications of the prestressing steel certification for the ratio of initial stress to characteristic tensile strength ( $\sigma_{p0}/f_{pk}$ ).  $\sigma_{p0} = \sigma_{pg0}$  may be used as the initial stress, with  $\sigma_{pg0}$  referring to the initial prestressing steel stress from prestressing and the permanent action.

## Check Internal Forces

The calculation of load cases results in a set of internal forces for each load case at the check location (e.g.  $N_x$ ,  $M_y$ ). The check internal forces are then determined from the results of the load cases with the combination rules relevant for the ultimate limit state, fatigue and serviceability limit state. One of the following methods can be selected in the analysis settings:

- *Min/Max combination*  
The results of a load case are added to the set of internal forces with the minimum or maximum of an internal force, if this increases the amount of the extreme value. Result sets from traffic actions in which the control variable is less than the threshold  $10^{-3}$  are not combined. The min/max combination delivers a constant number of sets regardless of the number of load cases and thus represents a particularly economical solution for the checks.
- *Complete combination*  
To determine the evidence internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The number of records increases exponentially with the number of inclusive load cases and can therefore result in high time and memory requirements for the checks.

For beams, design objects and axisymmetric elements, the resulting sets of internal forces are used directly in the checks. For area elements, *design internal forces* are derived from this, as will be described in more detail in the following section.

The internal forces relevant for the checks are documented in the detailed check listing. Regardless of the selection made, the results of the min/max combination are saved for the graphical representation. The load cases involved in the combination can be displayed using the *Combination information* context function.

The differences between the two combination methods mentioned before can be seen from the following example of a uniaxially stressed beam. The load cases 2, 3 and 4 shown can act simultaneously (inclusive). All safety and combination factors are assumed to be 1 for the example.

Action	$N_x$	$M_y$	Load case
G - permanent	-15	40	1
Q - variable	0	20	2
	5	10	3
	0	-10	4

*Internal forces of the load cases*

Extreme value	$N_x$	$M_y$	Combination
min $N_x$	-15	40	L1
max $N_x$	-10	50	L1+L3
min $M_y$	-15	30	L1+L4
max $M_y$	-10	70	L1+L2+L3

*Results of min/max combination*

Set	$N_x$	$M_y$	Combination
1	-15	40	L1
2	-15	60	L1+L2
3	-10	50	L1+L3
4	-15	30	L1+L4
5	-10	70	L1+L2+L3
6	-15	50	L1+L2+L4
7	-10	40	L1+L3+L4
8	-10	60	L1+L2+L3+L4

*Results of complete combination*

## Design internal forces for area elements

With area elements, the design internal forces correspond to the plasticity approach from Wolfensberger and Thürlimann. This approach takes into account how much the reinforcement deviates from the crack direction. Due to the current lack of precise data regarding the combined load of reinforced concrete shell structures from bending and normal force, the design internal forces for bending and normal force are calculated independently according to the static limit theorem of the plasticity theory and then used together as the basis for the design in the two reinforcement directions. This approach should always lead to results that are on the safe side.

Depending on the type of area element and reinforcement configuration, the variants of design internal forces listed below are taken into account for the checks.

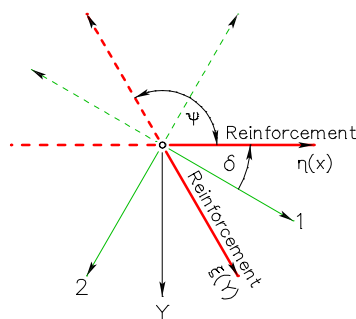
### Orthogonal area reinforcement

Slabs	$m_x \pm  m_{xy} $	
	$m_y \pm  m_{xy} $	
Plain stress elements	$n_x \pm  n_{xy} $	
	$n_y \pm  n_{xy} $	
Shells	$m_x \pm  m_{xy} $ and	$n_x \pm  n_{xy} $
	$m_y \pm  m_{xy} $ and	$n_y \pm  n_{xy} $

### Oblique area reinforcement

The bending design of slabs with oblique reinforcement assemblies is carried out according to Kuyt or Rüsçh. Here the design moments are calculated with the help of the principal moments  $m_1$ ,  $m_2$  according to the equations given in Book 166 DAfStB.

For load case combinations, the calculation is based on the extreme values of  $m_1$ ,  $m_2$ . For combined loads (bending and longitudinal force), both the design moments and the normal design forces are independently derived from  $n_1$ ,  $n_2$ . The normal design forces are then used together as the basis for the design. This should also result in an upper limit for the load.



Coordinate systems

Extreme values (principal bending moments):

$$m_{1,2} = \frac{1}{2}(m_x + m_y) \pm \frac{1}{2}\sqrt{(m_x - m_y)^2 + 4m_{xy}^2}$$

with  $m_1 \geq m_2$

The angle  $\delta$  assigned to  $m_1$  is:

$$\tan \delta = \frac{2 \cdot m_{xy}}{(m_x - m_y) + \sqrt{(m_x - m_y)^2 + 4 \cdot m_{xy}^2}}$$

### Design moments:

$$m_\eta = \frac{1}{\sin^2 \psi} \left[ m_1 \sin^2(\delta + \psi) + m_2 \cos^2(\delta + \psi) \pm |m_1 \sin \delta \sin(\delta + \psi) + m_2 \cos \delta \cos(\delta + \psi)| \right]$$

$$m_\xi = \frac{1}{\sin^2 \psi} \left[ m_1 \sin^2 \delta + m_2 \cos^2 \delta \pm |m_1 \sin \delta \sin(\delta + \psi) + m_2 \cos \delta \cos(\delta + \psi)| \right]$$

The formulas apply accordingly for the normal design forces.

## Checks in the Ultimate Limit States

The following checks are available:

- Bending with or without normal force or normal force only (EN 1992-1-1, Chapter 6.1).
- Minimum reinforcement against failure without warning (Chapter 5.10.1 (5)P and 9.2.1.1).
- Lateral force (Chapter 6.2).
- Torsion and combined stressing (Chapter 6.3).
- Shear joint (Chapter 6.2.5).
- Punching shear (Chapter 6.4).

## Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.4.3, the following combinations are taken into account in the ultimate limit states:

- For the combination of the permanent and temporary design situation either Equation (6.10) or the most unfavorable equation from (6.10a) and (6.10b) is permitted.

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10)$$

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10a)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10b)$$

For the coefficient  $\xi$  the value of  $\xi = 0.85$  results from Table A.1.2(B).

DIN EN 1990, ÖENORM B 1990:

Equation (6.10) is used for the combination.

SS EN 1990 (EKS 11):

Equations (6.10a) and (6.10b) apply with following modifications:

$$\sum_{j \geq 1} \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P \quad (6.10aSS)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_d \cdot \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_d \cdot \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10bSS)$$

Assuming reliability class 3, factor  $\gamma_d$  is set to 1. (see Section A, Article 11 and 14). The coefficient  $\xi$  is set to the value of  $\xi = 0.89$ .

BS EN 1990:

The coefficient  $\xi$  in Equation (6.10b) is set to the value of  $\xi = 0.925$ .

- Combination for accidental design situations

$$\sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.11b)$$

$\psi_{1,1} \cdot Q_{k,1}$  is used by the program for this combination.

ÖENORM B 1990-1:

$\psi_{2,1} \cdot Q_{k,1}$  is decisive.

- Combination for design situations caused by earthquakes

$$\sum_{j \geq 1} G_{k,j} + P + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.12b)$$

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

## Stress-Strain Curves

The following characteristics are used for section design:

- Concrete: Parabola-rectangle diagram according to EN 1992-1-1, Figure 3.3. Note that the design value for concrete compressive strength  $f_{cd}$  in Equation (3.15) is defined as  $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$  with  $\alpha_{cc} = 1$  for normal concrete and  $\alpha_{cc} = 0.85$  for lightweight concrete.

DIN EN 1992-1-1:

$\alpha_{cc} = 0.85$  for normal concrete and  $\alpha_{cc} = 0.75$  for lightweight concrete.

SS EN 1992-1-1:

$\alpha_{cc} = 1$  for normal and lightweight concrete.

BS EN 1992-1-1:

According to NA to 3.1.6 (1)P conservatively,  $\alpha_{cc} = 0.85$  is always assumed for normal concrete and lightweight concrete.

- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be  $k \cdot f_{yk} / \gamma_s$  with  $k = 1.05$  as per Table C.1, class A.

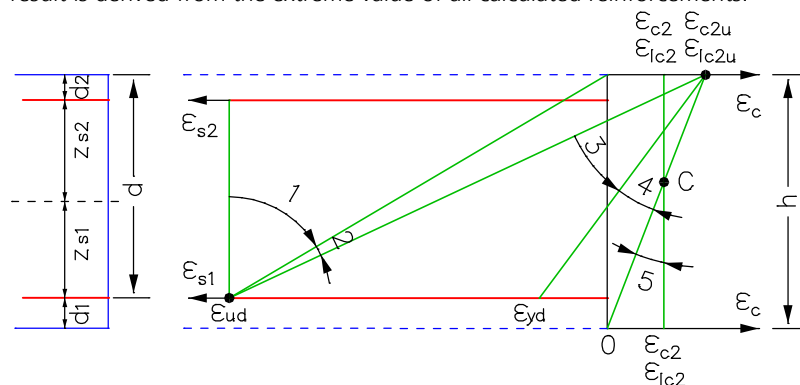
DIN EN 1992-1-1:

The maximum stress is assumed to be  $1.05 \cdot f_{yk} / \gamma_s$  for ductility class A according to DIN 488-1.

- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of  $f_{pd} = f_{p;0,1k} / \gamma_s$ .

## Design for Bending With or Without Normal Force or Normal Force Only

The design for longitudinal force and bending moment is performed according to EN 1992-1-1, Chapter 6.1. The reinforcement required for each internal force combination at the reinforced concrete section is determined iteratively based on the formulation of equilibrium conditions as well as the limit strain curve depicted in the illustration below. The final result is derived from the extreme value of all calculated reinforcements.



Strain areas for the design with  $\varepsilon_{ud} = 0.9 \varepsilon_{uk}$  and  $\varepsilon_{uk} = 0.025$  as per Table C.1.

DIN EN 1992-1-1:

$\varepsilon_{ud} = 0.025$

You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes. For sections subject to a compressive normal force, the minimum eccentricity defined in Chapter 6.1 (4) is taken into account. Concrete compression according to Chapter 6.1 (5) cannot be checked.

### Standard Mode

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tensile section to the greatest degree possible. Given ratios between certain reinforcement layers in the tension or compression zone are maintained as far as possible, unless this is deselected in the design specifications. The procedure in strain areas 4 and 5 is the same as with symmetrical design. The required transverse reinforcement of slab as per Section 9.3.1.1 (2) is considered during design according to user specification. However, the provision for horizontal reinforcement of walls as per Section 9.6.3 (1) is not taken into account.

DIN EN 1992-1-1:

The referenced compressive zone height  $x_d / d$  is limited according to Chapter 5.4 (NA.5) and NA.11.5.2 (1) as follows:

$$x_d / d \leq 0.45 \text{ for concrete strength classes up to C50/60.}$$

$$\leq 0.35 \text{ for concrete strength class C55/67 or higher and for lightweight concrete.}$$

### Symmetrical Mode

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected unless this is deselected in the design specifications.

### Compression member Mode

The design is performed symmetrically. In addition, the minimum reinforcement required by EN 1992-1-1, Chapter 9.5.2 (2), will be calculated:

$$A_{s,\min} = 0.10 | N_{Ed} | / f_{yd} \text{ or } 0.002 A_c, \text{ depending on which value is greater} \quad (9.12N)$$

with

$N_{Ed}$  Design value of the longitudinal force to be absorbed.

$f_{yd}$  Design value for the reinforcing steel strength at the yield strength.

DIN EN 1992-1-1:

$$A_{s,\min} = 0.15 N_{Ed} / f_{yd} \quad (9.12DE)$$

OENORM B 1992-1-1:

$$A_{s,\min} = 0.13 N_{Ed} / f_{yd} \geq 0.0026 A_c \quad (30AT)$$

SS EN 1992-1-1:

$$A_{s,\min} = 0.002 A_c \quad (\text{Article 28})$$

### Inclusion of tendons with bond

When designing beams and design objects, the internal forces of the concrete section is reduced by the statically determined portions which result from prestressing minus the losses from creep, shrinkage and prestressing steel relaxation (CSR). Situations prior to the grouting of the tendons are excluded. So only the restraint portions from 'P+CSR' and the external loads are contained in the remaining internal forces for the composite section. If necessary, the reinforcing steel positioned by the user will be increased until the composite internal forces can be absorbed. In the design mode *compression member* the prestressing steel area is taken into account when determining the minimum reinforcement according to Chapter 9.5.2 (2).

The position of the tendon groups in the section, the prestressing losses from CSR, the statically determined portions and the internal forces of the concrete section and the composite section are written to the detailed log.

As a separation into statically determined and undetermined shares of the internal forces from prestressing is not possible for shell structures, the prestressing is taken into account fully on the action side when designing the longitudinal reinforcement. As a result, on the resistance side only mild steel and concrete are considered whereas the strain reserves of the tendons with bond are not used.



## Minimum Reinforcement Against Failure Without Warning

With respect to prestressed concrete structures, a component failure without warning may not be caused by a tendon failure according to EN 1992-1-1, Chapter 5.10.1 (5)P. The failure can be prevented by adding the minimum reinforcement as described in Chapter 9.2.1 or any other measure listed in Section (6).

The minimum reinforcement is to be dimensioned according to Chapter 9.2.1 with Equation (9.1N) (also applies to reinforced concrete components). To account for this in the program, specify a base reinforcement in the reinforcing steel description.

Alternatively, you can select the minimum reinforcement in the section dialog based on the methods specified for prestressed concrete bridges in EN 1992-2, Chapter 6.1 (109) (robustness reinforcement). This reinforcement is determined based on Equation (6.101a):

$$A_{s,\min} = M_{\text{rep}} / (z_s \cdot f_{yk}) \quad (6.101a)$$

with

$M_{\text{rep}}$  Crack moment without allowance for prestressing force and under the assumption that the edge tensile stress corresponds to  $f_{\text{ctm}}$ . According to Chapter 9.2.1.1 (4), the 1.15-fold crack moment is used for components with unbonded tendons or with external prestressing.

$z_s$  Lever arm of the internal forces in the ultimate limit state.

According to EN 1992-2, Chapter 6.1 (110), the minimum reinforcement should be added in areas where tensile stresses in the concrete occur under the characteristic action combination. This process should take into account the statically undetermined prestressing effect and ignore the statically determined effect.

The program determines all stresses at the gross section. The statically determined prestressing effect can only be subtracted for beams and design objects. For area elements the prestress is alternatively reduced by a user-defined reduction factor. The lever arm  $z_s$  of the internal forces is assumed as  $0.9 \cdot d$  for the sake of simplicity. The calculated reinforcement is evenly distributed to the reinforcement layers in the tensile zone. In the design mode *symmetrical* reinforcement is also applied to the remaining layers. This will not affect the predefined relationships between the individual reinforcement layers. For sections with mode *compression member* the robustness reinforcement is not checked because minimum reinforcement is already determined during the design for bending with longitudinal force.

DIN EN 1992-1-1:

To ensure a ductile component behavior, the above-mentioned robustness reinforcement must be added instead of the minimum reinforcement from Equation (9.1N) for components with or without prestressing. The reinforcement must be evenly distributed in the tensile zone. As it is not specified in more detail, this reinforcement is determined with the characteristic combination as described above. The option to take tendons into account is not used by the program.

SS EN 1992-1-1:

According to Article 13, method D (proofs concerning the reliability of the tendons), in combination with at least one of the other methods, should be used. The second condition can be covered by adding the minimum reinforcement as described in Chapter 9.2.1 (method A) or by use of the above-mentioned robustness reinforcement.

## Surface Reinforcement

To prevent concrete spalling, a surface reinforcement may be necessary according to EN 1992-1-1, Chapter 9.2.4. For more information, refer to Annex J. The reinforcement determined in this manner can be incorporated into the program by specifying a base reinforcement in the reinforcing steel description.

OENORM B 1992-1-1:

The guidelines set forth in Annex J are not normative.

## Design for Lateral Force

Lateral force design involves determining the lateral force reinforcement and includes a concrete strut check according to EN 1992-1-1, Chapter 6.2. The following special conditions apply:

- The angle of the lateral force reinforcement is assumed to be 90°.
- The value for  $\cot \Theta$  can be selected by the user within the permissible national limits of Equation (6.7N).  
 DIN EN 1992-1-1:  
 In the calculation, the specified value for  $\cot \Theta$  is limited to the range permitted in accordance with Equation (6.7aDE) (method with load-dependent strut angle), unless the check with a constant value is selected in the section dialog. The actual effective angle of the concrete struts is logged for each check location.
- The minimum reinforcement according to Chapter 9.2.2 (5) of the standard is included in the calculated stirrup reinforcement. For areas, the minimum reinforcement as per Chapter 6.2.1 (4) will only be determined if the lateral force reinforcement is necessary for computation. For beams no minimum reinforcement is calculated for the direction with  $M = Q = 0$ .
- For beams and design objects, the shear design is performed separately for the  $Q_y$  and  $Q_z$  lateral forces.
- Slab and shell elements are designed for lateral force  $q_r = \sqrt{(q_{x^2} + q_{y^2})}$ . Depending on which has a negative effect, either the principal compressive force or principal tensile force is used for the associated longitudinal force.  
 DIN EN 1992-1-1:  
 If selected, the check will be carried out separately for the reinforcement directions x and y in accordance with Chapter 6.2.1 (10). In this case, the normal force in reinforcement direction is used for the associated longitudinal force. If lateral force reinforcement is necessary, it must be added from both directions.
- There is no reduction of the action from loads near supports as specified in Chapter 6.2.1 (8) of the standard.
- For beams and design objects, the decisive values of the equivalent rectangle are determined by the user independently of the normal section geometry. The coefficients for calculating the inner lever arm  $z$  based on the effective width and effective height must also be specified. For area elements, the calculation is generally performed with the lever arm  $z = 0.9 d$ .
- DIN EN 1992-1-1:  
 According to 6.2.3 (1), the inner lever arm is limited to the maximum value derived from  $z = d - c_{v,1} - 30 \text{ mm}$  and  $z = d - 2c_{v,1}$ . Note that  $c_{v,1}$  refers to the extent to which longitudinal reinforcement is laid in the concrete compressive zone.
- For beam sections with internal prestressing, the design value of lateral load-bearing capacity  $V_{Rd,max}$  according to Chapter 6.2.3 (6) is determined using the nominal value  $b_{w,nom}$  of the section width.
- The necessity of a lateral force reinforcement is analyzed according to Chapter 6.2.2 (1) of the standard. The special conditions listed in Sections (2) through (7) are not used in this case.
- BS EN 1992-1-1:  
 The shear strength of concrete of strength classes higher than C50/60 is limited to the value of class C50/60, according to NA to 3.1.2 (2)P. The concrete compressive strength  $f_{cd}$  according to Eq. (3.15) is determined conservatively with  $\alpha_{cc} = 0.85$ .

The used formulas from EN 1992-1-1 that are used are listed below.

### Components without computationally necessary lateral force reinforcement

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}] b_w \cdot d \quad (6.2a)$$

with at least

$$V_{Rd,c} = (v_{\min} + k_1 \cdot \sigma_{cp}) b_w \cdot d \quad (6.2b)$$

For lightweight concrete applies:

$$V_{IRd,c} = [C_{IRd,c} \cdot \eta_1 \cdot k \cdot (100 \rho_1 \cdot f_{lck})^{1/3} + k_1 \cdot \sigma_{cp}] b_w \cdot d \geq (\eta_1 \cdot v_{1,\min} + k_1 \cdot \sigma_{cp}) b_w \cdot d \quad (11.6.2)$$

where

$f_{ck}, f_{lck}$  is the characteristic concrete strength [N/mm<sup>2</sup>].

$k = 1 + \sqrt{200/d} \leq 2.0$  with  $d$  specified in mm.

$\rho_1 = A_{sl} / (b_w \cdot d) \leq 0.02$ .

$A_{sl}$  is the area of the tensile reinforcement that extends at least  $(l_{bd} + d)$  beyond the analyzed section (see Figure 6.3).

- $b_w$  is the smallest section width in the tensile zone of the section [mm].
- $\sigma_{cp} = N_{Ed} / A_c < 0.2 f_{cd}$  [N/mm<sup>2</sup>].
- $N_{Ed}$  is the normal force in the section due to loading or prestressing [N]  
( $N_{Ed} > 0$  for compression). The influence of the forced deformations on  $N_{Ed}$  can be ignored.
- $A_c$  is the entire area of the concrete section [mm<sup>2</sup>].
- $V_{Rd,c}, V_{lRd,c}$  is the design value of the lateral force resistance [N].
- $\eta_1$  is the reduction coefficient for lightweight concrete according to Eq. (11.1).

The recommended values are:

- $C_{Rd,c} = 0.18 / \gamma_c$  for normal concrete.
- $C_{lRd,c} = 0.15 / \gamma_c$  for lightweight concrete.
- $k_1 = 0.15$
- $v_{min} = 0.035 k^{3/2} \cdot f_{ck}^{1/2}$  for normal concrete. (6.3N)
- $v_{l,min} = 0.028 k^{3/2} \cdot f_{lck}^{1/2}$  for lightweight concrete according to 11.6.1 (1).

DIN EN 1992-1-1:

- $C_{Rd,c} = C_{lRd,c} = 0.15 / \gamma_c$
- $k_1 = 0.12$
- $v_{min} = (\kappa_1 / \gamma_c) k^{3/2} \cdot f_{ck}^{1/2}$
- $v_{l,min} = (\kappa_1 / \gamma_c) k^{3/2} \cdot f_{lck}^{1/2}$
- with
- $\kappa_1 = 0.0525$  for  $d < 600$  mm
- $\kappa_1 = 0.0375$  for  $d > 800$  mm

For  $600 \text{ mm} < d \leq 800 \text{ mm}$  can be interpolated.

### Components with computationally necessary lateral force reinforcement

The angle  $\theta$  between the concrete struts and the component axis perpendicular to the lateral force must be limited:

$$1 \leq \cot \theta \leq 2.5 \quad (6.7N)$$

DIN EN 1992-1-1:

$$1.0 \leq \cot \theta \leq (1.2 + 1.4 \sigma_{cp} / f_{cd}) / (1 - V_{Rd,cc} / V_{Ed}) \leq 3.0 \quad (6.7aDE)$$

with

$$V_{Rd,cc} = c \cdot 0.48 \cdot f_{ck}^{1/3} (1 - 1.2 \sigma_{cp} / f_{cd}) \cdot b_w \cdot z \quad (6.7bDE)$$

The individual parts of which are

$$c = 0.5$$

$\sigma_{cp}$  The design value of the concrete longitudinal stress at the level of the centroid of the section with  $\sigma_{cp} = N_{Ed} / A_c$  in N/mm<sup>2</sup>.

$N_{Ed}$  The design value of the longitudinal force in the section caused by external actions ( $N_{Ed} > 0$  as longitudinal compressive force).

For lightweight concrete the strut angle is to be limited to  $\cot \theta = 2$  in accordance with Eq. (6.7aDE). The input value  $V_{Rd,cc}$  from Eq. (6.7bDE) is to be multiplied by  $\eta_1$  according to Eq. (11.1).

OENORM B 1992-1-1:

$$0.6 \leq \tan \theta \leq 1.0 \quad (3AT)$$

If the section is in compression, then  $\theta$  in the range

$$0.4 \leq \tan \theta \leq 1.0 \quad (4AT)$$

may be selected.

SS EN 1992-1-1:

According to Article 15 and differing from Equation (6.7N), for prestressed components the condition  $1.0 \leq \cot \Theta \leq 3.0$  applies.

For components with lateral force reinforcement perpendicular to the complement axis, the lateral force resistance  $V_{Rd}$  is the smaller value from

$$V_{Rd,s} = (A_{sw} / s) \cdot z \cdot f_{ywd} \cdot \cot \theta \quad (6.8)$$

and

$$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta) \quad (6.9)$$

where

$A_{sw}$  is the section area of the lateral force reinforcement.

$s$  is the distance of the stirrups to each other.

$f_{ywd}$  is the design value for the yield strength of the lateral force reinforcement.

$v_1$  is a reduction coefficient for the concrete strength when shear cracks occur.

$\alpha_{cw}$  is a coefficient for taking into account the stress state in the compression chord.

The recommended values are:

$$v_1 = v$$

with

$$v = 0.6 (1 - f_{ck} / 250) \quad \text{for normal concrete } (f_{ck} \text{ in N/mm}^2) \quad (6.6N)$$

$$v = 0.5 \eta_1 (1 - f_{lck} / 250) \quad \text{for lightweight concrete } (f_{lck} \text{ in N/mm}^2) \quad (11.6.6N)$$

$$\alpha_{cw} = 1 \quad \text{for non-prestressed components}$$

$$= (1 + \sigma_{cp} / f_{cd}) \quad \text{for } 0 < \sigma_{cp} \leq 0.25 f_{cd} \quad (6.11aN)$$

$$= 1.25 \quad \text{for } 0.25 f_{cd} < \sigma_{cp} \leq 0.5 f_{cd} \quad (6.11bN)$$

$$= 2.5 (1 - \sigma_{cp} / f_{cd}) \quad \text{for } 0.5 f_{cd} < \sigma_{cp} \leq 1.0 f_{cd} \quad (6.11cN)$$

where

$\sigma_{cp}$  is the average compressive stress in the concrete (indicated as a positive value) as a result of the design value for the normal force.

The maximum effective section area of the lateral force reinforcement  $A_{sw,max}$  for  $\cot \theta = 1$  is derived from:

$$(A_{sw,max} \cdot f_{ywd}) / (b_w \cdot s) \leq \frac{1}{2} \alpha_{cw} \cdot v \cdot f_{cd} \quad (6.12)$$

The additional tensile force in the longitudinal reinforcement due to lateral force according to Eq. (6.18) is

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \Theta - \cot \alpha). \quad (6.18)$$

DIN EN 1992-1-1:

$$v_1 = \eta_1 \cdot 0.75 \cdot \min(1.0; 1.1 - f_{ck} / 500)$$

$$\eta_1 = 1.0 \text{ for normal concrete and as per Eq. (11.1) for lightweight concrete.}$$

$$\alpha_{cw} = 1.0$$

Equation (6.12) is not applied.

BS EN 1992-1-1:

$$v_1 = v \cdot (1 - 0.5 \cos \alpha)$$

The further regulations of NDP to 6.2.3 (3) are not taken into account.

**Lateral force reinforcement (Standard design)**

The lateral force reinforcement level is derived from Equation (9.4):

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) \tag{9.4}$$

where

$\rho_w$  is the reinforcement level of the lateral force reinforcement. In general, this level may not be smaller than

$$\rho_{w,min}$$

$A_{sw}$  is the section area of the lateral force reinforcement per length  $s$ .

$s$  is the distance of the lateral force reinforcement as measured along the component axis.

$b_w$  is the web width of the component.

$\alpha$  is the angle between the lateral force reinforcement and the component axis.

The recommended value for the minimum reinforcement is:

$$\rho_{w,min} = 0.08 \sqrt{f_{ck}} / f_{yk} \tag{9.5N}$$

DIN EN 1992-1-1:

$$\rho_{w,min} = 0.16 f_{ctm} / f_{yk} \tag{9.5aDE}$$

With respect to slabs, the value can vary between zero and the above value as described in Chapter 9.3.2 (2).

For structured sections with prestressed tension chord, the following applies:

$$\rho_{w,min} = 0.256 f_{ctm} / f_{yk} \tag{9.5bDE}$$

ÖENORM B 1992-1-1:

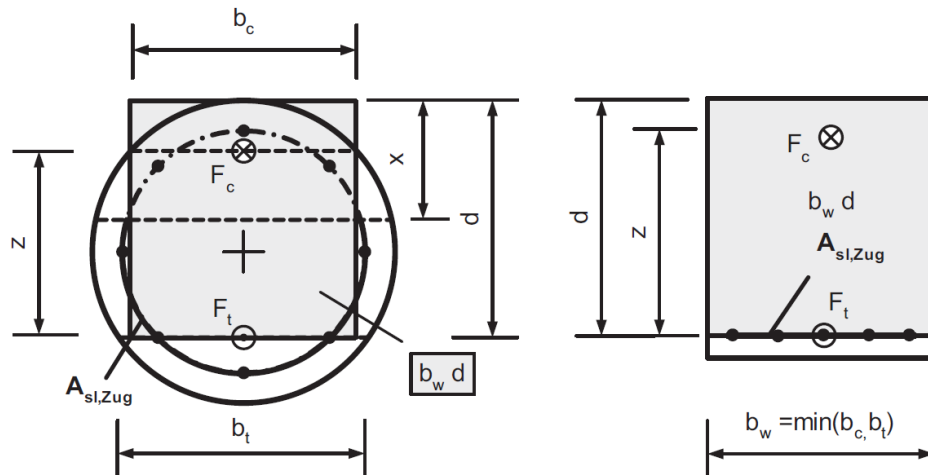
$$\rho_{w,min} = 0.15 f_{ctm} / f_{yd} \tag{24AT}$$

For slabs with a calculated shear reinforcement at least the 0.6-fold value of the minimum shear reinforcement of beams is to be applied.

**Lateral force design for circular and annular cross-sections according to Bender et al.**

For circular and annular cross sections with annular single stirrups or helixes, the lateral force design is optionally carried out according to Bender et al. (2010) for the resulting shear force  $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$ .

In its interpretation of 1 June 2012 of Chapter 10.3 of DIN 1045-1:2008, the German Committee for Structural Engineering (NABau) recommends using the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces for the effective width  $b_w$  (see following figure). The values for the width  $b_w$ , the effective height  $d$  and the inner lever arm  $z$  are defined in the cross-section dialog.



Definition of the effective width  $b_w$  as per NABau (2012) [Fig. from: Bender et al. (2006), p. 87]

For structural members without shear reinforcement, the resistance  $V_{Rd,ct}$  is given according to Bender et al. (2006),

Equ. (1), in accordance with DIN 1045-1:2008, Equ. (70). Therefore, the program uses the above equations (6.2a), (6.2b) and (11.6.2) of EN 1992-1-1 with the selected value for  $b_w$ .

For structural members with shear reinforcement, the design is carried out according to Bender et al. (2010):

$$V_{Rd,sy} = \alpha_k \cdot A_{sw} / s_w \cdot f_{yd} \cdot z \cdot \cot \Theta \cdot \sin \alpha \quad (10)$$

$$V_{Rd,max} = \alpha_k \cdot b_w \cdot z \cdot \alpha_c \cdot f_{cd} \cdot \cot \Theta / [(\cot \Theta \cdot \cot \alpha)^2 + 1] \quad (11)$$

where

$\alpha_k$  is an efficacy factor, which is stress-dependent ( $0.715 \leq \alpha_k \leq 0.785$ ) according to Bender et al. (2010), p. 422, and can be assumed with the mean value  $\alpha_k = 0.75$ .

$A_{sw}$  is the section area of the lateral force reinforcement per length  $s_w$ .

$s_w$  is the distance of the lateral force reinforcement as measured along the component axis.

$b_w$  is the effective cross-section width.

$z$  is the inner lever arm.

$\Theta$  is the inclination of the concrete compressive struts.

$\alpha$  is the angle between the lateral force reinforcement and the component axis (helix inclination).

$f_{yd}$  is the design value for the yield strength of the lateral force reinforcement.

$f_{cd}$  is the design value of the concrete compressive strength.

$\alpha_c$  is a coefficient to account for the stress state in the compression chord.

The additional tensile force in the longitudinal reinforcement due to lateral force  $Q_r$  is determined according to equation (6.18) of the standard. In case of simultaneous loading by lateral force and torsion, the torsion design is carried out according to the standard for vertical stirrups assuming a square torsion box.

The design results are stored separately from the standard design results.

## Design for Torsion and Combined Stressing

The design for torsion is carried out according to EN 1992-1-1, Chapter 6.3. It includes the calculation of the diagonal tensile reinforcement and the longitudinal reinforcement based on Equation (6.28) and the concrete strut check under lateral force based on Formula (6.29) of the standard.

The equivalent section on which this design is based is defined by the user independently of the normal section geometry.

### Strut angle

According to 6.3.2 (2), the rules set forth in Chapter 6.2.3 (2) for lateral force also apply for the strut angle.

DIN EN 1992-1-1:

For combined stress from torsion and proportional lateral force,  $V_{Ed}$  in Equation (6.7aDE) must include the shear force of the wall  $V_{Ed,T+V}$  based on Equation (NA.6.27.1) and  $b_w$  in Equation (6.7bDE) must include the effective thickness of wall  $t_{ef}$ . The check for both lateral force and torsion must be carried out using the selected angle  $\Theta$ . The reinforcements determined in this manner are to be added together.

$$V_{Ed,T+V} = V_{Ed,T} + V_{Ed} \cdot t_{ef} / b_w \quad (NA.6.27.1)$$

Alternatively a strut angle of  $45^\circ$  for torsion according to Chapter 6.3.2 (2) or a constant value  $\cot \Theta$  for lateral force and torsion (cf. interpretation No. 24 of NABau for DIN 1045-1) can be chosen in the section dialog.

### Torsion reinforcement

The necessary reinforcement is to be determined according to Chapter 6.3.2 (3):

$$\Sigma A_{sl} \cdot f_{yd} / u_k = T_{Ed} / 2A_k \cdot \cot \Theta \quad (6.28)$$

or

$$A_{sw} \cdot f_{yd} / s = T_{Ed} / 2A_k \cdot \tan \Theta$$

where

$A_{sl}$  is the section area of the longitudinal torsional reinforcement. The possibility of reducing the longitudinal torsional reinforcement in compression chords is not used.

- $A_{sw}$  is the section area of the torsion reinforcement perpendicular to the component axis.  
 $u_k$  is the perimeter of area  $A_k$ .  
 $s$  is the distance of the torsion reinforcement as measured along the component axis.  
 $A_k$  is the area enclosed by the center lines of the walls.

For approximately rectangular full sections, only the minimum reinforcement defined in Section (5) is necessary if the condition expressed by Equation (6.31) is met:

$$T_{Ed} / T_{Rd,c} + V_{Ed} / V_{Rd,c} \leq 1.0 \quad (6.31)$$

where

$T_{Rd,c}$  is the torsion crack moment which, according to Zilch (2006, p. 290), is defined as  $T_{Rd,c} = f_{ctd} \cdot W_T$ .

$V_{Rd,c}$  is the lateral force resistance according to Equation (6.2).

DIN EN 1992-1-1:

The condition (6.31) is supplemented with the following equations:

$$T_{Ed} \leq \frac{V_{Ed} \cdot b_w}{4.5} \quad (NA.6.31.1)$$

$$V_{Ed} \left[ 1 + \frac{4.5 T_{Ed}}{V_{Ed} \cdot b_w} \right] \leq V_{Rd,c} \quad (NA.6.31.2)$$

### Strut load-bearing capacity

To avoid exceeding the strut load-bearing capacity of a component subject to torsion and lateral force, the following condition must be met:

$$T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max} \leq 1.0 \quad (6.29)$$

where

$T_{Ed}$  is the design value of the torsion moment.

$V_{Ed}$  is the design value of the lateral force.

$T_{Rd,max}$  is the design value of the absorbable torsion moment based on

$$T_{Rd,max} = 2 v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t_{ef,i} \cdot \sin \Theta \cdot \cos \Theta \quad (6.30)$$

with  $\alpha_{cw}$  according to Equation (6.9) and  $v$  according to Eq. (6.6N) for normal concrete and according to Eq. (11.6.6N) for lightweight concrete.

DIN EN 1992-1-1:

For compact sections, the interaction Equation (NA.6.29.1) is used:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \quad (NA.6.29.1)$$

In Equation (6.30)  $v = \eta_1 \cdot 0.75$  is used for box sections and  $v = \eta_1 \cdot 0.525 \cdot \min(1.0; 1.1 - f_{ck} / 500)$  in all other cases with

$\eta_1 = 1$  for normal concrete and as per Eq. (11.1) for lightweight concrete.

OENORM B 1992-1-1:

For full sections the following interaction equation can be used:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \quad (9AT)$$

## Shear Joint Check

The shear joint check is performed for beam elements and design objects in accordance with EN 1992-1-1, Chapter 6.2.5. It is carried out for the shear force in the z-direction of the cross-section and is only useful for components that are mainly stressed in this direction. The transmission of shear force through the joint is ensured if the following conditions are met:

$$v_{Edi} \leq v_{Rdi} \quad (6.23)$$

**Design value of the shear stress in the joint  $v_{Edi}$ :**

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) \quad (6.24)$$

where

$\beta$  is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered. The ratio factor  $\beta$  is calculated depending on whether the shear joint is in the compression or tension zone and whether reinforcement in the existing concrete or in the concrete supplement was determined from the other checks with the internal forces associated with  $V_{Ed}$  in state II as follows (see also Booklet 600 to 6.2.5 (1)):

Outer edge of concrete supplement	Shear joint	Tensile reinforcement		$\beta$
		Concrete supplement	Existing Concrete	
Tension	Ten. / Comp.	-	-	1
Compression	Compression	-	-	$0 < F_{cdi} / F_{cd} < 1$
	Tension	Yes	Yes	$0 < F_{sdi} / (F_{sd} + F_{sdi}) < 1$
		Yes	No	0 (No check)
		No	Yes	1
No	No	0 (No check)		

$z$  is the lever arm of the composite section. The lever arm is assumed as in the lateral force check according to the specifications in the shear section.

$b_i$  is the width of the joint.

$F_{cdi}$  is the concrete compression force in the concrete supplement in the compression zone.

$F_{cd}$  is the total concrete compression force in the compression zone.

$F_{sdi}$  is the tension force of the reinforcing steel layers in the concrete supplement in the tensile zone.

$F_{sd}$  is the total tension force of the reinforcing steel layers in the tensile zone.

**Design value of the shear resistance in the joint  $v_{Rdi}$ :**

$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} (\mu \cdot \sin \alpha + \cos \alpha) \leq 0.5 \cdot v \cdot f_{cd} \quad (6.25)$$

DIN EN 1992-1-1:

The load bearing component of the transverse reinforcement from the shear friction in Eq. (6.25) may be increased to  $\rho \cdot f_{yd} (1.2 \cdot \mu \cdot \sin \alpha + \cos \alpha)$ .

For very smooth joints with external compression normal force perpendicular to the joint, the friction component in Eq. (6.25) may be taken into account up to the limit  $\mu \cdot \sigma_n \leq 0.1 \cdot f_{cd}$  according to NPD 6.2.2 (6).

where

$c, \mu$  are factors which depend on the roughness of the joint (see 6.2.5 (2)). Under dynamic or fatigue loads, the value  $c$  is halved according to 6.2.5 (5).

DIN EN 1992-1-1:

For very smooth joints under dynamic or fatigue loads,  $c = 0$  is to be assumed.

$f_{ctd}$  is the design value of the concrete tensile strength according to 3.1.6 (2)P.

$\sigma_n$  is the stress caused by the minimum normal force perpendicular to the joint, which can act simultaneously with the lateral force (positive for compression with  $\sigma_n < 0.6 f_{cd}$  and negative for tension). If  $\sigma_n$  is a tensile stress  $c \cdot f_{ctd}$  should usually be set to 0.



$$\rho = A_s / A_i$$

$A_s$  is the area of reinforcement crossing the joint, including ordinary shear reinforcement, with adequate anchorage at both sides of the interface.

$A_i$  is the area of the joint.

$\alpha$  the angle of inclination of the transverse reinforcement. This is set at 90° by the program.

$v$  is a strength reduction factor according to 6.2.2 (6).

OENORM B 1992-1-1:

If reinforcement is required, the following minimum reinforcement must be inserted perpendicular to the joint for beam-like components:

$$\rho_{\min} = 0.2 \cdot f_{\text{ctm}} / f_{\text{yk}} \geq 0.001$$

## Punching Shear

The load-bearing safety check against punching shear is carried out according to EN 1992-1-1, Chapter 6.4. This check is used to determine the necessary punching reinforcement. The following special conditions apply:

- The orthogonal effect directions labeled as  $y$  and  $z$  in the standard are indicated below as well as in the dialog and the listing as  $x$  and  $y$  in order to comply with the area reinforcement directions commonly used in the program.
- The average effective static height  $d$  results from the input parameters  $d_x$  and  $d_y$  with  $d = (d_x + d_y) / 2$ . These parameters are to be selected as shown in Figure 6.12, 6.16 or 6.17.
- No check is carried out for pad footings.
- The action can be entered directly or taken from the analyzed design situation at the ultimate limit state. In this case,  $V_{\text{Ed}}$  is set to the maximum support force  $R_z$  for each corresponding action combination.
- BS EN 1992-1-1:  
The shear strength of concrete of strength classes higher than C50/60 is limited to the value of class C50/60, according to NA to 3.1.2 (2)P.

The check is considered fulfilled if:

1. For slabs without punching reinforcement

$$v_{\text{Ed}} \leq v_{\text{Rd,max}}$$

$$v_{\text{Ed}} \leq v_{\text{Rd,c}}$$

2. For slabs with punching reinforcement

$$v_{\text{Ed}} \leq v_{\text{Rd,max}}$$

$$v_{\text{Ed}} \leq v_{\text{Rd,cs}}$$

DIN EN 1992-1-1, OENORM B 1992-1-1:

3. The minimum longitudinal reinforcement is maintained

with

$$v_{\text{Ed}} = \frac{\beta \cdot V_{\text{Ed}}}{u_1 \cdot d} \tag{6.38}$$

where

$v_{\text{Ed}}$  is the maximum acting lateral force per area unit.

$V_{\text{Ed}}$  is the design value of the entire lateral force to be absorbed. For foundation slabs the lateral force may be reduced due to the favorable action of the soil pressure according to 6.4.3 (8). For the reduction the program assumes the area within  $u_1$  unless the national annex contains a different rule.

$\beta$  is the load increase factor for taking into account eccentric load introduction according to Equation (6.39). The value is specified by the user.

DIN EN 1992-1-1, OENORM B 1992-1-1:

Values smaller than 1.10 are not permitted.

$d$  is the average effective height of the slab, which can be assumed as  $(d_x + d_y) / 2$ , with:

$d_x, d_y$  is the effective static height of the slab in the  $x$  or  $y$  direction in the section area of the analyzed perimeter.

- $u_i$  is the circumference of the analyzed perimeter.
- $v_{Rd,c}$  is the design value of the punching resistance per area unit for a slab without punching reinforcement.
- $v_{Rd,cs}$  is the design value of the punching resistance per area unit for a slab with punching reinforcement.
- $v_{Rd,max}$  is the design value of the maximum punching resistance per area unit.

The load discharge areas and check sections as per Chapter 6.4.2, Section (1) to (7), are taken into consideration. The user-specified opening dimensions are used to calculate the check sections.

### Punching resistance without punching reinforcement

The punching resistance of a slab without punching reinforcement is calculated as

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \quad (6.47)$$

For lightweight concrete applies

$$v_{lRd,c} = C_{lRd,c} \cdot k \cdot \eta_1 \cdot (100 \rho_1 \cdot f_{lck})^{1/3} + k_2 \cdot \sigma_{cp} \geq (\eta_1 \cdot v_{l,min} + k_2 \cdot \sigma_{cp}) \quad (11.6.47)$$

where

$f_{ck}$  is the characteristic concrete strength [N/mm<sup>2</sup>]

$k = 1 + \sqrt{(200 / d)} \leq 2.0$ ;  $d$  [mm]

$\rho_1 = \sqrt{(\rho_{lx} \cdot \rho_{ly})} \leq 0.02$

$\rho_{lx}, \rho_{ly}$  is the reinforcement level based on the fixed tensile reinforcement in the x or y direction. The values  $\rho_{lx}$  and  $\rho_{ly}$  are normally calculated as average values given a slab width based on the column measurements plus  $3 \cdot d$  for each side.

$\sigma_{cp} = (\sigma_{cx} + \sigma_{cy}) / 2$

where

$\sigma_{cx}, \sigma_{cy}$  are the normal stresses in the concrete in the x and y directions in the critical section (MN/m<sup>2</sup>, positive for pressure):

$$\sigma_{cx} = N_{Ed,x} / A_{cx} \text{ and } \sigma_{cy} = N_{Ed,y} / A_{cy}$$

$N_{Ed,x}, N_{Ed,y}$  are the normal forces acting on internal columns in the entire section area of the analyzed perimeter and the normal forces acting on the edge and corner columns in the area of the analyzed perimeter. These forces result from loads or prestressing.

$A_c$  is the section area of the concrete according to the definition of  $N_{Ed}$ .

The recommended country-specific values are:

$C_{Rd,c} = 0.18 / \gamma_c$  for normal concrete

$C_{lRd,c} = 0.15 / \gamma_c$  for lightweight concrete

$k_1 = 0.1$

$k_2 = 0.08$

$v_{min}$  derived from Equation (6.3N) for normal concrete:

$$v_{min} = 0.035 \cdot k^{3/2} \cdot f_{ck}^{1/2} \quad (6.3N)$$

$v_{l,min}$  derived from Chapter 11.6.1 (1) for lightweight concrete:

$$v_{l,min} = 0.028 \cdot k^{3/2} \cdot f_{lck}^{1/2}$$

For column foundations and foundation slabs the following applies according to Chapter 6.4.4 (2):

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{1/3} \cdot 2 \cdot d/a \geq (v_{min} \cdot 2 \cdot d/a) \quad (6.50)$$

$$v_{lRd,c} = C_{lRd,c} \cdot k \cdot \eta_1 \cdot (100 \rho_1 \cdot f_{lck})^{1/3} \cdot 2 \cdot d/a \geq (\eta_1 \cdot v_{l,min} \cdot 2 \cdot d/a) \quad (11.6.50)$$

$a$  Distance from the column edge to the decisive perimeter. Within the perimeter the soil pressures deducting the foundation dead load are allowed for relieving.

DIN EN 1992-1-1:

$v_{\min}$  as in Section 6.2.2 (1)

For internal columns of flat slabs with  $u_0/d < 4$  the following applies according to Book 600 of the DAfStb:

$$C_{Rd,c} = C_{IRd,c} = 0.18 / \gamma_c \cdot (0.1 \cdot u_0/d + 0.6) \geq 0.15 / \gamma_c \quad (\text{H.6-16})$$

For column foundations and foundation slabs the following applies:

$a$  The program does not iterate over the distance  $a$  according to NCI of 6.4.4 (2). If a constant perimeter with  $a = 1.0 d$  is assumed, only 50% of the soil pressures are allowed for relieving. This is taken into account during determination of the resistance.

$$C_{Rd,c} = C_{IRd,c} = 0.15 / \gamma_c$$

In all other cases the recommended value for  $C_{Rd,c}$  applies.

The bending reinforcement level  $\rho_1$  must also be limited to  $\rho_1 \leq 0.5 f_{cd} / f_{yd}$ .

OENORM B 1992-1-1:

The allowable reinforcement level for determining  $v_{Rd,c}$  must not exceed

$$\rho_1 = 0.4 \cdot f_{cd} / f_{yd} \leq 0.02.$$

For foundation slabs the program does not iterate over the distance  $a$  according to the supplement to 6.4.4 (2). For simplification, a constant perimeter with  $a = 1.0 d$  can be assumed.

### Punching resistances with punching reinforcement

1) The punching resistances with punching reinforcement are calculated as

$$v_{Ed} = \beta \cdot V_{Ed} / (u_0 \cdot d) \leq v_{Rd,max} \quad \text{for normal concrete} \quad (6.53)$$

$$v_{Ed} = V_{Ed} / (u_0 \cdot d) \leq v_{IRd,max} \quad \text{for lightweight concrete} \quad (11.6.53)$$

where

$u_0$  For an internal column  $u_0 = \text{Circumference of the column}$

For an edge column  $u_0 = c_2 + 3 \cdot d \leq c_2 + 2 \cdot c_1$

For a corner column  $u_0 = 3 \cdot d \leq c_1 + c_2$

$c_1, c_2$  are the column dimensions as shown in Figure 6.20. For circular columns, the values  $c_1, c_2$  in the equations for edge and corner columns are assumed to be  $c_1 = c_2 = 0.25 \cdot \text{column circumference}$ . For wall ends and wall corners,  $u_0$  is determined corresponding to edge and corner columns. Defined openings are taken into account by reducing  $u_0$  as for the critical perimeter.

OENORM B 1992-1-1: The possibility of simplification in case of round edge and corner columns is not used.

$\beta$  See 6.4.3 (3), (4) and (5).

$v_{Ed}$  is the lateral force to be absorbed at the column section per area unit.

The recommended country-specific values are:

$$v_{Rd,max} = 0.4 \cdot v \cdot f_{cd} \quad \text{with } v \text{ as per Eq. (6.6N)}$$

$$v_{IRd,max} = 0.4 \cdot v \cdot f_{lcd} \quad \text{with } v \text{ as per Eq. (11.6.6N)}$$

DIN EN 1992-1-1:

The maximum punching resistance is determined within the critical perimeter  $u_1$ :

$$v_{Ed,u1} \leq v_{Rd,max} = 1.4 \cdot v_{Rdc,u1} \quad (\text{NA.6.53.1})$$

Deviating from NDP of 6.4.5 (3),  $v_{Rdc,u1}$  is set to  $v_{Rd,c}$  according to Eq. (6.47).

OENORM B 1992-1-1:

For slabs or foundations with small load introduction areas with  $u_0/d < 4$ , the maximum punching resistance  $v_{Rd,max}$  shall be limited to the recommended values.

SS EN 1992-1-1:

$$v_{Rd,max} \leq 0.50 \cdot v \cdot f_{cd} \quad (\text{Article 16})$$

$$v_{Rd,max} \leq 0.50 \cdot v \cdot f_{lcd} \quad (\text{Article 36a})$$

BS EN 1992-1-1:

$$v_{Rd,max} = 0.5 \cdot v \cdot f_{cd}$$

For lightweight concrete the recommended value applies. The concrete compressive strength  $f_{cd}$  according to Eq. (3.15) is determined conservatively with  $\alpha_{cc} = 0.85$ .

2) The first reinforcement row is specified with a distance of  $0.5 \cdot d$  from the column edge; the other reinforcement rows are specified with a distance of  $s_r \leq 0.75 \cdot d$  from each other (see Figure 9.10). The reinforcement is determined using the following equation:

$$v_{Rd,cs} = 0.75 \cdot v_{Rd,c} + 1.5 \cdot (d / s_r) A_{sw} \cdot f_{ywd,ef} \cdot (1 / (u_1 \cdot d)) \sin \alpha \leq k_{max} \cdot v_{Rd,c} \quad [\text{MN/m}^2] \quad (6.52)$$

where

$A_{sw}$  is the section area of the punching reinforcement in a reinforcement row around the column [mm<sup>2</sup>].

$s_r$  is the radial distance of the punching reinforcement rows [mm].

$f_{ywd,ef}$  is the effective design value for the yield strength of the punching reinforcement according to  $f_{ywd,ef} = 250 + 0.25 \cdot d \leq f_{ywd}$  [MN/m<sup>2</sup>]

$u_1$  is the circumference of the critical perimeter [mm].

$d$  is the average value of the effective static heights in the orthogonal directions [mm].

$\alpha$  is the angle between the punching reinforcement and the slab plane.

$k_{max}$  is the factor for limiting the maximal load-bearing capacity with punching reinforcement. The recommended value is 1.5.

$$\left| \text{BS EN 1992-1-1: } k_{max} = 2.0 \right.$$

$$\left| \text{SS EN 1992-1-1: } k_{max} = 1.6 \right.$$

For bent-down reinforcement  $d / s_r = 0.67$  is used according to Section 6.4.5 (1).

DIN EN 1992-1-1:

Within the perimeter  $u_{out}$  as per Section 6.4.5 (4), a minimum of two reinforcement rows is always required. For the first two reinforcement rows of flat slabs,  $A_{sw}$  is to be increased by the factors  $k_{sw,1} = 2.5$  resp.  $k_{sw,2} = 1.4$ . For bent-down punching shear reinforcement  $d / s_r = 0.53$  is used. The bent-down reinforcement can be exploited with  $f_{ywd,ef} = f_{ywd}$ . For foundation slabs the stirrup reinforcement is calculated according to the following Equation:

$$\beta \cdot V_{Ed,red} \leq V_{Rd,s} = A_{sw,1+2} \cdot f_{ywd,ef} \quad (\text{NA.6.52.1})$$

For bent-down reinforcement:

$$\beta \cdot V_{Ed,red} \leq V_{Rd,s} = 1.3 A_{sw,1+2} \cdot f_{ywd} \sin \alpha \quad (\text{NA.6.52.2})$$

In order to determine  $V_{Ed,red}$ , the reduction value  $\Delta V_{Ed}$  in Eq. (6.48) is calculated using the area  $A_{crit}$  according to Figure NA.6.12.1 for the first two reinforcement rows and  $A_i > A_{crit}$  for the following rows  $i > 2$ , whereas in each case only 50% of the soil pressures are applied relieving. For the first two rows, 50% of the reinforcement amount determined with Eq. (NA.6.52.1) are required whereas 33% should be installed in each of the following rows. The first reinforcement row is specified with a distance of  $0.3 \cdot d$  from the column edge. For the first three rows the distance  $s_r$  between the rows should be limited to  $0.5 \cdot d$ .

OENORM B 1992-1-1:

For each of the first two rows  $A_{sw}$  is to be increased by 60%. The coefficient  $k_{max}$  in Eq. (6.52) is assumed to be  $k_{max} = 1.65$ . According to OENORM, it is assumed that the punching reinforcement comprises the respective lower layer of the bending reinforcement or consists of bent-up bars. For the given normal force  $N_{Ed}$  it is assumed that it does not include a compressive normal force from prestressing.

3) The perimeter  $u_{out,ef}$ , which does not need any punching reinforcement, is normally calculated based on Equation (6.54):

$$u_{out,ef} = \beta \cdot V_{Ed} / (v_{Rd,c} \cdot d) \quad (6.54)$$

In general, the outermost row of the punching reinforcement must not be farther from  $u_{out,ef}$  than  $1.5 \cdot d$ .

DIN EN 1992-1-1:

$v_{Rd,c}$  is determined as the lateral force resistance according to Chapter 6.2.2 (1).

4) For the minimum required punching reinforcement  $A_{sw,min}$  of the internal check sections, the following applies:

$$A_{sw,min} \cdot (1.5 \cdot \sin \alpha + \cos \alpha) / (s_r \cdot s_t) \geq 0.08 \cdot \sqrt{f_{ck}} / f_{yk} \quad (9.11)$$

where

$\alpha$  is the angle between the punching reinforcement and the longitudinal reinforcement (i.e., for vertical stirrups  $\alpha = 90^\circ$  and  $\sin \alpha = 1$ ).

$s_r$  is the radial distance of the stirrups of the punching reinforcement.

$s_t$  is the tangential distance of the stirrups of the punching reinforcement.

$f_{ck}$  in N/mm<sup>2</sup>.

DIN EN 1992-1-1:

$$A_{sw,min} = A_s \cdot \sin \alpha = 0.08 / 1.5 \cdot \sqrt{f_{ck}} / f_{yk} \cdot s_r \cdot s_t \quad (9.11DE)$$

5) Minimum longitudinal reinforcement

DIN EN 1992-1-1:

The minimum reinforcement is found according to Chapter 6.4.5 (NA.6) based on the design of the minimum moments:

$$m_{Ed,x} = \eta_x \cdot V_{Ed} \text{ and } m_{Ed,y} = \eta_y \cdot V_{Ed} \quad (NA.6.54.1)$$

with

$\eta_x, \eta_y$  the moment coefficient as per Table NA.6.1.1 for the x or y direction.

OENORM EN 1992-1-1:

The minimum reinforcement is determined according to Equation (28AT):

$$a_{s,min} = \frac{V_{Ed}}{0.9 \cdot d \cdot f_{yd}} \cdot \frac{e}{b_{eff}} \quad (28AT)$$

with

$e/b_{eff}$  the relative eccentricity as per Table 14AT.

## Checks Against Fatigue

The following checks according to EN 1992-1-1 are available:

- Fatigue of longitudinal reinforcement, shear reinforcement and prestressing steel (Chapter 6.8.5, 6.8.6)
- Fatigue of concrete under compressive stress (Chapter 6.8.7)
- Fatigue of the concrete compressive struts under lateral force and torsion (Chapter 6.8.7 (3))

The user can select two alternative methods for design:

- Simplified check for the frequent action combination according to EN 1992-1-1, Chapter 6.8.6 (2), and EN 1990, Eq. (6.15b), taking the relevant traffic loads at serviceability limit state into account.
- Check with damage equivalent stress ranges for the fatigue combination according to EN 1992-1-1, Chapter 6.8.3, Eq. (6.69), considering the specific fatigue load  $Q_{fat}$ .

The concrete compressive stresses are determined for both cases in state II. Differing from Chapter 5.10.9 the variation of prestressing is not taken into account.

## Design Combinations

For the check against fatigue two alternative action combinations can be used:

- Frequent combination for simplified checks according to EN 1992-1-1, Chapter 6.8.6 (2) in conjunction with EN 1990, Chapter 6.5.3.

$$\sum_{j \geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.15b)$$

- Fatigue combination for checks with damage equivalent stress ranges.

$$\left( \sum_{j \geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right) + Q_{fat} \quad (6.69)$$

In this equation  $Q_{k,1}$  and  $Q_{k,i}$  are non-cyclic, non-permanent actions whereas  $Q_{fat}$  defines the relevant fatigue load.

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

## Stress-Strain Curves

For checks against fatigue the following characteristics apply:

- Concrete: Stress-strain curve according to EN 1992-1-1, Figure 3.2, where a horizontal curve is assumed for strains of  $\epsilon_{c1}$  or higher (cf. Rossner, Graubner 2012, p. 511 and Nguyen et al. 2015, p. 158).
- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be  $k \cdot f_{yk}$  with  $k = 1.05$  as per Table C.1, class A.

DIN EN 1992-1-1:

The maximum stress is assumed to be  $1.05 \cdot f_{yk} / \gamma_s$  for ductility class A according to DIN 488-1.

- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of  $f_{p;0,1k}$ .

## Fatigue of Longitudinal Reinforcement, Shear Reinforcement and Prestressing Steel

The fatigue check is carried out according to EN 1992-1-1, Chapter 6.8. The steel stresses are calculated for longitudinal reinforcement from bending and longitudinal force as well as for prestressing steel in beams and design objects under the assumption of a cracked concrete section. For shear and longitudinal reinforcement from lateral force and torsion, the stresses are calculated according to 6.8.2 (3) based on a truss model with the strut angle  $\tan \Theta_{\text{fat}} = \sqrt{\tan \Theta} \leq 1$  acc. to Eq. (6.65). Where  $\Theta$  is the angle between the concrete compression struts and the beam axis used in the corresponding ultimate limit state design. The prestressing steel stresses in area elements are determined at the uncracked concrete section. Tendons without bond and external tendons are not checked.

DIN EN 1992-1-1:

The strut angle is to be determined according to Eq. (H.6-26) of Book 600 of the DAfStb.

### Simplified check

According to Chapter 6.8.6, adequate fatigue resistance may be assumed if the stress range under the frequent action combination does not exceed 70 MN/m<sup>2</sup> for unwelded reinforcing bars and 35 MN/m<sup>2</sup> for welded bars. The condition described in Chapter 6.8.6 (3) for couplings in prestressed components is not examined by the program.

DIN EN 1992-1-1:

The simplified check is not permitted for welded reinforcing bars.

### Check with damage equivalent stress ranges

According to Chapter 6.8.5 (3), for reinforcing and prestressing steel adequate fatigue resistance should be assumed if Eq. (6.71) is satisfied:

$$\gamma_{F,\text{fat}} \cdot \Delta\sigma_{s,\text{equ}}(N^*) \leq \Delta\sigma_{\text{Rsk}}(N^*) / \gamma_{s,\text{fat}} \quad (6.71)$$

with

$\gamma_{F,\text{fat}}$  = 1 according to Chapter 2.4.2.3 and 6.8.4 (1).

$\gamma_{s,\text{fat}}$  = 1.15 for reinforcing and prestressing steel according to Chapter 2.4.2.4.

$\Delta\sigma_{\text{Rsk}}(N^*)$  Permitted characteristic stress range at  $N^*$  load cycles based on the S-N curves specified in Tab. 6.4N for prestressing steel or Tab. 6.3N for reinforcing steel.

$\Delta\sigma_{s,\text{equ}}(N^*)$  Damage equivalent stress range with  $N^*$  load cycles. For building construction this value may be approximated by  $\Delta\sigma_{s,\text{max}}$ .

$\Delta\sigma_{s,\text{max}}$  Maximum stress range from the fatigue combination.

### Calculation method

The maximum from the robustness, crack and bending reinforcement is taken as the existing bending reinforcement. If as a result the load from the fatigue combination in state II cannot be absorbed, the design will be repeated using the existing reinforcement and the check internal forces.

The maximum stress range per steel layer that results from the strain state in state II or the truss model is determined separately for each check situation. For longitudinal reinforcement the varying bond behavior of reinforcing and prestressing steel is taken into account by increasing the steel stress by the coefficient  $\eta$  from Eq. (6.64). If for longitudinal and shear reinforcement the resulting stress range exceeds the permitted stress range, the necessary reinforcement will be iteratively increased until the check succeeds for all situations. In the *Symmetrical* and *Compression member* design modes the longitudinal reinforcement is applied at all predefined locations. This will not affect the predefined relationships between the individual reinforcement layers.

The permitted stress ranges and the coefficient  $\eta$  are specified by the user in the Section dialog.

The decisive reinforcement used for the check, which may have been increased, is recorded in the check log and saved for graphical representation.

## Fatigue of Concrete Under Longitudinal Compressive Stress

The fatigue check for concrete that is subject to compressive stress is performed for bending and longitudinal force at the cracked section as described in EN 1992-1-1, Chapter 6.8.7. This check takes into account the final longitudinal reinforcement and may include an increase applied during the fatigue check for reinforcing steel.

### Simplified check

Adequate fatigue resistance may be assumed if the following condition is satisfied:

$$\frac{\sigma_{c,\max}}{f_{cd,\text{fat}}} \leq 0.5 + 0.45 \frac{\sigma_{c,\min}}{f_{cd,\text{fat}}} \leq 0.9 \text{ for } f_{ck} \leq 50 \text{ MN/m}^2 \quad (6.77)$$

$$f_{cd,\text{fat}} \leq 0.8 \text{ for } f_{ck} > 50 \text{ MN/m}^2$$

where

$\sigma_{c,\max}$  is the maximum compressive stress at a fibre under the frequent load combination (compression measured positive).

$\sigma_{c,\min}$  is the minimum compressive stress at the same fibre where  $\sigma_{c,\max}$  occurs ( $\sigma_{c,\min} = 0$  if  $\sigma_{c,\min}$  is a tensile stress).

$f_{cd,\text{fat}}$  is the design fatigue strength of concrete according to Eq. (6.76). This value is entered by the user in the Section dialog.

$$f_{cd,\text{fat}} = k_1 \cdot \beta_{cc}(t_0) \cdot f_{cd} \cdot (1 - f_{ck}/250) \quad (6.76)$$

with  $\beta_{cc}(t_0)$  as per Eq. (3.2) and  $f_{cd}$  as per Eq. (3.15)

$$k_1 = 0.85$$

DIN EN 1992-1-1, OENORM B 1992-1-1, SS EN 1992-1-1:

$$k_1 = 1.0$$

### Check with damage equivalent concrete compressive stresses

According to Chapter 6.8.7 (1), a satisfactory fatigue resistance may be assumed for concrete under compression, if Eq. (6.72) is fulfilled:

$$E_{cd,\max,\text{equ}} + 0.43 \sqrt{1 - R_{\text{equ}}} \leq 1 \quad (6.72)$$

where

$R_{\text{equ}} = E_{cd,\min,\text{equ}} / E_{cd,\max,\text{equ}}$  is the stress ratio.

$E_{cd,\min,\text{equ}} = \sigma_{cd,\min,\text{equ}} / f_{cd,\text{fat}}$  is the minimum compressive stress level.

$E_{cd,\max,\text{equ}} = \sigma_{cd,\max,\text{equ}} / f_{cd,\text{fat}}$  is the maximum compressive stress level.

$\sigma_{cd,\min,\text{equ}}$  is the lower stress of the ultimate amplitude for  $N = 10^6$  cycles.

$\sigma_{cd,\max,\text{equ}}$  is the upper stress of the ultimate amplitude for  $N = 10^6$  cycles.

$f_{cd,\text{fat}}$  is the design fatigue strength of concrete according to Eq. (6.76).



## Fatigue of the Concrete Compressive Struts Under Lateral Force and Torsion

Fatigue of the concrete compressive struts is examined for beams and design objects. The check differentiates between components with and without calculatory required lateral force. In the case of combined loads from lateral force and torsion, the supplementary condition according to Chapter EN 1992-1-1, 6.3.2 (5) is checked in addition to the condition in Chapter 6.2.1 (5).

DIN EN 1992-1-1: In addition, the equations according to NCI for 6.3.2 (5) are evaluated.

### Components with required lateral force reinforcement

The fatigue check for concrete under compressive stress as per Chapter 6.8.7, is also applicable for verifying the concrete compressive struts of components with required lateral force reinforcement as per Chapter 6.8.7 (3).

In the case of vertical stirrups ( $\alpha = 90^\circ$ ), the design values  $\sigma_{cd,max}$  and  $\sigma_{cd,min}$  of the maximal and minimal compressive stress may be determined according to the following equations while assuming an identical compressive strut angle  $\Theta$  for lateral force and torsion as well:

$$\sigma_{cd,T} = \frac{T_{Ed}}{2 \cdot A_k \cdot t_{ef}} \cdot (\cot \Theta + \tan \Theta)$$

$$\sigma_{cd,V} = \frac{V_{Ed}}{b_w \cdot z} \cdot (\cot \Theta + \tan \Theta)$$

$$\sigma_{cd,max} = \begin{cases} \max \sigma_{cd,T} + \text{cor} \cdot \sigma_{cd,V} \\ \max \sigma_{cd,V} + \text{cor} \cdot \sigma_{cd,T} \end{cases}$$

$$\sigma_{cd,min} = \begin{cases} \min \sigma_{cd,T} + \text{cor} \cdot \sigma_{cd,V} \\ \min \sigma_{cd,V} + \text{cor} \cdot \sigma_{cd,T} \end{cases}$$

The program performs the check depending on the user's selection either according to the simplified method as per Eq. (6.77), for the frequent combination or by using the damage equivalent stress range as per Eq. (6.72), for the fatigue combination given in Chapter 6.8.3, Eq. (6.69).

When performing the simplified check under pure lateral force load, the concrete strength  $f_{cd,fat}$  should be reduced by the factor  $v \cdot \eta_1$  as per Chapter 6.2.2 (6). In case of combined stressing from lateral force and torsion, the reduction factor  $v \cdot \alpha_{cw} \cdot \eta_1$  with  $\alpha_{cw}$  as per Eq. (6.9) applies. The coefficient  $\eta_1$  should be set to 1 for normal concrete and according to Eq. (11.1), for light weight concrete.

DIN EN 1992-1-1:

The following factors apply:

$$v_1 = 0.75 \cdot v_2 \cdot \eta_1 \quad \text{in case of pure lateral force as per NDP 6.2.3 (3).}$$

$$v_2 = (1.1 - f_{ck} / 500) \leq 1.0 \quad \text{acc. to NCI 6.8.7 (3).}$$

$$v \cdot \alpha_{cw} \cdot \eta_1 = 0.525 \cdot \eta_1 \quad \text{in case of combined stressing as per NDP 6.2.2 (6).}$$

### Components without required lateral force reinforcement

For components without required lateral force reinforcement at ultimate limit state, adequate fatigue resistance against lateral force load may be assumed according to Chapter 6.8.7 (4), if the following conditions are satisfied:

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} \geq 0: \frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5 + 0.45 \cdot \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \leq 0.9 \text{ for concrete up to } C50/60 \quad (6.78)$$

$$\frac{|V_{Ed,min}|}{|V_{Rd,c}|} \leq 0.8 \text{ for concrete } C55/67 \text{ or higher}$$

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} < 0: \frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5 - \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \quad (6.79)$$

where

$V_{Ed,max}$  is the design value of the maximum lateral force under the frequent action combination.

$V_{Ed,min}$  is the design value of the minimum lateral force under the frequent action combination at the cross-section where  $V_{Ed,max}$  occurs.

$V_{Rd,c}$  is the design value of the absorbable lateral force without shear reinforcement as per Eq. (6.2a).

For performing the check, the program selects automatically the simplified method with the frequent action combination.

## Special Characteristic of Shell Structures

In shell structures the strain state at the cracked concrete section under general stress cannot be determined unambiguously. The design is therefore carried out separately for the reinforcement directions x and y with the design internal forces from Wolfensberger/Thürlimann or Rüsç as described above. The reinforcement calculated in this manner yields a reliable load-bearing capacity.

When calculating the stress range for reinforcing steel and concrete, this method can lead to unrealistic results in the case of torsional or shear stresses as shown in the following example:

Assume two identical sets of slab internal forces:

Set	$m_x$ [kNm/m]	$m_y$ [kNm/m]	$m_{xy}$ [kNm/m]
1	300	200	100
2	300	200	100

According to Wolfensberger/Thürlimann, this results in design variants for the x direction:

Set	Variant	$m$ [kNm/m]
1	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$
2	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$

The torsional moments generate a variation of the design moments and thus a calculatory stress range. This may lead to a necessary reinforcement increase in the fatigue check due to apparent overstressing. For normal design forces, this applies correspondingly to the shear forces.

Selecting **Limit design variants** in the Section dialog allows you to avoid the described effect. In this case only the corresponding variants are compared when determining the stress range, i.e. only the first and second variants of both sets in this example. Assuming constant stress, the stress range is thus correctly determined to be zero.

**This alternative, however, does not ensure that all conceivable stress fluctuations are analyzed. You should therefore be particularly careful when assessing the results. For this purpose the detailed log indicates the main variants and design internal forces used for the check.**

When determining the design internal forces according to Rüsç for inclined reinforcement, the described relationships apply accordingly.

## Checks in the Serviceability Limit States

The following checks are performed:

- Limiting the concrete compressive stresses (EN 1992-1-1, Chapter 7.2).
- Limiting the reinforcing steel stresses (Chapter 7.2).
- Limiting the prestressing steel stresses (Chapter 7.2).
- Decompression check (Chapter 7.3.1).
- Minimum reinforcement for crack width limitation (Chapter 7.3.2).
- Crack with calculation (Chapter 7.3.4).
- Crack width by limitation of the bar distances (Chapter 7.3.3 (2)).
- Limiting deformations (Chapter 7.4).

## Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.5.3, the following combinations are taken into account in the serviceability limit states:

- Combination for characteristic situations

$$\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i > 1} \psi_{0,i} \cdot Q_{k,i} \quad (6.14b)$$

- Combination for frequent situations

$$\sum_{j \geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.15b)$$

- Combination for quasi-continuous situations

$$\sum_{j \geq 1} G_{k,j} + P + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.16b)$$

For each combination you can define different design situations for the construction stages and final states. If necessary, the combination required by the check will automatically be determined from the section specifications. Each check is carried out for all the situations of a combination.

## Stress-Strain Curves

For checks in the serviceability limit states the following characteristics apply:

- Concrete: Stress-strain curve according to EN 1992-1-1, Figure 3.2, where a horizontal curve is assumed for strains of  $\epsilon_{c1}$  or higher (cf. Interpretation No. 098 of the NABau for DIN TR 102).
- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be  $k \cdot f_{yk}$  with  $k = 1.05$  as per Table C.1, class A.

DIN EN 1992-1-1:

The maximum stress is assumed to be  $1.05 \cdot f_{yk}$  for ductility class A according to DIN 488-1.

- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of  $f_{p;0,1k}$ .

## Stress Analysis

For uncracked concrete sections, the program assumes that concrete and steel under tensile and compressive stress behave elastically. As for cracked concrete sections, the concrete compressive stresses are determined using the aforementioned stress-strain curve.

### Area elements

For area elements the concrete stresses are calculated at the gross section. The steel stress check is carried out for reinforcing steel by determining the strain state at the cracked concrete section and for the prestressing steel at the uncracked concrete section.

### Beams and design objects

The action combination stresses that can be determined without checks are always calculated at the gross section.

Conversely, in the checks the stresses are determined as follows and are graphically displayed or logged:

- When checking the crack reinforcement and crack width, the concrete stress is calculated at the gross section
- When checking the decompression and concrete compressive stresses, the concrete stress is calculated
  - without internal tendons at the gross section
  - with internal tendons without bond at the net section
  - with internal tendons with bond for situations before being grouted at the net section or otherwise at the ideal section
- The reinforcing and prestressing steel stresses are checked by determining the strain state at the cracked concrete section

OENORM B 1992-1-1:

If the stresses according to Chapter 7.2 are calculated at the cracked section the different bonding behavior of concrete and prestressing steel is to be taken into account. The increase of tension force  $\Delta F_{tp}$  in the prestressing steel is to be calculated as follows:

$$\Delta F_{tp} = \xi_1^2 \cdot A_p \cdot \varepsilon(y_p) \cdot E_p \quad (13AT)$$

where

$\xi_1$  is the bond coefficient according to Eq. (7.5); the value can be entered in the crack width check dialog;

$A_p$  is the section area of the tendon with bond;

$\varepsilon(y_p)$  is the strain of the concrete section at the location  $y_p$  of the tendon;

$E_p$  is the elasticity modulus of the tendon.

For beams and design objects this rule is taken into account by the program for situations after grouting. For area elements it is not used because prestressing steel is only checked at the uncracked section.

## Limiting the Concrete Compressive Stresses

The concrete compressive stress check is carried out according to EN 1992-1-1, Chapter 7.2. As described in Chapter 7.1 (2), a cracked section is assumed if the tensile stress calculated in the uncracked state exceeds  $f_{ctm}$ .

The calculation in the cracked state is performed by determining the strain state with the final longitudinal reinforcement (maximum from robustness, crack and bending reinforcement including a possible increase from the fatigue check). For beams and design objects, the tendons with bond are taken into account on the resistance side provided that they are grouted in the check situation. For area elements, the compressive stress for both reinforcement directions is determined separately and the extreme value is checked because the general strain state cannot be determined unambiguously.

In the construction stages and final states, for members of exposure classes *XD*, *XF* and *XS* the concrete compressive stress  $\sigma_c$  as defined in Chapter 7.2 (1) is to be limited to  $0.60 f_{ck}$  under the characteristic combination. If stress in the concrete under quasi-continuous combination does not exceed the limit  $0.45 f_{ck}$ , linear creep can be assumed according to 7.2 (3). If this is not the case, non-linear creep must be taken into account. Both conditions are considered based on the user's specifications.

In prestressed concrete components as per Chapter 5.10.2.2, the maximum concrete compressive stress must be limited to  $0.60 f_{c(t)}$  when entering the average prestressing value. If the concrete compressive stress exceeds the value  $0.45 f_{c(t)}$ , the nonlinearity of the creep must be taken into account.  $f_{c(t)}$  indicates the average value of the concrete compressive strength at time  $t$  when the prestressing is entered.

The program assumes the time of introducing the prestressing to coincide with situation *G+P*. If a situation *G+P* is defined in the combination selected above, the concrete stress is checked against the limit value  $0.45 f_{c(t)}$  or  $0.60 f_{c(t)}$  for this situation depending on the user's specification. The value for  $f_{c(t)}$  is also defined in the dialog.

## Limiting the Reinforcing and Prestressing Steel Stresses

### Reinforcing steel

For reinforcing steel, the limitation of steel stress under the characteristic combination is checked for  $0.8f_{yk}$  or  $1.0f_{yk}$  depending on the user's selection according to EN 1992-1-1, Chapter 7.2 (5). The increased limit is permissible for stresses from indirect actions. In this check the reinforcement corresponds to the maximum value from the robustness, crack and bending reinforcement, including a possible increase as a result of the fatigue check. The determination of the strain state is performed at the cracked concrete section. If for beams and design objects tendons with bond are grouted in the check situation, they will be taken into account on the resistance side.

SS EN 1992-1-1:

According to Article 19, the limit  $1.0f_{yk}$  is generally assumed.

### Prestressing steel

For tendons with bond, the limitation of steel stress is checked at the cracked concrete section for beams and design objects and at the uncracked concrete section for area elements. This check is based on the limit  $0.75f_{pk}$  under the characteristic action combination.

DIN EN 1992-1-1:

The check is carried out for the quasi-continuous combination with the limit  $0.65f_{pk}$ . In addition, the stresses are checked against the minimum of  $0.9f_{p0,1k}$  and  $0.8f_{pk}$  under the characteristic combination.

For situations before prestressing and for tendons without bond, the stress  $\sigma_{pm0}(x)$  is checked according to Equation (5.43). External tendons are not checked.

## Decompression Check

This check is to be carried out for prestressed components of exposure classes XC2-XC4, XD1-XD3 and XS1-XS3 as per Table 7.1N in Chapter 7.3.1 of EN 1992-1-1. According to this, all parts of the tendon with bond or the duct must be located in the overcompressed concrete at a depth of at least 25 mm. The decisive action combination results from the selected exposure class or according to the user specification.

For beams and design objects, the analysis is carried out for stresses resulting from bending and normal force. A cracked section according to Chapter 7.1 (2) is assumed in this analysis in case the tensile stress under the decisive action combination exceeds  $f_{ctm}$ . In addition, the rules for stress analysis indicated above apply.

For area sections, an uncracked section is assumed. The 2D concrete stress applied in the direction of the tendon is decisive for the check.

The result is indicated as the 'compression depth' which refers to the shortest distance between the tendon or duct and the tensile zone or section edge. This value is negative if the tendon is in the tensile zone.

DIN EN 1992-1-1:

Table 7.1DE is decisive. The limit state of decompression is maintained if the concrete section around the tendon is under compressive stresses in the range of 100 mm or 1/10 of the section height. The higher range is decisive. The stresses are to be checked in state II.

For structures that are to be designed according to the DAfStb guideline for waterproof components, a compressive stress of  $0.5 \text{ MN/m}^2$  should remain in the component after deducting the loss of prestress according to Chapter 8.4 (2) of the guideline. This can be verified in the graphical representation of the determined check stresses.

OENORM B 1992-1-1:

Table 8AT is decisive.

## Minimum Reinforcement for Crack Width Limitation

The minimum reinforcement for crack width limitation is defined in EN 1992-1-1, Chapter 7.3.2. According to 7.3.2 (1), the minimum reinforcement is to be applied in areas where tension is expected. Tension areas can be defined in the section dialog by choosing either an action combination or a restraint (bending, central tension). Reinforcing steel layers that are not under tension are also provided with reinforcement in the *symmetrical* and *compression member* design modes. This will not affect the predefined relationships between the individual reinforcement layers.

For profiled sections, each subsection (web or flange) should be checked individually in accordance with Section (2). This cannot be done if any polygonal section geometries are taken into consideration. For this reason, the program always determines the minimum reinforcement based on the entire section. The coefficient  $k_c$  is calculated according to user specification either as per Eq. (7.2) or as per Eq. (7.3), optionally different for the top and bottom of the cross-section.

SS EN 1992-1-1:

The permissible crack widths are defined in Article 20, Table D-2, for the quasi-continuous action combination depending on the exposure class, the service life class and the corrosion susceptibility. If the tensile stress does not exceed  $f_{ctk} / \zeta$  with  $\zeta$  as per Table D-3, the concrete may be regarded as uncracked. In this case no minimum reinforcement is determined. The program assumes  $f_{ctk} = f_{ctk;0.05}(t) = 0.7 \cdot f_{ctm}(t) = 0.7 \cdot f_{ct,eff}$  with  $f_{ct,eff}$  according to Equation (7.1).

### Determining the minimum reinforcement

Minimum reinforcement  $A_{s,min}$  is determined using Equation (7.1) of the standard:

$$A_{s,min} \cdot \sigma_s = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} \quad (7.1)$$

where

$A_{ct}$  is the area of the concrete tensile zone during initial crack formation in state I. To determine the value, the program scales the bending moment of the action combination until the maximum edge stress in state I corresponds to the value  $f_{ct,eff}$ .

$\sigma_s$  is the maximum permitted stress in the reinforcing steel reinforcement in relation to the limit diameter of the reinforcing steel.

$k$  is the coefficient for factoring in nonlinearly distributed tensile stresses based on the user's input, which can vary between 0.65 and 1.0 depending on the section height.

DIN EN 1992-1-1:

In case of restraint within the component, these values can be multiplied by 0.8 and for tensile stresses due to restraint generated outside of the component,  $k = 1.0$  shall be assumed.

SS EN 1992-1-1:

According to Article 4a, the value can be assumed to be between 0.50 and 0.90.

$f_{ct,eff}$  is the effective concrete tensile strength at the time of crack formation based on the user's input. The tensile strength is assumed to be  $f_{ctm}$  or lower in case the crack formation is expected to occur within the first 28 days. The tensile strength, which depends on the age of the concrete, is defined in Equation (3.4) of Chapter 3.1.2.

DIN EN 1992-1-1:

If it is not certain whether crack formation will occur within the first 28 days, a tensile strength of at least 3 MN/m<sup>2</sup> for normal concrete and 2.5 MN/m<sup>2</sup> for lightweight concrete should be assumed.

$k_c$  is the coefficient for consideration of stress distribution prior to crack formation.

$k_c = 1.0$  for tension only

For rectangular sections and webs of box girders or T sections:

$$k_c = 0.4 (1 - \sigma_c / (k_1 \cdot h / h^*) / f_{ct,eff}) \leq 1 \quad (7.2)$$

For flanges of box girders and T sections:

$$k_c = 0.9 \cdot F_{cr} / A_{ct} / f_{ct,eff} \geq 0.5 \quad (7.3)$$

$\sigma_c$  is the average concrete stress in the analyzed part of the section with

$$\sigma_c = N_{Ed} / (b \cdot h) \quad (7.4)$$

$N_{Ed}$  is the normal stress in the analyzed part of the section (compressive force positive) under the decisive action combination.

$h^* = \min(h; 1.0 \text{ m})$ .

$k_1$  is the coefficient for taking into account the effects of normal force  $N_{Ed}$  on the stress distribution:

$k_1 = 1.5$  for compressive normal force

$$k_1 = 2 h^* / (3h) \quad \text{for tensile normal force}$$

$F_{cr}$  is the absolute value of the tensile force in the chord directly before crack formation. The tensile force is generated through the integration of tensile stresses within area  $A_{ct}$ .

The largest existing bar diameter  $\phi_s$  is specified in the Section dialog (where it is labeled with  $d_s$ ). It is used in the following equations to determine the limit diameter  $\phi_s^*$  as an input value for Table 7.2N:

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot h_{cr} / (2(h-d)) \quad \text{for bending} \quad (7.6N)$$

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot h_{cr} / (8(h-d)) \quad \text{for central tension} \quad (7.7N)$$

where

$h$  is the overall section height.

$d$  is the static effective height up to the centroid of the outermost reinforcement layer.

$h_{cr}$  is the height of the tensile zone directly before crack formation under the decisive action combination.

The limit diameter  $\phi_s^*$  and the permissible crack width  $w_{max}$  are used to determine the permissible reinforcing steel stress  $\sigma_s$  for Equation (7.1) according to Table 7.2N. The values within the table are interpolated linear, beyond the table they are extrapolated linear for  $w_k$  and quadratic for  $\sigma_s$ .

When determining the minimum reinforcement under central tension, the steel stress  $\sigma_s$  is calculated with the minimum of the permissible crack width and the maximum of the other quantities, provided they have been defined differently for the cross-section edges.

If the crack width check is to be carried out at the same time, the program will determine whether the specified crack width according to Chapter 7.3.4 is maintained by inserting the calculated minimum reinforcement. If necessary, the minimum reinforcement is increased iteratively until the check limit is reached. The increased reinforcement is indicated by an exclamation mark "!" in the log.

DIN EN 1992-1-1:

The limit diameter  $\phi_s^*$  for Table 7.2DE is determined using the following equations:

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot k \cdot h_{cr} / (4(h-d)) \geq \phi_s^* \cdot f_{ct,eff} / 2.9 \quad \text{for bending} \quad (7.6DE)$$

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot k \cdot h_{cr} / (8(h-d)) \geq \phi_s^* \cdot f_{ct,eff} / 2.9 \quad \text{for central tension} \quad (7.7DE)$$

The steel stress  $\sigma_s$  is calculated with the equation from Table 7.2DE and limited to the mean yield strength  $f_{yk}$  of the steel layers to be dimensioned.

Based on Chapter 7.3.2 (NA.5), the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined according to Equation (NA.7.5.1). It is not necessary to insert more reinforcing steel as results from Equation (7.1). The rules specified before will be used, if the option is selected by the user, whereas the possibility of lower reinforcement for slowly hardening concrete according to Section (NA.6) will not be used. The evaluation of Figure NA7.1d to determine the effective tensile zone  $A_{c,eff}$  is performed with the smallest edge distance  $d_1$  of the reinforcement.

OENORM B 1992-1-1:

Table 7.2N is replaced by Table 8AT. The steel stress  $\sigma_s$  is determined according to Equation (19AT) and limited to the mean yield strength  $f_{yk}$  of the steel layers to be dimensioned. The limit diameter is to be modified as follows:

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot k \cdot h_{cr} / (4(h-d)) \geq \phi_s^* \cdot f_{ct,eff} / 2.9 \quad (21AT)$$

For central tension  $h_{cr} = h / 2$  for each member side is applied.

For members under central restraint the minimum reinforcement for the crack width limitation can be determined according to Equation (16AT). This rule will be used, if the option is selected by the user, assuming the smallest edge distance of the reinforcement  $d_1$  for  $h-d$ . The program does not take into account the possibility of reducing the reinforcement for slowly hardening concrete.

### Special characteristic of prestressed concrete structures

According to the guidelines set forth in Chapter 7.3.2 (3), tendons with bond in the tensile zone may be added to the minimum reinforcement as long as their axis distance to the reinforcing steel layer does not exceed 150 mm. To include the tendons, add the term

$$\xi_1 \cdot A_p' \cdot \Delta\sigma_p$$

on the left side of Equation (7.1). In this formula

$A_p'$  is the section area of the tendons with bond located in  $A_{c,eff}$

$A_{c,eff}$  is the effective area of the reinforcement according to Figure 7.1. The section after the next describes how  $A_{c,eff}$  is determined.

$\xi_1$  is the adjusted ratio of bond strengths between reinforcing steel and prestressing steel according to Equation (7.5).

$\Delta\sigma_p$  is the stress change in the tendons.

For beams and design objects, the tendon layers with bond can be added using the  $\xi_1$  value specified in the Section dialog as long as they are grouted in the check situation. For area elements, prestressing steel can never be taken into account.

According to Section (4) of Chapter 7.3.2, prestressed concrete components do not require a minimum reinforcement in sections where the absolute value of concrete tensile stress  $\sigma_{ct,p}$  under the characteristic action combination and characteristic prestressing is less than  $f_{ct,eff}$ . This condition is automatically checked by the program.

DIN EN 1992-1-1:

According to Section (4), components with subsequent bond do not require a minimum reinforcement if the absolute value of concrete compressive stress under the characteristic combination on the section edge is greater than 1 N/mm<sup>2</sup>. This condition is also checked by the program.

OENORM B 1992-1-1:

The value is specified as  $\sigma_{ct,p} = 0.0$  N/mm<sup>2</sup>.

SS EN 1992-1-1:

The value is specified as  $\sigma_{ct,p} = f_{ctk} / \zeta$  with  $\zeta$  the crack safety factor according to Article 21, Table D-3.

The program assumes  $f_{ctk} = f_{ctk;0.05}(t) = 0.7 \cdot f_{ctm}(t) = 0.7 \cdot f_{ct,eff}$  with  $f_{ct,eff}$  according to Equation (7.1).

To delimit the areas where no minimum reinforcement is required, the concrete compressive stresses in state I are calculated at the gross cross-section with the mean characteristic prestress. The affected structural areas can be evaluated in the graphical stress representation for the characteristic combination. In the remaining areas, minimum reinforcement is determined if concrete tensile stresses occur in the selected check combination.

## Special features for waterproof concrete structures

DIN EN 1992-1-1:

For components that are to be designed according to the waterproof concrete guideline (WU-Richtlinie) of the German Committee for Reinforced Concrete (DAfStb), the permissible crack widths according to Table 2 of the guideline apply. The checks for limiting the crack width shall be lead for the frequent action combination in accordance with Chapter 8.5.1 (1) of the directive.

OENORM B 1992-1-1:

For components that are to be designed according to the oevb guideline for waterproof concrete structures, the permissible crack widths according to Chapter 4.5.1 of the guideline apply depending on the design class according to table 3-2. When using the design model "White tank optimized" ("Weiße Wanne optimiert"), the minimum crack-limiting reinforcement can be avoided if the requirements according to Chapter 4.5.2.2 of the guideline are met and if a minimum reinforcement  $A_{s,min}$  is dimensioned as follows per component side and reinforcement direction:

$$A_{s,min} = 0.07 \cdot A_c \cdot f_{ctm} / f_{yk}$$

## Crack Width Calculation

The crack width check is performed through direct calculation in accordance with EN 1992-1-1, Chapter 7.3.4, for all sections where tensile stresses in state I occur under the action combination that is based on the exposure class specified in the Table 7.1N. The bar diameter  $\phi$  ( $d_s$  in the dialog) of the reinforcing steel reinforcement and the decisive  $f_{ct,eff}$  concrete tensile strength are defined in the section dialog.

SS EN 1992-1-1:

The check is performed according to Article 20 for the quasi-continuous action combination.



The program performs the check according to the following steps:

- Determine strain state II under the check combination with the stress-strain curve shown in Figure 3.2. For beams and design objects, all tendons with bond are considered on the resistance side.
- Define the effective area of reinforcement  $A_{c,eff}$  shown in Figure 7.1 (see next section), determine the reinforcing steel layers and prestressing steel layers within  $A_{c,eff}$

- Calculate reinforcement level:

$$\rho_{p,eff} = (A_s + \xi_1^2 \cdot A_p') / A_{c,eff} \quad (7.10)$$

$$\rho_{tot} = (A_s + A_p') / A_{c,eff}$$

$\xi_1$  Bond coefficient according to user specification.

$A_s, A_p'$  Reinforcing steel and prestressing steel within  $A_{c,eff}$ .

- Determine individually for each reinforcing steel layer:

#### Difference of the average strain for concrete and reinforcing steel

$$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - k_t \cdot f_{ct,eff} / \rho_{p,eff} (1 + \alpha_e \cdot \rho_{p,eff})] / E_s \geq 0.6 \sigma_s / E_s \quad (7.9)$$

where

$$\alpha_e = E_s / E_{cm}$$

$\sigma_s$  is the reinforcing steel stress from strain state II.

DIN EN 1992-1-1:

$$\sigma_s = \sigma_{s2} + 0.4 f_{ct,eff} (1 / \rho_{p,eff} - 1 / \rho_{tot})$$

$\sigma_s$  is limited to  $f_{yk}$  in the program

$\sigma_{s2}$  = Reinforcing steel stress from strain state II

(NA. 7.5.3)

$f_{ct,eff}$  is the effective concrete tensile strength as per specifications.

$k_t$  is the factor for the duration of the load action:  
0.6 for short-term and 0.4 for long-term load action.

#### Maximum crack spacing

$$s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot \phi / \rho_{p,eff} \quad (7.11)$$

where

$\phi$  is the bar diameter specified by the user.

$c$  is the concrete cover with respect to the longitudinal reinforcement. The concrete cover is set to  $d_1 - \phi / 2$  in the program, where  $d_1$  is the smallest axis distance of the reinforcing steel reinforcement of the section edge within  $A_{c,eff}$

$k_1$  is the coefficient for consideration of the bond properties of the reinforcement. The coefficient is set to 0.8 in the program, which is the recommended value for good bond properties.

$k_2$  is the coefficient for taking strain distribution into account:  
0.5 for bending and 1.0 for pure tension.

$k_3, k_4$  The recommended national values are  $k_3 = 3.4$  and  $k_4 = 0.425$ .

DIN EN 1992-1-1:

$$k_1 \cdot k_2 = 1, k_3 = 0 \text{ and } k_4 = 1 / 3.6$$

$$s_{r,max} \leq \sigma_s \cdot \phi / (3.6 \cdot f_{ct,eff})$$

OENORM B 1992-1-1:

$$k_3 = 0 \text{ and } k_4 = 1 / (3.6 \cdot k_1 \cdot k_2) \leq \rho_{p,eff} \cdot \sigma_s / (3.6 \cdot k_1 \cdot k_2 \cdot f_{ct,eff}) \quad (22AT)$$

$$s_{r,max} = \phi / (3.6 \cdot \rho_{p,eff}) \leq \sigma_s \cdot \phi / (3.6 \cdot f_{ct,eff}) \quad (23AT)$$

SS EN 1992-1-1:

$$k_3 = 7 \phi / c \quad (\text{Article 22})$$

If an upper limit for the crack spacing in Equation (7.11) was specified in the section dialog, this allows the special features of Equations (7.13) and (7.14) and sections (4) and (5) of Chapter 7.3.4 to be taken into consideration.

### Calculated value of the crack width

$$w_k = s_{r,\max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (7.8)$$

The layer with the largest calculated crack width is shown in the log. If selected in the cross-section dialog, a constant mean steel strain within  $A_{c,\text{eff}}$  is assumed during calculation.

- For sections completely under tension, the check is performed separately for each of the two effective tensile zones. The maximum value is shown in the log.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the crack width. For that purpose a design is carried out using the decisive check combination for calculating the crack width. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The crack width is checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check) and saved for graphical representation together with the decisive reinforcing steel stress.

## Crack Width Check by Limitation of the Bar Distances

As an alternative to the direct crack width calculation described in EN 1992-1-1, Section 7.3.4, you can choose the simplified check according to Section 7.3.3 (2) through limitation of the bar spacing as shown in Table 7.3N in the cross-section dialog.

The program performs the check according to the following steps:

- Determine strain state II under the check combination defined by the requirement class with the stress-strain curve according to Figure 3.2. For beams and design objects, all tendons in a bond are considered on the resistance side.
- Determine the reinforcing steel stress  $\sigma_s$  for each reinforcement layer. If selected in the cross-section dialog, a constant mean steel stress within  $A_{c,\text{eff}}$  is assumed for calculating.

DIN EN 1992-1-1:

$$\sigma_s = \sigma_{s2} + 0,4 f_{ct,\text{eff}} (1/\rho_{p,\text{eff}} - 1/\rho_{\text{tot}}) \quad (\text{NA. 7.5.3})$$

$\sigma_{s2}$  = Reinforcing steel stress from strain state II

- Compare the value entered in the dialog (**max. s**) with the table value (**perm. s**), which is derived from the calculated steel stress  $\sigma_s$  and the permissible crack width  $w_k$ . In the log, the location with the largest quotient (**max. s / perm. s**) is checked.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the permissible bar spacing. For this purpose, a design is carried out with the decisive action combination for the check. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The bar spacings are then checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check).

### Note

According to Section 7.3.3 (2), the simplified check can only be used in the event of crack formation resulting from mostly direct actions (restraint). In addition, Table 7.3N should only be applied for single-layer tensile reinforcement with  $d_1 = 4$  cm (cf. Zilch, Rogge (2002), p. 277; Fingerloos et al. (2012), p. 109; Book 600 of the DAfStb (2012), p. 127).

OENORM B 1992-1-1:

The method is applicable for single-layer reinforcement with bar spacings according to Table 10AT resp. 11AT. These are valid for concrete covers  $25 \text{ mm} \leq c_{\text{nom}} \leq 40 \text{ mm}$  with bar diameters  $8 \text{ mm} \leq d_s \leq 20 \text{ mm}$ .

The user is responsible for the evaluation of these requirements.

## Determining the Effective Area $A_{c,eff}$

According to EN 1992-1-1, Figure 7.1, the effective area of reinforcement  $A_{c,eff}$  defines the area of a rectangular, uniaxially stressed concrete section in which the model assumptions in Book 466 of the German Committee for Reinforced Concrete (DAfStb) are applicable. Although the program can apply this model to any section and stress situation, the user has the responsibility and discretion to do so.

When determining  $A_{c,eff}$  the following steps are performed by the program:

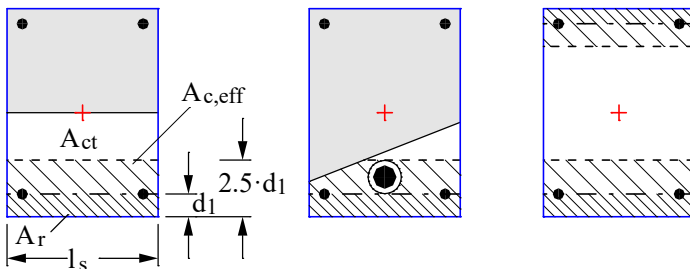
- Determine tensile zone  $A_{ct}$  in state I: When calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the exposure class. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Define the centroid line of the reinforcement as a regression line through the reinforcing steel layers in the tensile zone. In 2D frameworks and for area elements, a horizontal line through the centroid of the reinforcing layers under tension is assumed.
- Determine the truncated residual area  $A_r$  to the edge and the sum of section lengths  $l_s$ . The average edge distance is then assumed as  $d_1 = A_r / l_s$ , but not less than the smallest edge distance of the reinforcing steel layers in the tensile zone.
- Shift the centroid line in parallel by  $1.5 \cdot d_1$ . Assuming  $h - d = d_1$ , the height of  $A_{c,eff}$  is determined as per 7.3.2 (3) by  $h_{c,ef} = 2.5 \cdot (h - d) \leq h / 2$ . According to DIN EN 1992-1-1 and OENORM B 1992-1-1, Section 7.3.2 (3), this value is limited to  $(h - x) / 2$  ( $x$  = compressive zone height in state I).

DIN EN 1992-1-1 and OENORM B 1992-1-1:

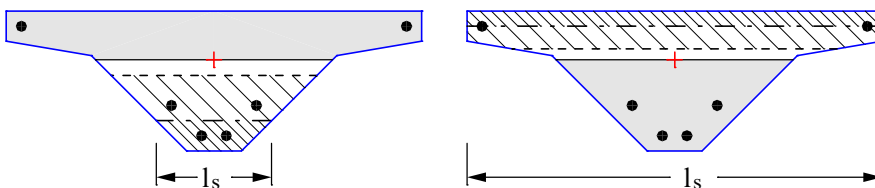
If the minimum reinforcement for thicker components under central restraint is selected in the section dialog, the height of  $A_{c,eff}$  is  $h_{c,ef} \geq 2.5 d_1$  according to Figure NA.7.1 d) or Eq. (16AT). In the crack width check, this increase of  $h_{c,ef}$  does not apply (see comments in Book 600 for Chapter 7.3.2 (NA.5) and 7.3.4 (2)).

- The resulting polygon is intersected with the tensile zone and then defines the effective area  $A_{c,eff}$ .
- If all the reinforcing steel layers of the section are under tension, then two zones will be determined; one for the layers above the centroid and the other for layers below the centroid. The area of each zone is limited to  $A_c / 2$ .

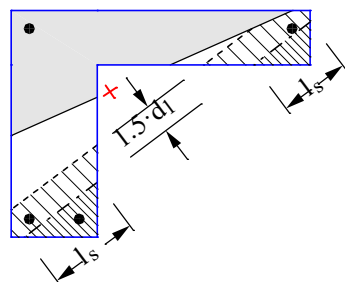
The following illustrations show the effective areas determined by the program for a few representative situations. The last case (edge beam) deviates from the model assumptions in Book 466 to such a degree that it is questionable as to whether it should be used.



Effective area of the reinforcement at a rectangular section under uniaxial bending, normal force with double bending and central tension



Effective area of the reinforcement at a bridge section under uniaxial bending



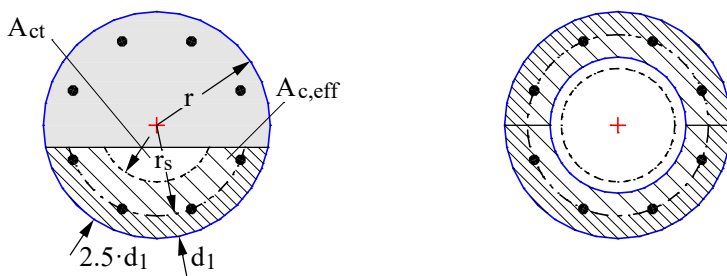
Effective area of the reinforcement at an edge beam under uniaxial bending

### Ring-shaped determination of $A_{c,eff}$

For circular solid and hollow cross-sections, the cross-section dialog allows that the effective area of the reinforcement  $A_{c,eff}$  for checking of the minimum reinforcement and the crack width is determined ring-shaped according to Wiese et al. (2004). This can be used e.g. for considering the specifics of bored piles and spun concrete columns. In order to determine  $A_{c,eff}$ , the following steps are performed by the program:

- Determine tensile zone  $A_{ct}$  in state I: When calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the exposure class. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Calculate the mean radius  $r_s$  of the reinforcing layers within the tensile zone. Assuming the circular radius  $r$  of the outer edge, the mean edge distance is determined by  $d_1 = r - r_s$ .
- The effective area  $A_{c,eff}$  is then assumed to be ring-shaped with a width of  $2.5 \cdot d_1$  and finally intersected with the tensile zone  $A_{ct}$ .
- If all the reinforcing steel layers of the section are under tension, then two ring-shaped zones will be determined; one for the layers above the centroid and the other for layers below the centroid.

The following figures show ring-shaped effective areas exemplarily.



*Effective area of the reinforcement at a solid section under bending with normal force as well as a hollow section under central tension.*

## Limiting Deformations

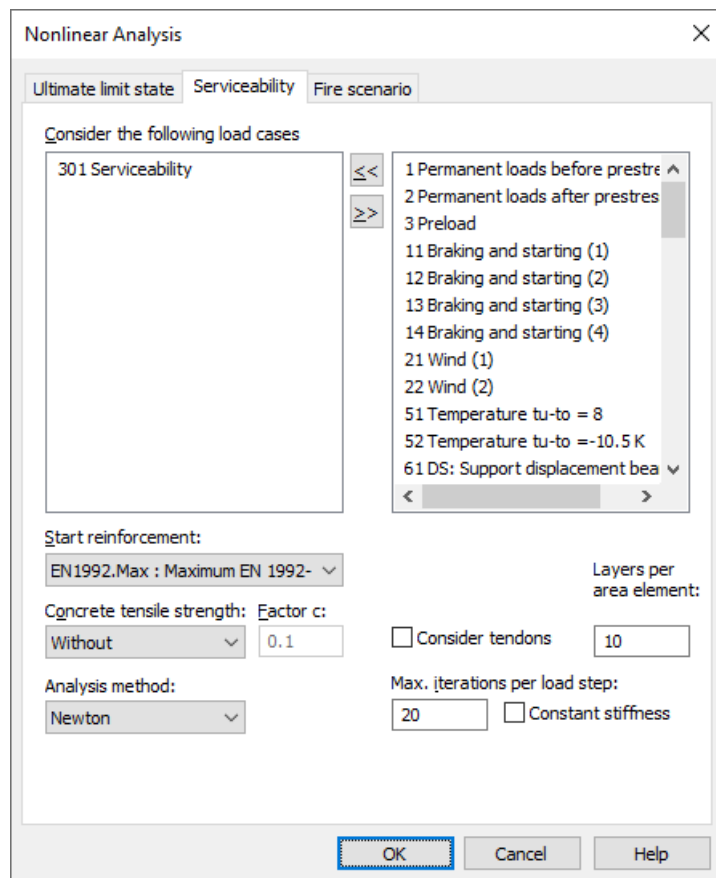
According to EN 1992-1-1, Chapter 7.4.1, the deformations of a component or structure may not impair its proper functioning or appearance. Consequently, a beam, slab or cantilever under the quasi-continuous action combination should not sag more than 1/250th of the span as specified in Section (4). To avoid damage to adjacent components, their deformation should be limited to 1/500th of the span as specified in Section (5).

The standard does not include a method for direct calculation of deformations in accordance with Chapter 7.4.3.

The InfoCAD program system allows you to perform a realistic check as part of a nonlinear system analysis for beam and shell structures that takes geometric and physical nonlinearities into account. The resistance of the tendons within the bond is currently not included in the calculation.

Editing is performed in the following steps:

- Define the check situation using the *Load Group* function in the Load dialog by grouping the decisive individual load cases. The variable loads must first be weighted with the combination coefficients  $\psi_2$  for the quasi-continuous combination.
- Select the check load cases in the *Nonlinear Analysis / Serviceability* dialog in the analysis settings for the FEM or framework analysis.
- Set the reinforcement determined in the ultimate limit state in the *Start reinforcement* selection field (maximum from bending, robustness, crack check and fatigue).
- Perform the FEM or framework analysis to determine the deformations in state II.
- Check the system deformations displayed graphically or in tabular form.



For a detailed description of nonlinear system analysis, refer to the relevant chapter of the manual.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The resulting bending, robustness and crack reinforcement, the decisive maximum value and the stirrup and torsion reinforcement are provided for the graphical representation as well.

The log shows the design internal forces and necessary reinforcements, checked stresses or crack widths at each result location. If the permissible limit values are exceeded, they are reported as warnings and indicated at the check location. The detailed log also lists the decisive combination internal forces of all design situations.

### Tendon reactions

$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$d_p, d_{p,\min}$	Depth of the tendons or ducts in the concrete compressive zone in the decompression check [mm].

### Stresses for beams and design objects

$\sigma_x$	Longitudinal stresses in the decompression and concrete compressive stress checks [MN/m <sup>2</sup> ].
$\sigma_s, \Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd}, \Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,y}, \Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from $Q_y$ and $Q_z$ [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,T}, \Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and longitudinal torsion reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{perm}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

### Stresses for area elements

$\sigma_T$	Concrete stress in the tendon direction in the decompression check [MN/m <sup>2</sup> ].
$\sigma_{sx}, \Delta\sigma_{sx}$	Stresses and stress ranges for reinforcing steel in the x direction [MN/m <sup>2</sup> ].
$\sigma_{sy}, \Delta\sigma_{sy}$	Stresses and stress ranges for reinforcing steel in the y direction [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd,x}, \Delta\sigma_{cd,x}$	Stresses and stress ranges in the concrete fatigue check under longitudinal compression in the x-
$\sigma_{cd,y}, \Delta\sigma_{cd,y}$	and y-direction [MN/m <sup>2</sup> ].
$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{perm}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

### Bending reinforcement

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams and design objects.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.
$a_{s\phi}$	Meridian reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.
$a_{s0}$	Ring reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.

### Reinforcement from lateral force

$a_{sbx}, a_{sby}, a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area elements from $q_x, q_y$ and $q_T$ .
$A_{sb,y}, A_{sb,z}$	Stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $Q_y$ and $Q_z$ .
$A_{sl}$ for $a_{sb}=0$	Longitudinal reinforcement [cm <sup>2</sup> ] of area elements.
$\Delta F_{tdy}, \Delta F_{tdz}$	Additional tensile force [kN] in the longitudinal reinforcement as a result of lateral force $Q_y$ and $Q_z$ .

**Torsional reinforcement**

$A_{sb,T}$	Torsional stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $M_x$ .
$A_{sl,T}$	Torsional longitudinal reinforcement [cm <sup>2</sup> ] of beams and design objects from $M_x$ .

**Design values**

$V_{Rd,ct}$ , $v_{Rd,ct}$	Absorbable design lateral force without shear reinforcement [kN or kN/m].
$v_{Rd,max}$	Absorbable design lateral force of concrete struts for area elements [kN/m].
$V_{Rd,max}$	Absorbable design lateral force of concrete struts for beams and design objects [kN].
$T_{Rd,max}$	Design value of the maximum absorbable torsion moment [kNm].
$Q/V_{Rd} + M_x/T_{Rd}$	For compact and box sections: $Q/V_{Rd,max} + M_x/T_{Rd,max}$ DIN EN 1992-1-1: For compact sections: $(Q/V_{Rd,max})^2 + (M_x/T_{Rd,max})^2$  OENORM B 1992-1-1: For full sections: $(Q/V_{Rd,max})^2 + (M_x/T_{Rd,max})^2$

**Crack width**

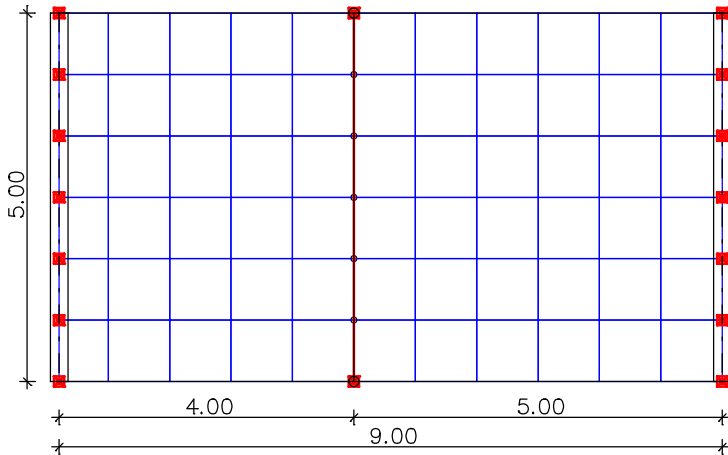
$w_{k,top}$ , $w_{k,bottom}$	Computed crack width at the top and bottom of the cross-section separately for the x and y reinforcement direction for area elements.
$w_k / w_{per}$	Crack width utilization.

## Examples

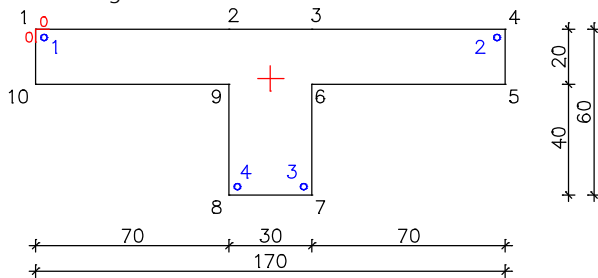
### Slab With Downstand Beam

In this example a rectangular slab ( $d = 20$  cm, C30/37-EN, BSt 500 S, exposure class XC2) with a downstand beam will be analyzed. This slab supported with joints will be subjected to its dead load and a traffic load of  $10 \text{ kN/m}^2$ .

The checks will be carried out for all possible combinations of load cases. This method is selected in the calculation settings and can take a very long time to complete if there is a large number of load cases.



The following image shows the dimensions of the downstand beam. The axis distance of the reinforcing steel from the section edge is 3 cm. The dead load of the downstand beam is reduced by the portion attributed to the slab.



Design specifications and reinforcing steel description of the slab (section 1):

- Edge distance of the reinforcing steel for the x and y direction of the upper (1st) and lower (2nd) layer: 0.03 m
- Bending design mode: *Standard* with 20% bending reinforcement in secondary direction as per Chapter 9.3.1.1 (2)
- Steel quality: 500S
- Effective height: 0.17 m
- Strut angle  $\cot \Theta$ : 1.0.
- Bending tensile reinforcement  $A_{s1}$  for the lateral force design:  $1.88 \text{ cm}^2$

Design specifications of the torsion-flexible downstand beam (section 2):

- Bending design mode: *Standard*
- Steel quality of the stirrups: 500S
- Shear section:
  - Width: 0.30 m
  - Effective height: 0.57 m
- Strut angle  $\cot \Theta$ : 1.0.
- Bending tensile reinforcement  $A_{s1}$  for the lateral force design:  $2.90 \text{ cm}^2$
- Check of the shear joint with default values but joint roughness: *Rough*



**EN 1992-1-1 actions****Standard design group****G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

**Load cases**

1 Dead load

**QN - Imposed load, traffic load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
 Working load - category A: Residential buildings  
 Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

**Load cases 1. Variante, inclusive**

2 Traffic span 1  
 3 Traffic span 2

**Design overview**

Se.	Expos. class	Prestress of component	Reinforc. M R B Q T	Fatigue B Q T P C V	Cr. wi.	De- co.	Stress C B P
1	XC4	Not prestressed	x x x x .	. . . . .	x	.	x x .
2	XC4	Not prestressed	x x x x .	. . . . .	x	.	x x .

(M) Nominal reinforcement to guarantee robustness.  
 (R) Nominal reinforcement for crack width limitation.  
 (B) Flexural reinforcement at ultimate limit state, fatigue and stress check.  
 (Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.  
 (T) Torsional reinforcement at ultimate limit and fatigue state.  
 (P) Prestressing steel at fatigue and stress check.  
 (C) Concrete comp. stress, concrete at fatigue check under long. compression.  
 (V) Concrete at fatigue check under lateral force.

**Settings for flexural and shear reinforcement**

M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 fyk Quality of stirrups.  
 Theta Angle of concrete truss.  
 Slabs Beams are designed like slabs.  
 Asl Given reinforcement according to picture 6.3, increase to maximum.  
 rhow Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).  
 as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).  
 Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den- sity [kg/m <sup>3</sup> ]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm <sup>2</sup> ] Pic. 6.3	Factor rhow	pre- str.
1	C30/37-EN	.	ST	500	1.00	.	1.88 0.00	1.00 0.00	.
2	C30/37-EN	.	ST	500	1.00	.	2.90 .	1.00 .	.

**Shear sections**

bw.nom Nominal width of the prestressed section according to 6.2.3(6).  
 h.nom Nominal height of the prestressed section according to 6.2.3(6).  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.  
 z1, z2 Height and width of the core section for torsion.  
 tef Thickness of the torsion box.  
 B. Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width [m]	Eff. width bw	Height [m] h	Eff.height d	Torsion. section [m] kd	z1	z2	tef
1	1.000	.	0.200	.	0.170 0.90	.	.	.
2	0.300	0.270	0.600	0.570	0.90	0.540	0.240	0.060

**Settings for the shear joint check**

Joint roug. Shear joint roughness: very smooth, smooth, rough, indented.  
 c Factor c acc. to 6.2.5 (2). Values deviation from the standard are marked with "!".  
 dz Distance of the shear joint from the top edge [m].  
 bi Shear joint width [m].  
 sigma Stress due to the minimum normal force perpendicular to the joint (compression negative) [MN/m<sup>2</sup>].  
 Dyn. Dynamic or fatigue load acc. to 6.2.5 (5).

Se.	Joint roug.	c	dz [m]	bi [m]	sigma [MN/m <sup>2</sup> ]	Dyn.
2	Rough	0.400	0.200	0.300	0.00	.

**1. Permanent and temporary situation**

Final state

G Dead load  
 QN Imposed load, traffic load

**1. Rare (characteristic) situation**

Final state

G Dead load  
 QN Imposed load, traffic load

**1. Quasi-continuous situation**

Final state

G Dead load  
 QN Imposed load, traffic load

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel [mm].  
 max.s Maximal given bar spacing of the reinforcing steel [mm].  
 sr,max Upper limit for the crack spacing from Eq. (7.11) [mm].  
 Xi1 Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 kt Coefficient for the duration of the load to calculate the crack width.  
 Fact. Reduction factor for  $f_{ctm}$  as per Chapt. 7.3.2 (As) resp. 7.3.4 (wk).  
 Comb. Combination for verifying the minimum reinf. (As) and crack width (wk):  
 CC, TC, QC = Characteristic, frequent, quasi-continuous combination,  
 CT, TT, TB = Central tension, tension on top side, tension on bottom,  
 CL = Action combination according to exposure class.  
 Method Check method for minimum reinf. (kc) and crack width (wk):  
 kc Determination of coefficient kc for webs/chords per Eq. (7.2/7.3).  
 auto = Eq. (7.2) for rectangular sections, Eq. (7.3) for others.  
 wk Calc. = Direct calculation of crack width as per Chapter 7.3.4,  
 Bar sp. = Limiting the bar spacing as per Table 7.3N,  
 Cal.(m) = Direct calculation for mean steel strain within  $A_{c,eff}$ ,  
 Spc.(m) = Lim. the bar spacing for mean steel strain within  $A_{c,eff}$ .  
 RS Ring-shaped determination of  $A_{c,eff}$  according to Wiese et al.,  
 Beton- und Stahlbetonbau 2004, Issue 4, p 253 ff.

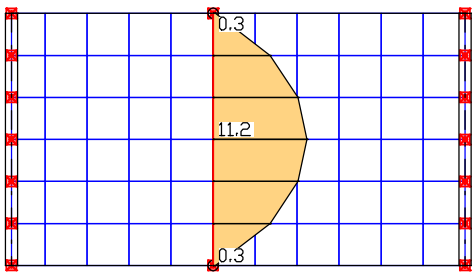
Se.	wmax	ds	max	sr	Coefficient	Fact.fctm	Comb.	Method	RS
	[mm]		s	max	Xi1 k kt	As wk	As wk	kc wk	
1	0.30	10	.	.	1.00 0.4	1.00 1.00	CL CL	auto calc.	.
2	0.30	10	.	.	1.00 0.4	1.00 1.00	CL CL	auto calc.	.

**Settings for the check of concrete and steel stresses**

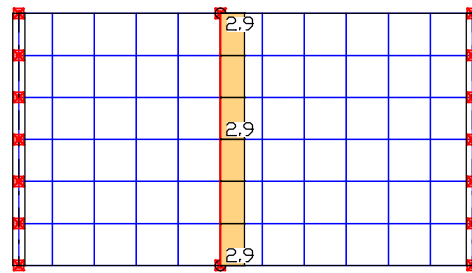
Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.s Reinforcing steel stress in the serviceability limit state.  
 (CC),(QC) Characteristic, quasi-continuous combination.  
 (TC),(CL) Frequent combination, combination according to exposure class.

Se.	fck(t)	per.sigma.c(t)	per.sigma.c	per.sigma.s	Decompression
	[MN/m²]	(CC, QC)	(CC)	(CC)	Comb. Stress
1	.	.	0.60 fck	0.80 fyk	.
2	.	.	0.60 fck	0.80 fyk	.

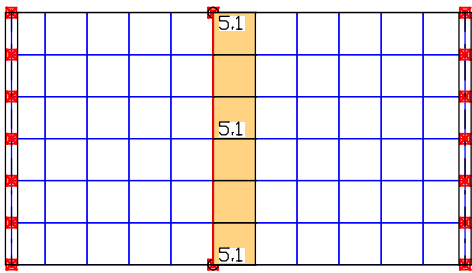
The calculated reinforcements are shown in the illustrations below.



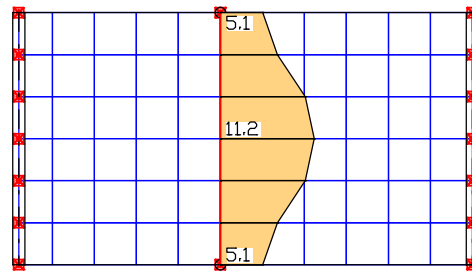
Bending reinforcement  $A_s$  of beams from ultimate limit state [cm²]



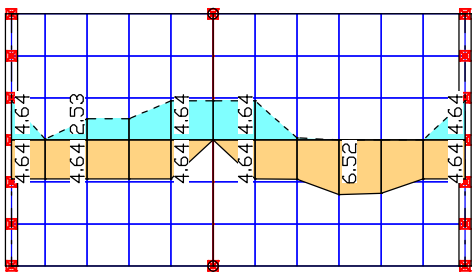
Bending reinforcement  $A_s$  of beams for ensuring robustness (ductility) [cm²]



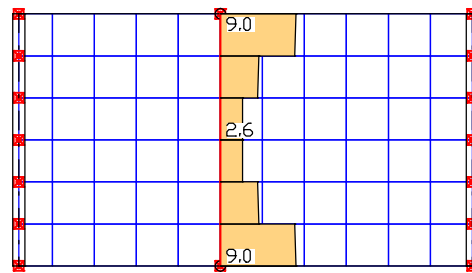
Bending reinforcement  $A_s$  of beams from crack width limitation check [cm²]



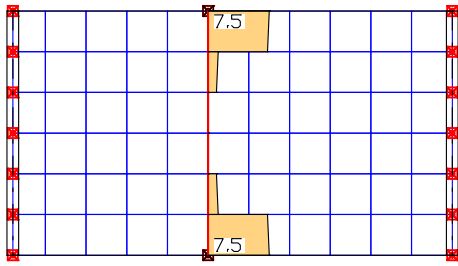
Maximum bending reinforcement  $A_s$  of beams [cm²]



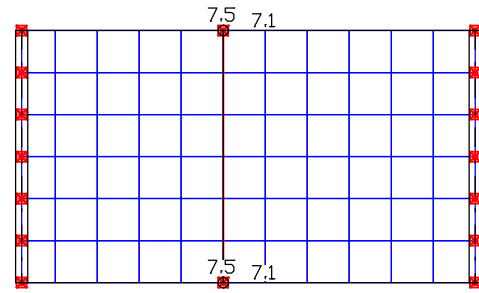
Maximum bending reinforcement  $a_{sx}$  of the slab [cm²/m]



Stirrup reinforcement  $A_{sb,z}$  of beams from ultimate limit state [cm²/m]



Transverse reinforcement shear joint check  $A_{sb}$  of beams from ultimate limit state [ $\text{cm}^2/\text{m}$ ]



Stirrup reinforcement  $a_{sb}$  of the slab from ultimate limit state [ $\text{cm}^2/\text{m}^2$ ] (node mean values)

An excerpt of the detailed log for the midspan of the downstand beam is provided below.

### Design of longitudinal reinforcement

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. for robustness as per EN 1992-2, 6.1 (109) (Charact. C.)  
 fctm Average centric concrete tensile strength [ $\text{MN}/\text{m}^2$ ]  
 zs,t/b Lever arm of inner strengths top/bottom with  $z_s=0,9*d$  [m]  
 fyk,t/b Strength of longitudinal reinforcement top/bottom [ $\text{MN}/\text{m}^2$ ]  
 max Sc Maximum concrete edge stress from Charact. C. [ $\text{MN}/\text{m}^2$ ]
- (R) Nominal/requ. reinforcement as per 7.3.2 for crack width limitation  
 Increase of reinforcement due to crack width check is marked by "!".  
 wmax Permissible crack width as per specification [mm]  
 ds Maximal given steel diameter [mm]  
 k Coefficient for consideration of non-linear distributed tensile stress  
 fct,eff Concrete strength at date of cracking [ $\text{MN}/\text{m}^2$ ]  
 kc Coefficient to consider stress distribution in tensile zone  
 acc. to Eq. (7.2) resp. Eq. (7.3)  
 max Sx Maximal concrete edge stress from action combination [ $\text{MN}/\text{m}^2$ ]
- (B) Design of reinforcement at ultimate limit state  
 In case of dominant bending, compression reinforcement is marked with "\*".  
 For section areas acc. to 6.1 (5) the concrete strain is not limited.  
 The minimum reinforcement acc. to 9.2.1.1 and 9.3.1.1 is not determined.  
 For compressive members the minimum reinf. acc. to 9.5.2 is considered.  
 fck Concrete strength for design of reinforcement [ $\text{MN}/\text{m}^2$ ]

Beam 70,  $x = 0.00$  m (Beam length 0.83 m)

Cross-section 2: Polygon - C30/37-EN

Steel 2; Design mode: Standard

(M) fctm=2.9; zs,t/b=0.513/0.513; fyk,t/b=500/500

(R) wmax=0.3; ds=10; kc per Eq. (7.3); k=1; fct,eff=2.9

(B) fck=30

Section properties	A [ $\text{m}^2$ ]	ys [m]	zs [m]	Iy [ $\text{m}^4$ ]	Iz [ $\text{m}^4$ ]	Iyz [ $\text{m}^4$ ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

#### 1. Characteristic (rare) combination (CC.1): G+QN, Final state

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
1 :	0.00	69.95	0.00
2 :	0.00	196.53	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1 :	L1
2 :	L1+L2+L3

#### 1. Quasi-continuous combination (QC.1): G+QN, Final state

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
2 :	0.00	107.92	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2 :	L1+0.30*L2+0.30*L3

#### 1. Permanent and temporary comb. (PC.1): G+QN, Final state

Relevant concrete internal forces from 8 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
2 :	0.00	284.31	0.00
5 :	0.00	69.95	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2 :	1.35*L1+1.50*L2+1.50*L3
5 :	L1

**Design of longitudinal reinforcement**

Reinforcement Lay.	Nx Type	My [kNm]	Mz [kNm]	max Sc [MN/m <sup>2</sup> ]	kc	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	Situation
1	M	0.00	69.95	0.00	.	.	0.00	CC.1,1
	R	0.00	107.92	0.00	4.25	.	0.00	QC.1,2
	B	0.00	69.95	0.00	.	.	0.00	PC.1,5
2	M	0.00	69.95	0.00	.	.	0.00	CC.1,1
	R	0.00	107.92	0.00	4.25	.	0.00	QC.1,2
	B	0.00	69.95	0.00	.	.	0.00	PC.1,5
3	M	0.00	196.53	0.00	7.73	.	1.44	CC.1,2
	R	0.00	107.92	0.00	4.25	0.50	2.53	QC.1,2
	B	0.00	284.31	0.00	.	.	5.60	PC.1,2
4	M	0.00	196.53	0.00	7.73	.	1.44	CC.1,2
	R	0.00	107.92	0.00	4.25	0.50	2.53	QC.1,2
	B	0.00	284.31	0.00	.	.	5.60	PC.1,2

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to Eq. (9.5N) is considered.

Ac	Section area for calculating the concrete stress from long. force [m <sup>2</sup> ]
bw	Effective width for calculation of shear stresses from Qz and Mx [m]
bn	Statically effective width for shear design using Qy [m]
kb	Factor to calculate the inner lever arm from bn
h	Effective height for calculation of shear stresses from Qy and Mx [m]
d	Statically effective height for shear design using Qz [m]
kd	Factor to calculate the inner lever arm from d
Angle	Angle cot Theta between the compressive strut and the beam axis
Asl giv.	Chargeable longitudinal reinf. acc. to Fig. 6.3 [cm <sup>2</sup> ]
rhow,min	Minimal percentage of lateral reinforcement acc. to Eq. (9.5N)
Qy, Qz	Lateral forces for design in y- and z-direction [kN]
VRdc	Absorbable lat. force without lat. reinf. per 6.2.2 (1) [kN]
VRdmax	Absorbable lateral force of comp. struts per 6.2.3 (3) [kN]
z	Inner lever arm z=kb*bn resp. z=kd*d [m]
Asb.y,z	Req. stirrup reinforcement from Qy resp. Qz [cm <sup>2</sup> /m]
Asl	Req. longitudinal reinf. acc. to Fig. 6.3 [cm <sup>2</sup> ] for req.Asb
Delta Ftd	Tensile force in long. reinf. from lateral force as per Eq. (6.18)

Beam 70, x = 0.00 m (Beam length 0.83 m)  
 Cross-section 2: Polygon - C30/37-EN  
 bw/bn/kb=0.3/0.27/0.9; h/d/kd=0.6/0.57/0.9  
 Ac=0.46; fyk=500; Asl giv./max=2.9/0; rhow,min=1\*(0.08\*fck<sup>1/3</sup>/fyk)

**1. Permanent and temporary comb. (PC.1): G+QN, Final state**

Relevant Set	concrete	internal forces from 8 sets of internal forces
2	:	Nx[kN] My[kNm] Mz[kNm] Mx[kNm] Qy[kN] Qz[kN]
2	:	0.00 284.31 0.00 0.00 0.00 -30.79

Load case combinations for the relevant sets of internal forces	
Set	Combination
2	: 1.35*L1+1.50*L2+1.50*L3

**Check of the shear reinforcement and the compressive struts**

Action	z	Q/	Asb.y	Asb.z	Asb.T	Asl.T	Asl	Situation
max	[m]	Angle	VRdc	[cm <sup>2</sup> /m]	[cm <sup>2</sup> /m]	[cm <sup>2</sup> /m]	[cm <sup>2</sup> ]	[cm <sup>2</sup> ]
Qy	: 0.24	1.00	0.00	.	.	.	.	-,-
Qz	: 0.51	1.00	0.47	.	2.63	.	2.90	PC.1,2
Action	z	Qy/	Qz/	Mx/	Q/VRd+	Delta	Ftd	Situation
max	[m]	Angle	VRdmax	VRdmax	TRdmax	Mx/TRd	[kN]	[cm <sup>2</sup> ]
Qy	: 0.24	1.00	0.00	.	.	.	0.00	-,-
Qz	: 0.51	1.00	.	0.04	.	.	15.40	PC.1,2

**Shear joint check**

bi	Width of the joint [m]
sigma.n	Stress perpendicular to the joint acc. to user specifications. Negative for compression [MN/m <sup>2</sup> ]
Roughness	Joint roughness (very smooth, smooth, rough, indented)
c, mue	Factors dependent on joint roughness according to 6.2.5(2)
z	Inner lever arm according to shear section [m]
beta	Ratio of longitudinal force in the new concrete and total longitudinal force in compression or tension zone. If the strain state cannot be clearly determined, 1.0 is assumed on the save side (marking *)
vEdi	Design value of shear stress in the joint acc. to Eq. (6.24) [MN/m <sup>2</sup> ]
vRdi	Design value of shear resistance in the joint acc. to Eq. (6.25) [MN/m <sup>2</sup> ]
vEdi/vRdi	Utilization rate
req. Asb	Area of the required transverse reinforcement [cm <sup>2</sup> /m]

Beam 70, x = 0.00 m (Beam length 0.83 m)  
 Cross-section 2: Polygon - C30/37-EN  
 bi = 0.3 m; sigma.n = 0 MN/m<sup>2</sup>  
 Roughness = Rough; c = 0.4; mue = 0.7

**1. Permanent and temporary comb. (PC.1): G+QN, Final state**

Relevant Set	concrete	internal forces from 8 sets of internal forces
2	:	Nx[kN] My[kNm] Mz[kNm] Qz[kN]
2	:	0.00 284.31 0.00 -30.79

Load case combinations for the relevant sets of internal forces	
Set	Combination
2	: 1.35*L1+1.50*L2+1.50*L3

**Shear joint check**

z	beta	vEdi	vRdi	vEdi/ vRdi	req. Asb	Situation
[m]	[-]	[MN/m <sup>2</sup> ]	[MN/m <sup>2</sup> ]		[cm <sup>2</sup> /m]	
0.51	1.00	0.2001	0.5413	0.37	0.00	PC.1,2

**Check of crack widths**

The check is led by direct calculation of the crack width.  
The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

(CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

wmax	Permissible crack width as per specification [mm]
ds	Maximal given steel diameter [mm]
fct,eff	Concrete strength at date of cracking [MN/m <sup>2</sup> ]
Sigma.c	Maximal concrete edge stress in state I [MN/m <sup>2</sup> ]
wk	Calculated value of crack width as per 7.3.4 [mm]
sr,max	Calculated / given maximal crack spacing as per 7.3.4 (3) [mm]
Ac,eff	Effective region of reinf. [m <sup>2</sup> ] acc. to Fig. 7.1
As,eff	Reinforcing steel within Ac,eff [cm <sup>2</sup> ]
Ap,eff	Prestressing steel with bond within Ac,eff [cm <sup>2</sup> ]
Sigma.s	Reinf. steel stress in state II [MN/m <sup>2</sup> ]
kt	Coefficient for the duration of load as per 7.3.4 (2)

Beam 70, x = 0.00 m (Beam length 0.83 m)  
Cross-section 2: Polygon - C30/37-EN  
wmax=0.3; ds=10; fct,eff=2.9; kt=0.4

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

**1. Quasi-continuous combination (QC.1): G+QN, Final state**

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
2 :	0.00	107.92	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2 :	L1+0.30*L2+0.30*L3

**Check of crack width for reinf. layer 3 (bottom)**

Nx	: 0.00 kN	As,eff	: 11.19 cm <sup>2</sup>
My	: 107.92 kNm	Ap,eff	: . cm <sup>2</sup>
Mz	: 0.00 kNm	Ac,eff	: 0.023 m <sup>2</sup>
Sigma.c	: 4.25 MN/m <sup>2</sup>	Sigma.s	: 175.61 MN/m <sup>2</sup>
Situation	: QC.1,2	sr,max	: 119.17 mm
		wk	: 0.09 wmax 0.30 mm

**Check of concrete compressive stress**

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked.

fck	Characteristic compressive concrete strength [MN/m <sup>2</sup> ]
Sigma.x,min	Total maximal longitudinal compressive stress [MN/m <sup>2</sup> ]
Sigma.x,per	= 0,60*fck for Charact. C. (CC) as per 7.2 (2)
top, bottom	Position of the edge point: above, below of centre

Beam 70, x = 0.00 m (Beam length 0.83 m)  
Cross-section 2: Polygon - C30/37-EN  
0.6\*fck=18

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

**1. Characteristic (rare) combination (CC.1): G+QN, Final state**

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
1 :	0.00	69.95	0.00
2 :	0.00	196.53	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1 :	L1
2 :	L1+L2+L3

**Check of compressive stress in concrete for the Characteristic (rare) combination**

Side	Se.- Pnt.	min [MN/m <sup>2</sup> ]	Sigma.x per. [MN/m <sup>2</sup> ]	Sigma.x [MN/m <sup>2</sup> ]	Period	Situation
top	1	(II)	-6.58	-18.00	Final	CC.1,2
bottom	7	(II)	0.00	-18.00	Final	CC.1,1

**Check of steel stress**

For the check, a cracked concrete section is assumed.

Type S Long. reinf. from N and M, layer number, Charact. C. (CC)  
 fck Concrete strength to determine the strain state [MN/m<sup>2</sup>]  
 Sigma.s,per = 0.80 \* fyk resp. 1.0 \* fyk (CK) as per 7.2 (5)

Beam 70, x = 0.00 m (Beam length 0.83 m)

Cross-section 2: Polygon - C30/37-EN  
 fck=30; Steel 2; 0.8\*fyk,t/b=400/400

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

**1. Characteristic (rare) combination (CC.1): G+QN, Final state**

Relevant concrete internal forces from 4 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
1 :	0.00	69.95	0.00
2 :	0.00	196.53	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1 :	L1
2 :	L1+L2+L3

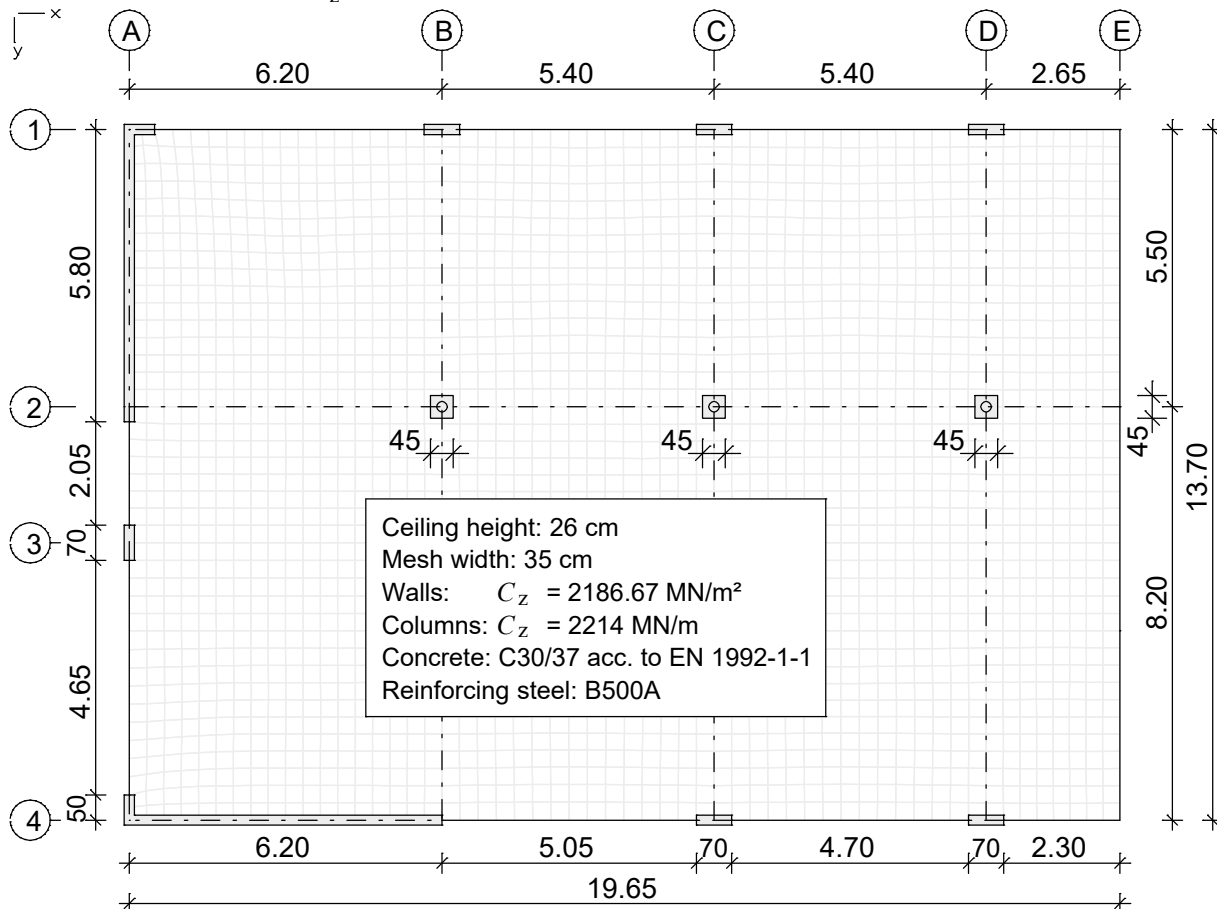
**Check of steel stress**

Steel Type	No.	Nx [kN]	My [kNm]	Mz [kNm]	As [cm <sup>2</sup> ]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]	Situation
S	1	0.00	69.95	0.00	0.00	.	400.00	CC.1,1
S	2	0.00	69.95	0.00	0.00	.	400.00	CC.1,1
S	3	0.00	196.53	0.00	5.60	319.92	400.00	CC.1,2
S	4	0.00	196.53	0.00	5.60	319.92	400.00	CC.1,2

## Flat Ceiling With Cantilever

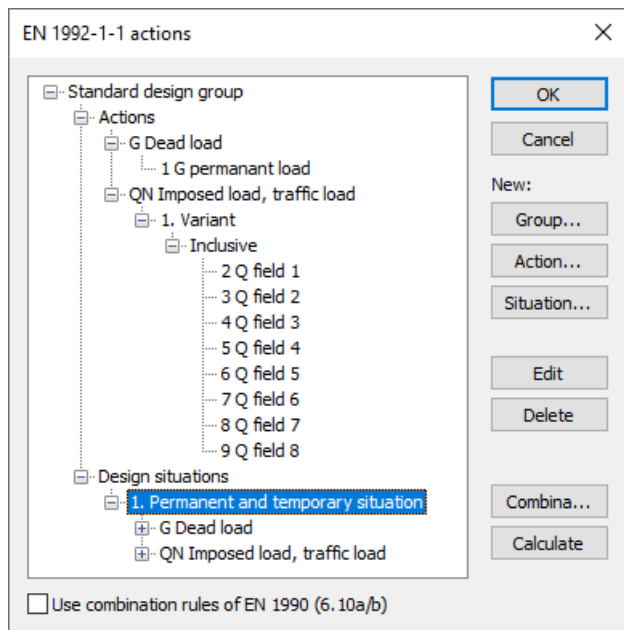
This example is a ceiling slab corresponding to Example 18 in the book 'Beispiele zur Bemessung nach Eurocode 2 – Band 2: Ingenieurbau'. The hinged slab is loaded with permanent loads and a traffic load applied span by span. The load cases are calculated linearly-elastically, combined to minimum and maximum design internal forces according to the permanent and temporary design situations and designed for the ultimate limit state. Furthermore, elastic calculations are performed for a deformation check.

The vertical spring stiffnesses  $C_z$  of the walls and columns are applied according to the literature.

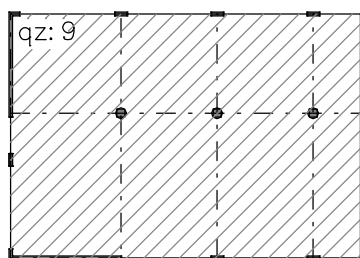


Element system with dimensions [m]

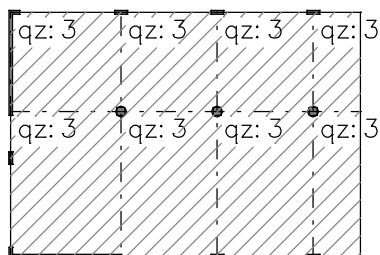
Design according to EN 1992-1-1



Loads



Load case 1, G permanent load [kN/m<sup>2</sup>]



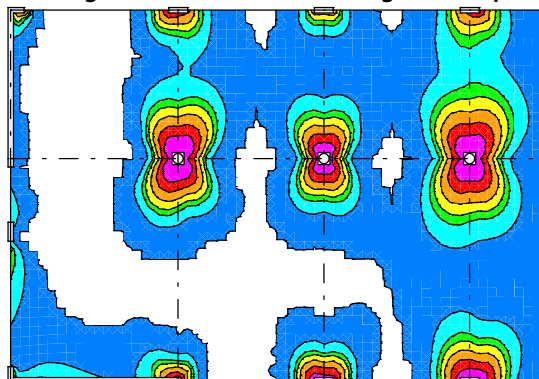
Load case 2 to 9, Q field 1 to 8 [kN/m<sup>2</sup>]

Reinforcement for area elements

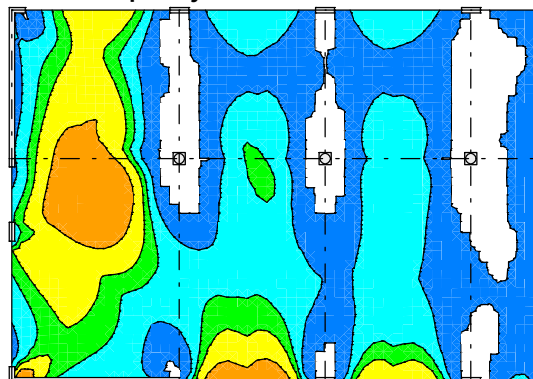
No.	Lay.	Qual.	d1x [m]	d2x [m]	asx [cm <sup>2</sup> /m]	d1y [m]	d2y [m]	asy [cm <sup>2</sup> /m]	as fix	Rolling
1	1	500M	0.045	0.035	0.000	0.035	0.025	0.000		Warm
	2	500M			0.000			0.000		Warm

- as Base reinforcement
- d1 Distance from the upper edge
- d2 Distance from the lower edge
- The z axis of the element system points to the lower edge

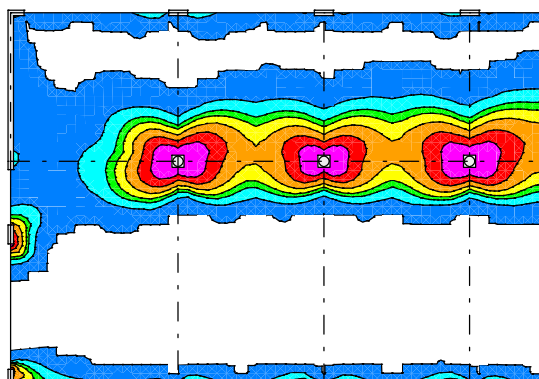
Bending reinforcement from design of the permanent and temporary situation



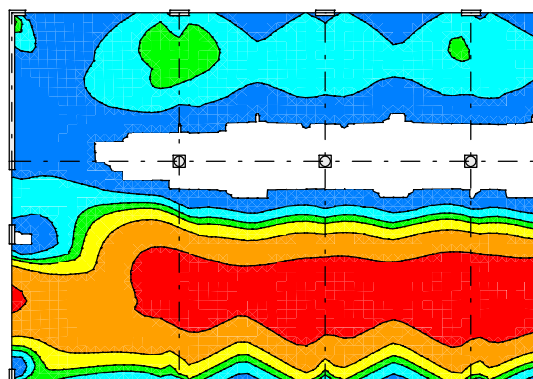
Bending reinforcement upper layer  $a_{sx,1}$  [cm<sup>2</sup>/m]



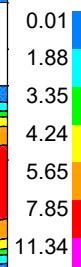
Bending reinforcement lower layer  $a_{sx,2}$  [cm<sup>2</sup>/m]



Bending reinforcement upper layer  $a_{sy,1}$  [cm<sup>2</sup>/m]



Bending reinforcement lower layer  $a_{sy,2}$  [cm<sup>2</sup>/m]



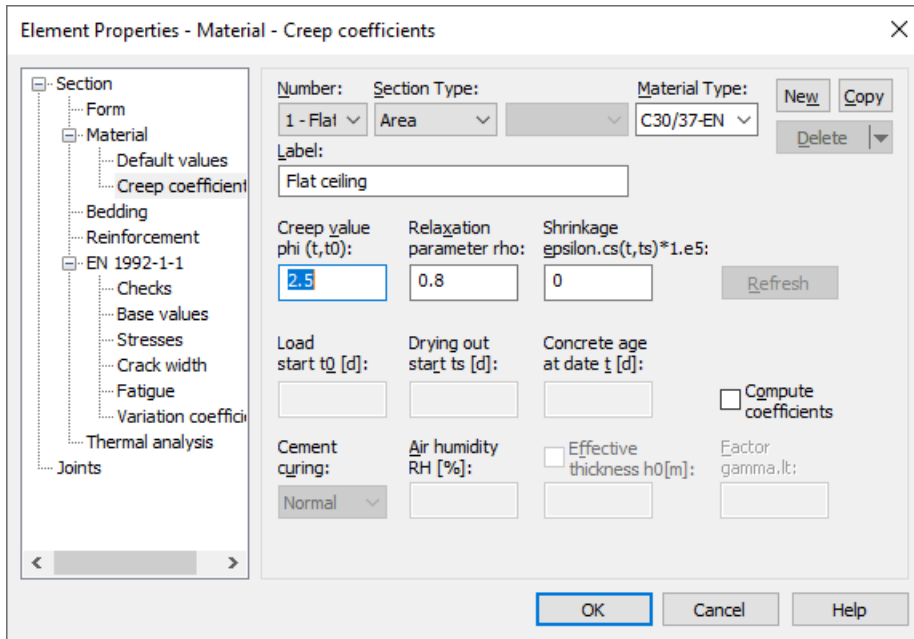
**Deformation  $u_z$**

The combination coefficient  $\psi_2$  of the quasi-continuous situation is assumed to be 0.3 in this example. For the calculation of the creep deformation a creep-generating permanent load case has to be defined (here superposition load case number 11).

**Load data load case 11: G+0.3·Q with elastic material behavior**

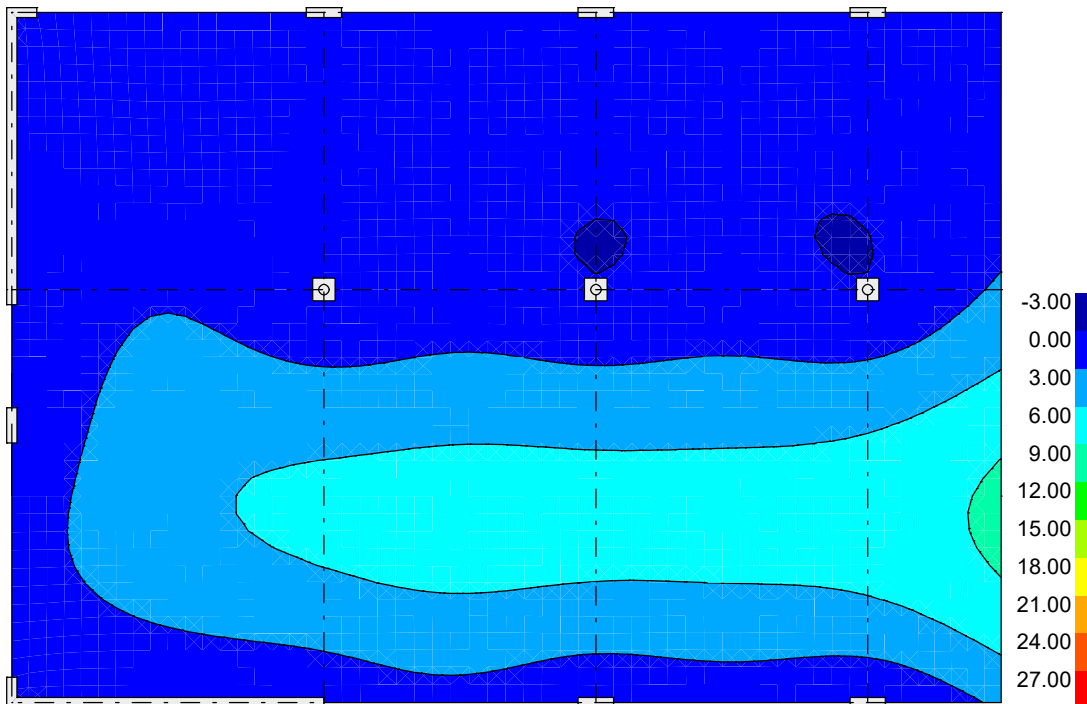
No.	Superposition of results (SUP)		weighting
	from	to	
1	1	1	1.000
2	2	9	0.300

The concrete creep is then calculated in load case 12 (load type *Creep and shrinkage*). In addition to the data in the load case, the creep and shrinkage coefficients of the concrete material specified in the cross-section are required. Since the shrinkage coefficient has no meaning for slab elements, it was set to zero in this example. Alternatively, the coefficients could have been calculated via the dialog according to the formulas of the standard.



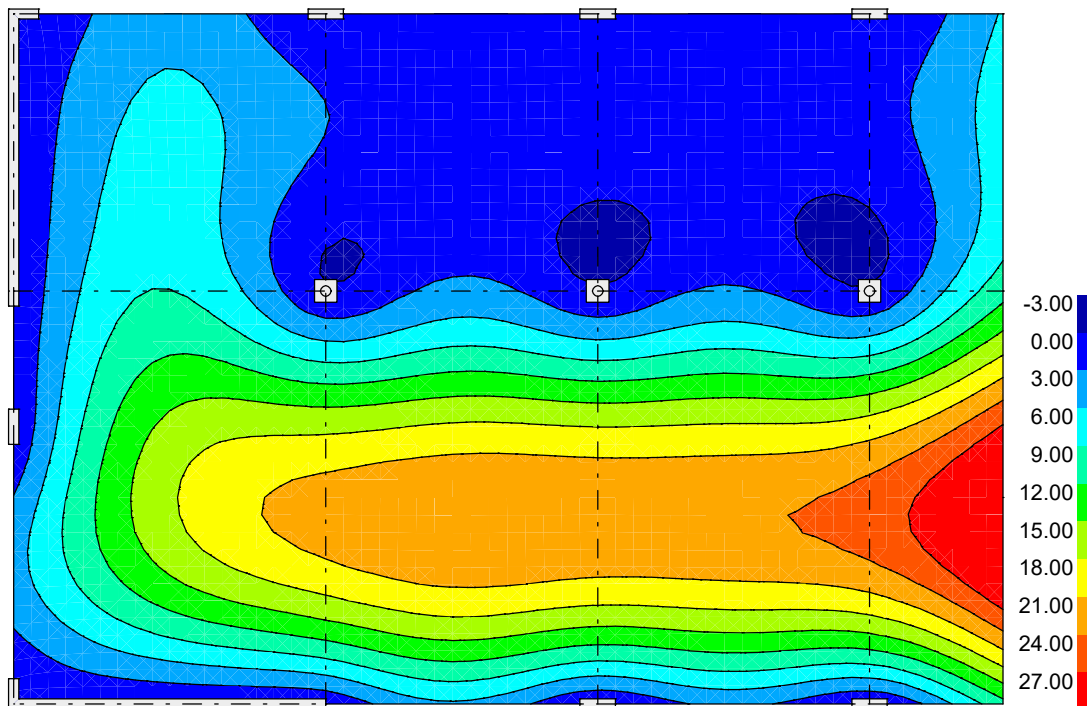
The internal forces due to creep were not taken into account in the design, since they generally remain insignificant for all elements with identical creep coefficients. However, relevant deformations due to creep are to be expected.

Below are some of the results.



Load case 11 (G+0.3·Q): Color gradient of the deformations  $u_z$  [mm] at the start of loading



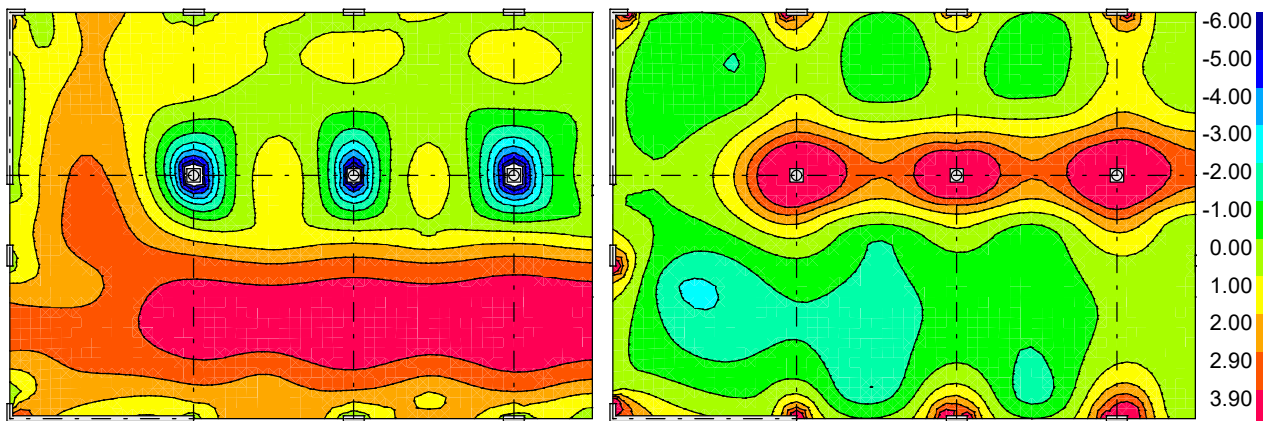


Load case 13: Color gradient of the deformations  $u_z$  [mm] at time  $t_\infty$

In summary, the following maximum deformations result in state I:

Load case	max $u_z$ [mm]	
	Calculation	Literature
11 G+0.3·Q with elastic material behavior	10	10
12 Creeping ( $\varphi=2,5$ by LC 11) with elastic material behavior	23	
13 Total (load case 11 + load case 12)	33	

**Principal tensile stresses  $\sigma_1$**



Load case 13: Color gradient of the principal tensile stresses  $\sigma_{1, \text{bottom}}$  and  $\sigma_{1, \text{top}}$  [MN/m<sup>2</sup>]

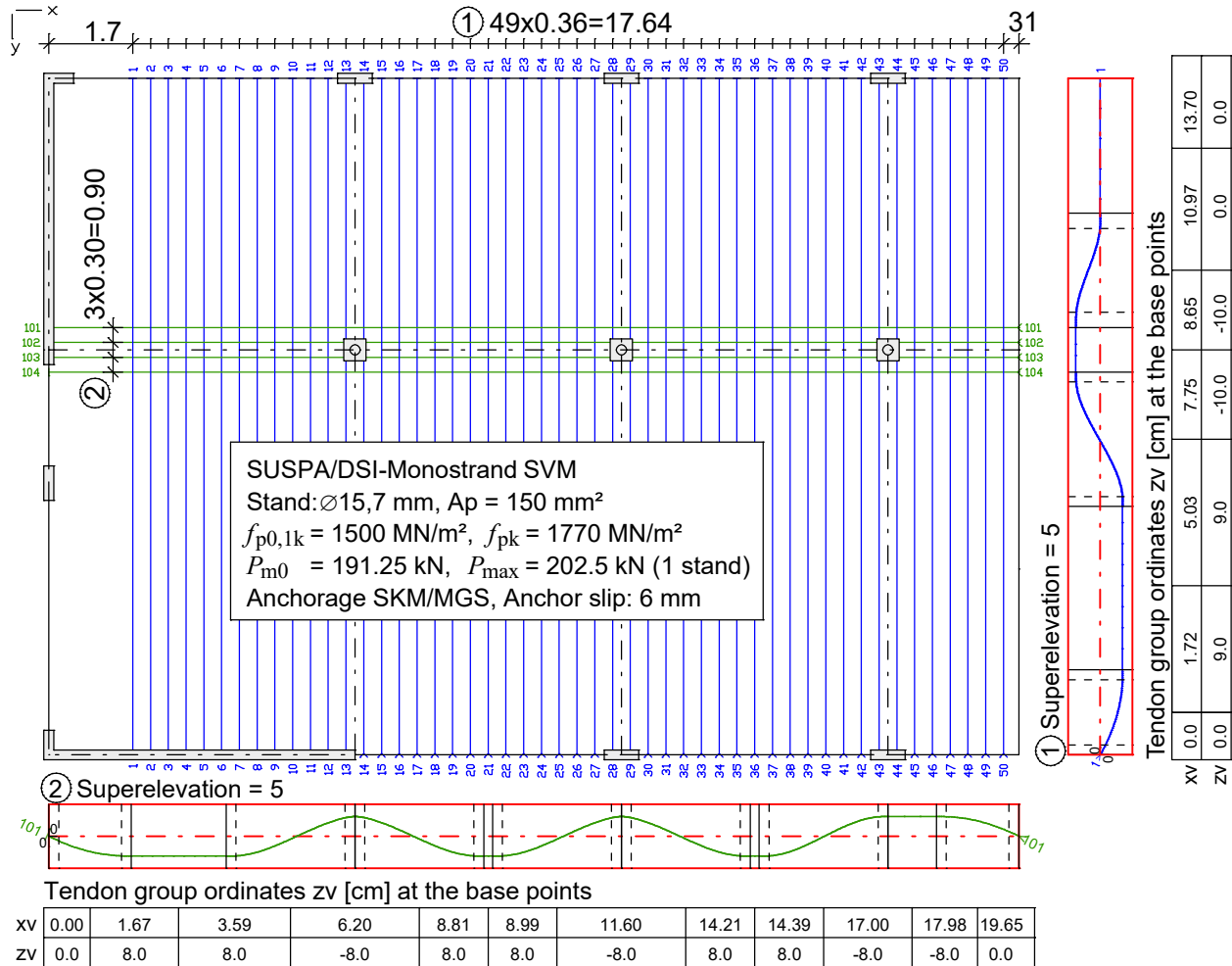
If Since the principal tensile stresses  $\sigma_1$  exceed the concrete tensile strength  $f_{ctm}$  of 2.9 MN/m in a large area, the calculation in state I is not suitable for the deformation check. In the manual section for the nonlinear system analysis, the deformations are therefore examined in detail in state II for this example.

## Flat Ceiling With Cantilever and Prestressing

This example is a variant of the previous example with prestressing without bond. For comparison, the required concrete reinforcement according to EN 1992-1-1 and the linear-elastic deformations are also determined. The prestressing is primarily intended to reduce the deflection of the slab.

Since a prestressed slab also results in normal forces in the area, shell elements must be used and care must be taken to ensure that the horizontal supports are as free of restraints as possible. Otherwise, large tensile stresses would result from the shrinkage of the concrete.

Monostrands of 7-strand prestressing steel St 1570/1770 with a corrosion protection system in free tendon position are used. The 50 tendons (1) in y-direction are uniformly distributed single monostrands. The four tendons (2) in the x-direction are double bundled monostrands.



System with arrangement of monostrands in free tendon position with their high and low points

### Loads

In addition to the load cases of the previous example, a prestressing load case (load type *Prestressing*) is required. The creep-generating permanent load case 11 is supplemented by the prestressing load case. Since the tendons have no bond with the concrete, the shrinkage of the concrete of  $\epsilon_{s,\infty} = -40 \cdot 10^{-5}$  assumed in the literature leads to an additional anchor slip of 7.9 mm in x direction and 5.5 mm in y direction. Thus, at time  $t_{\infty}$ , neglecting friction, a prestressing force of 178 kN per tendon (1) and 359 kN per tendon (2) results. The prestressing at an earlier time is not investigated in this example.

### Load data load case 10: P prestressing Load data load case 11: G+P+0.3·Q with elastic material behavior

No.	Superposition of results (SUP)			
	No.	load case from	to	weighting
1   Prestressing (VSPG)	1	1	1	1.000
	2	10	10	1.000
	3	2	9	0.300

**Design according to EN 1992-1-1**

The design-relevant actions are supplemented by the prestressing and the creep load case.

**EN 1992-1-1 actions**

**Standard design group**

**QN - Imposed load, traffic load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
Working load - category A: Residential buildings  
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

Load cases 1. Variant, inclusive

- 2 Q field 1
- 3 Q field 2
- 4 Q field 3
- 5 Q field 4
- 6 Q field 5
- 7 Q field 6
- 8 Q field 7
- 9 Q field 8

**G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

- 1 G permanent load

**P - Prestressing**

Gamma.sup / gamma.inf = 1 / 1

Load cases internal prestressing

- 10 P prestressing

**CSR1 - Creep, shrinkage, relaxation**

Load cases

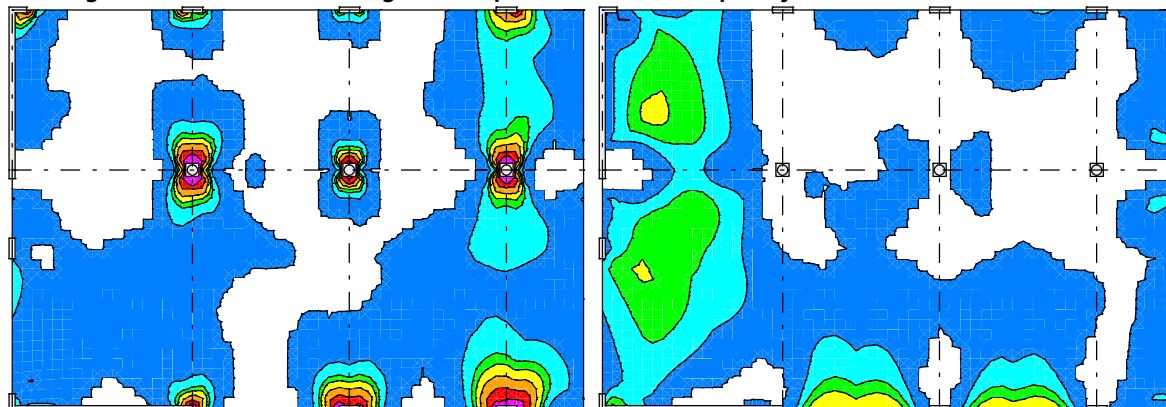
- 12 Creeping ( $\varphi=2.5$  by LC 11) with elastic material beh...

**1. Permanent and temporary situation**

Final state

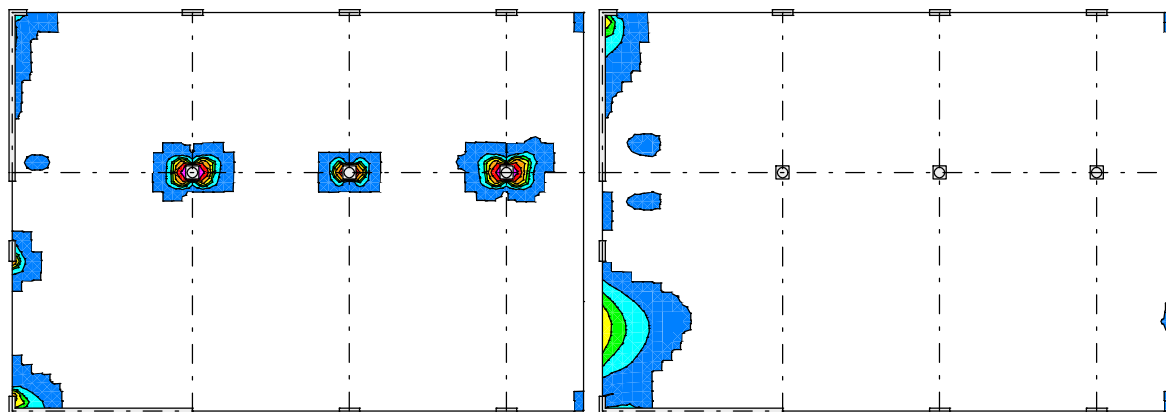
- G Dead load
- P Prestressing
- CSR1 Creep, shrinkage, relaxation
- QN Imposed load, traffic load

**Bending reinforcement from design of the permanent and temporary situation**



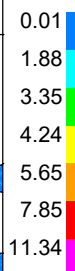
Bending reinforcement upper layer  $a_{sx,1}$  [cm<sup>2</sup>/m]

Bending reinforcement lower layer  $a_{sx,2}$  [cm<sup>2</sup>/m]

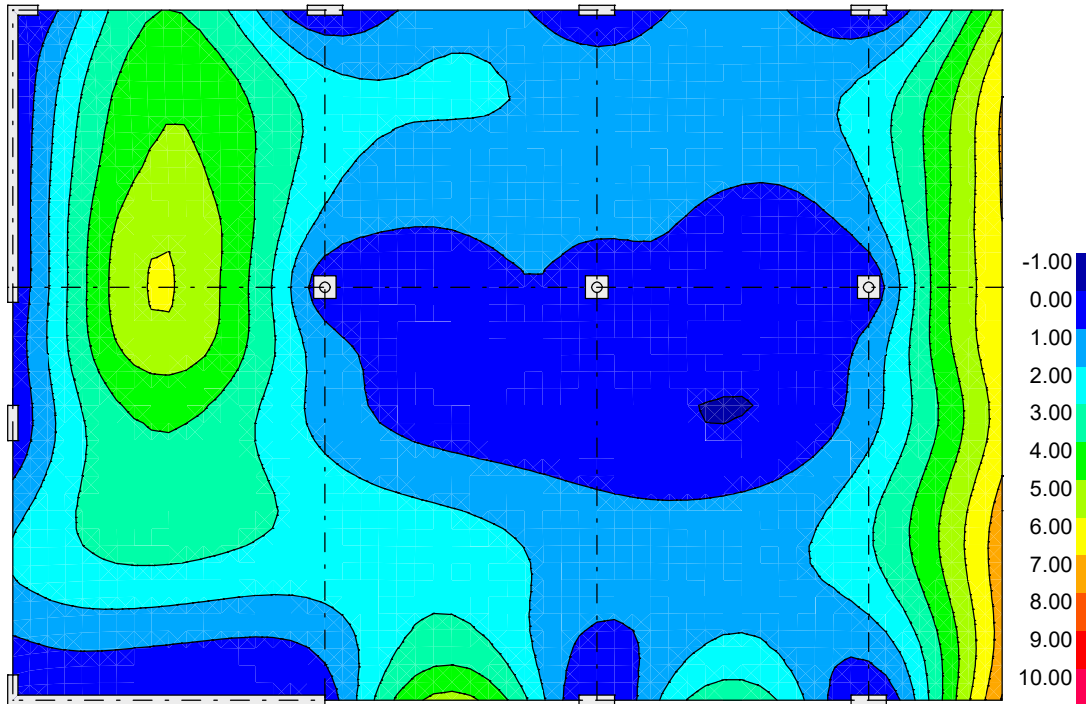


Bending reinforcement upper layer  $a_{sy,1}$  [cm<sup>2</sup>/m]

Bending reinforcement lower layer  $a_{sy,2}$  [cm<sup>2</sup>/m]



**Deformation  $u_z$**

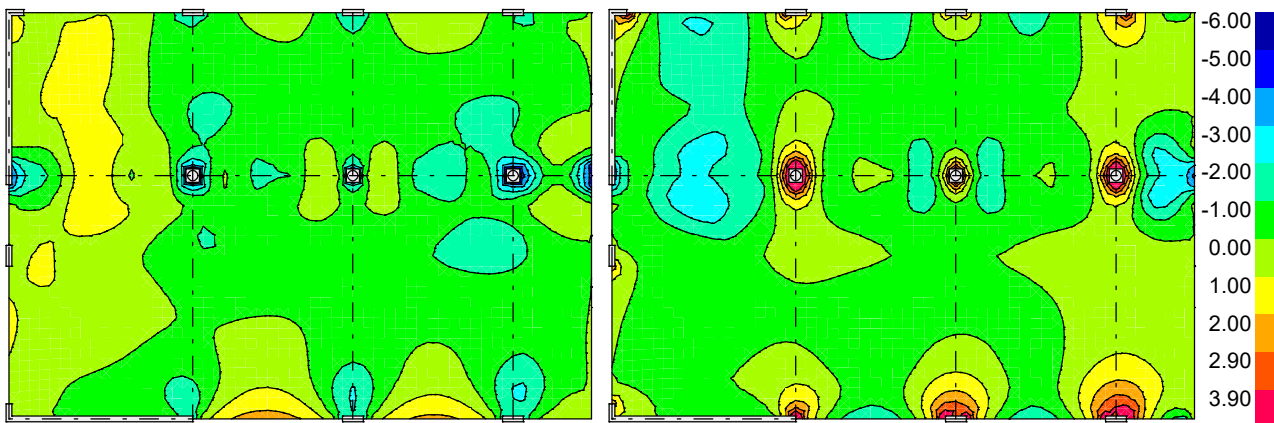


Load case 13: Color gradient of the deformations  $u_z$  [mm] at time  $t_\infty$

In summary, the following maximum deformations result in state I compared to the previous example without prestressing:

Load case	max $u_z$ [mm]	
	with prestress	without prestress
11 G+P+0.3·Q with elastic material behavior	3	10
12 Creeping ( $\varphi=2,5$ by LC 11) with elastic material behavior	5	23
13 Total (load case 10 + load case 11)	8	33

**Principal tensile stresses  $\sigma_1$**



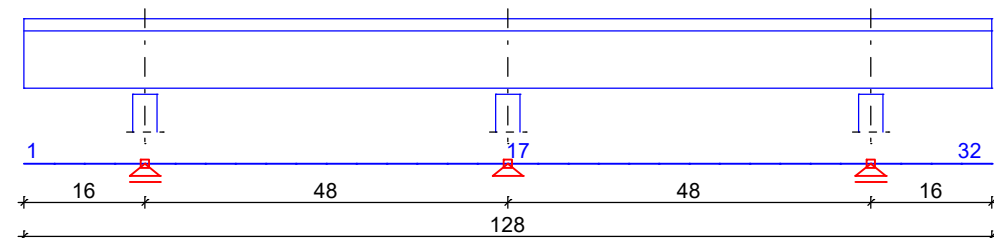
Load case 13: Color gradient of the principal tensile stresses  $\sigma_{1, \text{bottom}}$  and  $\sigma_{1, \text{top}}$  [MN/m<sup>2</sup>]

Since the principal tensile stresses  $\sigma_1$  exceed the concrete tensile strength  $f_{ctm}$  of 2.9 MN/m<sup>2</sup> only in small areas, an investigation in state II is not performed.

## Prestressed Roof Construction

This example involves the wide-spanned roof construction of an entrance hall that is represented as a continuous girder over two spans with a double-sided cantilever. A T-beam is selected as the section. The figure below shows the system in longitudinal and lateral section view.

Limited prestressing with subsequent bond is applied to the roof construction in the longitudinal direction. Prestressing in the lateral direction is not applied for reasons of economy. The construction is designed to meet exposure class XC1. According to Table 7.1N of the EN 1992-1-1, a decompression check is not necessary for this class.

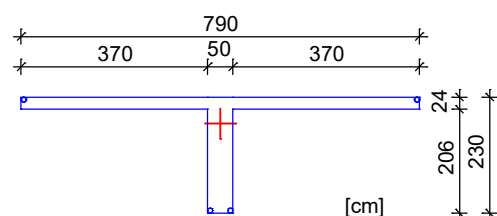


Static system and dimensions [m]

### Material

Concrete *C45/55-EN*  
Reinforcing steel *BSt 500*, axis distance from edge 5 cm

### Section



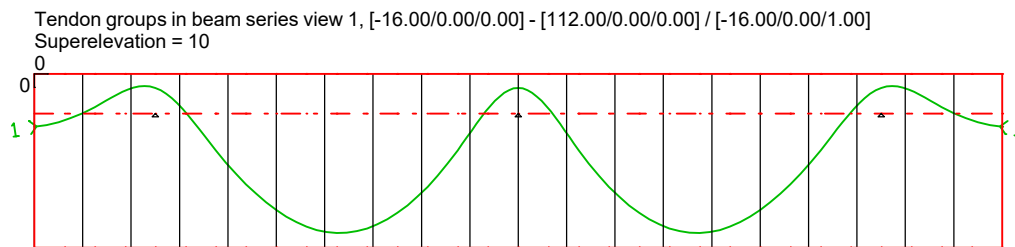
### Prestressing steel and prestressing system

Prestressing steel quality	St 1500/1770
Certification of the prestressing system	EC2
Number of tendons in the bundle	4
Section surface $A_p$	1800 mm <sup>2</sup>
E-modulus of the prestressing steel	195000 MN/m <sup>2</sup>
0.1% strain limit (yield strength) of the prestressing steel $f_{p0.1k}$	1500 MN/m <sup>2</sup>
Tensile strength of the prestressing steel $f_{pk}$	1770 MN/m <sup>2</sup>
Permissible prestressing force of a tendon $P_{m0}$	2295 kN
Prestressing loss from relaxation of prestressed steel	4.5 %
Friction coefficients when prestressing and releasing $\mu$	0.2
Unintentional deviation angle of a tendon $\beta'$	0.3 ‰/m
Slippage at prestressing anchor	6 mm
Duct diameter $d_h$	82 mm

### Variation coefficients of the internal prestressing

Construction stage ( $r_{sup} / r_{inf}$ )	1.1 / 0.9
Final state ( $r_{sup} / r_{inf}$ )	1.1 / 0.9

The tendon guide is shown in the next figure. 4 bundled tendons are arranged such that they stretch across the entire girder length and are prestressed at both girder ends. The prestressing system, prestressing procedure and prestressing curve for a tendon group are also shown.



Tendon group ordinates z<sub>v</sub> [cm] at the base points

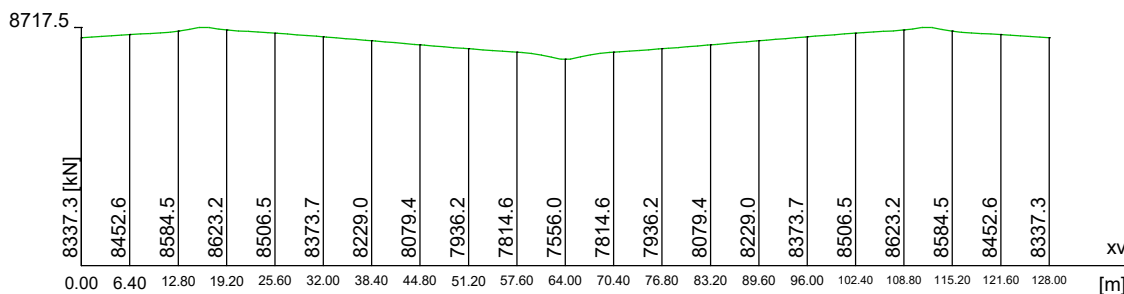
xv	0.00	6.40	12.80	19.20	25.60	32.00	38.40	44.80	51.20	57.60	64.00	70.40	76.80	83.20	89.60	96.00	102.40	108.80	115.20	121.60	128.00
1	70.0	52.0	19.2	42.0	120.2	180.1	208.8	201.5	157.4	78.3	18.5	78.3	157.4	201.5	208.8	180.1	120.2	42.0	19.2	52.0	70.0

Force function of tendon group 1 (4 tendon(s), l = 128.41 m)

Prestressing system 1 - Example. Certification according to EC2.  
 P<sub>m0</sub> = 2295.0 kN, A<sub>p</sub> = 1800.0 mm<sup>2</sup>, μ<sub>a</sub> = 0.20, Angle β' = 0.30 °/m  
 E-Modulus = 195000 MN/m<sup>2</sup>, A<sub>h</sub> = 5281.0 mm<sup>2</sup>, μ<sub>n</sub> = 0.20, Slippage = 6.00 mm

Prestressing procedure 1 - Example

Pre. anchor : Start End  
 Normal. force : 1.000 1.000  
 Pre. force [kN]: 2295.0 2295.0  
 Extension [mm]: 694.5 72.8



Tendon guide and prestressing curve in the longitudinal section (4 tendons).

**Loads**

- Load case 1 Dead load
- Load case 2 Additional dead load: q=11.06 kN/m
- Load case 3 Snow load: q=7.90 kN/m
- Load case 10 Prestressing
- Load case 15 Creep-generating permanent load: Dead load, additional dead load and prestressing
- Load case 20 Creep and shrinkage  
 Coefficients: φ<sub>t∞</sub> = 2.55; ρ = 0.8; ε<sub>t∞</sub> = -24.8 · 10<sup>-5</sup>  
 Creep-generating permanent load case: 15  
 The redistribution of internal forces between concrete and prestressing steel are taken into account.

**EN 1992-1-1 actions**

**Standard design group**

**G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

- 1 Dead load

**CSR1 - Creep, shrinkage, relaxation**

Prestressing loss from relaxation of prestressed steel: 4.5 %.

Load cases

- 20 Creep, shrinkage

**G - Additional dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

2 Additional dead load

**P - Prestressing**

Gamma.sup / gamma.inf = 1 / 1

Load cases internal prestressing

10 Prestressing

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures

Snow load - Places in CEN member states with more than 1000 m above sea level  
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.2

Load cases 1. Variante, inclusive

3 Snow load

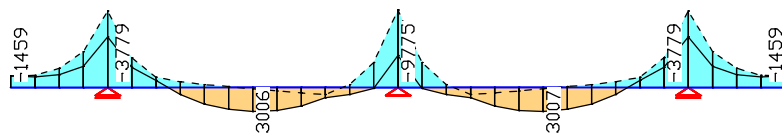
In this example all possible combinations of load cases are generated and designed. This method is selected in the calculation settings and can be very slow when applied for a large number of load cases.

Below you will find an example of the curve of bending moment  $M_y$  for design situations in the ultimate limit states.

**1. Permanent and temporary situation - Structural cond.**

Construction stage - UngROUTed

G Dead load  
P Prestressing

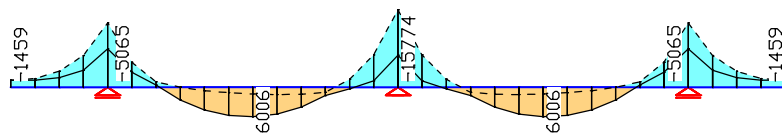


Bending moment  $M_y$  [kNm]

**2. Permanent and temporary situation - t0**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
QS Snow and ice load

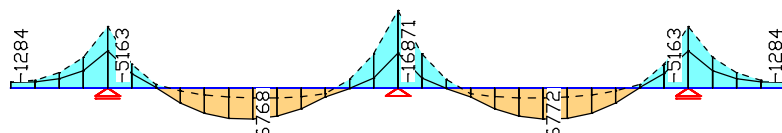


Bending moment  $M_y$  [kNm]

**3. Permanent and temporary situation - too**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
CSR1 Creep, shrinkage, relaxation  
QS Snow and ice load



Bending moment  $M_y$  [kNm]

**Design overview**

Se.	Expos. class	Prestress of component	Reinforc. M R B Q T	Fatigue B Q T P C V	Cr. wi. x	De- co. .	Stress C B P
1	XC4	Subsequent bond	x x x x .	. . . . .	x	.	x x x

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state, fatigue and stress check.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.
- (T) Torsional reinforcement at ultimate limit and fatigue state.
- (P) Prestressing steel at fatigue and stress check.
- (C) Concrete comp. stress, concrete at fatigue check under long. compression.
- (V) Concrete at fatigue check under lateral force.

**Dispersion of prestressing**

The variation of prestressing is considered at the following checks:  
 - Check of decompression and concrete compressive stress  
 - Nominal reinforcement for crack width limitation  
 - Check of crack width  
 All other checks are made using the mean value  $P_{m,t}$  of prestressing.

Se.	Prestressing of component	Const.period r.sup	r.inf	Final state r.sup	r.inf
1	Subsequent bond	1.10	0.90	1.10	0.90

**Settings for flexural and shear reinforcement**

M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 fyk Quality of stirrups.  
 Theta Angle of concrete truss.  
 Slabs Beams are designed like slabs.  
 Asl Given reinforcement according to picture 6.3, increase to maximum.  
 rho.w Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).  
 as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).  
 Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Dens- sity [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] Pic. 6.3 given max	Factor rho.w as	Red. pre- str.
1	C45/55-EN	.	ST	500	2.50	.	0.00	1.00	.

**Shear sections**

bw.nom Nominal width of the prestressed section according to 6.2.3(6).  
 h.nom Nominal height of the prestressed section according to 6.2.3(6).  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.  
 z1, z2 Height and width of the core section for torsion.  
 tef Thickness of the torsion box.  
 B. Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width [m]	Eff. width bw	Height [m] h	Eff.height d	Torsion. section [m] kd	z1	z2	tef	B.
1	0.500	0.500	0.450	0.90	2.300	2.300	2.250	0.90	2.200 0.400 0.100 .

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel [mm].  
 max.s Maximal given bar spacing of the reinforcing steel [mm].  
 sr,max Upper limit for the crack spacing from Eq. (7.11) [mm].  
 Xil Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 kt Coefficient for the duration of the load to calculate the crack width.  
 Fact. Reduction factor for factm as per Chapt. 7.3.2 (As) resp. 7.3.4 (wk).  
 Comb. Combination for verifying the minimum reinf. (As) and crack width (wk):  
 CC, TC, QC = Characteristic, frequent, quasi-continuous combination,  
 CT, TT, TB = Central tension, tension on top side, tension on bottom,  
 CL = Action combination according to exposure class.  
 Method Check method for minimum reinf. (kc) and crack width (wk):  
 kc Determination of coefficient kc for webs/chords per Eq. (7.2/7.3).  
 auto = Eq. (7.2) for rectangular sections, Eq. (7.3) for others.  
 wk Calc. = Direct calculation of crack width as per Chapter 7.3.4,  
 Bar sp. = Limiting the bar spacing as per Table 7.3N,  
 Cal.(m) = Direct calculation for mean steel strain within Ac,eff,  
 Spc.(m) = Lim. the bar spacing for mean steel strain within Ac,eff.  
 RS Ring-shaped determination of Ac,eff according to Wiese et al.,  
 Beton- und Stahlbetonbau 2004, Issue 4, p 253 ff.

Se.	wmax [mm]	ds	max s	sr	Coefficient Xil k	Fact.fctm As	Comb. wk	Method As wk kc	RS wk
1	0.20	20	.	.	0.38 0.65 0.4	1.00 1.00	CL CL	auto calc.	.

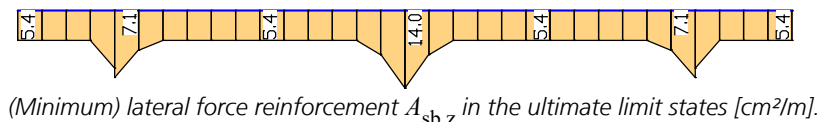
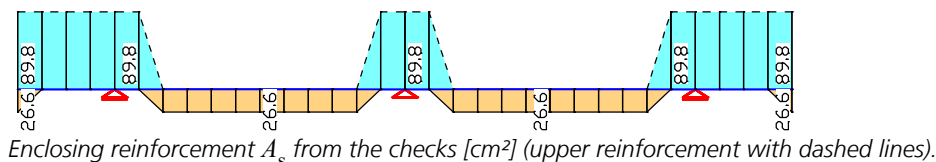
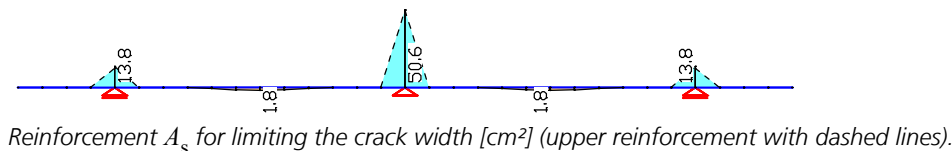
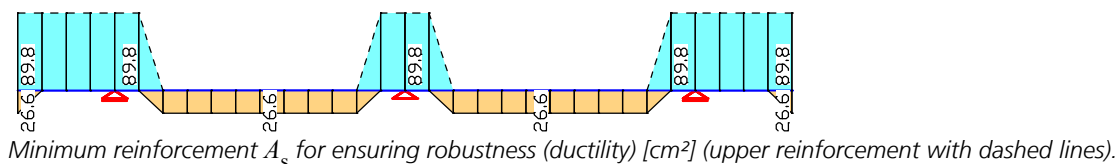
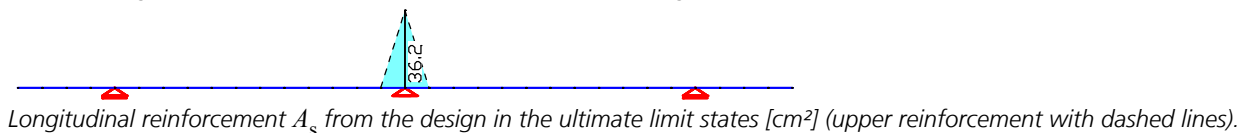


**Settings for the check of concrete and steel stresses**

fck(t) Compressive strength of concrete at the time t of prestressing.  
 Sigma.c(t) Concrete compressive stress at the time t of prestressing.  
 Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.s Reinforcing steel stress in the serviceability limit state.  
 (CC),(QC) Characteristic, quasi-continuous combination.  
 (TC),(CL) Frequent combination, combination according to exposure class.

Se.	fck(t)	per.sigma.c(t)	per.sigma.c	per.sigma.s	Decompression
	[MN/m <sup>2</sup> ]	(CC, QC)	(CC)	(CC)	Comb. Stress
1	45.00	0.45 fck(t)	0.60 fck	0.80 f <sub>yk</sub>	.

The following illustration shows the curve of the required bending and shear reinforcement.



The following pages contain excerpts from the detailed check log for beam 16 at location 2 (middle column).

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. for robustness as per EN 1992-2, 6.1 (109) (Charact. C.)
  - fctm Average centric concrete tensile strength [MN/m<sup>2</sup>]
  - zs,t/b Lever arm of inner strengths top/bottom with zs=0,9\*d [m]
  - fyk,t/b Strength of longitudinal reinforcement top/bottom [MN/m<sup>2</sup>]
  - max Sc Maximum concrete edge stress from Charact. C. [MN/m<sup>2</sup>]
  - without the statically determined part of prestressing
- (R) Nominal/requ. reinforcement as per 7.3.2 for crack width limitation
  - Increase of reinforcement due to crack width check is marked by "!".
  - wmax Permissible crack width as per specification [mm]
  - ds Maximal given steel diameter [mm]
  - k Coefficient for consideration of non-linear distributed tensile stress
  - fct,eff Concrete strength at date of cracking [MN/m<sup>2</sup>]
  - kc Coefficient to consider stress distribution in tensile zone
  - acc. to Eq. (7.2) resp. Eq. (7.3)
  - Ap' Part of prestr. steel area Xil\*Ap which was used to reduce req.As
  - Xil Bond coefficient for prestressing steel as per Eq. (7.5)
  - max Sx Maximal concrete edge stress from action combination [MN/m<sup>2</sup>]
- (B) Design of reinforcement at ultimate limit state
  - In case of dominant bending, compression reinforcement is marked with "\*\*".
  - For section areas acc. to 6.1 (5) the concrete strain is not limited.
  - The minimum reinforcement acc. to 9.2.1.1 and 9.3.1.1 is not determined.
  - For compressive members the minimum reinf. acc. to 9.5.2 is considered.
  - fck Concrete strength for design of reinforcement [MN/m<sup>2</sup>]
  - N0, M0 Statically determined forces of tendons with bond [kN, kNm]
  - fp0.1k Charact. value of the 0.1% strain limit of the prestr. steel [MN/m<sup>2</sup>]
  - fpk Charact. value of the tensile strength of the prestr. steel [MN/m<sup>2</sup>]

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond  
 Steel 1; Design mode: Standard  
 (M) fctm=3.8; zs,t/b=2.025/2.025; fyk,t/b=500/500  
 (R) wmax=0.2; ds=20; kc per Eq. (7.3); k=0.65; fct,eff=3.8; Xil=0.384  
 r.sup/inf(Constr.)=1.1/0.9; r.sup/inf(Final)=1.1/0.9  
 (B) fck=45

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.958	3.950	0.521	1.2596	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,lk [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7555.99	0.00

**1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted**

Relevant concrete internal forces from 1 sets of internal forces  
 Set Nx[kN] My[kNm] Mz[kNm]  
 1 : -7555.93 -4040.19 0.00

Load case combinations for the relevant sets of internal forces  
 Set Combination  
 1 : L1+L10

**2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups  

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.69	-.-	-.-	-.-	-.-	-.-	-.-

 Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces  

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6714.14	-9384.61	0.00	109.56	-11704.67	0.00

Load case combinations for the relevant sets of internal forces  
 Set Combination  
 2 : L1+L2+0.96\*L10+L20+L3

**1. Frequent combination (TC.1): G.1+P, Construction stage ungrouted**

No set of internal forces in this situation was relevant.

**2. Frequent combination (TC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Frequent combination (TC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Relevant concrete internal forces from 4 sets of internal forces  
 Set Nx[kN] My[kNm] Mz[kNm]  
 2 : -6042.73 -9624.61 0.00 r.inf

Load case combinations for the relevant sets of internal forces  
 Set Combination  
 2 : L1+L2+0.96\*L10+L20+0.50\*L3

**1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted**

Relevant concrete internal forces from 2 sets of internal forces  
 Set Nx[kN] My[kNm] Mz[kNm]  
 2 : -7555.93 -4040.19 0.00

Load case combinations for the relevant sets of internal forces  
 Set Combination  
 2 : L1+L10

**2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups  

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.69	-.-	-.-	-.-	-.-	-.-	-.-

 Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 8 sets of internal forces  

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6714.14	-16871.48	0.00	109.56	-19191.54	0.00

Load case combinations for the relevant sets of internal forces  
 Set Combination  
 2 : 1.35\*L1+1.35\*L2+0.96\*L10+L20+1.50\*L3

## Design of longitudinal reinforcement

Charact. c.: max Sc = 1.63 < 3.80 MN/m<sup>2</sup> => no minimum crack reinf. required

Reinforcement Lay. Type	Nx [kN]	My [kNm]	Mz [kNm]	max Sc [MN/m <sup>2</sup> ]	kc	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	Situation
1 M	109.56	-11704.67	0.00	4.93	.	.	44.91	CC.3,2
R	-6042.73	-9624.61	0.00	.	.	.	25.28!	TC.3,2
B	-6714.14	-16871.48	0.00	.	.	.	18.11	PC.3,2
2 M	109.56	-11704.67	0.00	4.93	.	.	44.91	CC.3,2
R	-6042.73	-9624.61	0.00	.	.	.	25.28!	TC.3,2
B	-6714.14	-16871.48	0.00	.	.	.	18.11	PC.3,2
3 M	0.06	-6609.23	0.00	.	.	.	0.00	CC.1,1
R	0.00	0.00	0.00	0.00	.	.	0.00	-, -
B	-7555.93	-4040.19	0.00	.	.	.	0.00	PC.1,2
4 M	0.06	-6609.23	0.00	.	.	.	0.00	CC.1,1
R	0.00	0.00	0.00	0.00	.	.	0.00	-, -
B	-7555.93	-4040.19	0.00	.	.	.	0.00	PC.1,2

## Design of shear reinforcement

The percentage of nominal reinforcement acc. to Equ. (9.5N) is considered.

Ac	Section area for calculating the concrete stress from long. force [m <sup>2</sup> ]
bw	Effective width for calculation of shear stresses from Qz and Mx [m]
bw.nom	Nominal value of the width when deducting the duct diameter [m]
bn	Statically effective width for shear design using Qy [m]
kb	Factor to calculate the inner lever arm from bn
h	Effective height for calculation of shear stresses from Qy and Mx [m]
h.nom	Nominal value of the height when deducting the duct diameter [m]
d	Statically effective height for shear design using Qz [m]
kd	Factor to calculate the inner lever arm from d
Angle	Angle cot Theta between the compressive strut and the beam axis
Asl.giv.	Chargeable longitudinal reinf. acc. to Fig. 6.3 [cm <sup>2</sup> ]
rhov,min	Minimal percentage of lateral reinforcement acc. to Eq. (9.5N)
Qy, Qz	Lateral forces for design in y- and z-direction [kN]
VRdc	Absorbable lat. force without lat. reinf. per 6.2.2 (1) [kN]
VRdmax	Absorbable lateral force of comp. struts per 6.2.3 (3) [kN]
z	Inner lever arm z=kb*bn resp. z=kd*d [m]
Asb.y,z	Req. stirrup reinforcement from Qy resp. Qz [cm <sup>2</sup> /m]
Asl	Req. longitudinal reinf. acc. to Fig. 6.3 [cm <sup>2</sup> ] for req.Asb
Delta Ftd	Tensile force in long. reinf. from lateral force as per Eq. (6.18)

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond  
 bw/bw.nom/bn/kb=0.5/0.5/0.45/0.9; h/h.nom/d/kd=2.3/2.3/2.25/0.9  
 Ac=2.926; fyk=500; Asl.giv./max=0/0; rhov,min=1\*(0.08\*fck<sup>2</sup>/fyk)

## 1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted

No set of internal forces in this situation was relevant.

## 2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted

Set	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
2	-7555.93	-15774.10	0.00	0.00	0.00	-3029.62

Load case combinations for the relevant sets of internal forces

Set	Combination
2	1.35*L1+1.35*L2+L10+1.50*L3

## 3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Set	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
2	-6714.14	-16871.48	0.00	0.00	0.00	-3073.60

Load case combinations for the relevant sets of internal forces

Set	Combination
2	1.35*L1+1.35*L2+0.96*L10+L20+1.50*L3

## Check of the shear reinforcement and the compressive struts

Action	z [m]	Q/VRdc	Asb.y [cm <sup>2</sup> /m]	Asb.z [cm <sup>2</sup> /m]	Asb.T [cm <sup>2</sup> /m]	Asl.T [cm <sup>2</sup> ]	Asl [cm <sup>2</sup> ]	Situation
Qy	0.41	2.50	0.00	.	.	.	.	PC.2,2
Qz	2.02	2.50	3.95	13.96	.	.	0.00	PC.3,2

Action	z [m]	Qy/VRdmax	Qz/VRdmax	Mx/TRdmax	Q/VRd+	Delta Ftd [kN]	Situation
Qy	0.41	2.50	0.00	.	.	0.00	PC.2,2
Qz	2.02	2.50	0.55	.	.	3842.00	PC.3,2

## Check of crack widths

The check is led by direct calculation of the crack width.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

(CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

wmax	Permissible crack width as per specification [mm]
ds	Maximal given steel diameter [mm]
fct,eff	Concrete strength at date of cracking [MN/m <sup>2</sup> ]
Sigma.c	Maximal concrete edge stress in state I [MN/m <sup>2</sup> ]
wk	Calculated value of crack width as per 7.3.4 [mm]
sr,max	Calculated / given maximal crack spacing as per 7.3.4 (3) [mm]

Ac,eff Effective region of reinf. [m<sup>2</sup>] acc. to Fig. 7.1  
 As,eff Reinforcing steel within Ac,eff [cm<sup>2</sup>]  
 Ap,eff Prestressing steel with bond within Ac,eff [cm<sup>2</sup>]  
 Sigma.s Reinf. steel stress in state II [MN/m<sup>2</sup>]  
 kt Coefficient for the duration of load as per 7.3.4 (2)  
 Xil Bond coefficient for prestressing steel as per Eq. (7.5)

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond  
 wmax=0.2; ds=20; fct,eff=3.8; kt=0.4; Xil=0.384  
 r.sup/inf(Constr.)=1.1/0.9; r.sup/inf(Final)=1.1/0.9

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.958	3.950	0.521	1.2596	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7555.99	0.00

**1. Frequent combination (TC.1): G.1+P, Construction stage ungrouted**

No set of internal forces in this situation was relevant.

**2. Frequent combination (TC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Frequent combination (TC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.69	-.	-.	-.	-.	-.	-.

Stat.determ.part (P+CSR)\*r.inf: Nx0=-6141.34 kN; My0=2088.05; Mz0=0.00 kNm

Relevant values from 4 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6042.73	-9624.61	0.00	98.61	-11712.67	0.00 r.inf

Load case combinations for the relevant sets of internal forces

Set	Combination
2	L1+L2+0.96*L10+L20+0.50*L3

**Check of crack width for reinf. layer 1 (top)**

Nx	: -6042.73 kN	As,eff	: 89.82 cm <sup>2</sup>
My	: -9624.61 kNm	Ap,eff	: 0.00 cm <sup>2</sup>
Mz	: 0.00 kNm	Ac,eff	: 0.988 m <sup>2</sup>
Sigma.c	: 1.96 MN/m <sup>2</sup>	Sigma.s	: 68.27 MN/m <sup>2</sup>
Situation	: TC.3,2	sr,max	: 509.80 mm
		wk	: 0.10 wmax 0.20 mm

**Check of concrete compressive stress**

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked.

fck Characteristic compressive concrete strength [MN/m<sup>2</sup>]  
 fck(t) Average compressive strength of concrete at time t of the beginning of prestressing (Situation G+P) as per 5.10.2.2 (5) [MN/m<sup>2</sup>]  
 Sigma.x,min Total maximal longitudinal compressive stress [MN/m<sup>2</sup>]  
 Sigma.x,per = 0,60\*fck for Charact. C. (CC) as per 7.2 (2)  
 top, bottom Position of the edge point: above, below of centre

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond  
 0.6\*fck=27; 0.45\*fck(t)=20.25  
 r.sup/inf(Constr.)=1.1/0.9; r.sup/inf(Final)=1.1/0.9

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.958	3.950	0.521	1.2596	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7555.99	0.00

**1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted**

Relevant concrete internal forces from 2 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
1	-8311.52	-2805.65	0.00 r.sup

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L10

**2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.69	-.-	-.-	-.-	-.-	-.-	-.-	-.-	-.-

Stat.determ.part (P+CSR)\*r.inf: Nx0=-6141.34 kN; My0=2088.05; Mz0=0.00 kNm

Relevant values from 4 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6042.73	-10509.41	0.00	98.61	-12597.47	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	L1+L2+0.96*L10+L20+L3

Check of compressive stress in concrete for the Characteristic (rare) combination

Side	Se.- Pnt.	min Sigma.x [MN/m <sup>2</sup> ]	per. Sigma.x [MN/m <sup>2</sup> ]	Period	Situation
top	2 (I)	-1.68	-20.25	Constr.	CC.1,1
bottom	9 (I)	-16.88	-27.00	Final	CC.3,2

Check of steel stress

For the check, a cracked concrete section is assumed.  
For tendon groups without bond and/or for situations before grouting, the prestressing steel stress is checked acc. to Eq. (5.43).

Type S Long. reinf. from N and M, layer number, Charact. C. (CC)  
Type P Prestressing steel, Tendon number, Charact. C. (CC)  
N0, M0 Statically determined forces of tendons with bond [kN, kNm]  
fck Concrete strength to determine the strain state [MN/m<sup>2</sup>]  
Sigma.s,per = 0.80 \* fyk resp. 1.0 \* fyk (CK) as per 7.2 (5)  
Sigma.p,per = 0.75 \* fpk as per 7.2 (5)

Beam 16, x = 4.00 m (Beam length 4.00 m)  
Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond  
fck=45; Steel 1; 0.8\*Fyk,t/b=400/400

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.958	3.950	0.521	1.2596	9.8822	0.0000

Tendon groups with bond

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7555.99	0.00

1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted

Relevant concrete internal forces from 2 sets of internal forces  
Relevant concrete internal forces from 1 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
1	-7555.93	-4040.19	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L10

2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted

No set of internal forces in this situation was relevant.

3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.69	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6714.14	-9384.61	0.00	109.56	-11704.67	0.00

Load case combinations for the relevant sets of internal forces

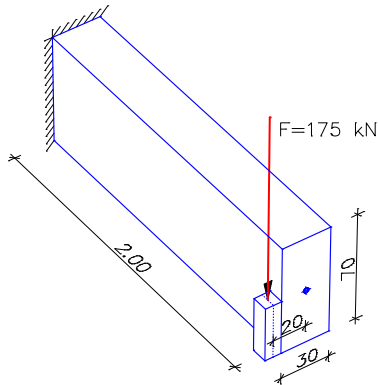
Set	Combination
2	L1+L2+0.96*L10+L20+L3

Check of steel stress

Steel Type	No.	Nx [kN]	My [kNm]	Mz [kNm]	As [cm <sup>2</sup> ]	Sigma.s [MN/m <sup>2</sup> ]	per. Sigma.s [MN/m <sup>2</sup> ]	Situation
S	1	-6714.14	-9384.61	0.00	44.91	43.29	400.00	CC.3,2
S	2	-6714.14	-9384.61	0.00	44.91	43.28	400.00	CC.3,2
S	3	-7555.93	-4040.19	0.00	0.00	.	400.00	CC.1,1
S	4	-7555.93	-4040.19	0.00	0.00	.	400.00	CC.1,1
P	1	.	.	.	72.00	1049.44	1275.00	CC.1,--

## Torsional Beam

The depicted cantilever is subjected to an eccentrically acting load  $F = 175$  kN. The required shear, torsion longitudinal and stirrup reinforcements are listed in the following log.



System drawing

### Design according to EN 1992-1-1:2014

#### Settings for flexural and shear reinforcement

M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 fyk Quality of stirrups.  
 Theta Angle of concrete truss.  
 Slabs Beams are designed like slabs.  
 Asl Given reinforcement according to picture 6.3, increase to maximum.  
 rhow Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).  
 as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).  
 Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den- sity [kg/m <sup>3</sup> ]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm <sup>2</sup> ] Pic. 6.3 given max	Red. Factor rhow as	pre- str.
1	C35/45-EN	.	.	500	1.00	.	0.00	1.00	.

#### Shear sections

bw.nom Nominal width of the prestressed section according to 6.2.3(6).  
 h.nom Nominal height of the prestressed section according to 6.2.3(6).  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.  
 z1, z2 Height and width of the core section for torsion.  
 tef Thickness of the torsion box.  
 B. Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width [m]	Eff. width bw [m]	Eff. width bn [m]	Height [m] h	Height [m] h.nom	Eff.height d [m]	Eff.height kd	Torsion section [m] z1	Torsion section [m] z2	Torsion section [m] tef	B.	
1	0.300	.	0.245	0.90	0.700	.	0.645	0.90	0.590	0.190	0.110	.

#### Design of shear reinforcement

The percentage of nominal reinforcement acc. to Eq. (9.5N) is considered.

Ac Section area for calculating the concrete stress from long. force [m<sup>2</sup>]  
 bw Effective width for calculation of shear stresses from Qz and Mx [m]  
 bn Statically effective width for shear design using Qy [m]  
 kb Factor to calculate the inner lever arm from bn  
 h Effective height for calculation of shear stresses from Qy and Mx [m]  
 d Statically effective height for shear design using Qz [m]  
 kd Factor to calculate the inner lever arm from d  
 z1, z2 Height and width of the core section Ak for torsion [m]  
 tef Wall thickness of the torsion section [m]  
 Angle cot Theta between the compressive strut and the beam axis  
 Asl.giv. Chargeable longitudinal reinf. acc. to Fig. 6.3 [cm<sup>2</sup>]  
 rhow,min Minimal percentage of lateral reinforcement acc. to Eq. (9.5N)  
 Qy, Qz Lateral forces for design in y- and z-direction [kN]  
 VRdc Absorbable lat. force without lat. reinf. per 6.2.2 (1) [kN]  
 VRdmax Absorbable lateral force of comp. struts per 6.2.3 (3) [kN]  
 z Inner lever arm z=kb\*bn resp. z=kd\*d [m]  
 Asb.y,z Req. stirrup reinforcement from Qy resp. Qz [cm<sup>2</sup>/m]  
 Asl Req. longitudinal reinf. acc. to Fig. 6.3 [cm<sup>2</sup>] for req.Asb  
 Delta FtD Tensile force in long. reinf. from lateral force as per Eq. (6.18)  
 Mx Torsional moment for design [kNm]  
 TRdmax Maximum absorbable torsional moment as per 6.3.2 (4) [kNm]  
 Asb.T Req. stirrup reinforcement from torsion [cm<sup>2</sup>/m]  
 Asl.T Req. longitudinal reinforcement from torsion [cm<sup>2</sup>]  
 fctd Design value of the tensile strength for TRd,c in Equ. (6.31) [MN/m<sup>2</sup>]

#### Location 1

Beam 1, x = 0.00 m (Beam length 2.00 m)  
 Cross-section 1: Polygon - C35/45-EN  
 Block section z1/z2=0.59/0.19; tef=0.11; fctd=1.49333

1. Permanent and temporary comb. (PC.1): G, Final state

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
Nx-	0.00	-350.00	0.00	35.00	0.00	175.00
Nx+	0.00	-350.00	0.00	35.00	0.00	175.00
My-	0.00	-472.50	0.00	47.25	0.00	236.25
My+	0.00	-350.00	0.00	35.00	0.00	175.00
Mz-	0.00	-350.00	0.00	35.00	0.00	175.00
Mz+	0.00	-350.00	0.00	35.00	0.00	175.00
Mx-	0.00	-350.00	0.00	35.00	0.00	175.00
Mx+	0.00	-472.50	0.00	47.25	0.00	236.25
Qy-	0.00	-350.00	0.00	35.00	0.00	175.00
Qy+	0.00	-350.00	0.00	35.00	0.00	175.00
Qz-	0.00	-350.00	0.00	35.00	0.00	175.00
Qz+	0.00	-472.50	0.00	47.25	0.00	236.25

Check of the shear reinforcement and the compressive struts

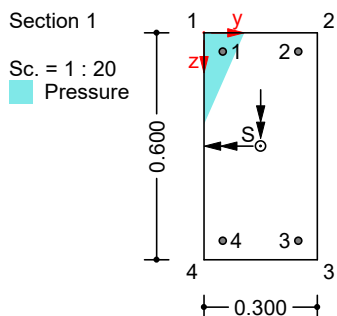
Action	z [m]	Angle	Q/VRdc	Asb.y [cm <sup>2</sup> /m]	Asb.z [cm <sup>2</sup> /m]	Asb.T [cm <sup>2</sup> /m]	Asl.T [cm <sup>2</sup> ]	Asl [cm <sup>2</sup> ]	Situation
max, cor.	0.22	1.00	0.00	.	.	.	.	.	-,-
Qy, Mx	0.22	1.00	0.00	0.00	.	4.85	7.56	0.00	PC.1,Qz+
Qz, Mx	0.58	1.00	3.04	.	9.36	4.85	7.56	0.00	PC.1,Qz+
Mx, Qz	0.58	1.00	3.04	.	9.36	4.85	7.56	0.00	PC.1,Qz+

Action	z [m]	Angle	Qy/VRdmax	Qz/VRdmax	Mx/TRdmax	Q/VRd+ Mx/TRd	Delta Ftd [kN]	Situation
max	0.22	1.00	0.00	.	.	.	0.00	-,-
Qy	0.22	1.00	0.00	.	.	.	118.13	PC.1,Qz+
Qz	0.58	1.00	.	0.23	.	.	.	PC.1,Qz+
Mx	.	1.00	.	.	0.32	.	.	PC.1,Qz+
Qy "+" Mx	0.22	1.00	0.00	.	0.32	0.32	.	PC.1,Qz+
Qz "+" Mx	0.58	1.00	.	0.23	0.32	0.54	.	PC.1,Qz+

## Single Design Reinforced Concrete

A single rectangular section is designed under bending and normal force.

### Pos. 1 - Reinforced concrete design per EN 1992-1-1: 2014



Action N = 10.00 kN; My = 67.50; Mz = 27.00 kNm  
 Resistance N = 10.00 kN; My = 67.50; Mz = 27.00 kNm  
 Force system ys / zs = 0.150 / 0.300 m  
 Strength C25/30-EN; gamma.c = 1.50; gamma.s = 1.15  
 Design mode Standard  
 Reinforcement 3.51 cm<sup>2</sup>; 0.19 %; Concrete area = 1800.00 cm<sup>2</sup>  
**Remark The concrete compression cannot be checked according to Chapter 6.1 (5). The minimum reinforcement acc. to Chapter 9.2.1.1 (1) is not included.**

Concrete section Point	y [m]	z [m]	eps[%]	sigma[MPa]	Inner Forces	y [m]	z [m]	F [kN]
1	0.000	0.000	-3.50	-16.67	Compr.	0.030	0.069	-145.43
	0.107	0.000	0.00	0.00	Tension	0.212	0.518	155.43
2	0.300	0.000	6.31	0.00	Lev. arm	0.181	0.449	
3	0.300	0.600	14.98	0.00				
4	0.000	0.600	5.18	0.00				
	0.000	0.242	0.00	0.00				

Reinforcement Point	y [m]	z [m]	d1 [m]	Es,fyk [MPa]	Zv0 [kN]	eps[%]	sigma[MPa]	As [cm <sup>2</sup> ]	
1	0.050	0.050	0.050	200000	500	0.0	-1.14	-228.48	0.00
2	0.250	0.050	0.050	200000	500	0.0	5.40	437.85	0.23
3	0.250	0.550	0.050	200000	500	0.0	12.63	444.74	2.60
4	0.050	0.550	0.050	200000	500	0.0	6.09	438.51	0.68

## Single Design Prestressed Concrete

In this example the results of the prestressed concrete design according to EN 1992-1-1 of the example *Prestressed roof construction* shall be reproduced using the single design according to EN 1992-1-1.

The values relevant for the design can be taken from the detailed listing for beam 16 at location 2 (middle column) of the example.

### Location 2

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond  
 Steel 1; Design mode: Standard  
 (B) fck=45

### Tendon groups with bond

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1500	1770	3.950	0.185	7200	82	7555.99	0.00

### 3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.69		-.		-.		-.

Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 8 sets of internal forces

Set	Concrete section			Bond section		
	Nx [kN]	My [kNm]	Mz [kNm]	Nx [kN]	My [kNm]	Mz [kNm]
2	-6714.14	-16871.48	0.00	109.56	-19191.54	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	1.35*L1+1.35*L2+0.96*L10+L20+1.50*L3

### Design of longitudinal reinforcement

Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	max Sc [MN/m <sup>2</sup> ]	kc	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	Situation
1	B	-6714.14	-16871.48	0.00	.	.	.	18.11	PC.3,2
2	B	-6714.14	-16871.48	0.00	.	.	.	18.11	PC.3,2
3	B	-7555.93	-4040.19	0.00	.	.	.	0.00	PC.1,2
4	B	-7555.93	-4040.19	0.00	.	.	.	0.00	PC.1,2

## Calculation procedure for the check program:

- The statically determined part of prestressing with creep and shrinkage ((P + CSR) · cos α · centroid distance) is subtracted from the concrete internal forces.
- From this the bond internal forces result (statically undetermined part of P + CSR with the internal forces from outer loads).
- The design is carried out with the bond internal forces. Thereby the prestressing steel together with the loss of prestressing from CSR is taken into account on the resistance side.

## Single Design according to EN 1992-1-1

- For the single design a new section with an additional steel layer at the position of the tendon is necessary. For this purpose section 1 is initially copied to get section 2.
- Subsequently the new steel layer is added to section 2. The values  $E$ ,  $f_{yk} = f_{p0,1k}$ ,  $y$ ,  $z$  and  $A_s = A_p$  for the check location can be found in the listing. For the prestressing force  $Z_{v0}$  the absolute value of the statically determined part  $N_{x0}$  from the listing is entered.

### Reinforcement of beam elements 2

	E-Modulus [MN/m <sup>2</sup> ]	fyk [MN/m <sup>2</sup> ]	y [m]	z [m]	As [cm <sup>2</sup> ]	Zv0 [kN]
1	210000	500	0.050	0.050	0.000	0.00
2	210000	500	7.850	0.050	0.000	0.00
3	210000	500	4.150	2.250	0.000	0.00
4	210000	500	3.750	2.250	0.000	0.00
5	195000	1500	3.950	0.185	72.000	6823.71

The e-modulus is used for prestressed steel layers only.

y, z coordinates of reinforcement

As Base reinforcement

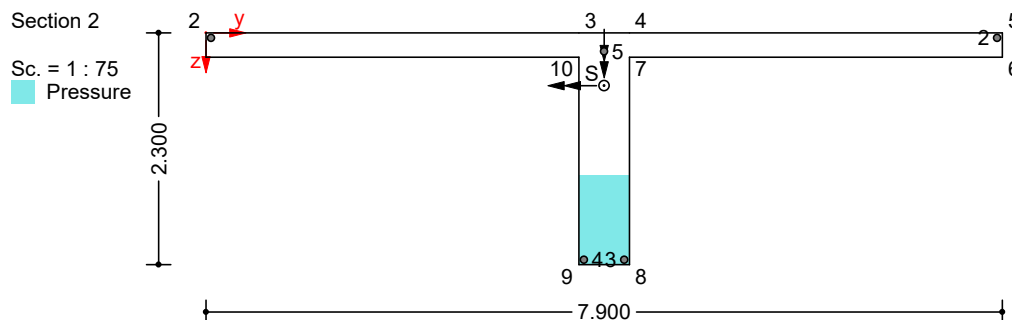
Zv0 Prestressing force of a prestressed steel layer



3. The bond internal forces with the **statically undetermined** part of P + CSR are necessary for the single design. They can also be taken from the listing.

Single design per EN 1992-1-1 [EBEM_EN1992]						
	Section	Combination	Nsd [kN]	Mysd [kNm]	Mzsd [kNm]	Mode
1	2	0	109.56	-19191.54	0.00	Standard
2						

**Pos. 1 - Reinforced concrete design per EN 1992-1-1: 2014**



Action N = 109.56 kN; My = -19191.54; Mz = 0.00 kNm  
 Resistance N = 109.56 kN; My = -19191.54; Mz = 0.00 kNm  
 Force system ys / zs = 3.950 / 0.525 m  
 Strength C45/55-EN; gamma.c = 1.50; gamma.s = 1.15; gamma.p = 1.15  
 Design mode Standard  
 Reinforcement 108.24 cm<sup>2</sup>; 0.37 %; Concrete area = 29260.00 cm<sup>2</sup>  
 Prestress P0 N0 = -6823.71 kN; My0 = 2318.83; Mz0 = -0.00 kNm  
 Resistance + P0 N+N0 = -6714.15 kN; My+My0 = -16872.71; Mz+Mz0 = -0.00 kNm

**Remark** The concrete compression cannot be checked according to Chapter 6.1 (5).  
 The minimum reinforcement acc. to Chapter 9.2.1.1 (1) is not included.

Concrete section Point	y [m]	z [m]	eps[‰]	sigma[MPa]	Inner Forces	y [m]	z [m]	F [kN]
1	-0.000	0.240	4.56	0.00	Compr.	3.950	1.928	-10868.30
2	0.000	0.000	5.49	0.00	Tension	3.950	0.165	10977.86
3	3.700	0.000	5.49	0.00	Lev. arm	0.000	1.762	
4	4.200	0.000	5.49	0.00				
5	7.900	0.000	5.49	0.00				
6	7.900	0.240	4.56	0.00				
7	4.200	0.240	4.56	0.00				
	4.200	1.405	0.00	0.00				
8	4.200	2.300	-3.50	-30.00				
9	3.700	2.300	-3.50	-30.00				
	3.700	1.405	0.00	0.00				
10	3.700	0.240	4.56	0.00				

Reinforcement Point	y [m]	z [m]	d1 [m]	Es,fyk [MPa]	Zv0 [kN]	eps[‰]	sigma[MPa]	As [cm <sup>2</sup> ]	
1	0.050	0.050	0.050	200000	500	0.0	5.30	437.76	18.12
2	7.850	0.050	0.050	200000	500	0.0	5.30	437.76	18.12
3	4.150	2.250	0.050	200000	500	0.0	-3.30	-435.86	0.00
4	3.750	2.250	0.050	200000	500	0.0	-3.30	-435.86	0.00
5	3.950	0.185	0.185	195000	1500	6823.7	4.77	1304.35	72.00

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# ENV 1992-1-1 Design

## Basics

The design according to ENV 1992-1-1 (Eurocode 2) determines the structurally required reinforced concrete reinforcement for spatially stressed polygon or area sections. The checks are carried out at the ultimate limit states for bending with longitudinal force, for lateral forces and under torsion. To take into account differences in the lateral force design, the version of the Eurocode 2

- DIN V ENV 1992-1-1: 1991 (Germany) or
- UNI ENV 1992-1-1: 1991 (Italy)

can be set (analysis, settings).

The design is carried out subsequent to the static analysis by combining the calculated load cases into logical actions which are assigned partial safety factors and combination coefficients for the various design situations. This allows the program to automatically calculate the decisive design values.

Design specifications and reinforcement data are entered when defining the section. The design only applies to elements with materials *C12/15* to *C50/60*. You can control the actions, safety factors and combination coefficients as well as the scope of the log using the *Settings* option in the *Analysis* menu.

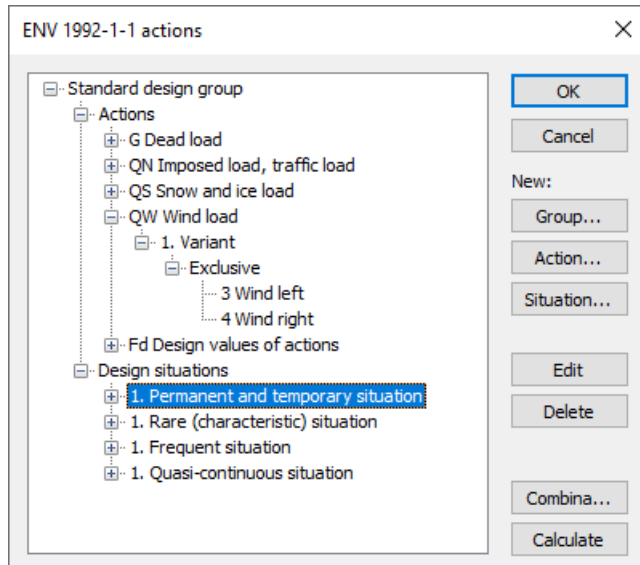
The design is normally carried out for the entire system. In the *ENV 1992-1-1 Design* folder of the database you can also perform a single design for user-defined section polygons or composite sections.

# Input

## Actions and Design Situations

The design values of the load are calculated based on the internal forces of the individual load cases and load case combinations. For this the existing load cases and load case combinations must be assigned to actions. This results in the desired design situations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH, P)
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Design values of actions (Fd)

The assigned load cases should contain a design-relevant set of loads with partial safety factors and combination coefficients (e.g. for nonlinear effects). They are combined exclusively.

### Group...

Open the dialog for entering a new design group. Optionally, particular actions and design situations can be defined for specific components (sections).

### Situation...

Open the dialog for entering new design situations.

### Edit

Open the Edit dialog for the selected action or situation.

### Delete

Delete the selected action or situation.

### Combinations...

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

### Calculate

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all combinations in the database. This allows you to evaluate the results without having to execute the checking module.

Each time you execute the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

## Definition of an Action

The illustration below shows an example of the dialog field for entering variable actions. The dialog fields for the other action types have a similar appearance.

### Label

User-defined label for the action.

### Gamma.sup, Gamma.inf

Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$

### Combination coefficients psi for:

Input fields for selecting the combination coefficients for variable actions. The button allows you to view and modify the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .

### Load cases

List of the possible load cases or load case combinations. Select items by highlighting them and clicking the button or use drag & drop.

### Multi-select

Load cases and combinations can be added to the actions more than once.

### Exclusive variants

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants by clicking the or buttons.

### Inclusive load cases

Selected load cases and combinations that can have a simultaneous effect.

### Exclusive load cases

Selected load cases and combinations that are mutually exclusive.

## Partial Safety Factors

The partial safety factors are preset to the default values according to ENV 1992-1-1. Different values can be stored for the partial safety factors of the construction materials if necessary. In version 6.12 or higher, the partial safety factors for actions are specified when defining the actions. These factors appear in the table for compatibility reasons only and therefore cannot be modified.

## Section Input

### Base Values

The section input includes all specific definitions for the bending and lateral force design. Depending on the analysis setting, the input dialog *ENV-D* (Germany) or *ENV-I* (Italy) is available according to the national edition of the ENV 1992-1-1 that was selected.

An overview of the design specifications can be accessed in the *ENV 1992-1-1 Design* section of the database.

### Design for M, N

Perform bending design for this section.

### Design for lateral force

Perform lateral force design for this section.

### Design for torsion

Perform torsional design for this section.

### Mode

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined ratios between the reinforcement layers will not be affected.
- *Compression member*: For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to ENV 1992-1-1.

### Alternative concrete

This value is necessary to perform a design according to the standard if the material type *Beton* is selected.

### Effective height

Effective static height for the lateral force design of area sections [m].

### Truss angle cot Theta

Concrete strut angle according to ENV 1992-1-1, Chapter 4.3.2.4.4 (method with variable strut angle).

### Bending reinforcement Asl according to picture 4.12

The bending tensile reinforcement to be taken into account according to Chapter 4.3.2.3, Figure 4.12 [cm<sup>2</sup>].

### Automatic extension of Asl to

You can optionally specify a maximum value for areas and the program will automatically increase the above input value until that maximum value is reached in order to avoid stirrup reinforcement [cm<sup>2</sup>].

### Quality of stirrups

- *420S*: Reinforcing rod with  $f_{yk} = 420$  MN/m<sup>2</sup>
- *500A*: Reinforcing rod with  $f_{yk} = 500$  MN/m<sup>2</sup>
- *500M*: Reinforcing mesh with  $f_{yk} = 500$  MN/m<sup>2</sup>
- *General*: Freely definable steel quality [MN/m<sup>2</sup>]

### Design like slabs

Beams are treated as slabs, meaning that, according to Chapter 4.3.2.3, no minimum lateral force reinforcement is determined.

### More than 50 % of the field reinforcement is staggered.

Switch for determining the coefficient  $k$  in Chapter 4.3.2.3, Equation (48.1).

## Shear Section

For polygon sections, additional section dimensions are required for the lateral force and torsion design according to ENV 1992-1-1. These dimensions are explained in the following. In case of sections with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.

### Width

Section width for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Height

Section height for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Effective height

Effective static height for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Effective width

Effective static width for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Core section $A_k = z_1 * z_2$

Area that is enclosed by the center line of a thin-walled hollow section (including hollow inner areas) for the calculation of the torsional reinforcement [m<sup>2</sup>].

### teff

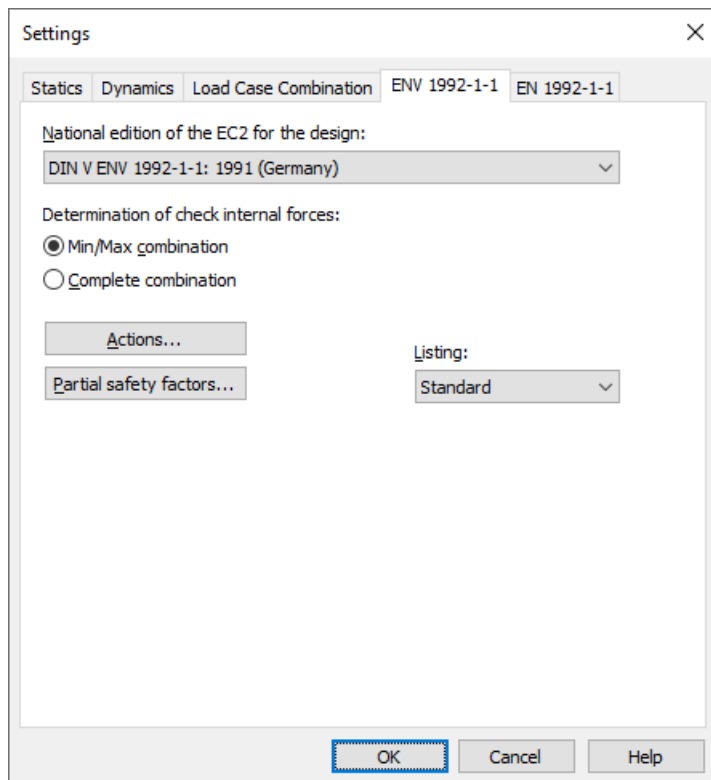
The equivalent wall thickness  $t$  of the torsion section according to Section 4.3.3.1 [m].

### Box section

Switch for determining the effectiveness factor  $\nu$  for torsion and combined load according to Section 4.3.3.1 or 4.3.3.2.

## Analysis Settings

The following dialog field can be accessed using the *Settings* option of the *Analysis* menu.





**National edition of the ENV 1992-1-1 for the design**

To take into account differences in the lateral force design, the version of the Eurocode 2

- DIN V ENV 1992-1-1: 1991 (Germany) or
- UNI ENV 1992-1-1: 1991 (Italy)

can be set.

**Actions...**

Open the dialog for describing actions.

**Determination of the check internal forces**

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account.

The differences between the two methods are explained in more detail in the section *EN 1992-1-1 Design > Check internal forces*.

**Partial safety factors...**

Open the dialog for modifying partial safety factors.

**Listing**

- *No*: No log is generated by the design program.
- *Standard*: A log with a tabular output of design internal forces and the required reinforcements is created during calculation.
- *Detailed*: This log differs from the standard log by additionally listing the internal forces of the check combination.

**Single Design**

The single design function allows you to analyze individual section polygons separately from the whole system using predefined internal forces. The following data needs to be entered in the *Single Design* table, which is located in the *ENV 1992-1-1 Design* folder of the database.

**Section**

Number of the section to be designed.

**fck**

Concrete strength according to ENV 1992-1-1 in [MPa]  $\equiv$  [MN/m<sup>2</sup>].

**Combination**

Design situation according to Table 2.3.

- *0*: Permanent and temporary design situation.
- *1*: Accidental design situation.

**Nsd, Mysd, Mzsd**

Internal forces that are designed.

### **Mode**

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to Section 5.4.1.2.
- *Strains*: Determine strain state for existing reinforcing steel layers.
- *Inactive*: Design disabled.

The calculation can be carried out while the input table is open using the *Single Design* or *Page Preview* menu item.

## Checks at the Ultimate Limit States

The checks at the ultimate limit state are carried out according to ENV 1992-1-1, Section 2.3.2.1 using a comparison of the design value of the load  $S_d$  with the corresponding design value of the resistance  $R_d$ . The requirements according to the safety concept ENV 1991 are considered to be met when

$$S_d \leq R_d \quad (1)$$

The design values are combined as follows:

- permanent and temporary design situations (fundamental combination)

$$\sum \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (2)$$

- accidental design situations

$$\sum \gamma_{GA,j} G_{k,j} + A_d + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i} \quad (3)$$

with

$G_{k,j}$	Characteristic values of the permanent actions.
$Q_{k,1}$	Characteristic value of the variable leading action (for instance, wind, snow, traffic load).
$Q_{k,i}$	Characteristic value of other variable actions (accompanying actions).
$A_d$	Design value of the accidental action.
$\gamma_{G,j}$	Partial safety factors for permanent actions $j$ .
$\gamma_{GA,j}$	Same as $\gamma_{G,j}$ , however for accidental design situations.
$\gamma_{Q,i}$	Partial safety factors for variable actions $i$ .
$\Psi_0, \Psi_1, \Psi_2$	Combination coefficients.

Information regarding the magnitude of the partial safety factors can be found in ENV 1992-1-1, Sections 2.3.3.1 and 2.3.3.2. The combination coefficients are given in the guideline for application of Eurocode 2, Deutscher Ausschuss für Stahlbeton (German Committee for Reinforced Concrete).

The design value of the resistance  $R_d$  is, along with the section geometry, dependent on the material strengths and the corresponding safety factors:

$$R_d = f(\text{section}; f_{ck} / \gamma_c; f_{yk} / \gamma_s)$$

with

$f_{ck}, f_{yk}$	Characteristic strengths of concrete and reinforcing steel.
$\gamma_c, \gamma_s$	Partial safety factors for concrete and reinforcing concrete.

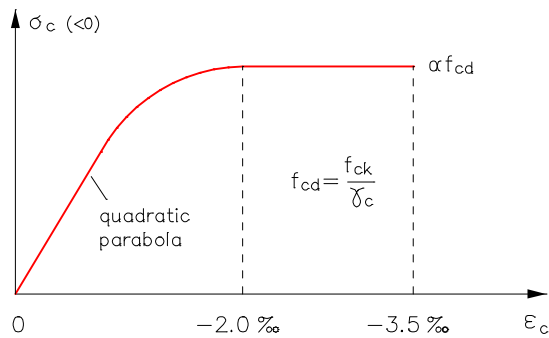
The combination of the actions according to equations (2) and (3) is carried out each for the entire internal force space. The minimum and maximum values of all internal force components and their corresponding components are thus available as results. The calculation of the design internal forces is carried out in the same way as the computation for the EN 1992-1-1. The design internal forces are determined separately for the fundamental and accidental combination. The reinforcement is calculated as the extreme value of both combinations.

The following checks are available:

- Bending with longitudinal force (chapter 4.3.1).
- Lateral force (chapter 4.3.2).
- Torsion and combined stressing (chapter 4.3.3).

## Material Properties

The design program makes use of the parabola-rectangle diagram, which represents the preferred idealization of the characteristic curve of concrete.



Stress-strain curve for concrete

The limit compression  $\epsilon_{cu}$  is  $-3.5 \text{‰}$ . The maximum value of the concrete compressive stress is determined by

$$\alpha \cdot f_{cd} = \alpha \cdot \frac{f_{ck}}{\gamma_c}$$

with

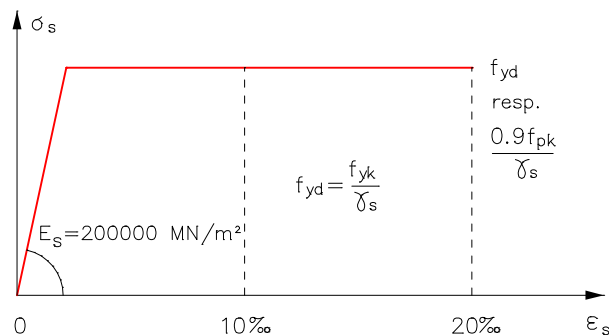
$\alpha$  Reduction factor that accounts for the long-term actions on the compressive strength as well as other unfavorable actions that originate from this type of load. A value of 0.85 is to be assumed.

$f_{ck}$  Characteristic cylinder compressive strength of the concrete (ENV 1992-1-1, Table 3.1).

$f_{cd}$  Design value of the concrete compressive stress.

$\gamma_c$  Partial safety factor for concrete (Table 2.3).

The characteristic curve for the reinforcing or prestressing steel shown in the following illustration is based on the ideal elastic-plastic method. According to ENV 1992-1-1 the steel strain is not limited in this case. The given limit strain of  $\epsilon_s = 20 \text{‰}$  is specified in the DAfStb's (German Committee of Reinforced Concrete) guideline for the application of Eurocode 2.



Stress-strain curve for reinforcing or prestressing steel

With

$f_{pk}$  Characteristic tensile strength of the prestressing steel.

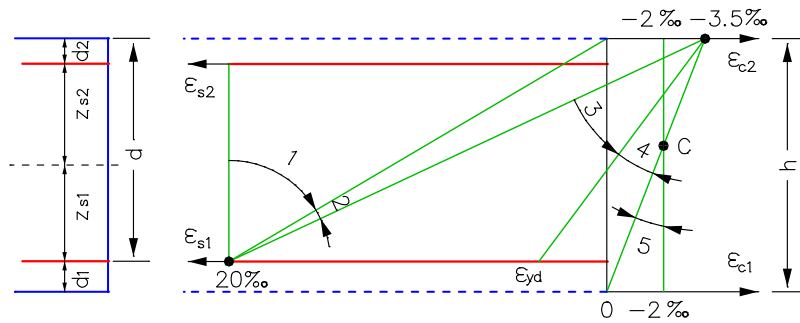
$f_{yd}$  Design value for the reinforcing steel strength at the yield strength.

$f_{yk}$  Characteristic strength of the reinforcing steel at the yield strength.

$\gamma_s$  Partial safety factor for reinforcing or prestressing steel (Table 2.3).

## Design for Bending and Longitudinal Force

The reinforcement required is determined iteratively based on the formulation of equilibrium conditions as well as the limit strain curve depicted in this illustration.



Strain areas for the design

You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes:

### Mode Standard

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tensile section to the greatest degree possible. For reasons of economy and to limit the compressive zone height according to ENV 1992-1-1, Section 2.5.3.4, the compressive reinforcement is determined in strain area 3. As a result, the following applies to the steel strain  $\epsilon_{s1}$ :

$$\epsilon_{s1} \geq \max\left(\epsilon_{yd}, \frac{3.5}{x/d} - 3.5\right) \quad [\text{‰}]$$

with

$\epsilon_{yd}$  The steel strain associated with the design value for strength  $f_{yd}$ .

$x/d$  Referenced compressive zone height.  
 $\leq 0.45$  for concrete qualities of C12/15 to C35/45.  
 $\leq 0.35$  for concrete qualities of C40/50 and higher.

The procedure in strain areas 4 and 5 is the same as with symmetrical design.

### Mode Symmetrical

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected.

### Mode Compression member

The design is performed symmetrically. Additionally, the required minimum reinforcement for columns and compression members according to Section 5.4.1.2 is determined:

$$A_{s,\min} = 0.15 N_{sd} / f_{yd} \geq |0.003| A_c$$

with

$f_{yd}$  Design value for the reinforcing steel strength at the yield strength.

$N_{sd}$  Design value of the longitudinal force to be absorbed.

$A_c$  Total area of the concrete section.

## Design for Lateral Force and Torsion

The design for lateral force and torsion includes the determination of the lateral force reinforcement and the check of the concrete struts according to ENV 1992-1-1, Chapters 4.3.2 and 4.3.3.

The following special conditions apply:

- The angle of the lateral force reinforcement is assumed to be 90°.
- The minimum reinforcement according to Section 5.4.2.2 of the standard is included in the calculated stirrup reinforcement of beams. For areas no minimum reinforcement is determined.
- For beams and design objects, the shear design is performed separately for the  $Q_y$  and  $Q_z$  lateral forces.
- Slab and shell elements are designed for the lateral force  $q_r = \sqrt{(q_x^2 + q_y^2)}$ . Depending on which has a negative effect, either the principal compressive force or principal tensile force is used for the associated longitudinal force.
- There is **no** reduction of the action from loads near supports as specified in Chapter 4.3.2.2, Section (9) through (11).
- For beam elements, the decisive values of the equivalent rectangle are determined by the user independently of the normal section geometry.
- According to Chapter 4.3.2.4.4 (2), the value  $z$  (inner lever arm) is assumed to be  $z = 0.9 d$ .
- A reduction of  $V_{rd2}$  according to equation (4.15) is **not** carried out.
- For torsion with bending and/or longitudinal force, the principal compressive stresses are **not** checked according to Chapter 4.3.3.2.2 (2).

## Serviceability Limit States

No checks are carried out for the serviceability limit state. If defined, the program does calculate, however, the design values for the following situations:

- Rare (characteristic) situations

$$\sum_{j \geq 1} G_{k,j} + P_k + Q_{k,1} + \sum_{i > 1} \psi_{0,i} \cdot Q_{k,i}$$

- Frequent situations

$$\sum_{j \geq 1} G_{k,j} + P_k + \psi_{1,1} \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i}$$

- Quasi-continuous situations

$$\sum_{j \geq 1} G_{k,j} + P_k + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i}$$

## Results

The extremal values of the internal forces, support reactions, deformations and soil pressures are saved for all design situations.

The log lists design internal forces and required reinforcement for each result location. If a compression reinforcement results for primarily bending, then this is marked with a "\*\*".

### Bending Reinforcement

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.
$a_{s\phi}$	Meridian reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.
$a_{s\psi}$	Ring reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.

### Reinforcement from lateral force

$a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area and axisymmetric shell elements.
$A_{sb,y}$	Stirrup reinforcement of beams [cm <sup>2</sup> /m] from $Q_y$ .
$A_{sb,z}$	Stirrup reinforcement of beams [cm <sup>2</sup> /m] from $Q_z$ .
$A_{sl}$ for $a_{sb}=0$	Longitudinal reinforcement [cm <sup>2</sup> ] of area elements.

### Torsion reinforcement

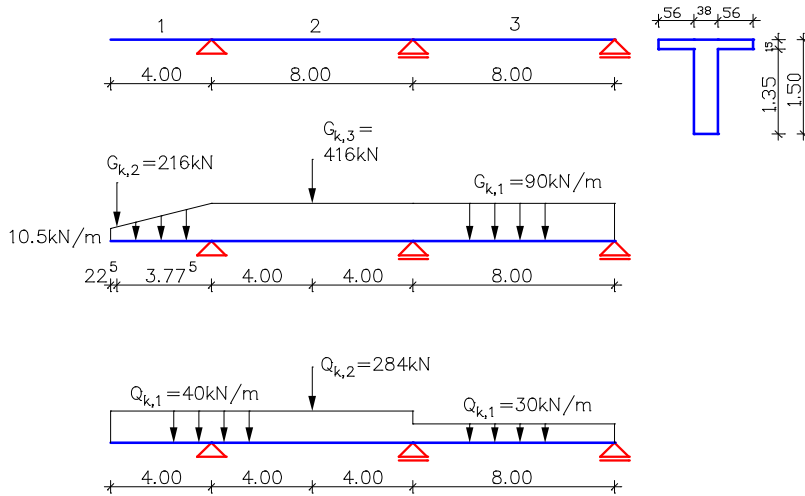
$A_{sb,T}$	Torsional stirrup reinforcement of beams [cm <sup>2</sup> /m] from $M_x$ .
$A_{sl,T}$	Torsional longitudinal reinforcement of beams [cm <sup>2</sup> ] from $M_x$ .

### Design values

$V_{Rd1}$	Absorbable design lateral force without shear reinforcement [kN].
$V_{Rd2}$	Absorbable design lateral force of the concrete struts [kN].
$T_{Rd1}$	Absorbable design torsional moment of the concrete struts [kNm].
$Q + T$	$= [\max(Q_y, Q_z) / V_{Rd2}]^2 + [M_x / T_{Rd1}]^2$ according to ENV 1992-1-1, Equation (4.47).

## Example - Two-Span Girder with Cantilever

The following example illustrates the analysis of the two-span girder with cantilever shown. The system corresponds to example 6 from 'Beispiele zur Bemessung von Betontragwerken nach EC2' (Examples of Design of Concrete Structures According to EC2), Deutscher Beton-Verein.



System and load of the reinforced concrete girder

The characteristic values of the permanent loads  $G_{k,1}$  to  $G_{k,3}$  and the variable loads  $Q_{k,1}$  and  $Q_{k,2}$  are provided in the illustration. It is important to note that the variable loads are effective independent of one another. The partial safety factors and combination coefficients used as well as the definition of the actions with respect to the seven individual load cases can be taken from the following program printout.

### Reinforced concrete design per DIN V ENV 1992-1-1: 1991 Eurocode 2

#### Partial safety factors material

	Concrete gamma.c	Reinforcing steel; gamma.s
Fundamental combination	1.50	1.15
Accidental combination	1.30	1.00

#### ENV 1992-1-1 actions

##### Standard design group

##### G - Dead load 1

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

1 Dead load Gk,1

##### G - Dead load 2

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

2 Dead load Gk,2

##### G - Dead load 3

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

3 Dead load Gk,3

##### QN - Imposed load, traffic load 1

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
Traffic loads on ceilings - Living rooms; Office rooms; Sales rooms to 50 m<sup>2</sup>;  
to 50 m<sup>2</sup>; Halls; Balconies; Rooms in hospitals  
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

Load cases 1. Variante, inclusive

4 Traffic cantilever  
5 Traffic span 1  
6 Traffic span 2



**QN - Imposed load, traffic load 2**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
 Traffic loads on ceilings - Assembly rooms; Garages and multi-storey car parks;  
 Gymnasiums; Bleachers; Halls in teaching buildings; Libraries, archives  
 Psi.0 / Psi.1 / Psi.2 = 0.8 / 0.8 / 0.5

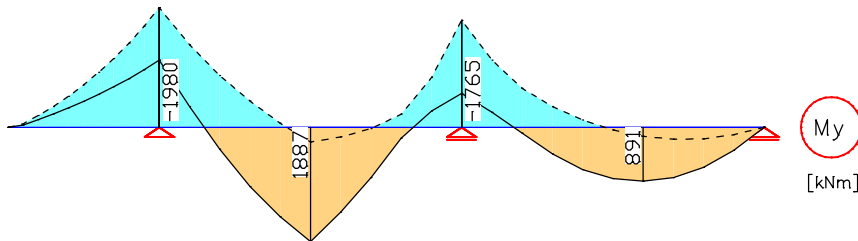
Load cases 1. Variante, inclusive

7 Traffic Qk,2

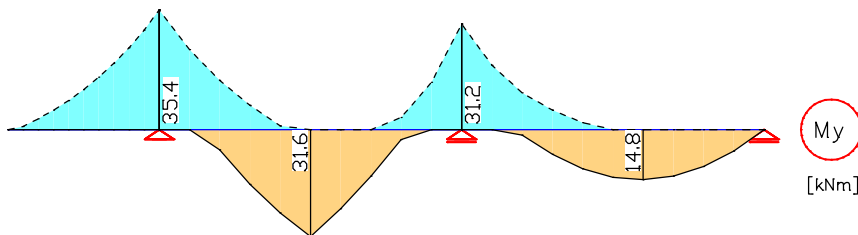
**1. Permanent and temporary situation**

- G - Dead load 1
- G - Dead load 2
- G - Dead load 3
- QN - Imposed load, traffic load 1
- QN - Imposed load, traffic load 2

Designing occurs for the extreme values of actions.



Moment boundary curves [kNm], linear internal forces calculation without redistribution



Corresponding bending reinforcement [cm²]

The above illustrations show the calculated moment boundary curves of the girder for a linear internal force calculation and the resulting  $A_s$  curves. There has been no redistribution of the column moments. The design settings and results are listed below.

**Properties for design**

fyk Quality of stirrups.  
 Theta Angle of concrete truss.  
 (!) Constant angle as per interpretation no. 24 of NABau for DIN 1045-1.  
 (\*) Theta.V as per Eq. (73), Theta.T = 45° according to 10.4.2(2).

Se.	Beton	Flexural design	fyk [MPa]	St.	Truss an.	Design	Asl	Pic.	4.12	> 50%	Asl
1	C30/37	Standard	.	.	.	.	.	.	.	.	staggered

**Section geometry and beam reinforcement**

Se.	Point	Concrete		Reinforcement						
		y [m]	z [m]	Es, fyk [MPa]	y [m]	z [m]	dl [m]	As [cm²]		
1	1	0.000	0.000	200000	500	0.030	0.090	0.030	0.00	
	2	0.560	0.000	200000	500	1.470	0.090	0.030	0.00	
	3	0.940	0.000	200000	500	0.910	1.410	0.030	0.00	
	4	1.500	0.000	200000	500	0.590	1.410	0.030	0.00	
	5	1.500	0.150							
	6	0.940	0.150							
	7	0.940	1.500							
	8	0.560	1.500							
	9	0.560	0.150							
	10	0.000	0.150							

**Beam bending reinforcement from ultimate limit state design**

Compression reinforcement is marked in the case of predominant bending through "\*\*".

Beam	Lo.	Layer	Nx [kN]	My [kNm]	Mz [kNm]	y [m]	z [m]	As [cm <sup>2</sup> ]
1	1	1	.	.	.	0.030	0.090	0.00
		2	.	.	.	1.470	0.090	0.00
		3	.	.	.	0.910	1.410	0.00
		4	.	.	.	0.590	1.410	0.00
1	11	1	0.00	-1980.39	0.00	0.030	0.090	17.69
		2	0.00	-1980.39	0.00	1.470	0.090	17.69
		3	.	.	.	0.910	1.410	0.00
		4	.	.	.	0.590	1.410	0.00
2	1	1	0.00	-1980.39	0.00	0.030	0.090	17.69
		2	0.00	-1980.39	0.00	1.470	0.090	17.69
		3	.	.	.	0.910	1.410	0.00
		4	.	.	.	0.590	1.410	0.00
2	6	1	.	.	.	0.030	0.090	0.00
		2	.	.	.	1.470	0.090	0.00
		3	0.00	1887.23	0.00	0.910	1.410	15.78
		4	0.00	1887.23	0.00	0.590	1.410	15.78
2	11	1	0.00	-1765.05	0.00	0.030	0.090	15.59
		2	0.00	-1765.05	0.00	1.470	0.090	15.59
		3	.	.	.	0.910	1.410	0.00
		4	.	.	.	0.590	1.410	0.00
3	1	1	0.00	-1765.05	0.00	0.030	0.090	15.59
		2	0.00	-1765.05	0.00	1.470	0.090	15.59
		3	.	.	.	0.910	1.410	0.00
		4	.	.	.	0.590	1.410	0.00
3	7	1	.	.	.	0.030	0.090	0.00
		2	.	.	.	1.470	0.090	0.00
		3	0.00	891.16	0.00	0.910	1.410	7.38
		4	0.00	891.16	0.00	0.590	1.410	7.38
3	11	1	.	.	.	0.030	0.090	0.00
		2	.	.	.	1.470	0.090	0.00
		3	.	.	.	0.910	1.410	0.00
		4	.	.	.	0.590	1.410	0.00

## References

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# OENORM B 4700 Design

## Basics

The reinforced concrete and prestressed concrete design as specified in OENORM B 4700, B 4750 and B 4753 is applicable for both building and bridge structures. Permitted structure models include 2D and 3D beam and area constructions. Prestressed structures can only be checked in the FEM module.

Differing components can be combined in a Structure model:

- Non-prestressed components
- Prestressed components with subsequent bond
- Prestressed components without bond
- Components with external prestressing
- Mixed-construction components

The design is carried out after the static calculation. As a part of this process, the calculated load cases are combined into logical action groups. The program will take into account the preset safety factors and combination coefficients for the defined action combinations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements.

The actions and check selection dialogs can be opened from the analysis settings. Detailed check specifications and reinforcement data must be entered during section definition.

The checks are limited to elements with materials *ÖC16/20* to *ÖC50/60*, *ÖNB40* and the general material type *ÖNBeton*.

For beams and design objects, all checks are carried out at the polygon section. For general notes on using design objects, refer to the relevant chapter in the manual.

In the *OENORM B 4700 Design* folder of the database you can also perform a single design for user-defined section polygons or composite sections.

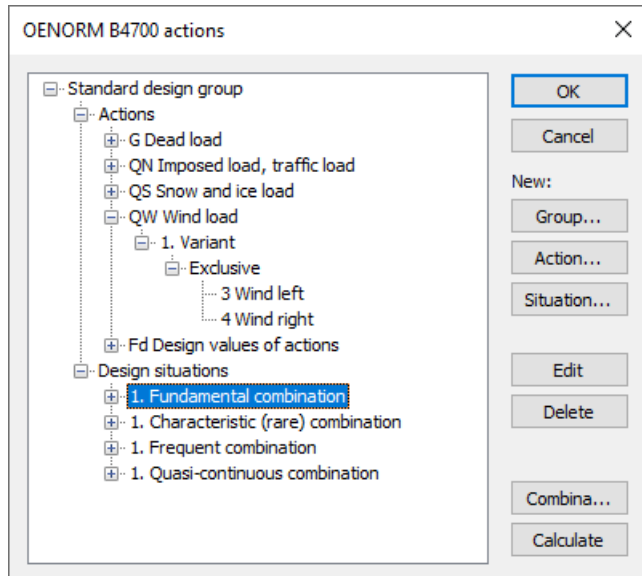
The following paragraphs primarily cite the guidelines of standards B 4700 and B 4750. Reference to OENROM B 4753 is only made if it contains a different or complementary rule.

# Input

## Actions and Design Situations

The design values of the load are calculated based on the internal forces of individual load cases and load case combinations. For this the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired action combinations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Prestressing (P)
- Creep and shrinkage, relaxation (CSR1, CSR2)  
These actions are only available if a P action has been defined. In the combinations they are treated, along with P, as a single action.
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Design values of actions (Fd)  
The assigned load cases should contain a design-relevant set of loads with partial safety factors and combination coefficients such as for example a load group to take into account nonlinear effects. The selected load cases are combined exclusively.

### Group...

Open the dialog for entering a new design group. According to e.g. standard OENORM B 4015, Table 7, certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

### Situation...

Open the dialog for entering new action combinations. The combination must be classified as either a construction stage or a final state before starting any corresponding checks. For prestressed concrete structures with subsequent bond, you can specify that the tendons are still ungrouted.

### Edit

Open the Edit dialog for the selected action or combination.

### Delete

Delete the selected action or combination.

### Combinations...

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

## Calculate

Calculate the defined action combinations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all combinations in the database. This allows you to evaluate the results without having to execute the checking module. Each time you execute the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

The following table demonstrates how the action combinations are used in the various checks. The numbers refer to the chapters of the standards.

Situation	Ultimate limit state	Serviceability limit state	B4700	B4750	B4753
Fundamental, Accidental	Longitudinal reinf.		3.4.2	12.2	12.2
	Lateral reinf.		3.4.4	12.3	12.3
	Torsional reinf.		3.4.6	12.4	12.4
Characteristic (rare)		Concrete compr. stress		11.4.5(1)	11.4.2(1)
		Reinforcing steel stress		11.4.5(3)	11.4.2(7)
		Prestressing steel stress		11.4.5(4)	11.4.2(4)
		Decompression Class A		11.3	11.3
		Crack width Class B		11.5	11.5
Frequent	Robustness reinf.	Crack width Class C		12.5	12.5
		Decompression Class B		11.3	11.3
		Crack width Class D		11.5	11.5
		Crack width Class C		11.5	11.5
Quasi-continuous		Concrete compr. stress		11.4.5(2)	11.4.2(2)
		Prestressing steel stress		11.4.5(4)	11.4.2(5)
		Decompression Class C		11.3	11.3
		Crack width Class E	4.2	11.5	11.5
		Deformations	4.3	11.6	11.6
Fatigue	Concrete				13.2
	Reinforcing steel				13.3-4
	Prestressing steel				13.3

## Definition of an Action

The illustration below shows an example of the dialog field for entering variable actions. The dialog fields for the other action types have a similar appearance.


### Label

User-defined label for the action.


### Gamma.sup, Gamma.inf

Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$ .

**Combination coefficients psi for:**

Input fields for selecting the combination coefficients for variable actions. The  button allows you to view and modify the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ . Depending on the selection, the values according to B 4750, table 5, or B 4753, table 4, will apply.



**Load cases**

List of possible load cases or load case combinations. You can choose an item from the list by selecting it and then clicking the  button or by using drag & drop.

**Multi-select**

Load cases and combinations can be added to the actions more than once.

**Exclusive variants**

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants by clicking the  or  buttons.

**Inclusive load cases**

Selected load cases and combinations that can have a simultaneous effect.

**Exclusive load cases**

Selected load cases and combinations that exclude each other.

**Prestressing loss from relaxation of prestressing steel**

The prestressing loss is defined as a constant percentage reduction of prestress.

**CS as constant reduction of prestress**

As an alternative to defining CS load cases, you can allow for the effect of creep and shrinkage by defining a constant percentage reduction of prestress.

**Internal prestressing**

Selected load cases that describe internal prestressing. The reactions of the individual load cases are added together.

**External prestressing**

Selected load cases that describe external prestressing. The reactions of the individual load cases are added together.

## Partial Safety Factors

The partial safety factors of the construction materials are preset with the default values specified by OENORM B 4700, Tables 1 and 2, and can be modified if necessary. In version 6.12 or higher, the partial safety factors for actions are specified when defining the actions. These factors appear in the Partial Safety Factors dialog for compatibility reasons only and therefore cannot be modified.

## Section Input

The section inputs contain all of the specific settings made for checks in the ultimate limit and serviceability states. In addition to these specifications, the selected material properties and the properties of the reinforcing steel are also relevant for the design. An overview of the design settings can be accessed in the *OENORM B 4700 Design* section of the database.

## Checks

The following dialog is used to define which ultimate limit state and serviceability checks are available for the section. The analysis settings allow to override this selection for the entire structure.

### Prestress of component

The type of prestressing can be selected for each section separately:

- *not prestressed*
- *subsequent bond*
- *without bond*
- *external*
- *mixed construction*

### Requirement class

The check conditions for the decompression and crack width check are defined in OENORM B 4750, Chapter 3.7.3, Table 1, based on requirement classes A through F. OENORM B 4700 makes no reference to requirement classes, but instead defines the serviceability checks for the quasi-continuous combination. This check corresponds to a requirement class E classification.

## Base Values

The base values apply for ultimate limit state and serviceability checks.

The screenshot shows a software dialog box titled "Properties for element 2 - OENORM B 4700 - Base values". On the left is a tree view with categories like Section, Material, Bedding, and OENORM B 4700. The main area contains several input fields and dropdown menus:

- Number:** 1
- Section Type:** Polygon
- Material Type:** ÖNB40
- Label:** Roof girder
- Design mode for bend and longitudinal force:** Standard
- Eff. height [m]:** (empty field)
- Truss angle tan  $\beta$ :** 1
- Alternative concrete:** Null
- Quality of stirrups:** General, 550
- Chargeable As according to picture 23 [cm<sup>2</sup>]:** (empty field)
- 50 % of the biggest bottom reinforcement is unbroken up to support
- Design like slabs

Buttons at the bottom include OK, Cancel, and Help. A warning icon and text "Properties concern further elements." are also present.

### Design mode

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined ratios between the reinforcement layers will not be affected.
- *Compression member*: For compression members, a symmetrical design is carried out with allowance for the minimum reinforcement according to OENORM B 4700, Chapter 3.4.9.2, Eq. (60).

### Alternative concrete

This value is necessary to perform a design according to the standard if the material type *Beton* or *ÖNBeton* are selected.

### Effective height

Effective static height for the shear design of area elements [m].

### Truss angle tan $\beta$

Concrete strut angle according to OENORM B 4700, Chapter 3.4.4.2 (7).

### 50 % of the biggest bottom reinforcement is unbroken up to support

Setting for components without diagonal tensile reinforcement (e.g., slabs) according to B 4700, Chap. 3.4.4.4.

### Chargeable bending reinforcement according to picture 23

Bending reinforcement to be added according to OENORM B 4700, Figure 23 [cm<sup>2</sup>].

### Quality of stirrups

- *420S*: Reinforcing steel BSt 420 according to OENORM B 4200-7 and B 4700, Tab. 5
- *500A*: Reinforcing steel BSt 500 according to OENORM B 4200-7 and B 4700, Tab. 5
- *500M*: Reinforcing steel M 500 according to OENORM B 4200-7 and B 4700, Tab. 5
- *General*: User-definable steel quality [MN/m<sup>2</sup>].

### Design like slabs

Beams or design objects are treated like slabs.



## Shear Section

For polygon sections, additional section dimensions are required for the lateral force and torsion design according to OENORM. These dimensions are explained in the following. In case of sections with internal prestressing or with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.

Properties for element 2 - OENORM B 4700 - Shear section

Number: 1 - Roc Section Type: Polygon Material Type: ÖNB40

Label: Roof girder

Height [m]: 1.5

Eff. width [m]: 0.3

Width [m]: 0.38

Nom. width [m]:

def [m]: 0.063

Core section  $A_k = b_k * d_k$

$d_k$  [m]: 1.437  $b_k$  [m]: 0.317

Box section:

Qy, Qz, Mx

OK, Cancel, Help

### Width

Section width for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Height

Section height for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Effective height

Effective static height for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Effective width

Effective static width for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Nom. width, nom. height

The nominal width or height of internally prestressed components as per OENORM B 4750, Chapter 12.3.4 (4) for including the duct diameter in the calculation of the lateral load-bearing capacity.

### Core section $A_k = d_k * b_k$

Dimensions of the core section for calculating the torsion reinforcement [m].

### def

The effective wall thickness of the torsion section according to OENORM B 4700, Figure 29 [m].

### Box section

Setting for checking the principal compressive stress according to OENORM B 4750, Chapter 12.4 (3).

## Stresses

Properties for element 2 - OENORM B 4700 - Stresses

Section: 1 - Roc, Section Type: Polygon, Material Type: ÖNB40

Label: Roof girder

Concrete compressive stresses

Rare comb.	Quasi-continuous c.
perm. sigma.c: <input checked="" type="radio"/> 0.60 fck	perm. sigma.c: <input type="radio"/> 0.45 fck
<input type="radio"/> 0.55 fck	

Reinforcing steel stresses

Rare comb.
perm sigma.g: <input checked="" type="radio"/> 0.8 fyk
<input type="radio"/> 0.7 fyk

Check of decompression

Stress: [dropdown], Conc. tensile strength in constr. stages: 1 \* fctk

Buttons: OK, Cancel, Help

### Concrete compressive stresses

The concrete compressive stress  $\sigma_c$  must be limited to  $0.60 \cdot f_{ck}$  under the rare (characteristic) action combination according to OENORM B 4750, Chapter 11.4.5 (1). According to OENORM B 4753, Eq. (12), for uniaxial bending  $\sigma_c$  must be limited to  $0.55 \cdot f_{ck}$ . To prevent disproportionate creep deformations, you should also maintain the limit  $0.45 \cdot f_{ck}$  under the quasi-continuous action combination as per B 4750, Chapter 11.4.5 (2).

### Reinforcing steel stresses

The concrete steel stress  $\sigma_s$  must be limited to  $0.80 \cdot f_{yk}$  under the rare (characteristic) action combination as per OENORM B 4750, Chapter 11.4.5 (3). A limitation to  $0.70 \cdot f_{yk}$  is required according to Eq. (18) of OENORM B 4753.

### Decompression, Stress

Decisive stress for the decompression check for area elements ( $\sigma_1, \sigma_x, \sigma_y$ ).

### Decompression, Concrete tensile strength

Decisive tensile strength for the check of concrete tensile stresses during construction stages according to OENORM B 4750, Chapter 3.7.5, and B 4753, Chapter 11.4.2 (3).

## Crack Width

These specifications apply to the minimum crack reinforcement calculation and the crack width check through beam diameter limitation.

### **wk**

Calculation value of the crack width according to OENORM B 4700, Tables 8a and 8b or 9 and 10 [mm]. If this value deviates from the default values 0.15 mm or 0.30 mm, the program interpolates between the tables.

### **max. ds**

Largest existing bar diameter of the reinforcing steel reinforcement according to OENORM B 4700, Chapter 4.2.3 (3) [mm].

### **Action combinations acc. to B 4753**

For railway bridges the action combinations according to OENORM B 4753, Table 1, are decisive. It provides the use of the rare combination for requirement class C instead of the frequent combination as described in B 4750.

### **Determination of the concrete tensile zone**

You can specify the tensile section where the minimum crack reinforcement as per OENORM B 4700, Chapter 4.2.2 will be placed by selecting either an action combination (AC) or a restraint (bending, central tension).

### **Coefficient $k_p$**

The bond coefficient  $k_p$  according to OENORM B 4750, Chapter 11.5 (6), defines the extent to which prestressing steel as per Chapter 11.5 (5) can be taken into account for the minimum crack reinforcement. Data input is blocked for area elements since prestressing steel is normally not taken into account here.

### **Reduction coeff.**

This factor allows you to group all of the reductions of reinforcement level  $\rho_t$  according to OENORM B 4700, Chapter 4.2.2, Sections (5) through (7).

## Fatigue

Properties for element 2 - OENORM B 4700 - Fatigue

Section: Form, Shear stresses, Material, Bedding, OENORM B 4700, Checks, Base values, Shear section, Stresses, Crack width, **Fatigue**, Thermal analysis, General, Eccentricity

Number: 1 - Roc, Section Type: Polygon, Material Type: ÖNB40

Label: Roof girder

Permissible characteristic stress fatigue ranges

Reinforcement	Prestressed steel
dSigma.Rsk,s: 220 MN/m <sup>2</sup>	dSigma.Rsk,p: 110 MN/m <sup>2</sup>

Limit design variants

OK, Cancel, Help

### **dSigma.Rsk,s**

Permissible characteristic stress range  $\Delta\sigma_{Rsk}$  of the longitudinal reinforcement and shear reinforcement according to OENORM B 4753, Chapter 13.3 / 13.4 [MN/m<sup>2</sup>]. The value specified in Chapter 13.3 (3) is suggested in the dialog.

### **dSigma.Rsk,p**

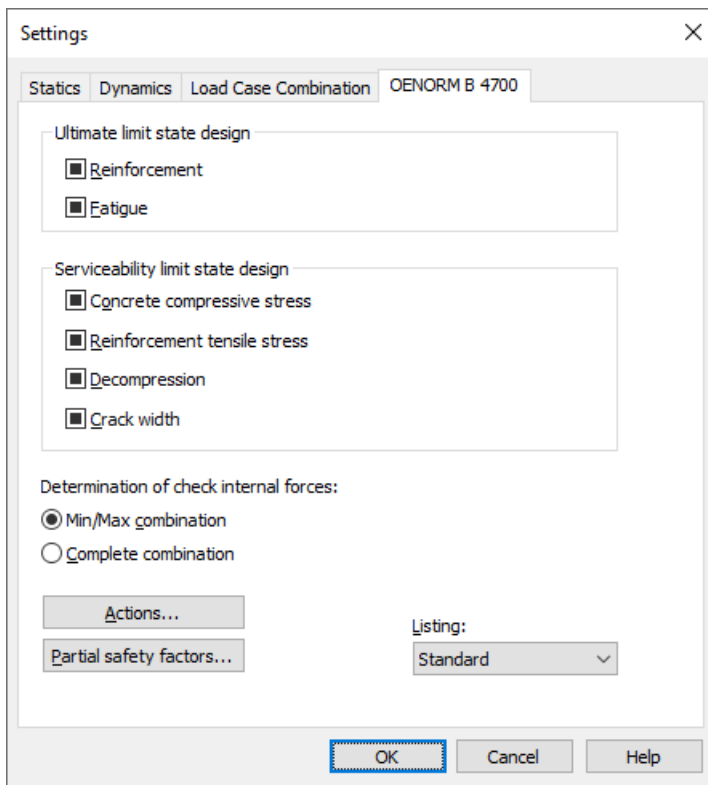
Permissible characteristic stress range  $\Delta\sigma_{Rsk}$  of the prestressing steel according to OENORM B 4753, Chapter 13.3 (4) [MN/m<sup>2</sup>]. The value for tendons in steel ducts is suggested in the dialog.

### **Limit design variants**

For area elements, the variants for determining the stress range can be limited to the corresponding sets of design internal forces. For more information see chapter '*Check Against Fatigue > Special Characteristic of Shell Structures*'.

## Analysis Settings

The *OENORM B 4700* dialog page can be opened using the *Settings* function of the *Analysis* menu.



### Check selection

When selecting checks, the following cases are to be distinguished:

- or  The check is performed according to the settings in the section dialog (see *Section inputs*).
- or  The check is performed for all sections of the structure.
- or  The check is performed for no sections of the structure.

Corresponding section settings are bundled as follows:

Reinforcement	Bend and longitudinal force Lateral force Torsion Robustness
Crack width	Minimum crack reinforcement Crack width

An overview of the checks can be accessed using the *Design Settings* function in the *OENORM B 4700 Design* folder of the database.

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *EN 1992-1-1 Design > Check internal forces*.

### Actions...

Open the dialog for describing actions.

**Partial safety factors...**

Open the dialog for modifying partial safety factors.

**Listing**

- *No*: No log is generated by the checking program.
- *Standard*: Log with tabular output of results.
- *Detailed*: Additional output of the decisive combination internal forces at the check locations.
- *Standard > permissible*: Standard log limited to check locations where the permissible limit values are exceeded.
- *Detailed > permissible*: Detailed log limited to check locations where the permissible limit values are exceeded.

## Single Design

The single design function allows you to analyze individual section polygons separately from the whole system using predefined internal forces. The calculation is carried out from the opened input table via the *Single Design* item in the *Analysis* menu or the *Print Preview* function.

The following data needs to be entered in the *Single Design* table, which is located in the *OENORM B 4700 Design* folder of the database.

**Section**

Number of the section to be designed.

**fck**

Concrete strength according to OENORM B 4700, Table 4 [MN/m<sup>2</sup>].

**Combination**

Combination according to OENORM B 4700, Table 1.

- *0*: Fundamental combination
- *1*: Accidental combination

**Nsd, Mysd, Mzsd**

Longitudinal design force [kN], bending design moments around the y and z axes [kNm].

**Mode**

- *Standard*: Standard design for bending with longitudinal force.
- *Symmetrical*: Design for symmetrical reinforcement.
- *Compression member*: Compression member design.
- *Inactive*: Design disabled.
- *Strains*: Determine the strain state for existing reinforcing steel layers.
- *Strains SLS*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A linear strain-stress curve of the concrete is used in the compression zone to determine the strain state.

## Punching Shear Check

When you select a check node, the key data for the checks is displayed in a dialog field. This field is divided into four sections.

### 1. Column

The column forms *Rectangle*, *Round* and *General* with the viewpoints *Intern*, *Edge in x-direction*, *Edge in y-direction* and *Corner* are available. When you enter a new column, the program will suggest the dimensions of existing columns. The edge distances  $a_x$  and  $a_y$  are used to calculate the circumference  $u$  of the critical perimeter. When selecting the column form *General*, you must enter the circumference  $u$ .

### 2. Slab

This section shows the material properties, the existing bending reinforcement for calculating the reinforcement level  $\rho$ , and additional coefficients for calculating the punching resistances.

### 3. Action

You can either add a support force from a previous design according to OENORM or enter an action directly. All bedding forces reduce the design value of the lateral force.

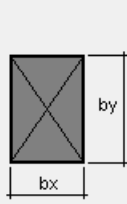
### 4. Results

This section shows the calculated punching resistances, the necessary punching reinforcement (if applicable) and the minimum bending reinforcement. You can call up an improved bending reinforcement by clicking the *Proposal...* button.

### Example

OENORM B 4700 Punching Shear Check Node 1659

Column:  
 Rectangle, inside  
 Width  $b_x$ : 0.24 m  
 Height  $b_y$ : 0.24 m  
 m  
 m



Slab  
 Steel: BSt 550  $a_{sx}$ : 6 cm<sup>2</sup>/m Eff. height: 0.17 m  
 Concr.: C20/25  $a_{sy}$ : 6 cm<sup>2</sup>/m Eff. height: 0.17 m  
 $\kappa_s$ : 0.5 Span length  $l$ : 4 m  
 $\kappa_e$ : 1.15 Truss angle  $\alpha$ : 90 °

Action  
 V<sub>Sd</sub> from calculation  
 Enter V<sub>Sd</sub>

	Fundamental combination	V <sub>Sd</sub> [kN]	Bedding force [kN]
		208.976	0
	Accidental combination	0	0

Results of the fundamental combination

V <sub>Sd,max</sub>	VR <sub>dc</sub> [kN]	VR <sub>ds</sub> [kN]	
240.32	269.08		✓
$a_{sx,min}$	$a_{sy,min}$	As <sub>v</sub> [cm <sup>2</sup> ]	
4.11	4.11		Proposal...

Design OK Cancel

**Punching shear check node 1659**

Check per OENORM B 4700 edition: 2001-06-01, section 3.4.5.

**1. Measurements, situation and material**

Rectangular column with width  $b_x = 0.24$  m and height  $b_y = 0.24$  m  
 Situation: Inside column;  $\kappa_e = 1.15$   
 Critical perimeter  $u = 2.56$  m

Effective height of the slab  $d_x = 0.170$  m;  $d_y = 0.170$  m  
 Available longitudinal reinforcement  $a_{sx} = 6.00$  cm<sup>2</sup>/m;  $a_{sy} = 6.00$  cm<sup>2</sup>/m  
 Truss angle  $\alpha = 90.0^\circ$ ; Span length  $l = 4.00$  m;  $\kappa_s = 0.50$

Concrete: ÖC20/25;  $\gamma_c = 1.50$

Reinforcement: BSt 550;  $\gamma_s = 1.15$

**2. Action**

$V_{Sd} = 208.98$  kN from fundamental combination

$V_{Sd,max} = 240.32$  kN

**3. Punching resistance without punching reinforcement**

$$V_{Rdc} = 1,2 \cdot \tau_d \cdot \kappa_c \cdot \left(1,2 + 2000 \cdot \frac{1}{l/d} \cdot \rho\right) \cdot u \cdot d$$

with	$\tau_d = 0.24$ N/mm <sup>2</sup>	$\kappa_c = 1.43$
	$l/d = 23.53$	$\rho = 0.0035$
	$d = 0.170$ m	$u = 2.56$ m

$$V_{Rdc} = 269.08 \text{ kN}$$

**4. Check**

$$V_{Sd,max} \leq V_{Rdc} \quad \text{The check is OK!}$$

**5. Minimum longitudinal reinforcement after section 3.4.5.3 (4)**

$$a_{s_{x,min}} = \frac{V_{Sd,max}}{0,9 \cdot d_x \cdot f_{yd}} \cdot e_x / b_{ef} \quad a_{s_{y,min}} = \frac{V_{Sd,max}}{0,9 \cdot d_y \cdot f_{yd}} \cdot e_y / b_{ef}$$

with	$f_{yd} = 478.26$ N/mm <sup>2</sup>	$e_x/b_{ef} = 0.125$
	$d = 0.170$ m	$e_y/b_{ef} = 0.125$

$$a_{s_{x,min}} = 4.11 \text{ cm}^2/\text{m} \leq a_{sx} \quad a_{s_{y,min}} = 4.11 \text{ cm}^2/\text{m} \leq a_{sy}$$



# Prestressed Structures

## Internal Prestressing

For internal prestressing, the tendon groups as well as the prestressing system and procedures are entered using the *Prestressing* function of the *Structure* menu. To include them in the FEM calculation, you then need to define a load case with the *Prestressing* load type. For more information, refer to the *Prestressed Concrete* section of the manual.

Prestressing with bond and prestressing without bond are differentiated in the section inputs and the specifications for the *Creep and shrinkage* load case. For prestressed components with subsequent bond the tendons can be set ungrouted for the respective design situation in the action dialog.

## External Prestressing, Mixed Construction

External prestressing can be taken into account by entering the external forces directly in the program. For mixed construction, the additional tendons with bond must be entered as described above.

## Variation of Prestressing

For checks in the ultimate limit state, the following applies for the design value of the prestressing force according to OENORM B 4750, Chapter 12.1 (3):

$$P_d = \gamma_p \cdot P_{m,t}$$

with

$P_{m,t}$  Mean value of prestressing force at time  $t$  including prestressing losses from friction, slippage, creep, shrinkage and relaxation.

$\gamma_p$  Partial safety factor of the prestressing force;  $\gamma_p = 1$  is normally assumed.

In the serviceability limit state, two characteristic values for the prestressing force are defined in OENORM B 4750, Chapter 11.2 (3):

$$P_{k,sup} = r_{sup} \cdot P_{m,t} \quad \text{Upper characteristic value acc. to Eq. (22).}$$

$$P_{k,inf} = r_{inf} \cdot P_{m,t} \quad \text{Lower characteristic value acc. to Eq. (23).}$$

According to the standard, the coefficients  $r_{sup}$  and  $r_{inf}$  equal  $r_{sup} = 1.05$  and  $r_{inf} = 0.95$  for tendons with subsequent bond, and for tendons with immediate bond or without bond as well as for effects from external prestressing  $r_{sup} = r_{inf} = 1.0$ .

Chapter 11.2 (3) of the standards B 4750 and B 4753 specifies for which checks variation is to be taken into account:

- Minimum crack reinforcement and crack width check (B 4753)
- Decompression check and concrete compressive stress check (B 4750 and B 4753)
- Reinforcing steel and prestressing steel stress check (B 4750)

The program takes variation into account in all of the above checks.

## Creep and Shrinkage

Similar to prestressing, creep and shrinkage can be taken into account by defining the appropriate load case (*Creep and shrinkage* load type) in the FEM calculation. Besides the creep-generating permanent load case, you also need to specify whether the internal forces relocation between concrete and prestressing steel is to be taken into account. This option is only useful in the case of tendons with bond.

The decisive creep and shrinkage coefficients for calculating the *Creep and shrinkage* load case are entered in the Section dialog. Alternatively, this dialog can be used to calculate the coefficient as per OENORM B 4750, Chapter 8.2 and 8.3.

The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1+\rho \cdot \varphi} (\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,s})$$

With

- $\sigma_b(t)$  Concrete stress from creep and shrinkage at time  $t$ .
- $E_b$  E-modulus of the concrete.
- $\rho$  Relaxation coefficient according to Trost for time  $t$  (normally  $\rho = 0.80$ ).
- $\varphi$  Creep coefficient for time  $t$ .
- $\varepsilon_b(t)$  Concrete strain from creep and shrinkage at time  $t$ .
- $\varepsilon_{b,0}$  Concrete strain from creep-generating continuous load.
- $\varepsilon_{b,s}$  Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, yield the internal forces and deformations of the concrete. The resulting stress changes in the prestressing steel are also determined provided they are selected in the load case. Any influence from the relaxation of the prestressing steel will be ignored in this case. According to Zilch/Rogge (2002, p. 256), this influence can be calculated separately (see following section) and combined with the changes from creep and shrinkage for all time-dependent prestressing losses:

$$\Delta\sigma_{p,csr} = \Delta\sigma_{pr} + E_p \cdot \Delta\varepsilon_{cpt}$$

with

- $\Delta\sigma_{pr}$  Prestressing loss from relaxation of the prestressing steel.
- $\Delta\varepsilon_{cpt}$  Concrete strain change from creep and shrinkage.
- $E_p$  E-modulus of the prestressing steel.

## Relaxation of Prestressing Steel

According to OENORM B 4750, Chapter 8.4, the stress change  $\Delta\sigma_{pr}$  in the tendons at position  $x$  caused by relaxation must be taken into account in addition to the stress loss from concrete creep and shrinkage. Unless specified otherwise by the manufacturer, this change may be taken from Figure 10 with an initial stress of

$$\sigma_p = \sigma_{pg0} - 0.3 \Delta\sigma_{p,csr} \quad (11)$$

with

- $\Delta\sigma_{p,csr}$  Stress change in the tendons due to creep, shrinkage and relaxation at position  $x$  at time  $t$ .
- $\sigma_{pg0}$  Initial stress in the tendons from prestressing and permanent actions.

Since the entire stress loss cannot be known in advance, the input value  $\Delta\sigma_{p,csr}$  must be estimated for Eq. (11) according to the standard, e.g.,  $\sigma_p = 0.95 \cdot \sigma_{pg0}$ .

You can define the stress losses in the CSR actions of the *OENORM B 4700 Actions* dialog.

## Checks in the Ultimate Limit States

The following checks according to OENORM are available:

- Bending with longitudinal force (B 4700, Chap. 3.4.2, and B 4750, Chap. 12.2).
- Minimum reinforcement against failure without warning (B 4750, Chap. 12.5).
- Lateral force (B 4700, Chap. 3.4.4, and B 4750, Chap. 12.3).
- Torsion and combined strain (B 4700, Chap. 3.4.6, and B 4750, Chap. 12.4).
- Punching shear (B 4700, Chap. 3.4.5).
- Fatigue check (B 4753, Chap. 13).

## Design Combinations

The decisive action combinations are calculated using the following formulas from OENORM B 4700, Chapter 3.2.2, in conjunction with B 4750, Chapter 12.1:

- Fundamental combination

$$L_d = \sum_j (\gamma_{G,j} \cdot G_{k,j}) + (\gamma_P \cdot P_k) + 1.50 \cdot Q_{k,i} \quad \text{for } i = 1 \quad (4)$$

$$L_d = \sum_j (\gamma_{G,j} \cdot G_{k,j}) + (\gamma_P \cdot P_k) + 1.35 \sum_{i=1}^n Q_{k,i} \quad \text{for } n > 1 \quad (5)$$

- Accidental combination

$$L_d = \sum_j G_{k,j} + P_k + \sum_i (\psi_{2,i} \cdot Q_{k,i}) + F_{\text{ex}} \quad (6)$$

According to B 4753, Chapter 13.1, the fatigue check uses a special

- Fatigue combination

$$L_k = \sum_j G_{k,j} + P_k + (\psi_{1,1} \cdot Q_{k,1}) + \sum_{i>1} (\psi_{2,i} \cdot Q_{k,i})$$

which is based on the frequent action combination in the serviceability state according to B 4750, Eq. (20).

The individual parts have the following meaning:

$\gamma_{G,j}, \gamma_P$	Partial safety factor for permanent actions and prestressing.
$G_{k,j}, P_k$	Characteristic values of the permanent actions and prestressing.
$Q_{k,i}$	Characteristic values of the variable actions.
$n$	Number of variable actions.
$\psi_{2,i}$	Combination coefficients of B 4750, Table 5, or B 4753, Table 4.
$F_{\text{ex}}$	Accidental action.

Different variants in the construction stages and final states can be defined for each combination. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

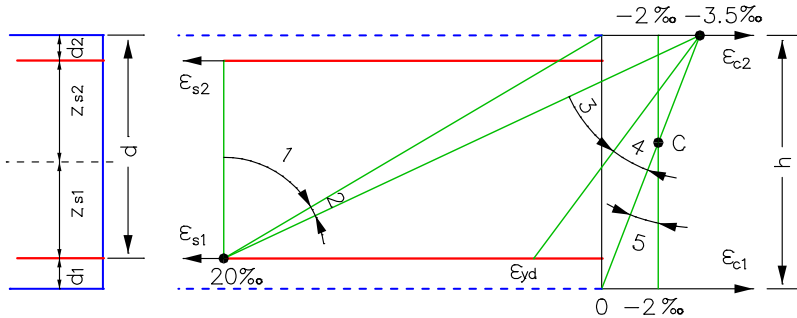
## Stress-Strain-Curves

The following characteristics are used for section design:

- *Concrete*: Parabola-rectangle diagram according to OENORM 4700, Figure 7.
- *Reinforcing steel*: Stress-strain curves according to OENORM B 4700, Figure 9.
- *Prestressing steel*: Stress-strain curve according to OENORM B 4750, Figure 11b, with upper horizontal branch according to Chapter 10.1.2 (2) of the standard.

## Design for Normal Force, Bending Moment (N, M, N and M)

The design for normal force and bending moment is performed according to OENORM B 4700, Chapter 3.4.2. The design internal forces are calculated in the same way as for the design according to EN 1992-1-1. The reinforcement required for each internal force combination at the reinforced concrete section is determined iteratively based on the formulation of equilibrium conditions as well as the limit strain curve depicted in the illustration below. The final result is derived from the extreme value of all calculated reinforcements.



Strain areas for the design

The steel strain is limited to  $\epsilon_s = 20\text{‰}$  for the purpose of numerical stability. You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes.

### Mode Standard

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tensile section to the greatest degree possible. Given ratios between certain reinforcement layers in the tension or compression zone are maintained as far as possible. For reasons of economy, if the steel strain  $\epsilon_{sd}$  related to the steel strength  $f_{sd}$  is exceeded, compressive reinforcement is determined.

The procedure in strain areas 4 and 5 is the same as with symmetrical design. The requirements for the minimum reinforcement for the less stressed direction of slabs and walls according to Chapter 6.6.2 are not taken into account during bending design.

### Mode Symmetrical

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected.

### Mode Compression member

The design is performed symmetrically. Additionally, the required minimum reinforcement for columns and compression members according to Chapter 3.4.9.2 of B 4700 is determined:

$$A_{s,\min} = 0.15 \cdot N_{sd} / f_{yd} \geq 0.0028 \cdot A_c \tag{60}$$

with

$f_{yd}$  Design value for the reinforcing steel strength at the yield strength.

$N_{sd}$  Design value of the longitudinal force to be absorbed.

$A_c$  Total area of the concrete section.

### Inclusion of tendons with bond

When designing beams and design objects, the internal forces of the concrete section is reduced by the statically determined portions which result from prestressing minus the losses from creep, shrinkage and prestressing steel relaxation (CSR). Situations prior to the grouting of the tendons are excluded. So only the restraint portions from  $P+CSR$  and the external loads are contained in the remaining internal forces for the composite section. If necessary, the reinforcing steel positioned by the user will be increased until the composite internal forces can be absorbed. In the design mode *compression member* the prestressing steel area is taken into account when determining the minimum reinforcement according to Chapter 3.4.9.2.

The position of the tendon groups in the section, the prestressing losses from CSR, the statically determined portions and the internal forces of the concrete section and the composite section are written to the detailed log.

As a separation into statically determined and undetermined shares of the internal forces from prestressing is not possible for shell structures, the prestressing is taken into account fully on the action side when designing the longitudinal reinforcement. As a result, on the resistance side only mild steel and concrete are considered whereas the strain reserves of the tendons with bond are not used.

## Minimum Reinforcement Against Failure Without Warning

Prestressed structures must be checked to ensure that structure failures do not occur without warning during initial crack formation. According to OENORM B 4753, Chapter 12.5, you can fulfill this requirement by simply following the guidelines in B 4700, Chapter 3.4.9.4 (1), for the minimum bending reinforcement.

OENORM B 4750, Chapter 12.5, on the other hand, requires that you apply a special minimum reinforcement (robustness reinforcement). The assumption here is that the prestressing force will be fictitiously reduced until bending cracks occur under the frequent action combination. The reduced prestressing force  $P_{\text{red}}$  is:

$$P_{\text{red}} = \frac{M - f_{\text{ctm}} \cdot W_{\text{c1}} + N \cdot k_1}{e_p + k_1} \quad (60)$$

with

$M$  Bending moment from the frequent action combination without prestressing force.

$N$  Normal force from the frequent action combination without prestressing force.

$f_{\text{ctm}}$  Average concrete tensile strength according to OENORM B 4700, Table 4.

$k_1$  Core width, based on the bending tensile edge fiber,  $k_1 = W_{\text{c1}} / A_c$ .

$W_{\text{c1}}$  Resistance moment based on the bending tensile edge fiber 1.

$A_c$  Section area.

$e_p$  Eccentricity of the prestressing force.

The reinforcement must be designed according to B 4750, Eq. (35). As part of the process, the normal force  $N = -P_{\text{red}}$  must be factored in to determine the auxiliary values in Eq. (34) and (36).

The program determines the robustness reinforcement according to B 4750. Irrespective of the result, you must observe the rules for the minimum bending reinforcement as per B 4700, Chapter 3.4.9.4 (1). These rules can be incorporated into the program by specifying a base reinforcement in the reinforcing steel description.

## Design for Lateral Force

The design for lateral force involves determining the diagonal tensile reinforcement and includes a concrete strut check according to OENORM B 4700, Chapter 3.4.4.2 in conjunction with B 4750, Chapter 12.3.4. The following special conditions apply:

- The angle of the diagonal tensile reinforcement is assumed to be 90°.
- The calculated stirrup reinforcement takes into account the minimum reinforcement based on Equation (62) of B 4700.
- For beams and design objects, the shear design is performed separately for the  $Q_y$  and  $Q_z$  lateral forces.
- Slab and shell elements are designed for lateral force  $q_r = \sqrt{q_x^2 + q_y^2}$ . Depending on which has a negative effect, either the principal compressive force or principal tensile force is used for the associated longitudinal force.
- In B 4700, Formula (27b) (calculation of the reduced concrete strut reinforcement), the section areas of the non-prestressed reinforcement in the bending compression chord (compression reinforcement)  $A_{s2}$  is assumed to be zero to be on the safe side.
- There is no reduction of the action from loads near supports as specified in Chapter 3.4.4.2, Section (12) or (13) of B 4700.
- For beams and design objects, the decisive web width and the effective height of the girder are defined by the user independently of the normal section geometry.
- For sections with internal prestressing, the design value of lateral load-bearing capacity  $V_{\text{Rdc}}$  as per B 4750, Chapter 12.3.4 (4), is determined using the nominal value  $b_{\text{w,nom}}$  of the section width.
- For prestressed sections, the angle  $\beta$  of the concrete struts is limited to  $\tan \beta \leq 1.0$  according to B 4750, Chapter 12.3.4 (1).
- The lever arm  $z$  of the internal forces is generally assumed as  $z = 0.9 d$ .

Slab and shell elements (and beam elements, if required by the user) can be checked to determine whether a diagonal tensile reinforcement is necessary according to Chapter 3.4.4.4 of B 4700. As in the previous case, **no** reduction of the action from loads near supports occurs.

The formulas from OENORM B 4700 that are used:

$$V_{Rdc} = b_w \cdot z \cdot v \cdot f_{cd} \cdot \frac{1}{\cot \beta + \tan \beta} \quad (27)$$

with

$V_{Rdc}$  Design value of the lateral force resistance if the strength of the concrete strut angle plays a decisive role (concrete failure under diagonal pressure).

$b_w$  Smallest width of the web.

$z$  Lever arm of the internal forces.

$d$  Effective height of the girder.

$$v = 0.7 - \frac{1.5 \cdot f_{cd}}{200} \geq 0.5 \quad (f_{cd} \text{ in N/mm}^2) \quad (25)$$

$\beta$  Angle of the concrete struts in relation to the girder axis with  $0.6 \leq \tan \beta \leq 1.6$ . (23)

$$V_{Rdc,red} = 1.67 \cdot V_{Rdc} \cdot \left( 1 - \frac{\sigma_{cd,eff}}{f_{cd}} \right) \leq V_{Rdc} \quad (27a)$$

with

$$\sigma_{cd,eff} = \frac{N_{sd} - f_{yd} \cdot A_{s2}}{A_c} \quad (27b)$$

$V_{Rdc,red}$  Reduced value of  $V_{Rdc}$  resulting from compressive force.

$\sigma_{cd,eff}$  Average effective concrete stress as a result of longitudinal force (positive for compression).

$N_{sd}$  Design value of the longitudinal force to be absorbed (positive for compression).

$A_{s2}$  Section area of the non-prestressed reinforcement in the bending compression chord (compression reinforcement).

$A_c$  Entire area of the concrete section.

$$V_{Rds} = A_{sw} / s \cdot z \cdot f_{yd} \cdot \cot \beta \quad (29)$$

with

$V_{Rds}$  Design value of the lateral force resistance if the strength of the diagonal tensile reinforcement plays a decisive role (diagonal tensile reinforcement failure).

$A_{sw}$  Section area of the diagonal tensile reinforcement in the effective area  $s$ .

$s$  Distance of the diagonal tensile reinforcement in the direction of the girder axis (e.g., stirrup distance).

$$a_{sw,min} = \frac{A_{sw}}{s} \geq \frac{15 \cdot f_{ctm}}{f_{yd}} \cdot b_w \quad (62)$$

with

$f_{ctm}$  Average concrete tensile strength according to B 4700, Table 4.

$b_w$  Smallest width of the web in cm.

$$V_{Rd1} = [\tau_d \cdot k \cdot (1.2 + 40 \cdot \rho) + 0.15 \cdot \sigma_{cd}] \cdot b_w \cdot d \quad (39)$$

with

$V_{Rd1}$  Absorbable lateral design force of a component without shear reinforcement.

$\tau_d$  Calculated value of shear stress according to B 4700, Table 4.

$k = (1.6 - d) \geq 1.0$

In components for which more than 50% of the largest span reinforcement is continuously extended to the support ( $d$  in m).

$k = 1.0$  if less than 50% of the largest span reinforcement is continuously extended to the support.

$$\rho = \frac{A_s}{b_w \cdot d} \leq 0.02$$

$A_s$  Bending tensile reinforcement that is extended by at least  $d+l_{b,crf}$  beyond the section under consideration (see B 4700, Figure 23),  $l_{b,crf}$  according to Formula (84).

$$\sigma_{cd} = N_{sd} / A_c$$

$N_{sd}$  Design value of a longitudinal normal force that simultaneously acts with the lateral force (positive for compression).

$A_c$  Concrete section area.

$b_w$  Smallest width of the section under consideration.

## Design for Torsion

The design for torsion is carried out according to OENORM B 4700, Chapter 3.4.6. This design involves determining the diagonal tensile reinforcement and the longitudinal reinforcement and includes a concrete strut check under maximum torsional stress according to formula (52) combined with a concrete strut check under lateral force according to formula (56) of the standard. Box girders are also checked as per B 4750, Chapter 12.4 (3) to ensure that the principle compressive stress does not exceed the calculated value  $f_{cd}$  of the concrete compressive strength.

The calculated longitudinal reinforcement takes into account the minimum reinforcement according to formula (62) of the standard. The equivalent hollow section on which this design is based is defined by the user independently of the normal section geometry.

The formulas from OENORM B 4700 that are used:

$$t_{Sd} = \frac{T_{Sd}}{2A_k} \quad (49)$$

with

$t_{Sd}$  Design value of the shear force to be absorbed in the lateral areas of the equivalent hollow section.

$T_{Sd}$  Design value of the torsion moment to be absorbed.

$A_k$  Areas enclosed by the center line of the equivalent hollow section.

$$t_{Rdc} = v \cdot f_{cd} \cdot d_{ef} \cdot \frac{1}{\cot \beta + \tan \beta} \quad (52)$$

with

$t_{Rdc}$  Design value of resistance against concrete failure due to diagonal compression.

$d_{ef} = d_0/6$  Wall thickness of the equivalent hollow section.

$d_0$  Diameter of the largest circle that can be drawn within the actual section.

$$t_{Rds} = A_{sw} / s \cdot f_{yd} \cdot \cot \beta \quad (53)$$

with

$t_{Rds}$  Design value of the resistance against failure due to diagonal tensile reinforcement yielding.

$A_{sw}$  Section area of the diagonal tensile reinforcement (stirrup beams) inserted to absorb torsion.

$s$  Distance of the diagonal tensile reinforcement beams in the direction of the girder axis.

$$F_t(T)_i = u_{k,i} \cdot t_{Sd} \cdot \cot \beta \geq 0 \quad (54)$$

$$F_t(T)_i \leq A_{si} \cdot f_{yd} \quad (55)$$

with

$F_t(T)_i$  Tensile force of the lateral wall under consideration in the longitudinal direction of the girder.

$u_{k,i}$  Length of the center line of the lateral wall under consideration in the equivalent hollow section.

$A_{si}$  Section of the longitudinal torsion reinforcement for the lateral wall under consideration.

$$\left( \frac{V_{Sd}}{V_{Rdc}} \right) + \left( \frac{t_{Sd}}{t_{Rdc}} \right) \leq 1 \quad (56)$$

## Punching Shear

The load-bearing safety check with respect to punching shear is carried out according to OENORM B 4700, Chapter 3.4.5. The check is fulfilled when the following condition is met (equations provided by the standard):

$$V_{Sd,max} \leq V_{Rd} \quad (42)$$

with

$$V_{Sd,max} = V_{Sd} \cdot \kappa_e \quad (43)$$

$V_{Sd}$  Design value of the lateral force exerted in the critical perimeter.

$\kappa_e$  Coefficient for taking load eccentricity into account.  $\kappa_e = 1$  for central punching shear.

$V_{Rd}$  Punching resistance according to equations (44) and (45).

Punching resistance without punching reinforcement is calculated as

$$V_{Rdc} = 0.12 \cdot \tau_d \cdot \kappa_c \cdot (1.2 + 2000 \cdot d/l \cdot \rho) \cdot u \cdot d \quad (44)$$

with

$\tau_d$  Calculated value of shear stress according to B 4700, Table 4.

$\kappa_c = 1.6 - d > 1$  ( $d$  in m)

$d = 0.5(d_x + d_y)$  average effective height.

$l$  for flat slabs, the maximum span adjoining to the column; for single foundations, twice the maximum ground plan dimensions. In this case note that  $l/d \geq 20$ .

$$\rho = \sqrt{\rho_x \cdot \rho_y} \leq 0.015$$

$\rho_x = \frac{a_{sx}}{d_x}$ ;  $\rho_y = \frac{a_{sy}}{d_y}$  Level of tensile reinforcement in the x and y direction.

$u$  Decisive circumference of the critical perimeter according to B 4700, Chapter 3.4.5.4.

Punching resistance with punching reinforcement is calculated as

$$V_{Rds} = V_{Rdc} + \kappa_s \cdot A_{sv} \cdot f_{yd} \cdot \sin \alpha \leq 1.4 \cdot V_{Rdc} \quad (45)$$

with

$\kappa_s$  Effect coefficient depending on the selected arrangement of the punching reinforcement.  $\kappa_s = 0.5$  is normally applicable.

$A_{sv}$  Steel section of the punching reinforcement between column edge and critical perimeter, with a minimum of

$$A_{sv,min} = 0.15 \frac{f_{ctm} \cdot u \cdot d}{f_{yd} \cdot \sin \alpha}$$

$\alpha \geq 35^\circ$  Angle between the effect direction of the reinforcement and the slab midplane.

Minimum bending reinforcement

$$a_{s,min} = \frac{V_{Sd}}{0.9 \cdot d \cdot f_{yd}} \cdot \frac{e}{b_{ef}} \quad (47)$$

$\frac{e}{b_{ef}}$  according to B 4700, Table 7.



## Check Against Fatigue

According to OENORM B 4750, fatigue checks are not normally required for building and road bridge construction. The fatigue strength of railway bridges, however, must always be checked as per B 4753. As outlined in Chapter 13, a check must be carried out for the special fatigue combination, which is based on the frequent action combination in the serviceability limit state and comprises the loads specified in Chapter 9.4.2 (load combination 2). The traffic load that has an impact on fatigue should be added to the program using a variable action  $Q_N$ .

## Fatigue of Longitudinal Reinforcement, Shear Reinforcement and Prestressing Steel

The fatigue check is carried out according to B 4753, Chapter 13.3. The steel stresses are calculated for longitudinal reinforcement from bending and longitudinal force as well as for prestressing steel in beams and design objects under the assumption of a cracked concrete section. The stresses for shear and longitudinal reinforcement from lateral force and torsion are calculated on the basis of a truss model using the strut angle  $\tan \beta$  from the shear design. The prestressing steel stresses in area elements are determined at the uncracked concrete section. Tendons without bond and external tendons are not checked.

According to chapter 13.3, Section (1), the actions from prestressing are to be reduced by 5% while the permanent actions and principal traffic loads are to be increased by 5% in the check combination to account for unavoidable variation. The program does not automatically make these modifications, which ultimately lead to an increase in the stress range. If necessary, you can alternatively specify a lower permissible stress range in accordance with Chapter 13.3, Sections (3) and (4).

The check is valid if the damage-equivalent stress range  $\Delta\sigma_{s, \text{equ}}$  of the steel does not exceed

$$\Delta\sigma_{s, \text{equ}} = \sigma_{sd, \text{max}} - \sigma_{sd, \text{min}} \leq \Delta\sigma_{Rsk} / \gamma_{fat} \quad (35)$$

with  $\gamma_{fat} = 1.15$ . Note that  $\Delta\sigma_{Rsk}$  is the permissible characteristic stress range specified by the user in the Section dialog for  $2 \cdot 10^6$  load changes. The compressive stresses of the non-prestressed reinforcement are not considered in the stress range calculation as permitted in the standard.

The maximum from the robustness, crack and bending reinforcement is assumed to be the existing bending reinforcement. If as a result the load from the fatigue combination in state II cannot be absorbed, a new design is carried out based on the existing reinforcement and the check internal forces. The maximum stress range per steel layer that results from the strain state in state II or the truss model is determined separately for each check situation. If this range exceeds the permitted stress range according to Eq. (35), the necessary reinforcement will be iteratively increased until the check succeeds for all situations. In the *Symmetrical* and *Compression member* design modes the reinforcement is applied at all predefined locations. This will not affect the predefined relationships between the individual reinforcement layers.

The decisive reinforcement used for the check, which may have been increased, is recorded in the check log and saved for graphical representation.

## Fatigue of Concrete Under Compressive Stress

The fatigue check for concrete under compressive stress is conducted for bending and longitudinal force according to Chapter 13.2 of OENORM B 4753 at the cracked section. It takes into account the final longitudinal reinforcement and may include an increase applied during the fatigue check for reinforcing steel. The struts of components subject to lateral force stress are not analyzed. The check is considered successful if compliance with OENORM B 4753, Eq. (34) is established.

$$\Delta\sigma_{c, \text{equ}} = \sigma_{cd, \text{max}} - \sigma_{cd, \text{min}} \leq 0.50f_{cd} - 0.55\sigma_{cd, \text{min}} \quad (34)$$

with

$\sigma_{cd, \text{max}}$  Largest longitudinal compressive stress of the concrete (compression > 0).

$\sigma_{cd, \text{min}}$  Smallest longitudinal compressive stress of the concrete in the same fiber in which  $\sigma_{cd, \text{max}}$  occurs. If tensile stresses occur, set  $\sigma_{cd, \text{min}} = 0$ .

## Special Characteristic of Shell Structures

In shell structures the strain state at the cracked concrete section under general stress cannot be determined unambiguously. The design is therefore carried out separately for the reinforcement directions x and y with the design internal forces from Wolfensberger/Thürlimann or Rüsç as described above. The reinforcement calculated in this manner yields a reliable load-bearing capacity.

When calculating the stress range for reinforcing steel and concrete, this method can lead to unrealistic results in the case of torsional or shear stresses as shown in the following example:

Assume two identical sets of slab internal forces:

Set	$m_x$ [kNm/m]	$m_y$ [kNm/m]	$m_{xy}$ [kNm/m]
1	300	200	100
2	300	200	100

According to Wolfensberger/Thürlimann, this results in design variants for the x direction:

Set	Variant	$m$ [kNm/m]
1	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$
2	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$

The torsional moments generate a variation of the design moments and thus a calculatory stress range. This may lead to a necessary reinforcement increase in the fatigue check due to apparent overstressing. For design normal forces, this applies for the shear forces in a corresponding manner.

Selecting **Limit design variants** in the Section dialog allows you to avoid the described effect. In this case only the corresponding variants are compared when determining the stress range, which means only the first and second variants of both sets in this example. Assuming constant stress, the stress range is thus correctly determined to be zero.

**This alternative, however, cannot ensure that all conceivable stress fluctuations will be analyzed. You should therefore be particularly careful when assessing the results. The main variants and design internal forces that are used for the check are indicated in the detailed log for this purpose.**

When determining the design internal forces according to Rüsç for inclined reinforcement, the described relationships apply accordingly.

## Checks in the Serviceability Limit States

The following checks specified by OENORM are performed:

- Limiting the concrete compressive stresses (B 4750, Chap. 11.4.5, and B 4753, 11.4.2).
- Limiting the reinforcing steel stresses (B 4750, Chap. 11.4.5, and B 4753, 11.4.2).
- Limiting the prestressing steel stresses (B 4750, Chap. 11.4.5, and B 4753, 11.4.2).
- Check of decompression (B 4750 and B 4753, Chapter 11.3)
- Crack-dispersing minimum reinforcement (B 4700, Chap. 4.2.2, and B 4750, 11.5)
- Limiting the crack width (B 4700, Chapter 4.2.3).
- Limiting deformations (B 4700, Chapter 4.3).

The concrete compressive stress, steel stress and decompression checks are only relevant for prestressed structures.

## Design Combinations

Based on OENORM B 4750, Chapter 11.2, the combinations listed below are taken into consideration in the serviceability limit states. The combination coefficients can be selected according to B 4750, Table 5, or B 4753, Table 4.

- Rare (characteristic) combination

$$L_0 = \sum_j G_{k,j} + P_k + Q_{k,1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} \quad (19)$$

- Frequent combination

$$L_1 = \sum_j G_{k,j} + P_k + \psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \quad (20)$$

- Quasi-continuous combination

$$L_2 = \sum_j G_{k,j} + P_k + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \quad (21)$$

Different variants of the construction stages and final states can be defined for each combination. If necessary, the combination required by the check will automatically be determined from the section specifications. Each check is carried out for all the variants of a combination.

## Stress Determination

For uncracked concrete sections, the program assumes that concrete and steel under tensile and compressive stress behave elastically. With respect to cracked concrete sections, concrete that is subjected to compression is assumed to have an elastic behavior yet without the capacity to absorb tensile stresses (cf. OENORM B 4750, Chapter 11.4).

### Area elements

For area elements the concrete stresses are calculated at the gross section. The steel stress check is carried out for reinforcing steel by determining the strain state at the cracked concrete section and for the prestressing steel at the uncracked concrete section.

### Beams and design objects

The action combination stresses that can be determined without checks are always calculated at the gross section.

Conversely, in the checks the stresses are determined as follows and are graphically displayed or logged:

- When checking the crack reinforcement and crack width, the concrete stress is calculated at the gross section
- When decompression and the concrete compressive stresses are checked, the concrete stress is calculated in state I
  - without internal tendons at the gross section
  - with internal tendons without bond at the net section
  - with internal tendons with bond for situations before being grouted at the net section or otherwise at the ideal section
- The reinforcing and prestressing steel stresses are checked by determining the strain state at the cracked concrete section

## Limiting the Concrete Compressive Stresses

The concrete compressive stresses are checked according to OENORM B 4750, Chapter 11.4.5 and B 4753, Chapter 11.4.2. Based on B 4750, Chapter 11.4 (2), the cracked state is assumed if the tensile stress calculated in the uncracked state under the rare action combination exceeds the value  $f_{ctm}$ . This assumption also takes into account the variation of prestressing force as per B 4750, Chapter 11.2 (3).

The calculation in the cracked state is carried out with a linear stress-strain curve by determining the strain state with the final longitudinal reinforcement (maximum from robustness, crack and bending reinforcement including a possible increase from the fatigue check). For beams and design objects, the tendons with bond are taken into account on the resistance side provided that they are grouted in the check situation. For area elements, the compressive stress for both reinforcement directions is determined separately and the extreme value is checked since the general strain state cannot be determined unambiguously.

The concrete compressive stress  $\sigma_c$  must be limited to  $0.60f_{ck}$  under the rare (characteristic) combination according to OENORM B 4750, Chapter 11.4.5 (1). According to OENORM B 4753, Eq. (12), the stress must be limited to  $0.55f_{ck}$  given uniaxial bending. Also, to prevent disproportionate creep deformations, the limit  $0.45f_{ck}$  should be maintained under the quasi-continuous combination as per B 4750, Chap. 11.4.5 (2). Both options are considered based on the user's specifications.

## Limiting the Reinforcing and Prestressing Steel Stresses

### Reinforcing steel

For reinforcing steel, the limit of  $0.80f_{yk}$  (B 4750, Chapter 11.4.5 (3)) or  $0.70f_{yk}$  (B 4753, Chapter 11.4.2 (7)) as set by the user for steel stress under the rare combination will be checked. In this check the reinforcement corresponds to the maximum value from the robustness, crack and bending reinforcement, including a possible increase as a result of the fatigue check. The determination of the strain state is performed at the cracked concrete section. If beam tendons with bond are grouted in the check situation, they will be taken into account on the resistance side.

### Prestressing steel

For tendons with bond, the limitation of steel stress as per OENORM B 4750, Chapter 11.4.5 (4), will be checked at the cracked concrete section for beams and design objects and at the uncracked concrete section for area elements. In either case the following limits apply:

- $0.70f_{pk}$  under the rare combination
- $0.65f_{pk}$  under the quasi-continuous combination

For situations prior to grouting and for tendons without bond, the stress  $\sigma_{pm0}$  is checked according to B 4750, Eq. (4) and (5). External tendons are not checked.

## Check of Decompression

This check is carried out for prestressed components of requirement classes A-C with the combinations specified in Table 1 of the OENORM B 4750. According to Chapter 11.2 (3) of the standard, the variation of the prestressing force is to be taken into account. The minimum requirement classes for the structure depend on the prestressing type as outlined in Table 2 of the standard. For area elements, the principal tensile stress  $\sigma_1$  or one of the longitudinal tensile stresses  $\sigma_x$  or  $\sigma_y$  is checked depending on the user's selection. The latter can be used to limit the check to the direction of the prestressing if the internal force systems are appropriately aligned. In all other cases, the rules listed above for stress analysis apply.

The permitted stress limits are defined as follows:

### Construction stage

According to OENORM B 4750, Chapter 3.7.3, road bridges in the construction stage must be checked to ensure that the characteristic tensile strength  $f_{ctk}$  as per B 4700, Table 4, is not exceeded under the decisive action combination. This also applies to railway bridges as per B 4753, Chapters 3.2 and 11.4.2 (3). The effective tensile strength for the construction stage is defined in the section dialog.

### Final state

The decompression limit state is reached according to OENORM B 4750, Chapter 3.7, when the stress of the edge fiber

reaches zero in the precompressed tensile zone (i.e., at the section edge facing the tendon). According to Chapter 11.3 of the standard, the action combinations of this limit state as defined by the requirement classes may not be exceeded. The program determines the above edge as follows:

- Beams and design objects: If the edge point next to the tendon is above the centroid, the stress on the upper side of the section will be checked. If not, the lower side of the section will be checked.
- Area elements: The check will be carried out for the upper or lower section edge if the tendon next to the check point is located above or below the centroid level of the element in question. Tendons outside of the element are taken into account at a distance of up to five times the section height.

If the tendon guide is ambiguous, the check will be carried out for both sides.

## Crack Dispersing Minimum Reinforcement

For all components in which crack formation through restraint is expected, a crack reinforcement should be distributed to the tensile edges based on OENORM B 4700, Chapter 4.2.2. The reinforcement  $A_s$  to be applied is determined according to Eq. (64):

$$A_s = \rho_t / 100 \cdot b_t \cdot h_t \quad (\rho_t \text{ in } \%) \quad (64)$$

The reinforcement level  $\rho_t$ , which is based on tensile zone  $b_t \cdot h_t$  and dependent on the specified limit diameter  $d_s = d_{sg}$  and the permissible crack width  $w_k$ , must be taken from Table 8a ( $w_k = 0.30 \text{ mm}$ ) or Table 8b ( $w_k = 0.15 \text{ mm}$ ). The calculated limit diameter  $d_{sr}$  serves as the input value and is determined according to the following relationship:

$$d_{sg} = d_{sr} \cdot k \quad (65)$$

with

$$k = h_t / 5 \quad h_1 \geq 1 \quad \text{for bending cracks } h_t < h. \quad (66)$$

$$k = h / 10 \quad h_1 \geq 1 \quad \text{for separation cracks.} \quad (67)$$

$b_t$  Average width of the concrete tensile zone.

$h_t$  Height of the tensile zone in state I when the concrete tensile strength is reached.

$h$  Height of the section or the section part under consideration.

$h_1$  Distance of the reinforcement's centroid from the tensile edge.

For concrete quality above B30, the limit diameter  $d_{sg}$  is to be reduced by a factor of  $30/f_{cwk}$  according to Chapter 4.2.2 (4). The program will accordingly increase the input value  $d_{sr}$  for Table 8a or 8b by this factor.

In the case of predominant compression, i.e., if  $h_t \leq h_1$ , the limitation of the bar diameter is not necessary according to Chapter 4.2.2 (3), Item b). The program uses the minimum reinforcement level as per Table 8a or 8b for Eq. (64) in such cases.

The reductions of reinforcement level  $\rho_t$  as permitted in Sections (5) through (7) can be specified in the Section dialog. The guidelines of Sections (8) through (10) are to be taken into account constructively by the user.

The tensile sections in which the crack reinforcement is to be inserted can be defined in the Section dialog by selecting either an action combination or a restraint (bending, central tension). In the design modes *Symmetrical* and *Compression member*, the remaining layers are also reinforced. This will not affect the predefined relationships between the individual reinforcement layers.

If the crack width check is to be carried out at the same time, the program will determine whether the specified permissible crack width is maintained by inserting the calculated crack reinforcement. If necessary, the crack reinforcement is increased iteratively until the check limit is reached. The increased reinforcement is indicated by an exclamation mark "!" in the log.

The necessary (constructive) minimum reinforcement as per B 4700, Chapter 3.4.9.4, and B 4750, Chapter 7.3.3, is not taken into consideration when determining the crack reinforcement. It can be specified as the base reinforcement in the reinforcing steel description.

### Special characteristic of prestressed concrete structures

In prestressed concrete structures, according to OENORM B 4750, Chapter 11.5 (5), tendons with bond in the tensile zone whose distance from the reinforcing steel reinforcement in the tensile zone does not exceed 300 mm can be accounted for with  $k_p$  times their prestressed steel area  $A_p$ . The calculated reinforcement level is then derived by

$$\rho_{tr} = 100 (A_s + k_p \cdot A_p) / (b_t \cdot h_t) \quad (31)$$

The bond coefficient  $k_p$  must be determined as per 11.5 (6) and specified for beams and design objects in the Section

dialog. The program does not take prestressing steel into account for area elements.

Influences from the plain stress effect and the change of the inner lever arm during transition from state I to state II can be taken into account by reducing the reinforcement level as per B 4750, Eq. (33):

$$\rho_t = \rho_{tr} \cdot \sqrt{k_c} \quad (33)$$

$$k_c = 1 + \frac{N}{k_h \cdot A_c \cdot f_{ctm}} \quad (34)$$

$$0 \leq k_c \leq 1$$

with

$N$  Permanent normal force (compression:  $N < 0$ ) or permanent prestressing force.

$f_{ctm}$  Average expected tensile strength of the concrete during initial cracking.

$k_h = 1.5$  for beams and slabs with  $h < 1$  m.

$k_h = 1.5 h$  for beams and slabs with  $h \geq 1$  m.

According to B 4750, Chapter 11.5 (8), the steel section area  $A_s$  used in formula (31) may not be less than

$$A_{s,min} = 0.0028 \cdot k_c \cdot b_t \cdot h_t \quad (35)$$

This value must be increased by 15% for B50 and 30% for B60.

The program does not take into account the special rules for tension chords with connected flange parts provided in Chapter 11.5 (8).

## Limiting the Crack Width

The crack width check is carried out according to OENORM B 4700, Chapter 4.2.3, by limiting the bar diameter  $d_s$  of the reinforcement in the tensile section to  $d_{sg}$ .

The bar diameter  $d_s$  is specified in the Section dialog. The limit diameter  $d_{sg}$  is determined from the calculated limit diameter  $d_{sr}$  based on formulas (65) through (67) of the standard. Note that  $d_{sr}$  which depends on the existing reinforcement level  $\rho_t$  and the steel stress  $\sigma_{sD}$  of the reinforcement, is taken from Table 9 ( $w_k = 0.30$  mm) or Table 10 ( $w_k = 0.15$  mm).

The reinforcement level  $\rho_t$  is determined according to Chapter 4.2.2 (3) with the tensile zone in state I upon reaching the concrete tensile strength. In the case of predominant compression, i.e., if the height of the tensile zone  $h_t$  results in  $h_t \leq h_1$ , the limit diameter is not checked according to Chapter 4.2.2 (3) (b).

The steel stress  $\sigma_{sD}$  is derived from the strain state II under the check combination defined by the requirement class and with allowance for prestressing variation. For beams and design objects, all tendons with bond are considered on the resistance side. The stress-strain curve of the concrete is assumed to be linear as per B 4750, Chapter 11.4 (1).

The concrete steel layer with the maximum value for the difference ( $d_s - d_{sg}$ ) is listed in the log.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine the reinforcement that is required to limit the crack width. A design with the decisive check combination is performed for this purpose. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The bar diameter limitation is then checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check).

## Limiting Deformations

According to OENORM B 4700, Chapter 4.3, or B 4750, Chapter 11.6, the deformations of a structure or component must be limited such as to prevent any negative effects with respect to its function or appearance. Chapter 4.3.1 (2) of B 4700 stipulates that in general, no component under continuous load should sag any more than 1/250th of the span.

Prestressed concrete structures of requirement classes A, B, and C are planned to remain in state I for the quasi-continuous action combination. This allows short-term deformations in condition I to be calculated on the basis of the elasticity theory in accordance with B 4750, Chapter 11.6 (7). Congruence with the actual deformation largely depends on how realistic the modulus of elasticity is defined. For structures classified as requirement classes D and E, the deformation analysis specified in Chapter 11.6 (9) should be conducted in accordance with B 4700, Chapter 4.3.3.

When using InfoCAD to perform a realistic check in state II, this can only be done as part of a nonlinear system analysis for beam and shell structures that takes geometric and physical nonlinearities into account. The resistance of the tendons with bond is currently not included in the calculation.

Editing is performed in the following steps:

- Define the check situation with the *Load group* function in the Load dialog by grouping the decisive individual load cases. The variable loads must first be weighted with the combination coefficients  $\psi_2$  for the quasi-continuous combination.
- Select the check load cases in the *Nonlinear Analysis / Serviceability* dialog of the analysis settings for the FEM or framework analysis.
- Set the reinforcement determined in the ultimate limit state in the *Start reinforcement* selection field (maximum from bending, robustness, crack check and fatigue).
- Perform the FEM or framework analysis to determine the deformations in state II.
- Check the system deformations displayed graphically or in tabular form.

For a detailed description of the nonlinear system analysis, refer to the relevant chapter of the manual.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The resulting bending, robustness and crack reinforcement, the decisive maximum value and the stirrup and torsion reinforcement are provided for the graphical representation as well.

The log shows the design internal forces and necessary reinforcements, checked stresses or crack widths at each result location. If the permissible limit values are exceeded, they are reported as warnings and indicated at the check location. The detailed log also lists the decisive combination internal forces of all design situations.

### Stresses for beams and design objects

$\sigma_x$	Longitudinal stresses in the decompression and concrete compressive stress checks [MN/m <sup>2</sup> ].
$\sigma_s, \Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd}, \Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete under longitudinal compression [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,y}, \Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from $Q_y$ and $Q_z$ [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,T}, \Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and longitudinal torsion reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{perm}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

### Stresses for area elements

$\sigma_x, \sigma_y, \sigma_1$	Longitudinal stress in x or y direction or principal tensile stresses in the decompression check [MN/m <sup>2</sup> ] (depending on user specification).
$\sigma_2$	Principal compressive stresses [MN/m <sup>2</sup> ].
$\tau_{xy}, \tau_{xz}, \tau_{yz}$	Shear stresses [MN/m <sup>2</sup> ].
$\sigma_{sx}, \Delta\sigma_{sx}$	Stresses and stress ranges [MN/m <sup>2</sup> ] for reinforcing steel in the x direction.
$\sigma_{sy}, \Delta\sigma_{sy}$	Stresses and stress ranges [MN/m <sup>2</sup> ] for reinforcing steel in the y direction.
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges [MN/m <sup>2</sup> ] for prestressing steel.

$\sigma_{cd,x}, \Delta\sigma_{cd,x}$	Stresses and stress ranges in the concrete fatigue check under longitudinal compression in the x-
$\sigma_{cd,y}, \Delta\sigma_{cd,y}$	and y-direction [MN/m <sup>2</sup> ].
$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{perm}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

### Bending reinforcement

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams and design objects.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.
$a_{s\phi}$	Meridian reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.
$a_{s\Omega}$	Ring reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.

### Reinforcement from lateral force

$a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area and axisymmetric shell elements.
$A_{sb,y}, A_{sb,z}$	Stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $Q_y$ and $Q_z$ .
$A_{sl}$ for $a_{sb}=0$	Longitudinal reinforcement [cm <sup>2</sup> ] of area elements.

### Torsion reinforcement

$A_{sb,T}$	Torsional stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $M_x$ .
$A_{sl,T}$	Torsional longitudinal reinforcement [cm <sup>2</sup> ] of beams and design objects from $M_x$ .

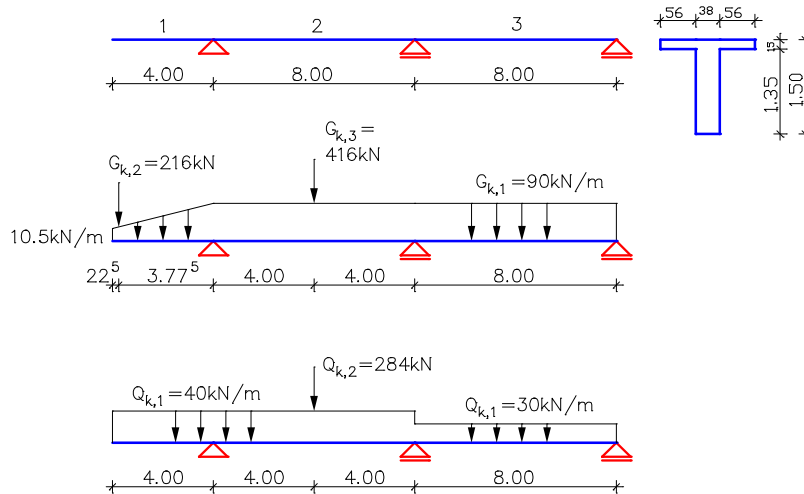
### Design values

$v_{Rd1}$	Absorbable design lateral force without shear reinforcement for area elements [kN/m].
$v_{Rdc}$	Absorbable design lateral force of concrete struts for area elements [kN/m].
$V_{Rd1}$	Absorbable design lateral force without shear reinforcement for beams and design objects [kN].
$V_{Rdc}$	Absorbable design lateral force of concrete struts for beams and design objects [kN].
$T_{Rdc}$	Design value of the maximum absorbable torsion moment [kNm].



## Example - Two-Span Girder with Cantilever

The following example illustrates the analysis of the two-span girder with cantilever shown. The system corresponds to example 6 from 'Examples for Applying the New OENORM B 4700, Reinforced Steel Structures - EUROCODE-Oriented Analysis, Design and Detailing', Österreichisches Normungsinstitut (ON), Vienna.



System and load of the reinforced concrete girder

The characteristic values of the permanent loads  $G_{k,1}$  to  $G_{k,3}$  and the variable loads  $Q_{k,1}$  and  $Q_{k,2}$  are provided in the illustration. It is important to note that the variable loads are effective independent of one another. The definition of actions with respect to the seven individual load cases can be taken from the following program printout.

### OENORM B4700 actions

#### Standard design group

##### G - Dead load

Gamma.sup / gamma.inf = 1.35 / 1

##### Load cases

1 Dead load Gk,1

##### G - Dead load from above on cantilever

Gamma.sup / gamma.inf = 1.35 / 1

##### Load cases

2 Dead load Gk,2

##### G - Dead load from above on span 1

Gamma.sup / gamma.inf = 1.35 / 1

##### Load cases

3 Dead load Gk,3

##### QN - Imposed load, traffic load

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Reinforced concrete and prestressed concrete structures  
Working loads on ceilings - Living rooms, lounges, office rooms including adjacent rooms, staircases and walks  
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

##### Load cases 1. Variant, inclusive

4 Traffic cantilever  
5 Traffic span 1  
6 Traffic span 2

**QN - Imposed load, traffic load from above on span 1**

$\gamma_{sup} / \gamma_{inf} = 1.5 / 0$

Combination coefficients for: Reinforced concrete and prestressed concrete structures  
 Working loads on ceilings - Living rooms, lounges, office rooms including adjacent rooms, staircases and walks  
 $\psi_{0.1} / \psi_{1.1} / \psi_{2.1} = 0.7 / 0.5 / 0.3$

Load cases 1. Variant, inclusive

7 Traffic Qk,2

**1. Fundamental combination**

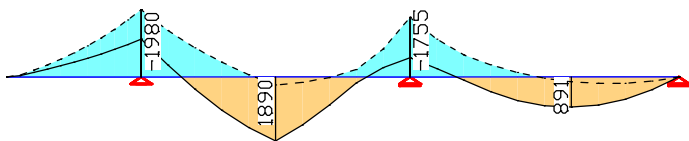
Final state

- G Dead load
- G Dead load from above on cantilever
- G Dead load from above on span 1
- QN Imposed load, traffic load
- QN Imposed load, traffic load from above ...

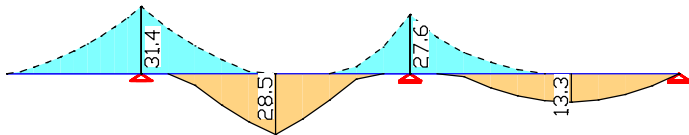
**1. Quasi-continuous combination**

Final state

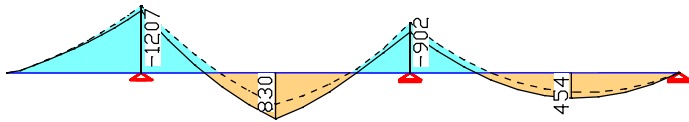
- G Dead load
- G Dead load from above on cantilever
- G Dead load from above on span 1
- QN Imposed load, traffic load
- QN Imposed load, traffic load from above ...



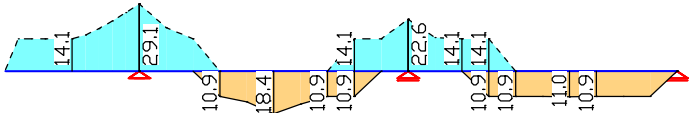
Moment boundary curves  $M_y$  [kNm], 1st fundamental combination, linear internal force determination without redistribution



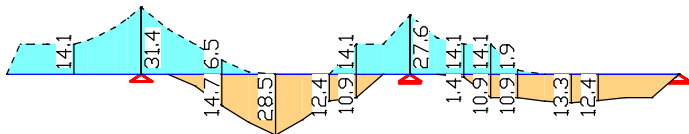
Bending reinforcement  $A_s$  from the load-bearing capacity [cm<sup>2</sup>]



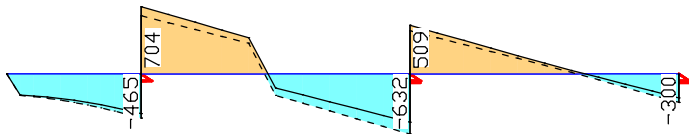
Moment boundary curves  $M_y$  [kNm], 1st quasi-continuous combination, linear internal force determination without redistribution



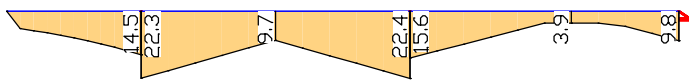
Bending reinforcement  $A_s$  from crack check [cm<sup>2</sup>]



Maximum bending reinforcement  $A_s$  [cm<sup>2</sup>]



Lateral force boundary curves  $Q_z$  [kN], 1st fundamental combination, linear internal force determination without redistribution



Associated lateral force reinforcement  $A_{sb}$  [cm<sup>2</sup>/m]

The preceding illustrations show the calculated boundary curves of the girder with respect to a linear internal force determination and the resulting  $A_s$  or  $A_{sb}$  curve. There has been no redistribution of the column moments.

Below you will find a printout of the design specifications and an excerpt of the design results.

**Design acc. to OENORM B 4700 (2001), B 4750 (2000), B 4753 (2003)**

The design is applicable for reinforced and prestressed concrete with and without bond. The actions are combined by use of the specific partial safety and combination factors according to OENORM B 4700, B 4750 und B 4753.

**All checks are made for the extreme values of actions.**

**Design overview**

Se.	Class, Prestress of component	Reinforc. M R B Q T	Fatig. S P C	Crack- width	De- comp.	Stress checks Comp. S P
1	E Not prestressed	. x x x .	. . . .	x	.	. . . .

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state.
- (T) Torsional reinforcement at ultimate limit state.
- (S) Reinforcing steel at stress and fatigue check.
- (P) Prestressing steel at stress and fatigue check.
- (C) Concrete at fatigue check.

**Settings for flexural and shear reinforcement**

f<sub>yk</sub> Quality of stirrups [MN/m<sup>2</sup>].  
 tan β Angle of concrete truss.  
 Slabs Beams are designed like slabs.  
 A<sub>s</sub> Given reinforcement according to B4700, picture 23.  
 50% A<sub>s</sub> More than 50% of the biggest bottom reinforcement is unbroken up to support.

Se.	Concrete	Design- mode for M and N	f <sub>yk</sub> St. [MN/m <sup>2</sup> ]	Truss- angle tan β	Design like Slabs	A <sub>s</sub> Pict.23 [cm <sup>2</sup> ]	50% A <sub>s</sub> up to support
1	ÖNB40	Standard	500	1.00	.	0.00	.

**Shear sections**

b<sub>w,nom</sub> Nominal width of the prestressed section acc. to B4750, 12.3.4(4).  
 h<sub>nom</sub> Nominal height of the prestressed section acc. to B4750, 12.3.4(4).  
 d<sub>k</sub>, b<sub>k</sub> Height and width of the core section for torsion.  
 def Thickness of the torsion box.  
 B. Box section.

Se.	Width [m]		Eff. width		Height [m]		Eff.height			Torsion section [m]		
	b <sub>w</sub>	b <sub>w,nom</sub>	b <sub>n</sub> [m]	h	h <sub>nom</sub>	d [m]	d <sub>k</sub>	b <sub>k</sub>	def	B.		
1	0.380	.	0.300	1.500	.	1.420	1.437	0.317	0.063	.		

**Settings for the check of crack widths**

k<sub>p</sub> Bond coeff. of prestr. steel for beam sections acc. to B4750, 11.5(6).  
 k<sub>rho</sub> Reduction coeff. for reinf. degree rho.t acc. to B4700, 4.2.2(5)-(7).  
 AC Use of the action combinations according to B4753, table 1.

Se.	Crack width wk,per [mm]	Steel diameter ds [mm]	Coeff. k <sub>p</sub>	k <sub>rho</sub>	AC B4753	Determination of tens. zone for A <sub>s,min</sub> Comb. acc. to class
1	0.30	20.00	1.00	.	.	Comb. acc. to class

**Section geometry and reinforcing steel for beams**

Se.	Point	Concrete		Reinforcing steel						
		y [m]	z [m]	Es, f <sub>yk</sub> [MN/m <sup>2</sup> ]	y [m]	z [m]	d <sub>l</sub> [m]	A <sub>s</sub> [cm <sup>2</sup> ]		
1	1	0.000	0.000	200000	550	0.080	0.080	0.070	0.00	
	2	0.560	0.000	200000	550	1.420	0.080	0.070	0.00	
	3	0.940	0.000	200000	550	0.860	1.420	0.080	0.00	
	4	1.500	0.000	200000	550	0.640	1.420	0.080	0.00	
	5	1.500	0.150							
	6	0.940	0.150							
	7	0.940	1.500							
	8	0.560	1.500							
	9	0.560	0.150							
	10	0.000	0.150							

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (R) Nominal/requ. reinforcement as per 4.2.2/3 for crack width limitation  
 Increase of reinforcement due to crack width check is marked by "!".
- (B) Design of reinforcement at ultimate limit state  
 In case of dominant bending, compression reinforcement is marked with "\*\*".

Beam No.	Se.	Lo.	Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	Ap' [cm²]	req.As [cm²]
1	1	1	1	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00
			2	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00
			3	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00
			4	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00
1	1	11	1	R	0.00	-1207.40	0.00	.	14.57!
				B	0.00	-1980.39	0.00	.	15.70
			2	R	0.00	-1207.40	0.00	.	14.57!
				B	0.00	-1980.39	0.00	.	15.70
			3	R	0.00	-1207.40	0.00	.	0.00
				B	0.00	-1111.40	0.00	.	0.00
			4	R	0.00	-1207.40	0.00	.	0.00
				B	0.00	-1111.40	0.00	.	0.00
2	1	6	1	R	0.00	829.68	0.00	.	0.00
				B	0.00	242.20	0.00	.	0.00
			2	R	0.00	829.68	0.00	.	0.00
				B	0.00	242.20	0.00	.	0.00
			3	R	0.00	829.68	0.00	.	9.18!
				B	0.00	1890.00	0.00	.	14.23
			4	R	0.00	829.68	0.00	.	9.18!
				B	0.00	1890.00	0.00	.	14.23
3	1	1	1	R	0.00	-902.05	0.00	.	11.29!
				B	0.00	-1755.00	0.00	.	13.79
			2	R	0.00	-902.05	0.00	.	11.29!
				B	0.00	-1755.00	0.00	.	13.79
			3	R	0.00	-902.05	0.00	.	0.00
				B	0.00	-562.80	0.00	.	0.00
			4	R	0.00	-902.05	0.00	.	0.00
				B	0.00	-562.80	0.00	.	0.00
3	1	7	1	R	0.00	453.86	0.00	.	0.00
				B	0.00	144.44	0.00	.	0.00
			2	R	0.00	453.86	0.00	.	0.00
				B	0.00	144.44	0.00	.	0.00
			3	R	0.00	453.86	0.00	.	5.52!
				B	0.00	891.16	0.00	.	6.66
			4	R	0.00	453.86	0.00	.	5.52!
				B	0.00	891.16	0.00	.	6.66
3	1	11	1	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00
			2	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00
			3	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00
			4	R	0.00	0.00	0.00	.	0.00
				B	0.00	0.00	0.00	.	0.00

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to B 4700, 3.4.9.4(2) is considered.

VRd, TRd Design value of maximum absorbable lateral force, torsional moment  
 Angle Angle tan Beta between the compressive strut and the beam axis  
 Asb,Asl.T Req. stirrup reinf. from lateral force and torsion, torsional reinf.  
 Asl Req. longitudinal reinf. acc. to B 4700, Fig. 23 for req. Asb.  
 Sigma2 Box section, max. principal compressive stress [MN/m²]  
 For the check of Sigma2/fcd, state II is marked with "\*"   
 fcd = alpha.cc \* fck / gamma.c, per. Sigma2 in state I [MN/m²]  
 = alpha.cc \* v1 \* alpha.cc \* fck / gamma.c, per. Sigma2 in state II

Beam No.	Loc.	Qy/VRd	Angle	Asb.y [cm²/m]	Qz/VRd	Angle	Asb.z [cm²/m]	Asl [cm²]	Q/VRd+ Mx/TRd	Asb.T, Asl.T [cm²/m, cm²]	Sigma2 /fcd
1	1	0.00	1.00	0.00	0.00	1.00	0.00	.	.	.	.
11	0.00	1.00	0.00	0.30	1.00	14.45	.	.	.	.	.
2	1	0.00	1.00	0.00	0.46	1.00	22.30	.	.	.	.
6	0.00	1.00	0.00	0.20	1.00	9.72	.	.	.	.	.
11	0.00	1.00	0.00	0.46	1.00	22.35	.	.	.	.	.
3	1	0.00	1.00	0.00	0.32	1.00	15.61	.	.	.	.
7	0.00	1.00	0.00	0.04	1.00	3.93	.	.	.	.	.
11	0.00	1.00	0.00	0.20	1.00	9.81	.	.	.	.	.

**Check of crack widths**

The check calculates the limiting steel diameter.  
 (CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

Beam No.	Se.	C.	Lo.	Rei. Lay.	Nx [kN]	My [kNm]	Mz [kNm]	Sigma.x [MN/m²]	ds [mm]	per. [mm]
1	1	QC	1	0	0.00	0.00	0.00	0.00	20.0	.
			11	2	0.00	-1207.40	0.00	4.33	20.0	25.6
2	1	QC	1	2	0.00	-1207.40	0.00	4.33	20.0	25.6
			6	3	0.00	829.68	0.00	4.51	20.0	65.9
			11	2	0.00	-902.05	0.00	3.23	20.0	33.8
3	1	QC	1	2	0.00	-902.05	0.00	3.23	20.0	33.8
			7	4	0.00	453.86	0.00	2.47	20.0	34.6
			11	0	0.00	0.00	0.00	0.00	20.0	.

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Concrete, reinforced and prestressed concrete structures – Part 1: Design and construction.  
Publisher: DIN Deutsches Institut für Normung e.V., Beuth Verlag GmbH, Berlin 2008.

Heft 166 - Berechnungstafeln für schiefwinklige Fahrbahnplatten von Straßenbrücken  
(Book 166 - Calculation Tables for Oblique-angled Roadway Slabs of Road Bridges).  
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OENORM B 4750:2000-11

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Publisher: Österreichisches Normungsinstitut  
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Spannbeton-Eisenbahnbrücken – EUROCODE-nahe Berechnung, Bemessung und konstruktive Durchbildung  
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Publisher: Österreichisches Normungsinstitut  
Österreichisches Normungsinstitut (ON), Vienna 2003.

Beispiele zur Anwendung der neuen ÖNORM B 4700

„Stahlbetontragwerke – EUROCODE-nahe Berechnung, Bemessung und konstruktive Durchbildung“,  
(Examples for applying the new OENORM B 4700 'Reinforced Steel Structures –  
EUROCODE-Orientated Analysis, Design and Detailing') as of 1 October 1995.  
Publisher: Österreichisches Normungsinstitut  
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Bemessung der Stahlbeton- und Spannbetonbauteile nach DIN 1045-1  
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Ernst & Sohn Verlag, Berlin 2002.

# SIA 262 Design

## Basics

The reinforced concrete and prestressed concrete design according to SIA 262:2017 is applicable for both building and bridge structures. Permitted structure models include beam, area and solid constructions. Prestressed structures can only be checked in the FEM module.

Differing components can be combined in a structure model:

- Non-prestressed components
- Prestressed components with subsequent bond
- Prestressed components without bond
- Components with external prestressing
- Mixed-construction components

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with SIA 260. The program will take into account the preset safety factors and combination coefficients for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements.

The actions and check selection dialogs can be opened from the analysis settings. Detailed check specifications and reinforcement data must be entered during section definition.

The checks are limited to elements with materials *SC12/15* to *SC50/60*, *SLC12/13* to *SLC50/55* and *SCX*.

For beams and design objects, all checks are carried out at the polygon or composite section. For general notes on using design objects, refer to the relevant chapter of the manual.

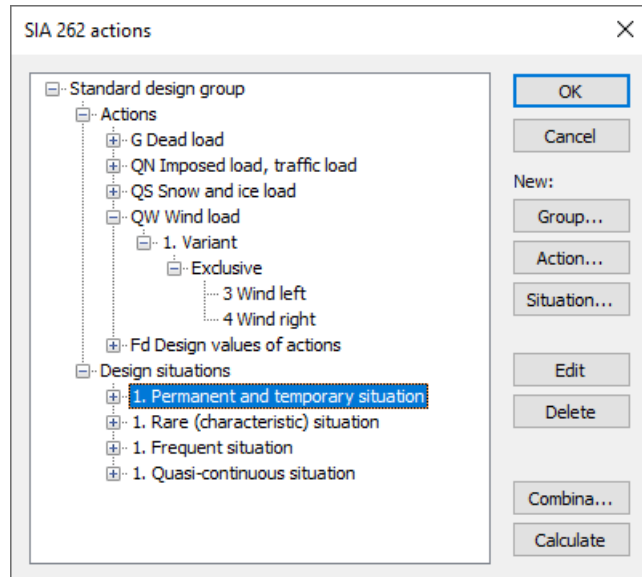
In the *SIA 262 Design* folder of the database you can also perform a single design for user-defined polygon sections or composite sections.

# Input

## Actions and Design Situations

The design values of the load are calculated based on the internal forces of the individual load cases and load case combinations. For this the existing load cases and load case combinations must be assigned to actions. This results in the desired design situations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Prestressing (P)
- Creep and shrinkage, relaxation (CSR1, CSR2)  
These actions are only available if a P action has been defined. In the combinations they are treated, along with P, as a single action.
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)  
The assigned load cases should contain a design-relevant set of loads with partial safety factors and combination coefficients such as for example a load group to take into account nonlinear effects. The selected load cases are combined exclusively.
- Cyclic fatigue actions (Qfat)

### Group...

Open the dialog for entering a new design group. According to e.g. standard SIA 261, Chapter 8.3.3, certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

### Situation...

Open the dialog for entering new design situations. Situations must be classified as either a construction stage or a final state in order to control the checking process. For prestressed concrete structures with subsequent bond, you can specify that the tendons are still ungrouted.

### Edit

Open the Edit dialog for the selected action or situation.

### Delete

Delete the selected action or situation.

**Combinations...**

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

The following example shows the total variants of the *permanent and temporary situation* according to Eq. (6.10) to be examined with the load cases (L1...L6) involved and their weighting factors.

Actions	Load cases	$\gamma_{sup}$	$\gamma_{inf}$	$\psi_0$
Dead load	1	1.35	1.0	-
Imposed load, traffic load	2, 3 (inclusive)	1.5	0	0.7
Wind load	4	1.5	0	0.6
$F_d$ Design values of actions	5, 6	1.0	1.0	-

The image shows two screenshots of the '1. Permanent and temporary situation' dialog box. Both dialogs show a table of combinations (24 of 24) with columns for combination number and load cases L1 through L6. The left dialog shows combinations 1 through 12, and the right dialog shows combinations 13 through 24. Each dialog has buttons for 'OK', 'Cancel', and 'Load group...'.

**Calculate**

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all combinations in the database. This allows you to evaluate the results without having to open the checking module. Each time you open the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

The following table demonstrates how the situations are used in the various checks. The numbers refer to the SIA 262 chapters.

Situation	Ultimate limit state	Chapter	Serviceability limit state	Chapter
Permanent and temp. Accidental Earthquake	Longitudinal reinf.	4.3.2		
	Lateral reinf.	4.3.3		
	Torsional reinf.	4.3.5		
Characteristic (rare)			Prevention of brittle failure	4.4.2
Frequent			Crack width limitation	4.4.2
Quasi-continuous			Concrete compr. stress	3.1.2.6.3 4.1.5.2.4
			Crack width limitation	4.4.2
Fatigue	Fatigue reinf. steel	4.3.8		
	Fatigue prestr. steel	4.3.8		
	Fatigue concrete	4.3.8		



## Definition of an Action

The illustration below shows an example of the dialog field for entering variable actions. The dialog fields for the other action types have a similar appearance.

### Label

User-defined label for the action.

### Gamma.sup, Gamma.inf

Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$

### Combination coefficients psi for:

Input fields for selecting the combination coefficients for variable actions. The button allows you to view and modify the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .

### Load cases

List of the possible load cases or load case combinations. Select items by highlighting them and clicking the button or use drag & drop.

### Multi-select

Load cases and combinations can be added to the actions more than once.

### Exclusive variants

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants by clicking the or buttons.

### Inclusive load cases

Selected load cases and combinations that can have a simultaneous effect.

### Exclusive load cases

Selected load cases and combinations that are mutually exclusive.

### Prestressing loss from relaxation of prestressing steel

The prestressing loss is defined as a constant percentage reduction of prestress.

### CS as constant reduction of prestress

As an alternative to defining load cases, you can allow for the effect of creep and shrinkage (CS) by defining a constant percentage reduction of prestress.

### Internal prestressing

Selected load cases that describe internal prestressing. The reactions of the individual load cases are added up.

## External prestressing

Selected load cases that describe external prestressing. The reactions of the individual load cases are added up.

## Partial Safety Factors

The partial safety factors for actions are determined by the definition of actions in accordance with SIA 260, Table 1, and can be modified if necessary. The partial safety factors of the construction materials are preset with the values specified by SIA 262, Section 2.3.2.5.

## Section Input

The section inputs contain all of the specific settings made for checks in the ultimate limit and serviceability states. In addition to these specifications, the selected material properties and the properties of the reinforcing steel are also relevant for the design. An overview of the design specifications can be accessed in the *SIA 262 Design* section of the database.

## Checks

The following dialog is used to define which ultimate limit state and serviceability checks are available for the section. For composite sections, the selection is limited to the load-bearing capacity checks. The analysis settings allow to override this selection for the entire structure.

### Prestress of component

The type of prestressing can be selected for each section separately:

- *not prestressed*
- *subsequent bond*
- *without bond*
- *external*
- *mixed construction*

### Requirement

The requirement for crack formation determines the actions and steel stress limits for crack width limitation according to SIA 262, Table 17 and Figure 31.

## Base Values

Unless otherwise specified, the base values apply for all checks in the ultimate, fatigue and serviceability limit states.

### Design mode

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary.
- *Compression member*: For compression members, a symmetrical design is carried out with allowance for the minimum reinforcement according to SIA 262, Chapter 5.5.4.

### Factor for as in secondary direction

According to SIA 262, Chapter 5.5.3.2, secondary longitudinal reinforcement of slabs should not be less than 20% of the principal reinforcement. The examination is carried out on the program side with the results of the bending design separately for the upper and lower side of the cross-section. The direction with the largest amount of reinforcement per cross-sectional side defines each principal reinforcement direction. The assignment of the factorized reinforcement in secondary direction then takes place via corresponding reinforcement layers.

### Reduction factor of prestr. for brittle failure

In the program, the regulations of EN 1992-2, Chapter 6.2 (110) are decisive for the arrangement of robustness reinforcement. Thus for the determination of the tensile zone the statically determined effect of prestressing is not taken into account. Because this cannot be determined for area elements, the prestress can alternatively be reduced by a reduction factor. The specification of an appropriate value is subject to the discretion of the user.

### Design without considering given reinforcement ratios

If selected, the reinforcement increase required in the design is performed without taking into account the reinforcement ratios specified by the basic reinforcement.

### Quality of stirrups

Steel quality for stirrup and longitudinal reinforcement from lateral force in MN/m<sup>2</sup>.

### Effective height

Effective static height for the shear design of area elements [m].

### Design like slabs

Beams or design objects are treated like slabs.

**Strain  $\epsilon_{s.v}$  for slabs**

- *Basic value:* Strain  $\epsilon_v$  according to Section 4.3.3.2.2 for calculation of the absorbable lateral force without lateral force reinforcement [%].
- *Max. grain  $D_{max}$ :* Maximum grain of the concrete for the calculation of  $k_g$  according to Equation (37).
- *Longitudinal reinforcement is graded:* Switch for the increase of  $\epsilon_v$  according to Section 4.3.3.2.3.

**Compression field angle Alpha [°]**

Angle of the concrete compressive field according to Section 4.3.3.2.

**Coefficient  $k_c$  for compress. strength**

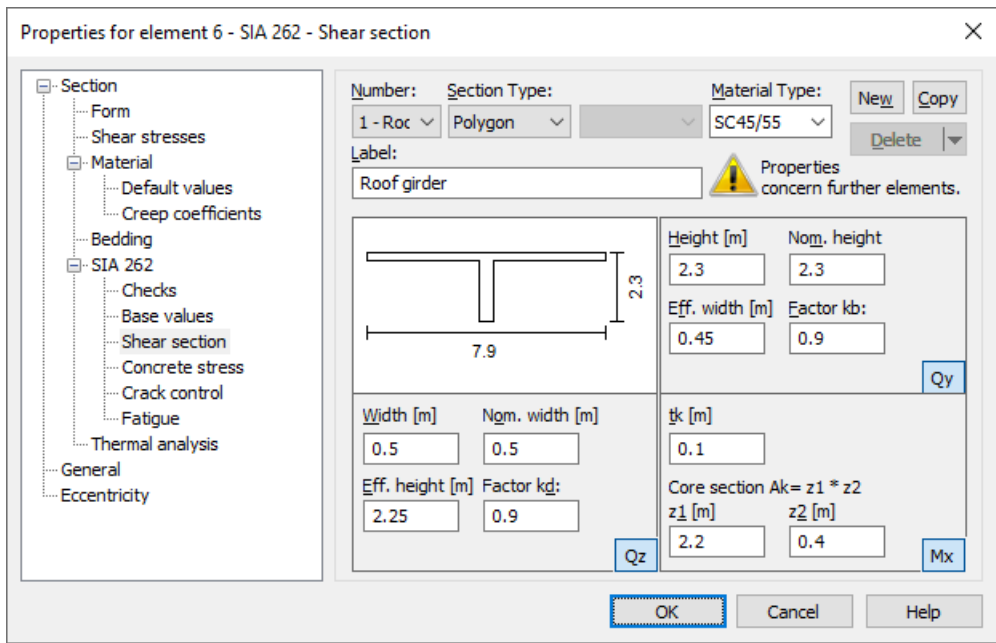
Coefficient for the concrete compressive strength according to Section 4.2.1.7.

**Design as circular cross-section**

For circular and annular cross-sections, the lateral force design according to Bender et al. (2010) can be selected as an alternative for the resulting shear force  $Q_T = \sqrt{Q_y^2 + Q_z^2}$ . The corresponding inputs are made on the *Shear Section* dialog page.

**Shear Section**

For polygon and composite sections, additional section dimensions are required for the lateral force and torsion design according to SIA 262. These dimensions are explained in the following. In case of sections with internal prestressing or with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.



**Width**

Section width for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

**Height**

Section height for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

**Effective height**

Effective static height for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

**Effective width**

Effective static width for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

**Nom. width, nom. height**

The nominal width or height of internally prestressed components as per SIA 262, Section 4.3.3.3.5, for including the duct

diameter in the calculation of the design value of the lateral load-bearing capacity  $V_{Rd,c}$ .

### Factor kb, Factor kd

Factor for calculating the inner lever arm  $z$  from the effective width  $b_n$  or effective height  $d$  in the lateral load-bearing capacity check for  $Q_y$  or  $Q_z$ .

### tk

The effective wall thickness of the torsion section [m].

### Core section Ak= z1 \* z2

Dimensions of the core section for calculating the torsion reinforcement [m].

## Circular and annular cross-section

If the circular design according to Bender et al. (2010) was selected for the resulting lateral force  $Q_r$  on the *Base values* dialog page, the equivalent cross sections for the shear design must be defined in the following dialog.

### Width bw

Effective section width for calculation of the lateral force bearing capacity for  $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$ . According to the recommendation of the German Committee for Standardization in Civil Engineering (NABau), the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces should be selected for the effective width  $b_w$ . For circular cross-sections, the program suggests the dimension of the square inscribed in the circle ( $R \cdot \sqrt{2}$ ) for  $b_w$ , and twice the wall thickness for annular cross-sections.

### Effective height d

Statically effective height for calculation of the lateral force bearing capacity for  $Q_r$ . The program suggests  $d = h - d_1$ , where the height is set to  $h = R \cdot \sqrt{2}$  and  $d_1$  indicates the edge distance of the outer reinforcement layer.

### Factor kd

Factor for calculating the inner lever arm  $z$  from the effective height  $d$  in the verification for  $Q_r$ .

### Efficacy factor

According to Bender et al. (2010), p. 422, the efficacy factor  $\alpha_k$  is stress-dependent ( $0.715 \leq \alpha_k \leq 0.785$ ) and can be assumed with the mean value  $\alpha_k = 0.75$ .

### Helix inclination

Angle between shear force reinforcement and component axis. When entering an inclination of  $90^\circ$ , annular single stirrups are assumed.

**z1, z2, tef**

The dimensions  $z_1$ ,  $z_2$  of the square core cross-section and the effective wall thickness  $t_{ef}$  of the torsion box are defined according to SIA 262, Figure 19. The design for torsion is carried out according to the standard for vertical stirrups.

**Concrete Stress**

Properties for element 6 - SIA 262 - Concrete stress

Section

- Form
- Shear stresses
- Material
  - Default values
  - Creep coefficients
- Bedding
- SIA 262
  - Checks
  - Base values
  - Shear section
  - Concrete stress
  - Crack control
  - Fatigue
  - Thermal analysis
- General
- Eccentricity

Number: 1 - Roc Section Type: Polygon Material Type: SC45/55

Label: Roof girder

Quasi-permanent combination

perm. sigma.c:

0.45 fck

At the time t of prestressing

perm. sigma.c(t):

0.45 fck(t)  0.60 fck(t)

Concr. strength fck(t) [MN/m²]: 45

Properties concern further elements.

OK Cancel Help

**perm. sigma.c**

In accordance with SIA 262, Section 3.1.2.6.3, the effect of the load level on the creep is to be taken into account for concrete stresses  $\sigma_c > 0.45 f_{ck}$ . The compliance of this stress limit is verified for the quasi-continuous combination referring to EN 1992-1-1, Section 7.2 (3).

**perm. sigma.c(t)**

Permissible concrete stress  $\sigma_c(t)$  at time  $t$  when prestressing is introduced according to SIA 262, Section 4.1.5.2.4, Equation (23). If the compressive stress exceeds the value  $0.45 \cdot f_{ck(t)}$ , the nonlinearity of the creep should be taken into account according to Section 3.1.2.6.3. The program assumes that prestressing is introduced in design situation G1+P.

**fck(t)**

Concrete strength at time  $t$  when prestressing is introduced as per Section 4.1.5.2.4 [MN/m²].

## Crack Control

These specifications apply to the check against brittle failure, the calculation of the crack reinforcement and the crack width limitation.

Properties for element 6 - SIA 262 - Crack control

Section  
 Form  
 Shear stresses  
 Material  
 Default values  
 Creep coefficients  
 Bedding  
 SIA 262  
 Checks  
 Base values  
 Shear section  
 Concrete stress  
 Crack control  
 Fatigue  
 Thermal analysis  
 General  
 Eccentricity

Number: 1 - Roc Section Type: Polygon Material Type: SC45/55  
 New Copy  
 Delete

Label: Roof girder Properties concern further elements.

Tensile strength: kt\*fctm Factor kt: 1

Restraint for crack reinforcement: Centr. Tens  Take prestr. steel into account

Max. bar diameter ds per Fig. 31 [mm]: 10

OK Cancel Help

### Tensile strength $f_{ctd}$

The concrete tensile strength  $f_{ctd}$  can be calculated either with SIA 262, Equation (98), or with Equation (100).

### Factor $k_t$

Reduction factor  $k_t$  for taking into account the dimension of the tension chord  $t$  according to Equation (99).

### Restraint for crack reinforcement

Selection of the tensile zone according to Section 4.4.2.3.6 for distribution of the crack reinforcement in case of restraint.

### Take prestr. steel into account

Bonded prestressing steel within the tensile zone is taken into account according to Section 4.4.2.3.10.

### Max. bar diameter $d_s$ per Fig. 31 [mm]

Largest existing bar diameter of the reinforcing steel for determination of the stress limits according to Figure 31.

## Fatigue

Properties for element 6 - SIA 262 - Fatigue

Section: Form, Shear stresses, Material, Bedding, SIA 262, Checks, Base values, Shear section, Concrete stress, Crack control, **Fatigue**, Thermal analysis, General, Eccentricity

Number: 1 - Roc, Section Type: Polygon, Material Type: SC45/55, New, Copy, Delete

Label: Roof girder, Properties concern further elements.

Reinforcing steel, prestressing steel

Long. reinf.	Shear reinf.	Prestr. steel
dSigma.sd,fat: 145	dSigma.bd,fat: 65	dSigma.pd,fat: 95 MN/m <sup>2</sup>

Increase factor  $k_{\xi}$ : 1

Limit design variants

OK, Cancel, Help

### $d\sigma_{sd,fat}$ , $d\sigma_{bd,fat}$ , $d\sigma_{pd,fat}$

Design values of the fatigue strength of the longitudinal reinforcement, the shear reinforcement and the prestressing steel according to SIA 262, Table 13. For calculation of the coefficient  $k_{\phi}$  according to Equation (85) for shear reinforcement, the mandrel diameter is taken to be  $d_1 = 4 \phi_s$ .

### Increase factor $k_{\xi}$

Increase factor  $k_{\xi}$  for the reinforcing steel stress of the longitudinal reinforcement. This factor is used to take into account the varying bonding behavior of concrete and prestressing steel as per Section 4.3.8.1.4.

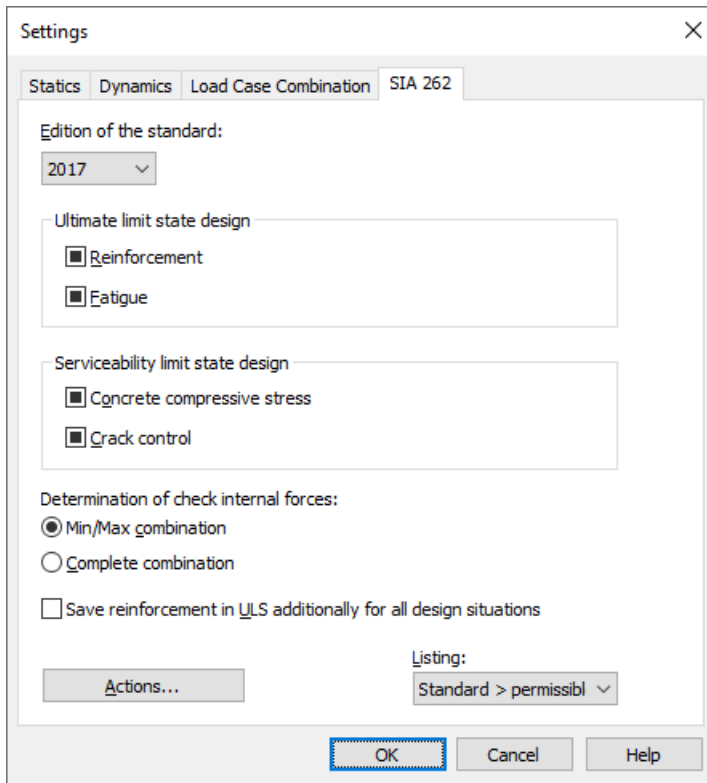
### Limit design variants

For area elements, the variants for determining the stress range can be limited to the corresponding sets of design internal forces. For more information see chapter 'Check Against Fatigue > Special Characteristic of Shell Structures'.



## Analysis Settings

The SIA 262 dialog page can be opened using the *Settings* function of the *Analysis* menu.



### Edition of the standard

The edition you select will be used for all subsequent calculations.

### Check selection

When selecting checks, the following cases are to be distinguished:

- or  The check is performed according to the settings in the section dialog (see *Section inputs*).
- or  The check is performed for all sections of the structure.
- or  The check is performed for no sections of the structure.

Corresponding section settings are bundled as follows:

Reinforcement	Bend and longitudinal force Lateral force Torsion Robustness
Crack control	Brittle failure Crack reinforcement from restraint Crack width limitation

An overview of the checks can be accessed using the *Design Settings* function in the *SIA 262 Design* folder of the database.

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *EN 1992-1-1 Design > Check internal forces*.

### Save reinforcement in ULS additionally for all design situations

In addition to the maximum required ultimate limit state reinforcement, the reinforcement is saved separately for each design situation in the ultimate limit state.

#### Actions...

Open the dialog for describing actions.

#### Listing

- *No*: No log is generated by the checking program.
- *Standard*: Log with tabular output of results.
- *Detailed*: Additional output of the decisive combination internal forces at the check locations.
- *Standard > permissible*: Standard log limited to check locations where the permissible limit values are exceeded.
- *Detailed > permissible*: Detailed log limited to check locations where the permissible limit values are exceeded.

## Single Design

The single design function allows you to analyze individual section polygons separately from the whole system using predefined internal forces. The calculation is carried out from the opened input table via the *Single Design* item in the *Analysis* menu or the *Print Preview* function. The entry table can be found in the *SIA 262 Design* folder of the database.

#### Section

Number of the section to be designed.

#### Concrete

Concrete class SC 12/15, ... SC 50/60 or LSC 12/13, ... LSC 50/55

#### Apparent density

Apparent density of the lightweight concrete [kg/m<sup>3</sup>].

#### Combination

Design situation according to SIA 261, Section 4.4.3.4 or 4.4.3.5.

- *0*: Permanent and temporary design situation.
- *1*: Accidental design situation.

#### Nsd, Mysd, Mzsd

Internal forces that are designed.

#### Mode

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members, a symmetrical design is carried out with allowance for the minimum reinforcement according to SIA 262, Section 5.5.4.
- *Strains*: Determine strain state for existing reinforcing steel layers.
- *Strains SLS*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. In the compression zone, a linear strain-stress curve of the concrete with the gradient  $\tan \alpha = E_{cm}$  is used.
- *Strains SLS2*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A nonlinear strain-stress curve of the concrete with a strength of  $f_{cm}$  is used. Note that a horizontal progression is assumed for strains exceeding  $\varepsilon_{c1}$ .
- *Load bearing capacity*: Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
- *Maximum bending moment My*: Determination of the maximum bearable bending moment  $M_y$ . The moment  $M_y$  is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.
- *Inactive*: Design disabled.

## Punching Shear Check

When you select a check node, the key data for the checks is displayed in a dialog field. This dialog is divided into three pages.

### 1a. Input data, column

The column forms *Rectangle* and *Round*, with the viewpoints *Intern*, *Edge parallel to x*, *Edge parallel to y* and *Corner* are available. When you enter a new column, the program will suggest the dimensions of existing columns. The edge distances  $a_x$  and  $a_y$  are used to calculate the perimeters  $u_i$  of the check sections for columns near to an edge or a corner.

### 1b. Input data, slab

This section presents the material properties, the existing bending reinforcement ( $a_{sx}$ ,  $a_{sy}$ ) as well as other coefficients for the calculation of the punching shear resistances:

$D_{\max}$  maximum grain of the concrete [mm]

$d_x, d_y$  static height for the x and y direction to determine the average static height  $d$  [m]

$d_v$  effective static height for the absorption of the shear force [m]

$\beta$  inclination of the reinforcement according to Figure 26 [°]

$d_{\text{int}}$  static height for the determination of the reinforcement for protection against total collapse [m]

$s_{\text{int}}$  sum of bar spacings of reinforcement for protection against total collapse [m]

$l_x, l_y$  spans according to Figure 24 [m]

### 1c. Input data, action

The action  $V_d$  and  $M_d$  can either be taken as a support reaction from a previous design according to SIA 262, or entered directly. Possible average soil pressures  $\sigma_0$  decrease the design value of the lateral force.

### 2. Aperture

This dialog page is used to define the geometry and location of an opening.

### 3. Results

This dialog page shows the calculated punching shear resistances, the necessary punching shear reinforcement (if applicable) and the minimum bending reinforcement.

### Example

**SIA 262 Punching Shear Check Node 4312**

Input data | Aperture | Results

Column: Rectangle, corner

Width  $b_x$ : 0.3 m  
Height  $b_y$ : 0.4 m  
Edge spacing  $a_x$ : 0.2 m  
Edge spacing  $a_y$ : 0.1 m

Concrete: SC20/25  $D_{\max}$  [mm]: 32  
Steel: B500B  
Figure 26:  $\beta$ : 0°  $d_{\text{int}}$  [m]: 0.14  $s_{\text{int}}$  [m]: 0.4  
Action:  From calculation  Enter  $V_d$  [kN]:  $M_d$  [kNm]:  $\sigma_{0d}$  [kN/m<sup>2</sup>]: 0

$a_{sx}$  [cm<sup>2</sup>/m]: 5  $a_{sy}$  [cm<sup>2</sup>/m]: 5  
Eff. h.  $d_x$  [m]: 0.17 Eff. h.  $d_y$  [m]: 0.17 Stat. h.  $d_v$  [m]: 0.17  
 $l_x$  [m]: 4  $l_y$  [m]: 5

Consider aperture

**SIA 262 Punching Shear Check Node 4312**

Input data | Aperture | Results

**Results: 1. Permanent and temporary situation**  
Available long. reinf.  $a_{sx} = 5.00$  cm<sup>2</sup>/m  $a_{sy} = 5.00$  cm<sup>2</sup>/m  
 $V_d = 587.0$  kN  $V_{Rd,c} = 344.7$  kN  $V_d/V_{Rd,c} = 1.70$   
 $A_{sw} = 6.75$  cm<sup>2</sup>  
 $u_{out} = 1.93$  m Distance = 0.59 m

The check is OK!

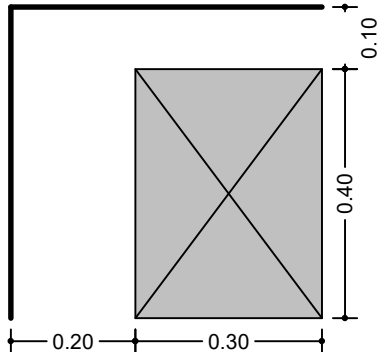
Proposal Undertake proposal 1 Undertake proposal 2

**Punching shear check node 146**

The check is performed according to SIA 262:2017.

## 1. Measurements, situation and material

Rectangular column with width  $b_x = 0.30$  m and height  $b_y = 0.40$  m  
 Situation: Corner column; Edge spacing  $a_x = 0.20$  m; Edge spacing  $a_y = 0.10$  m



Static height  $d_v = 0.170$  m

Critical perimeter  $u = 1.13$  m (distance = 0.09 m);  $A_{crit} = 0.34$  m<sup>2</sup>

Available long. reinf.  $a_{sx} / a_{sy} = 5.00 / 5.00$  cm<sup>2</sup>/m

Eff. height of the slab  $d_x / d_y = 0.170 / 0.170$  m;  $d = (d_x + d_y) / 2 = 0.170$  m

Spans  $l_x / l_y = 4.00 / 5.00$  m

Truss angle  $\alpha = 90.0^\circ$

Concrete: SC20/25;  $D_{max} = 32$  mm

$\tau_{cd} = 0.3 \cdot \eta_t \cdot \sqrt{f_{ck}} / \gamma_c = 0.3 \cdot 1.0 \cdot \sqrt{20.00} / 1.50 = 0.89$  N/mm<sup>2</sup>

$f_{bd} = 1.4 \cdot f_{ctm} / \gamma_c = 2.05$  N/mm<sup>2</sup>

Reinforcement: B500B

$E_s = 205000$  N/mm<sup>2</sup>;  $f_{sd} = f_{sk} / \gamma_s = 500.00 / 1.15 = 434.78$  N/mm<sup>2</sup>

Collapse protection as per Figure 26

$\beta = 0^\circ$ ;  $d_{int} = 0.140$  m;  $s_{int} = 0.40$  m

## 2. Action: 1. Permanent and temporary situation

$V_d = 587.00$  kN;  $M_d = 0.00$  kNm

## 3. Punching resistance without punching reinforcement

Coefficient  $k_e$  as per Chapter 4.3.6.2.4

$e_u = 0$ ;  $k_e = 1$

Distance  $r_s$  as per Chapter 4.3.6.4.4

$r_{s,x} = 0.22 \cdot l_x = 0.88$  m;  $r_{s,y} = 0.22 \cdot l_y = 1.10$  m

$b_s = \min(1.5 \cdot \sqrt{(r_{s,x} \cdot r_{s,y})}; l_{min}) = 1.48$  m

Bending resistance and comparison moment as per Chapter 4.3.6.4.7

$m_{Rd,x} = 36.79$  kNm;  $m_{Rd,y} = 36.79$  kNm

$m_{sd,x} = m_{sd,y} = \max(V_d \cdot (1/8 + |e_u| / 2b_s); V_d / 2) = 293.50$  kNm

Slab rotation  $\psi$  as per Chapter 4.3.6.4.1 at approximation level 2

$\psi_x = 1.5 \cdot r_{s,x} / d \cdot f_{sd} / E_s \cdot (m_{sd,x} / m_{Rd,x})^{3/2} = 0.37$

$\psi_y = 1.5 \cdot r_{s,y} / d \cdot f_{sd} / E_s \cdot (m_{sd,y} / m_{Rd,y})^{3/2} = 0.46$

$\psi = \max(\psi_x; \psi_y) = 0.46$

Coefficient  $k_r$  as per Chapter 4.3.6.3.2

$k_g = 48 / (16 + D_{max}) = 1.00$

$k_r = \min(1 / (0.45 + 0.18 \cdot \psi \cdot d \cdot k_g); 2) = 2.00$

Punching resistance as per Chapter 4.3.6.3.1

$$V_{Rd,c} = k_r \cdot \tau_{cd} \cdot d_v \cdot k_e \cdot u = 344.71 \text{ kN}$$

$$V_d / V_{Rd,c} = 587.00 / 344.71 = 1.70 > 1 \quad \Rightarrow \text{Punching reinforcement is required!}$$

#### 4. Punching reinforcement perpendicular to the slab plane

Design lateral force as per Chapter 4.3.6.5.2

$$V_{d,s} = \max(V_d - V_{Rd,c}; V_d / 2) = 293.50 \text{ kN}$$

Punching reinforcement as per Chapter 4.3.6.5.4

$$V_{Rd,s} = A_{sw} \cdot k_e \cdot \sigma_{sd} \cdot \sin 90^\circ$$

$$\varnothing_{sw} = 14 \text{ mm as per Table 20}$$

$$\sigma_{sd} = \min(E_s \cdot \psi / 6 \cdot (1 + f_{bd} / f_{sd}) \cdot d / \varnothing_{sw}); f_{sd}) = 434.78 \text{ N/mm}^2$$

$$A_{sw} = V_{d,s} / k_e / \sigma_{sd} = 6.75 \text{ cm}^2$$

Reinforcement arrangement as per Chapter 5.5.3.8, Figure 39 and Table 20

- The punching reinforcement should consist of two or more rows
- The first row should have a minimal distance of 0.06 m and max. 0.11 m from the edge of the supported area
- The radial distance of the reinforcing rows must not exceed 0.11 m
- The outmost row should have a distance of 0.51 m from the edge of the supported area
- In the second row, the tangential distance of the reinforcing elements must not exceed 0.26 m

Check of the concrete compressive strut at the supported area as per Chapter 4.3.6.5.7

$$V_{Rd,c} = \min(2 \cdot k_r; 3.5) \cdot \tau_{cd} \cdot d_v \cdot k_e \cdot u = 603.24 \text{ kN}$$

$$V_d / V_{Rd,c} = 587.00 / 603.24 = 0.97 \leq 1 \quad \Rightarrow \text{Check is OK!}$$

Punching shear check outside of the reinforced zone as per Chapter 4.3.6.5.9 and Figure 25

Check perimeter  $u_{out} = 1.93 \text{ m}$ ; Distance = 0.59 m

The static height is assumed to be  $d_v = 0.17 \text{ m}$

$$V_{Rd,c,out} = k_r \cdot \tau_{cd} \cdot d_v \cdot k_e \cdot u_{out} = 587.85 \text{ kN}$$

$$V_d / V_{Rd,c,out} = 587.00 / 587.85 = 1.00 \leq 1 \quad \Rightarrow \text{Check is OK!}$$

#### 5. Collaps protection

Check section as per Figure 26:  $u_{int} = s_{int} + \pi / 2 \cdot d_{int} = 0.62 \text{ m}$

$k_\beta = 0.37$  as per Table 12 for ductility class B

Residual resistance at check section as per Chapter 4.3.6.7.2

$$V_{Rd,res} = A_s \cdot f_{sd} \cdot k_\beta \leq 1.7 \cdot \tau_{cd} \cdot d_{int} \cdot u_{int}$$

$$A_s = \min(V_d; 1.7 \cdot \tau_{cd} \cdot d_{int} \cdot u_{int}) / (f_{sd} \cdot k_\beta) = 8.20 \text{ cm}^2$$

## Prestressed Structures

### Internal Prestressing

For internal prestressing, the tendon groups as well as the prestressing system and procedures are entered using the *Prestressing* function of the *Structure* menu. To include them in the FEM calculation, you then need to define a load case of the *Prestressing* load type. For more information, refer to the *Prestressed Concrete* section of the manual.

Prestressing with bond and prestressing without bond are differentiated in the section inputs and the specifications for the *Creep and shrinkage* load case. For prestressed components with subsequent bond the tendons can be set ungrouted for the respective design situation in the action dialog.

### External Prestressing, Mixed Construction

External prestressing can be taken into account by entering the external forces directly in the program. For mixed construction, the additional tendons with bond must be entered as described above.

### Creep and Shrinkage

Similar to prestressing, creep and shrinkage are taken into account by specifying a corresponding load case (*Creep and shrinkage* load type) in the FEM calculation. Besides the creep-generating permanent load case, you also need to specify whether the internal forces relocation between concrete and prestressing steel is to be taken into account. This option is only useful in the case of tendons with bond.

The decisive creep and shrinkage coefficients for calculating the *Creep and shrinkage* load case are entered in the section dialog.

The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1+\rho \cdot \varphi} (\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,S})$$

In this case:

- $\sigma_b(t)$  Concrete stress from creep and shrinkage at time  $t$ .
- $E_b$  E-modulus of the concrete.
- $\rho$  Relaxation coefficient according to Trost for time  $t$  (normally  $\rho = 0.80$ ).
- $\varphi$  Creep coefficient for time  $t$ .
- $\varepsilon_b(t)$  Concrete strain from creep and shrinkage at time  $t$ .
- $\varepsilon_{b,0}$  Concrete strain from creep-generating continuous load.
- $\varepsilon_{b,S}$  Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, yield the internal forces and deformations of the concrete. The resulting stress changes in the prestressing steel are also determined provided they are selected in the load case. Any influence from the relaxation of the prestressing steel will be ignored in this case. According to Zilch/Rogge (2002, p. 256), this influence can be calculated separately (see following section) and combined with the changes from creep and shrinkage for all time-dependent prestressing losses:

$$\Delta\sigma_{p,csr} = \Delta\sigma_{pr} + E_p \cdot \Delta\varepsilon_{cpt}$$

with

- $\Delta\sigma_{pr}$  Prestressing loss from relaxation of the prestressing steel.
- $\Delta\varepsilon_{cpt}$  Concrete strain change from creep and shrinkage.
- $E_p$  E-modulus of the prestressing steel.

## Relaxation of Prestressing Steel

According to SIA 262, Section 3.3.2.7.1, the design values of the stress loss from relaxation of the prestressing steel for a duration of 1000 h can be taken from Figure 8. Long time values of the stress loss can be determined by multiplying these values with the factor 3.

You can define the stress losses in the CSR actions of the *SIA 262 Actions* dialog.

## Checks in the Ultimate Limit States

The following checks are available:

- Bending and bending with normal force (SIA 262, Chapter 4.3.2)
- Lateral force (Chapter 4.3.3)
- Torsion and combined load (Chapter 4.3.5)
- Punching shear (Chapter 4.3.6)
- Fatigue (Chapter 4.3.8)

The following combinations in accordance with SIA 260, Chapter 4.4.3, are taken into account in the ultimate limit states:

- Permanent and temporary design situations

$$E_d = E \{ \gamma_G \cdot G_k, \gamma_P \cdot P_k, \gamma_{Q,1} \cdot Q_{k,1}, \psi_{0,i} \cdot Q_{k,i} \} \quad (16)$$

- Accidental design situations

$$E_d = E \{ G_k, P_k, A_d, \psi_{2,i} \cdot Q_{k,i} \} \quad (17)$$

- Design situations resulting from earthquakes ( $A_E$ ) according to SIA 261, Section 16.1.4.

$$E_d = E \{ G_k, P_k, A_E, \psi_{2,i} \cdot Q_{k,i} \}$$

- Fatigue combination according to SIA 262, Chapter 4.3.8, combined with EN 1992-1-1, Chapter 6.8.3, Equation (6.68).

$$E_d = E \{ \{ G_k, P_k, \psi_{1,1} \cdot Q_{k,1}, \psi_{2,i} \cdot Q_{k,i} \}; Q_{fat} \} \quad (6.68)$$

In this equation  $Q_{k,1}$  and  $Q_{k,i}$  are non-cyclic, non-permanent actions, whereas  $Q_{fat}$  defines the action of the relevant fatigue load.

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

## Stress-Strain-Curves

The following characteristics are used for section design:

- Concrete: parabola-rectangle diagram according to SIA 262, Figure 12 and Equation (28). The coefficient  $\eta_t$  in Equations (2) and (3) which considers the effect of load duration on concrete strength, is assumed to be  $\eta_t = 1$  according to Section 4.2.1.3.
- Reinforcing steel: stress-strain curve according to Figure 16, with rising upper branch and  $k_s = 1.05$ .
- Prestressing steel: stress-strain curve according to Figure 17, with horizontal upper branch according to Section 4.2.3.5.

The check against fatigue is carried out in the cracked state with a linear stress-strain curve according to Section 4.3.8.1.3.

## Design for Bending and Bending With Normal Force

The design for bending and bending with normal force is carried out using a detailed section analysis according to SIA 262, Section 4.3.2.3, where the coefficient for the concrete compressive strength  $k_c = 1.0$  is assumed. As a simplification, calculations are performed with  $k_s = 1.05$  and  $\varepsilon_{ud} = 0.020$  for all reinforcing steel types. The design includes slab, plain stress and shell elements with perpendicular or inclined reinforcement as well as beams. The design internal forces are calculated in the same way as for the design according to EN 1992-1-1. For each internal force combination, the necessary reinforcement due to the equilibrium conditions of the reinforced concrete section is determined iteratively. The final result is derived from the extreme value of all calculated reinforcements.

You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes.

### Mode Standard

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tensile section to the greatest degree possible. Given ratios between certain reinforcement layers in the tension or compression zone are maintained as far as possible, unless this is deselected in the design specifications. For reasons of economy, if the steel strain  $\varepsilon_{sd}$  part of the steel strength  $f_{sd}$  is exceeded, compressive reinforcement is determined. The required transverse reinforcement of slab as per Section 5.5.3.2 is considered during design according to user specification. However, the provision for horizontal reinforcement of walls as per Section 5.5.4.11 is not taken into account.

### Mode Symmetrical

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected unless this is deselected in the design specifications.

### Mode Compression member

The design is performed symmetrically. Additionally, the minimum reinforcement of 0.6% required according to Section 5.5.4.2 of the standard is determined. This calculation is based on the entire area of the concrete section. For beams and design objects with tendons with bond the prestressing steel area is taken into account.

## Design for Lateral Force

The design for lateral force includes the determination of lateral force reinforcement and the check of the resistance of the concrete compressive field according to SIA 262, Chapter 4.3.3. The following special conditions apply:

- The angle of the diagonal tensile reinforcement is assumed to be  $90^\circ$ .
- The minimum reinforcement according to Section 5.5.2.2 of the standard is included in the calculated stirrup reinforcement.
- For beams and design objects, the shear design is performed separately for the  $Q_y$  and  $Q_z$  lateral forces.
- Slab and shell elements are designed for the lateral force  $q_r = \sqrt{q_x^2 + q_y^2}$ .
- There is **no** limitation on the check locations according to Section 4.3.3.2.1 or 4.3.3.4.1 as well as **no** reduction of the action from loads near supports according to Section 4.3.3.2.7.
- For beams and design objects, the decisive values of the equivalent rectangle are determined by the user independently of the normal section geometry. The coefficients for calculating the inner lever arm  $z$  based on the effective width and effective height according to Section 4.3.3.4.2 must also be specified.
- For area elements, the calculation is normally performed with the lever arm  $z = 0.9 d$ .
- The coefficient  $k_c$  for the concrete compressive strength defined by the user is taken into account.

Formulas used from the standard:

### 4.3.3.2 Components without Lateral Force Reinforcement

4.3.3.2.1 Lateral force resistance of slabs without lateral force reinforcement

$$v_{Rd} = k_d \tau_{cd} d_v \quad (35)$$

$$k_d = \frac{1}{1 + \varepsilon_v d k_g} \quad (36)$$

$$k_g = \frac{48}{16 + D_{\max}} \quad (37)$$



$D_{\max}$  Maximum grain in the concrete.  $D_{\max} = 0$  for lightweight concrete.

$\tau_{cd}$  Design value of the shear stress limit.

$$\tau_{cd} = \frac{0.3 \eta_t \sqrt{f_{ck}}}{\gamma_c} \quad (3)$$

$d$  Static height, average static height given several reinforcement layers [mm].

$d_v$  Effective static height for absorbing the lateral force  $d_v = d$  is assumed.

$\eta_t$  Coefficient to take into account the effect of the load duration on the concrete strength. In accordance with Section 4.2.1.3,  $\eta_t = 1$  is assumed.

4.3.3.2.2 If the bending reinforcement remains in the elastic state:

$$\varepsilon_v = \frac{f_{sd}}{E_s} \frac{m_d}{m_{Rd}} \quad (38)$$

If plastic deformation of the bending reinforcement cannot be ruled out:

$$\varepsilon_v = 1.5 \frac{f_{sd}}{E_s} \quad (39)$$

$m_d$  Design value of the bending moment.

$m_{Rd}$  Design value of the bending resistance.

4.3.3.2.3 The strain  $\varepsilon_v$  is to be increased by 50% if the longitudinal reinforcement in the check area is staged.

4.3.3.2.4 The strain  $\varepsilon_v$  is to be increased with  $1/(\sin^4 \vartheta + \cos^4 \vartheta)$ , where  $\vartheta$  is the Angle between the principal reinforcement and principal direction of the lateral force ( $\arctan q_y / q_x$ ).

#### 4.3.3.3 Components with Lateral Force Reinforcement (Standard design)

4.3.3.4.3 Resistance of a vertical lateral force reinforcement

$$V_{Rd,s} = A_{sw} / s \cdot z \cdot f_{sd} \cot \alpha \quad (43)$$

4.3.3.3.2 The compression field angle can freely be chosen between the following limits:

$$\alpha_{\min} \leq \alpha \leq 45^\circ \quad (40)$$

The minimum compression field angle  $\alpha_{\min}$  is:

-  $\alpha_{\min} = 30^\circ$  in the normal case,

-  $\alpha_{\min} = 25^\circ$  if a significant longitudinal compressive force acts on the web,

-  $\alpha_{\min} = 40^\circ$  if a longitudinal tensile force acts on the web or plastic deformations of the chord in the observed part of the girder are expected.

4.3.3.4.6 Resistance of the concrete compressive field

$$V_{Rd,c} = b_w \cdot z \cdot k_c \cdot f_{cd} \cdot \sin \alpha \cdot \cos \alpha \quad (45)$$

$k_c$  Coefficient for determination of the concrete strength according to Section 4.2.1.7.

4.3.3.4.12 Longitudinal tensile force as a result of lateral force

$$F_{tVd} = V_d (\cot \alpha - \cot \beta) \quad (50)$$

$\beta$  Angle of the stirrup reinforcement (here  $90^\circ$ ).

$F_t = 0.5 \cdot |V_d| \cot \alpha$  (additional tensile force in the longitudinal reinforcement)

5.5.2.2 Minimum stirrup reinforcement of girders

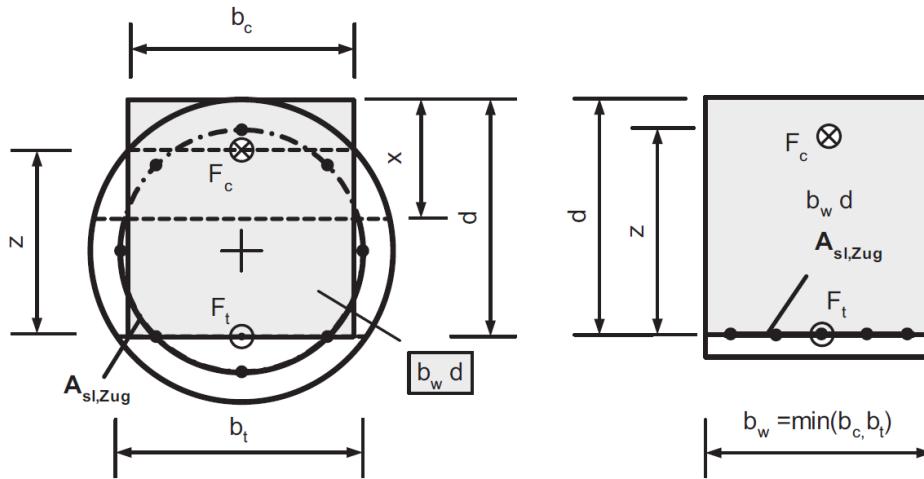
$$\rho_w = \frac{A_{sw}}{s b_w} \geq 0,001 \sqrt{\frac{f_{ck}}{30}} \frac{500}{f_{sk}} \quad (110)$$

5.5.3.4 The reinforcement content of slabs also has to satisfy the condition of Equation (110), if shear reinforcement is necessary.

### Lateral force design for circular and annular cross-sections according to Bender et al.

For circular and annular cross sections with annular single stirrups or helixes, the lateral force design is optionally carried out according to Bender et al. (2010) for the resulting shear force  $Q_r = \sqrt{Q_y^2 + Q_z^2}$ .

In its interpretation of 1 June 2012 of Chapter 10.3 of DIN 1045-1:2008, the German Committee for Structural Engineering (NABau) recommends using the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces for the effective width  $b_w$  (see following figure). The values for the width  $b_w$ , the effective height  $d$  and the inner lever arm  $z$  are defined in the cross-section dialog.



Definition of the effective width  $b_w$  as per NABau (2012) [Fig. from: Bender et al. (2006), p. 87]

For structural members without shear reinforcement, the resistance  $V_{Rd,ct}$  is given according to Bender et al. (2006), Equ. (1), in accordance with DIN 1045-1:2008, Equ. (70). Therefore, the program uses the above equations (35) of SIA 262 with the selected value for  $b_w$ .

For structural members with shear reinforcement, the design is carried out according to Bender et al. (2010):

$$V_{Rd,sy} = \alpha_k \cdot A_{sw} / s_w \cdot f_{yd} \cdot z \cdot \cot \Theta \cdot \sin \alpha \quad (10)$$

$$V_{Rd,max} = \alpha_k \cdot b_w \cdot z \cdot \alpha_c \cdot f_{cd} \cdot \cot \Theta / [(\cot \Theta \cdot \cot \alpha)^2 + 1] \quad (11)$$

where

$\alpha_k$  is an efficacy factor, which is stress-dependent ( $0.715 \leq \alpha_k \leq 0.785$ ) according to Bender et al. (2010), p. 422, and can be assumed with the mean value  $\alpha_k = 0.75$ .

$A_{sw}$  is the section area of the lateral force reinforcement per length  $s_w$ .

$s_w$  is the distance of the lateral force reinforcement as measured along the component axis.

$b_w$  is the effective cross-section width.

$z$  is the inner lever arm.

$\Theta$  is the inclination of the concrete compressive struts.

$\alpha$  is the angle between the lateral force reinforcement and the component axis (helix inclination).

$f_{yd}$  is the design value for the yield strength of the lateral force reinforcement.

$f_{cd}$  is the design value of the concrete compressive strength.

$\alpha_c$  is a coefficient to account for the stress state in the compression chord.

The additional tensile force in the longitudinal reinforcement due to lateral force  $Q_r$  is determined according to equation (50) of the standard. In case of simultaneous loading by lateral force and torsion, the torsion design is carried out according to the standard for vertical stirrups assuming a square torsion box.

The design results are stored separately from the standard design results.

## Design for Torsion and Combined Loads

The design for torsion is carried out according to SIA 262, Chapter 4.3.5. The stirrup reinforcement, the longitudinal reinforcement and the resistance of the concrete compressive field are determined. According to the standard, for simultaneously acting lateral force and torsional load, the combined utilization of the concrete compressive field

$$\frac{V_d}{V_{Rd,c}} + \frac{T_d}{T_{Rd,c}}$$

is checked. The ideal hollow section on which this design is based is defined by the user independently of the normal section geometry.

Formulas used from the standard:

### 4.3.5 Torsion and combined load

#### 4.3.5.2 Equivalent plain stress element forces

$$V_{d,i} = \frac{T_d}{2A_k} z_i \quad (54)$$

$V_{d,i}$  Design value of the lateral force in the plain stress element  $i$ .

$T_d$  Design value of the torsional moment.

$A_k$  Section area according to Figure 19.

$z_i$  Lever arm of the longitudinal force in the plain stress element  $i$ .

#### 4.3.5.3 Effective plain stress element thickness of solid sections

$$t_k \leq d_k / 8 \quad (55)$$

$d_k$  Maximum diameter that can be placed inside area  $A_k$ .

## Punching Shear

The check of the load-bearing safety with respect to punching shear is carried out according to SIA 262, Chapter 4.3.6. The necessary punching reinforcement and longitudinal reinforcement to protect against collapse are determined.

The following special conditions apply:

- The factor  $k_e$  is determined according to Section 4.3.6.2.4.
- The average static height  $d$  results from the input parameters  $d_x$  and  $d_y$  with  $d = (d_x + d_y) / 2$ . The static height  $d_v$  for absorption of lateral force according to Section 4.3.6.2.1 is to be specified by the user.
- The action can be entered directly or taken from the analyzed design situation at the ultimate limit state. In this case,  $V_d$  is set to the maximum support force  $R_z$  for each corresponding action combination. The moment for consideration of the load eccentricity  $M_d$  results from the support moments  $M_x$  and  $M_y$  with  $M_d = \sqrt{M_x^2 + M_y^2}$ . It is therefore important that the support is oriented correctly. The least favorable combination is logged.
- In accordance with Section 4.3.6.2.6 soil pressures within the check perimeter can be used to reduce the design value of the lateral force  $V_d$ . This is taken into account if the value  $\sigma_0$  is entered in the punching shear dialog.

The punching shear check is fulfilled when:

$$V_d \leq V_{Rd}$$

The following formulas are used:

#### 4.3.6.2.4 Reduction factor for the circumference of the check perimeter

$$k_e = \frac{1}{1 + \frac{e_u}{b}} \quad (56)$$

$e_u$  Load eccentricity of the column with:

$$e_u = |M_d / V_d|$$

$M_d$  Moment for calculation of the load eccentricity.  
 $b$  Diameter of a circle with the same area as the area of the column.

#### 4.3.6.3.1 Punching resistance without punching reinforcement

$$V_{Rd,c} = k_r \tau_{cd} d_v u \quad (57)$$

with

$\tau_{cd}$  Design Value of the Shear Stress Limit

$$\tau_{cd} = \frac{0.3 \eta_t \sqrt{f_{ck}}}{\gamma_c} \quad (3)$$

$d_v$  Static height.

$u$  Circumference of the perimeter under consideration according to Figure 21 and 22.

$\eta_t$  Coefficient to take into account the effect of the load duration on the concrete strength. In accordance with Section 4.2.1.3,  $\eta_t = 1$  is assumed.

$$4.3.6.3.2 \quad k_r = \frac{1}{0.45 + 0.18 \psi d k_g} \leq 2 \quad (58)$$

with

$$4.3.6.4.1 \quad \psi = 1.5 \frac{r_s}{d} \frac{f_{sd}}{E_s} \left( \frac{m_{sd}}{m_{Rd}} \right)^{3/2} \quad (59)$$

$r_s$  Radial distance between column axis and moment zero point according to Section 4.3.6.4.4:  
 $r_s = \text{Maximum} (0.22 l_x; 0.22 l_y)$

$d$  Effective static height.

$f_{sd}$  Design value for the yield criterion of reinforcing steel.

$E_s$  Elasticity modulus of the reinforcing steel.

$m_{sd}$  Average moment in the support band, determined for approximation level 2 according to Section 4.3.6.4.7 with Equations (61) to (64).

$m_{Rd}$  Average value of the bending resistance in the support band.

$k_g$  Coefficient for consideration of the maximum grain diameter according to Section 4.3.3.2.1.

#### 4.3.6.5.4 Punching resistance with punching reinforcement

$$V_{Rd,s} = \sum A_{sw} k_e \sigma_{sd} \sin \beta \quad (67)$$

with

$A_{sw}$  Area of the punching reinforcement.

$\sigma_{sd}$  Design value of the stress in the punching reinforcement

$$\sigma_{sd} = \frac{E_s \Psi}{6} \left( 1 + \frac{f_{bd}}{f_{sd}} \frac{d}{\varnothing_{sw}} \right) \leq f_{sd} \quad (68)$$

with

$f_{bd}$  Design value of the bond stress

$$f_{bd} = \frac{1.4 f_{ctm}}{\gamma_c} \quad (103)$$

$\varnothing_{sw}$  Diameter of the punching reinforcement elements. The limit value of Table 20 is used in the program.

4.3.6.5.2 The design of punching reinforcement is carried out using the following lateral force

$$V_{d,s} = V_d - V_{Rd,c} \quad (65)$$

$$V_{d,s} \geq V_d / 2 \quad (66)$$

4.3.6.5.7 Crack resistance of the concrete compressive diagonals

$$V_{Rd,c} = 2 k_r \tau_{cd} d_v u \leq 3.5 \tau_{cd} d_v u \quad (67)$$

4.3.6.5.9 An additional check at the perimeter  $u_{\text{out}}$  is performed to determine the size of the area with punching reinforcement.

$$V_d \leq V_{\text{Rd,c,out}}$$

$$V_{\text{Rd,c,out}} = k_r \tau_{\text{cd}} d_v u_{\text{out}}$$

For the static height  $d_v$ , the value entered in accordance with Section 4.3.6.2.1 is used for the sake of simplicity.

#### 4.3.6.7 Protection against collapse

Determination of the required longitudinal reinforcement on the side of the slab under bending compression.

$$V_{\text{Rd,res}} = \sum (A_s f_{\text{sd}} k_{\beta}) \leq 1.7 \tau_{\text{cd}} d_{\text{int}} u_{\text{int}} \quad (70)$$

$$u_{\text{int}} = \sum (s_{\text{int}} + \pi/2 d_{\text{int}}) \quad (71)$$

with

$d_{\text{int}}$  Distance between bending reinforcement and reinforcement against collapse according to Figure 26.

$s_{\text{int}}$  Distance between the outer reinforcing bars according to Figure 26.

$k_{\beta}$  Coefficient according to Table 12.

## Check Against Fatigue

### Fatigue of Longitudinal Reinforcement, Shear Reinforcement and Prestressing Steel

The fatigue check is carried out according to SIA 262, Chapter 4.3.8. The steel stresses are calculated for longitudinal reinforcement from bending and longitudinal force as well as for prestressing steel in beams and design objects under the assumption of a cracked concrete section. For shear and longitudinal reinforcement from lateral force and torsion, the stresses are calculated according to Section 4.3.8.1.7 based on a truss model with the strut angle  $\tan \alpha_{\text{fat}} = \sqrt{\tan \alpha} \leq 1$ .

Where  $\alpha$  is the angle between the concrete compression struts and the beam axis used in the corresponding ultimate limit state design. The prestressing steel stresses in area elements are determined at the uncracked concrete section. Tendons without bond and external tendons are not checked.

The check is carried out for the long-time strength according to Section 4.3.8.2.3, Equation (86):

$$\Delta\sigma_{\text{sd}}(Q_{\text{fat}}) \leq \Delta\sigma_{\text{sd,D}} \quad (86)$$

with

$$\Delta\sigma_{\text{sd,D}} \cong 0.8 \cdot \Delta\sigma_{\text{sd,fat}} \quad (86)$$

$\Delta\sigma_{\text{sd}}(Q_{\text{fat}})$  Stress range of fatigue action  $Q_{\text{fat}}$ .

$\Delta\sigma_{\text{sd,fat}}$  Design value of the fatigue strength according to Table 13.

In case of bonded prestressed tendons the stresses in the reinforcing steel are increased by the factor  $k_{\xi}$  according to Equation (81) to take into account the varying bond behavior of reinforcing and prestressing steel.

The values for  $\Delta\sigma_{\text{sd,fat}}$  and  $k_{\xi}$  are specified by the user in the Section dialog.

#### Calculation method

The maximum from the robustness, crack and bending reinforcement is taken as the existing bending reinforcement. If as a result the load from the fatigue action in state II cannot be absorbed, the design will be repeated using the existing reinforcement and the check internal forces.

The maximum stress range per steel layer that results from the strain state in state II or the truss model is determined separately for each check situation. Multiplying the coefficient  $k_{\xi}$  yields the stress range  $\Delta\sigma_{\text{sd}}$ . If for longitudinal and shear reinforcement this range exceeds the permitted stress range according to Equation (86), the necessary reinforcement will be iteratively increased until the check succeeds for all situations. In the *Symmetrical* and *Compression member* design modes the longitudinal reinforcement is applied at all predefined locations. This will not affect the predefined relationships between the individual reinforcement layers.

The decisive reinforcement used for the check, which may have been increased, is recorded in the check log and saved for graphical representation.

## Fatigue of Concrete Under Compressive Stress

The fatigue check for concrete that is subject to compressive stress is performed for bending and longitudinal force at the cracked section. This check takes into account the final longitudinal reinforcement and may include an increase applied during the fatigue check for reinforcing steel. The struts of components subject to lateral force stress are not analyzed.

The check according to Section 4.3.8.3.1 is proved, if Equation (91) is fulfilled:

$$|\sigma_{cd}|_{\max} \leq 0.5 k_c f_{cd} + 0.45 |\sigma_{cd}|_{\min} \leq 0.9 k_c f_{cd} \quad (91)$$

with

$|\sigma_{cd}|_{\max}$ ,  $|\sigma_{cd}|_{\min}$  Design values of the maximum and minimum concrete compressive stress for the fatigue action  $Q_{fat}$ .  
In the case of tensile stresses,  $|\sigma_{cd}|_{\min}$  is assumed to be zero.

$k_c$  The reduction coefficient for the concrete compressive strength is assumed to be  $k_c = 1.0$  according to Section 4.2.1.7.

## Special Characteristic of Shell Structures

In shell structures the strain state at the cracked concrete section under general stress cannot be determined unambiguously. The design is therefore carried out separately for the reinforcement directions x and y with the design internal forces from Wolfensberger/Thürlimann or Rüschi as described above. The reinforcement calculated in this manner yields a reliable load-bearing capacity.

When calculating the stress range for reinforcing steel and concrete, this method can lead to unrealistic results in the case of torsional or shear stresses as shown in the following example:

Assume two identical sets of slab internal forces:

Set	mx [kNm/m]	my [kNm/m]	mxy [kNm/m]
1	300	200	100
2	300	200	100

According to Wolfensberger/Thürlimann, this results in design variants for the x direction:

Set	Variant	m [kNm/m]
1	1	$mx +  mxy  = 400$
	2	$mx -  mxy  = 200$
2	1	$mx +  mxy  = 400$
	2	$mx -  mxy  = 200$

The torsional moments generate a variation of the design moments and thus a calculatory stress range. This may lead to a necessary reinforcement increase in the fatigue check due to apparent overstressing. For normal design forces, this applies correspondingly to the shear forces.

Selecting **Limit design variants** in the Section dialog allows you to avoid the described effect. In this case only the corresponding variants are compared when determining the stress range, i.e. only the first and second variants of both sets in this example. Assuming constant stress, the stress range is thus correctly determined to be zero.

**This alternative, however, does not ensure that all conceivable stress fluctuations are analyzed. You should therefore be particularly careful when assessing the results. For this purpose the detailed log indicates the main variants and design internal forces used for the check.**

When determining the design internal forces according to Rüschi for inclined reinforcement, the described relationships apply accordingly.

## Checks in the Serviceability Limit States

The following checks are performed:

- Limiting the concrete compressive stresses (SIA 262, Section 3.1.2.6.3 and 4.1.5.2.4).
- Minimum reinforcement against brittle failure (robustness reinforcement) (Chapter 4.4.2).
- Crack reinforcement in case of restraint (Chapter 4.4.2).
- Crack width limitation (Chapter 4.4.2).

In accordance with SIA 260, Section 4.4.4.4, the following combinations are taken into account in the serviceability limit states:

- Rare situations

$$E_d = E \{ G_k, P_k, \psi_{0,i} \cdot Q_{k,i} \} \quad (20)$$

- Frequent situations

$$E_d = E \{ G_k, P_k, \psi_{1,1} \cdot Q_{k,1}, \psi_{2,i} \cdot Q_{k,i} \} \quad (21)$$

- Quasi-continuous situations

$$E_d = E \{ G_k, P_k, \psi_{2,i} \cdot Q_{k,i} \} \quad (22)$$

## Limiting the Concrete Compressive Stresses

The concrete compressive stress check is carried out in state I. For area elements the concrete stresses are calculated at the gross section. For beams and design objects, the bending stress is calculated

- without internal tendons at the gross section,
- with internal tendons without bond at the net section,
- with internal tendons with bond for situations before being grouted at the net section or otherwise at the ideal section.

In accordance with SIA 262, Section 3.1.2.6.3, the influence of the load level on the creep behavior is to be taken into account for concrete stresses  $\sigma_c > 0.45 f_{ck}$ . If selected in the section dialog, this stress limit is verified for the quasi-continuous combination based on EN 1992-1-1, Section 7.2 (3).

In prestressed concrete components the concrete compressive stresses during and after the prestressing process may not exceed the value  $0.6 f_{ck}(t)$  at any location of the structure out of the anchoring area according to SIA 262, Section 4.1.5.2.4. If the concrete compressive stress also exceeds the value  $0.45 f_{ck}(t)$ , the nonlinearity of the creep must be taken into account.  $f_{ck}(t)$  indicates the characteristic value of the concrete compressive strength at time  $t$  when the prestressing is introduced.

The program assumes the time of introducing the prestressing to coincide with situation G1+P. If a quasi-continuous situation G1+P is defined, the concrete stress is checked against the limit value  $0.45 f_{ck}(t)$  or  $0.6 f_{ck}(t)$  for this situation depending on the user's specification. The value for  $f_{ck}(t)$  is also defined in the dialog.

## Minimum Reinforcement Against Brittle Failure

According to SIA 262, Chapter 4.4.2, brittle failure of concrete in the tensile zone must be prevented by installation of a minimum reinforcement. The minimum reinforcement (Robustness reinforcement) is calculated for the crack moment using the design values of the tensile strength  $f_{ctd}$  and the steel strength  $f_{sd}$ :

$$A_s = M_{cr} / (f_{sd} \cdot z)$$

with

$M_{cr}$  Crack moment by which a tensile stress of  $f_{ctd}$  occurs without prestressing effect at the section edge.

$z$  Lever arm of internal forces.

The crack moment results in  $M_{cr} = W_c \cdot f_{ctd}$ , the lever arm  $z$  of the internal forces is assumed to be  $0.9 \cdot d$  for the sake of simplicity. In accordance with Section 4.4.2.3.6 the minimum reinforcement is to be placed in the tensile zone of the components. Referring to EN 1992-2, Section 6.1 (110), the rare action combination is used to determine the tensile zone.

In this process the statically undetermined prestressing effect should be taken into account rather than the statically determined prestressing effect. The program determines all stresses at the gross section. The statically determined prestressing effect can only be subtracted for beams and design objects. For area elements the prestress is alternatively reduced by a user-defined reduction factor.

The calculated reinforcement is evenly distributed to the reinforcement layers in the tensile zone. In the design mode *symmetrical* reinforcement is also applied to the remaining layers. This will not affect the predefined relationships between the individual reinforcement layers. For sections with mode *compression member* the robustness reinforcement is not checked because minimum reinforcement is already determined during the design for bending with longitudinal force.

## Crack Reinforcement in Case of Restraint

The installation of a minimum reinforcement can be used to limit the crack width in case of imposed or obstructed deformations according to SIA 262, Section 4.4.2.3.7. In accordance with Section 4.4.2.3.6 the minimum reinforcement is to be placed in the tensile zone of the components. The tensile zone is defined by the user in the section dialog by selection of a restraint action (tension at the top/bottom, central tension).

The minimum reinforcement is calculated with the following equation:

$$A_s = k_s \cdot f_{ctd} \cdot A_{ct} / \sigma_{s,adm}$$

with

$k_s$  Coefficient for consideration of stress distribution prior to crack formation:

$$k_s = 1.0 \quad \text{for central restraint}$$

$$k_s = 0.4 \quad \text{for bending restraint of rectangular sections, deduced from SIA D 0182, Eq. (10.7)}$$

$$k_s = 0.9 \cdot F_{cr} / A_{ct} / f_{ctd} \geq 0.5 \quad \text{in all other cases according to EN 1992-1-1, Eq. (7.3)}$$

with the tensile force  $F_{cr}$  in the tension chord in state I directly before crack formation with the edge stress  $f_{ctd}$ .

The tensile force is calculated by integrating the tensile stresses over the area  $A_{ct}$ .

$f_{ctd}$  Design value of the concrete tensile strength. Depending on the selection in the section dialog one of the following equations is used:

$$f_{ctd} = k_t \cdot f_{ctm} \quad (98)$$

$$f_{ctd} = k_t \cdot f_{ctk0.95} \quad (100)$$

with

$$k_t = \frac{1}{1 + 0.5t} \quad (99)$$

$t$  Smallest dimension of the observed tension chord [m]. For slabs and rectangular sections under bending load,  $t = h/3$  applies.

The coefficient  $k_t$  can be defined in the section dialog. The suggested value is  $k_t = 1.0$ .

$A_{ct}$  Area of the concrete tensile zone at initial crack formation in state I.

$\sigma_{s,adm}$  Reinforcing steel strength according to SIA 262:2017, Equation (100a), depending on the selected requirement (Table 17) and the bar diameter of the longitudinal reinforcement.

$$\sigma_{s,adm} = \sqrt{\frac{9 \cdot E_s \cdot f_{ctm} \cdot w_{nom}}{\varnothing_s}} \quad (100a)$$

$w_{nom}$  Nominal crack width in the center of gravity of the reinforcement.

$\varnothing_s$  Diameter of a reinforcing bar.

If selected by the user, bonded prestressing steel within the tensile zone can optionally be taken into account for  $A_s$  according to Section 4.4.2.3.10.



## Crack Width Limitation

Limitation of crack width is performed by comparing the existing reinforcing steel stresses with the permitted steel stresses according to SIA 262:2017, Table 17 and Equation (100a). The reinforcing steel stresses are calculated in state II for the maximum of robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check. Depending on the selected requirement, the reinforcing steel stresses may not exceed the following values:

Requirement	Action combination	
	frequent	quasi-continuous
normal	-	$\sigma_s \leq f_{sd}$
raised	$\sigma_s \leq f_{sd} - 80 \text{ N/mm}^2$	$\sigma_s \leq \sigma_{s,adm}$ for $w_{nom} = 0.5 \text{ mm}$
high	$\sigma_s \leq f_{sd} - 80 \text{ N/mm}^2$	$\sigma_s \leq \sigma_{s,adm}$ for $w_{nom} = 0.2 \text{ mm}$

If the check cannot be proved with the existing reinforcement, the crack reinforcement will be increased.

## Limiting Deformations

According to SIA 262, Chapter 4.4.3, the deformations of a component or structure may not impair its proper functioning or appearance. Considering that, the deformation should not exceed the limits specified in SIA 260.

The InfoCAD program system allows you to perform a realistic check as part of a nonlinear system analysis for beam and shell structures that takes geometric and physical nonlinearities into account. The resistance of the tendons with bond is currently not included in the calculation.

Editing is performed in the following steps:

- Define the check situation with the *Load group function* in the Load dialog through grouping the decisive individual load cases. The variable loads must first be weighted with the combination coefficients of the combination specified in SIA 260.
- Select the check load cases in the *Nonlinear Analysis / Serviceability* dialog of the analysis settings for the FEM or framework analysis.
- Set the reinforcement determined in the ultimate limit state in the *Start reinforcement* selection field (maximum from bending, robustness, crack check and fatigue).
- Perform the FEM or framework analysis to determine the deformations in state II.
- Check the system deformations displayed graphically or in tabular form.

For a detailed description of the nonlinear system analysis, refer to the relevant chapter of the manual.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The resulting bending, minimum- and crack reinforcement, the decisive maximum value and the stirrup and torsion reinforcement are provided for the graphical representation as well.

The log shows the design internal forces and necessary reinforcements, checked stresses or crack widths at each result location. If the permissible limit values are exceeded, they are reported as warnings and indicated at the check location. The detailed log also lists the decisive combination internal forces of all design situations.

### Stresses for beams and design objects

$\sigma_x$	Longitudinal stress in the concrete compressive stress check [MN/m <sup>2</sup> ].
$\sigma_s, \Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd}, \Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete under longitudinal compression [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,y}, \Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from $Q_y$ and $Q_z$ [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,T}, \Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and for longitudinal torsion reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{perm}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

**Stresses for area elements**

$\sigma_x, \sigma_y$	Longitudinal stress in x or y direction [MN/m <sup>2</sup> ].
$\sigma_{sx}, \Delta\sigma_{sx}$	Stresses and stress ranges for reinforcing steel in the x direction [MN/m <sup>2</sup> ].
$\sigma_{sy}, \Delta\sigma_{sy}$	Stresses and stress ranges for reinforcing steel in the y direction [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd,x}, \Delta\sigma_{cd,x}$	Stresses and stress ranges in the concrete fatigue check under longitudinal compression in the x- and y-direction [MN/m <sup>2</sup> ].
$\sigma_{cd,y}, \Delta\sigma_{cd,y}$	
$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{perm}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

**Bending Reinforcement**

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.
$a_{sq}$	Meridian reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.
$a_{sD}$	Ring reinforcement [cm <sup>2</sup> /m] for axisymmetric shell elements.

**Reinforcement from lateral force**

$a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area and axisymmetric shell elements.
$A_{sb,y}$	Stirrup reinforcement of beams from $Q_y$ [cm <sup>2</sup> /m].
$A_{sb,z}$	Stirrup reinforcement of beams from $Q_z$ [cm <sup>2</sup> /m].
$a_{sq}$	Longitudinal reinforcement from the lateral force design of area elements [cm <sup>2</sup> /m].
$A_{sl,y}$	Longitudinal reinforcement of beams from $Q_y$ [cm <sup>2</sup> ].
$A_{sl,z}$	Longitudinal reinforcement of beams from $Q_z$ [cm <sup>2</sup> ].

**Torsion reinforcement**

$A_{sb}$	Stirrup reinforcement of beams from torsion [cm <sup>2</sup> /m].
$A_{sl,T}$	Longitudinal reinforcement of beams from torsion [cm <sup>2</sup> ].

**Design values**

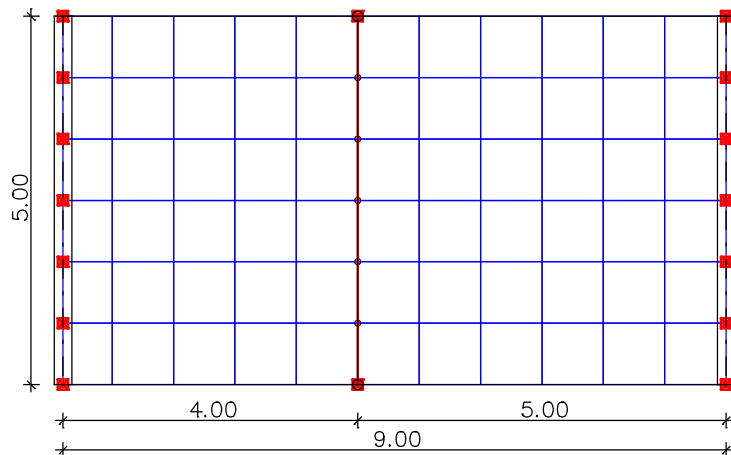
$V_{yRdc}$	Resistance of the concrete compressive field with respect to $Q_y$ [kN].
$V_{zRdc}$	Resistance of the concrete compressive field with respect to $Q_z$ [kN].
$Q/V_{Rdc} + M_x/T_{Rdc}$	Utilization of the concrete compressive field as a result of combined load from lateral force and torsion.
$T_{Rdc}$	Resistance of the concrete compressive field with respect to $M_x$ [kNm].

## Examples

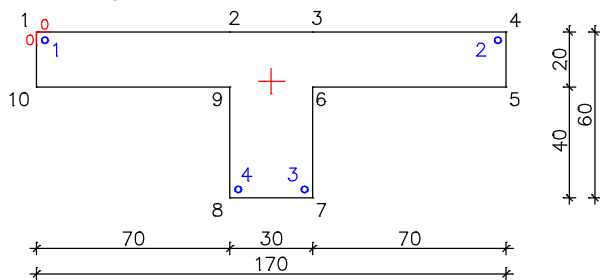
### Slab With Downstand Beam

In this example a rectangular slab ( $d = 20$  cm,  $SC25/30$ ) with a downstand beam is analyzed. This joint-supported slab will be subjected to a traffic load of  $10$  kN/m<sup>2</sup>.

The necessary design specifications and the stirrup reinforcements calculated for the slab and the downstand beam are shown.



The following image shows the dimensions of the downstand beam. The axis distance of the reinforcing steel from the section edge is  $3$  cm. The dead load of the downstand beam is reduced by the share attributed to the slab.



#### SIA 262 actions

##### Standard design group

##### G - Dead load

$\Gamma_{sup} / \Gamma_{inf} = 1.35 / 0.8$

Load cases

1 Dead load

##### QN - Imposed load, traffic load

$\Gamma_{sup} / \Gamma_{inf} = 1.5 / 0$

Combination coefficients for: Buildings  
Working load - category A - floor spaces  
 $\Psi_{1.0} / \Psi_{1.1} / \Psi_{1.2} = 0.7 / 0.5 / 0.3$

Load cases 1. Variant, inclusive

2 Traffic span 1  
3 Traffic span 2

##### 1. Permanent and temporary situation

Final state  
G - Dead load  
QN - Imposed load, traffic load

##### 1. Rare (characteristic) situation

Final state  
G - Dead load  
QN - Imposed load, traffic load

##### 1. Frequent situation

Final state  
G - Dead load  
QN - Imposed load, traffic load

**1. Quasi-continuous situation**

Final state  
 G - Dead load  
 QN - Imposed load, traffic load

**Design overview**

Se.	Prestress of component	Requi-remment	Reinforc. M R B Q T	Fatig. S P C	Crack width	Comp.-stress
1	Not prestressed	raised	x + x x .	. . .	x	.
2	Not prestressed	raised	x + x x .	. . .	x	.

- (M) Nominal reinforcement against brittle failure (robustness reinforcement).
- (R) Crack reinforcement from restraint (x), required reinf. due to load (+).
- (B) Flexural reinforcement at ultimate limit state.
- (Q) (Nominal)lateral force reinforcement at ultimate limit state.
- (T) Torsional reinforcement at ultimate limit state.
- (S) Reinforcing steel at fatigue check.
- (P) Prestressing steel at fatigue check.
- (C) Concrete at fatigue check.

**Settings for flexural and shear reinforcement**

M,N Design mode for bend and longitudinal force: (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 Red. Reduction factor of prestress for determining the tensile zone for distribution of reinf. against brittle failure for area elements.  
 Fac. as Factor for bending reinf. of slabs in secondary dir. per 5.5.3.2.  
 eta.t Coefficient for concrete strength as per 4.2.1.3.  
 fsk Quality of stirrups [MN/m<sup>2</sup>].  
 Alpha Compression field angle [°].  
 kc Reduction factor for concrete strength as per 4.2.1.7.  
 Slab Beams are designed like slabs.  
 eps.v Base value of strain as per 4.3.3.2.2 [o/oo].  
 Dmax Max. grain of concrete as per Eq. (37) [mm].  
 Long. Longitudinal reinforcecent is graded as per 4.3.3.2.3.

Se.	Concrete	Des. for M,N	Red. pre-str.	Fac. as	eta.t	fsk [MN/m <sup>2</sup> ]	Alpha [°]	kc	Des. like slabs	Base eps.v	Dmax [mm]	Long. reinf. graded
1	SC25/30	ST	.	0.20	1.00	500	45.00	0.55	.	3.00	32	.
2	SC25/30	ST	.	.	1.00	500	45.00	0.55	.	.	.	.

**Shear sections**

bw.nom Nominal width of the prestressed section according to 4.3.3.3.5.  
 h.nom Nominal height of the prestressed section according to 4.3.3.3.5.  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d according to 4.3.3.4.2.  
 z1, z2 Dimensions of the ideal hollow section for torsion as per 4.3.5.1.  
 tk Thickness of the ideal hollow section.

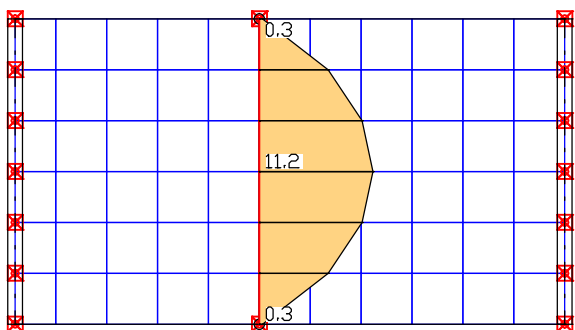
Se.	Width [m]	Eff. width	Height[m]	Eff.height	Torsion section [m]
	bw	bw.nom	bn [m]	kd	z1 z2 tk
1	1.000	.	.	0.200	0.170 0.90 . . .
2	0.300	.	0.270 0.90	0.600	0.570 0.90 0.540 0.240 0.060

**Settings for crack control**

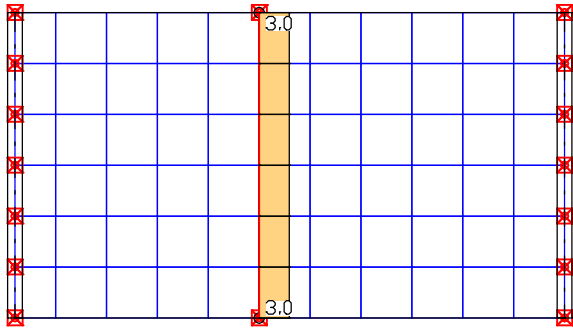
ds Max. given bar diameter of the reinforcing steel as per Fig. 31  
 fctd Design value of concrete tensile strength.  
 kt Reduction factor for concrete tensile strength.

Se.	Type of restraint	ds [mm]	kt	fctd	Regard pre-str. steel
1	.	10	1.00	kt*fctm	.
2	.	10	1.00	kt*fctm	.

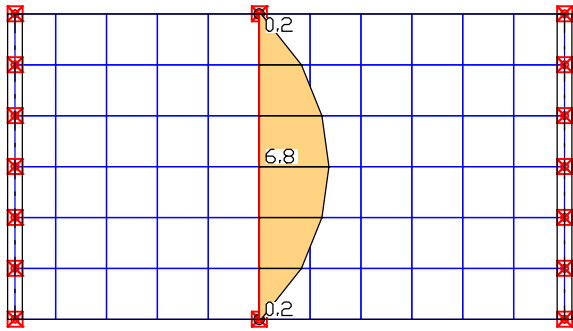
The calculated reinforcements are shown in the illustrations below.



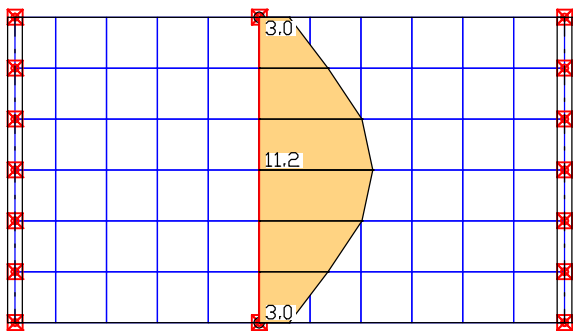
Longitudinal reinforcement of the beams in the ultimate limit state [cm<sup>2</sup>]



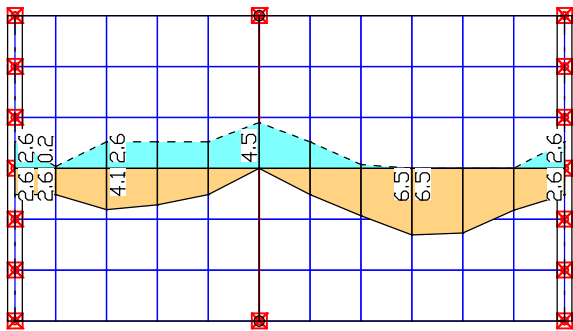
Longitudinal reinforcement of the beams to ensure robustness [cm<sup>2</sup>]



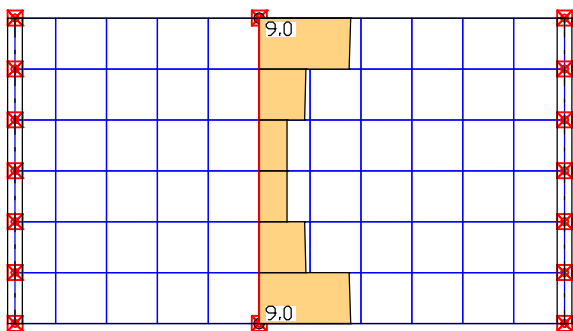
Longitudinal reinforcement of the beams to limit the crack width [cm<sup>2</sup>]



Maximum longitudinal reinforcement of the beams [cm<sup>2</sup>]



Maximum slab reinforcement in the intersection direction based on the robustness, crack width and design checks in the ultimate limit state [cm<sup>2</sup>/m]



Lateral force reinforcement of the beams [cm<sup>2</sup>/m]

An excerpt of the detailed log for the midspan of the downstand beam is provided below.

### Design of longitudinal reinforcement

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. against brittle failure acc. to Chapter 4.4.2 (Charact. C.).  
 fctd Design value of concrete tensile strength as per Chapter 4.4.1 [MN/m<sup>2</sup>]  
 zs,t/b Lever arm of inner strengths top/bottom with zs=0,9\*d [m]  
 S.s,adm Admissible steel stress as per Fig. 31 [MN/m<sup>2</sup>]  
 max Sc Maximum concrete edge stress from Charact. C. [MN/m<sup>2</sup>]  
 (R) Required reinforcement as per 4.4.2 for crack width limitation  
 Increase of reinforcement due to crack width check is marked by "!".  
 ds Maximal given steel diameter [mm]  
 (B) Design of reinforcement at ultimate limit state  
 In case of dominant bending, compression reinforcement is marked with "\*".  
 fck Concrete strength for design of reinforcement [MN/m<sup>2</sup>]

### Beam 70

#### Location 1

Beam 70, x = 0.00 m (Beam length 0.83 m)

Cross-section 2: Polygon - SC25/30

Steel 2; Design mode: Standard

(M) fctd=2.6; zs,o/u=0.513/0.513; Sigma.s,adm=435

(R) fctd=2.6; ds=10

(B) fck=25

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

#### 1. Characteristic (rare) combination (CC.1): G+QN, Final state

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]
Nx- :	0.00	69.95	0.00
Nx+ :	0.00	69.95	0.00
My- :	0.00	69.95	0.00
My+ :	0.00	196.53	0.00
Mz- :	0.00	69.95	0.00
Mz+ :	0.00	69.95	0.00

#### 1. Frequent combination (TC.1): G+QN, Final state

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]
Nx- :	0.00	69.95	0.00
Nx+ :	0.00	69.95	0.00
My- :	0.00	69.95	0.00
My+ :	0.00	133.24	0.00
Mz- :	0.00	69.95	0.00
Mz+ :	0.00	69.95	0.00

#### 1. Quasi-continuous combination (QC.1): G+QN, Final state

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]
Nx- :	0.00	69.95	0.00
Nx+ :	0.00	69.95	0.00
My- :	0.00	69.95	0.00
My+ :	0.00	107.92	0.00
Mz- :	0.00	69.95	0.00
Mz+ :	0.00	69.95	0.00

#### 1. Permanent and temporary comb. (PC.1): G+QN, Final state

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]
Nx- :	0.00	55.96	0.00
Nx+ :	0.00	55.96	0.00
My- :	0.00	55.96	0.00
My+ :	0.00	284.31	0.00
Mz- :	0.00	55.96	0.00
Mz+ :	0.00	55.96	0.00

### Design of longitudinal reinforcement

Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	max Sc [MN/m <sup>2</sup> ]	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	Situation
1	M	0.00	69.95	0.00	.	.	0.00	CC.1, Nx-
	R	0.00	0.00	0.00	0.00	.	0.00	-,-
	B	0.00	55.96	0.00	.	.	0.00	PC.1, Nx-
2	M	0.00	69.95	0.00	.	.	0.00	CC.1, Nx-
	R	0.00	0.00	0.00	0.00	.	0.00	-,-
	B	0.00	55.96	0.00	.	.	0.00	PC.1, Nx-
3	M	0.00	196.53	0.00	7.73	.	1.48	CC.1, My+
	R	0.00	133.24	0.00	.	.	3.41!	TC.1, My+
	B	0.00	284.31	0.00	.	.	5.58	PC.1, My+
4	M	0.00	196.53	0.00	7.73	.	1.48	CC.1, My+
	R	0.00	133.24	0.00	.	.	3.41!	TC.1, My+
	B	0.00	284.31	0.00	.	.	5.58	PC.1, My+

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to 5.5.2.2 is considered.

bw Effective width for calculation of shear stresses from Qz [m]  
 bn Statically effective width for shear design using Qy [m]  
 kb Factor to calculate the inner lever arm from bn  
 h Effective height for calculation of shear stresses from Qy and Mx [m]  
 d Statically effective height for shear design using Qz [m]  
 kd Factor to calculate the inner lever arm from d  
 fsk Strength of stirrup reinforcement [MN/m<sup>2</sup>]  
 kc Reduction coefficient to determine the concrete strength  
 Qy, Qz Lateral forces for design in y- and z-direction [kN]  
 VRd Lateral force resistance without lateral reinforcement [kN]  
 VRdc Lateral force resistance of the compression field [kN]  
 z Inner lever arm  $z=kb*bn$  resp.  $z=kd*d$   
 Alpha Compression filed angle [°]  
 Asb.y,z Req. stirrup reinforcement from aus Qy, Qz [cm<sup>2</sup>/m]  
 Asl.y,z Req. longitudinal reinforcement from Qy, Qz [cm<sup>2</sup>]

**Beam 70****Location 1**

Beam 70, x = 0.00 m (Beam length 0.83 m)  
 Cross-section 2: Polygon - SC25/30  
 bw/bn/kb=0.3/0.27/0.9; h/d/kd=0.6/0.57/0.9  
 fsk=500; kc=0.55

**1. Permanent and temporary comb. (PC.1): G+QN, Final state**

Concrete internal forces						
	Nx[kN]	My[kNm]	Mz[kNm]	Mx[kNm]	Qy[kN]	Qz[kN]
Nx- :	0.00	55.96	0.00	0.00	0.00	-5.41
Nx+ :	0.00	55.96	0.00	0.00	0.00	-5.41
My- :	0.00	55.96	0.00	0.00	0.00	-5.41
My+ :	0.00	284.31	0.00	0.00	0.00	-30.79
Mz- :	0.00	55.96	0.00	0.00	0.00	-5.41
Mz+ :	0.00	55.96	0.00	0.00	0.00	-5.41
Mx- :	0.00	55.96	0.00	0.00	0.00	-5.41
Mx+ :	0.00	94.43	0.00	0.00	0.00	-9.13
Qy- :	0.00	55.96	0.00	0.00	0.00	-5.41
Qy+ :	0.00	55.96	0.00	0.00	0.00	-5.41
Qz- :	0.00	284.31	0.00	0.00	0.00	-30.79
Qz+ :	0.00	55.96	0.00	0.00	0.00	-5.41

**Check of the shear reinforcement and the compressive struts**

Qy	: 0.00 kN	Alpha	: 45.00 °
Qy/VyRdc	: 0.00	z	: 0.24 m
Situation	: -, -	req.Asb.y	: 0.00 cm <sup>2</sup> /m
		req.As1.y	: 0.00 cm <sup>2</sup>
Qz	: -30.79 kN	Alpha	: 45.00 °
Qz/VzRdc	: 0.04	z	: 0.51 m
Situation	: PC.1,Qz-	req.Asb.z	: 2.74 cm <sup>2</sup> /m
		req.As1.z	: 0.35 cm <sup>2</sup>

**Check of crack widths**

The check is led by limiting the steel stress.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

(TC) Frequent, (QC) Quasi-continuous combination

ds Maximal given steel diameter [mm]  
 Sigma.c Maximal concrete edge stress in state I [MN/m<sup>2</sup>]  
 Sigma.s Reinf. steel stress in state II [MN/m<sup>2</sup>]  
 per. Permissible steel stress as per Table 17 and Fig. 31 [MN/m<sup>2</sup>]

**Beam 70****Location 1**

Beam 70, x = 0.00 m (Beam length 0.83 m)  
 Cross-section 2: Polygon - SC25/30  
 Requirement: raised; ds=10; fctm=2.6

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	0.460	0.850	0.178	0.0107	0.0828	0.0000

**1. Frequent combination (TC.1): G+QN, Final state**

Concrete internal forces			
	Nx[kN]	My[kNm]	Mz[kNm]
Nx- :	0.00	69.95	0.00
Nx+ :	0.00	69.95	0.00
My- :	0.00	69.95	0.00
My+ :	0.00	133.24	0.00
Mz- :	0.00	69.95	0.00
Mz+ :	0.00	69.95	0.00

1. Quasi-continuous combination (QC.1): G+QN, Final state

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]
Nx-	0.00	69.95	0.00
Nx+	0.00	69.95	0.00
My-	0.00	69.95	0.00
My+	0.00	107.92	0.00
Mz-	0.00	69.95	0.00
Mz+	0.00	69.95	0.00

Check of crack width for reinf. layer 3 (bottom) - Frequent combination

Nx [kN]	: 0.00	Sigma.c [MN/m <sup>2</sup> ]	: 5.24	
My [kNm]	: 133.24	As [cm <sup>2</sup> ]	: 5.58	
Mz [kNm]	: 0.00	Sigma.s [MN/m <sup>2</sup> ]	: 218.56	per. 355.00
Situation : TC.1, My+				

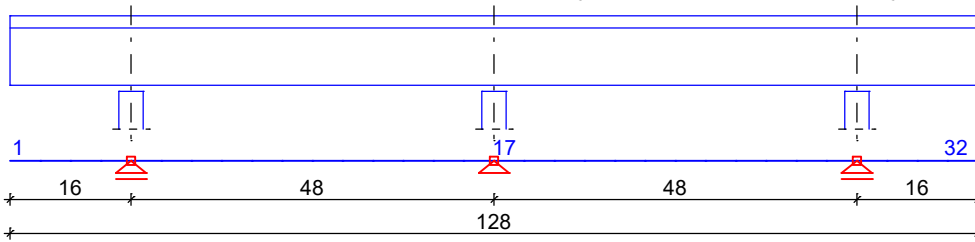
Check of crack width for reinf. layer 3 (bottom) - Quasi-continuous combination (w.nom = 0.5 mm)

Nx [kN]	: 0.00	Sigma.c [MN/m <sup>2</sup> ]	: 4.25	
My [kNm]	: 107.92	As [cm <sup>2</sup> ]	: 5.58	
Mz [kNm]	: 0.00	Sigma.s [MN/m <sup>2</sup> ]	: 176.93	per. 435.00
Situation : QC.1, My+				

## Prestressed Roof Construction

This example involves the wide-spanned roof construction of an entrance hall that is represented as a continuous girder over two spans with a double-sided cantilever. A T-beam is selected as the section. The figure below shows the system in longitudinal and lateral section view.

Limited prestressing with subsequent bond is applied to the roof construction in the longitudinal direction. Prestressing in the lateral direction is not applied for reasons of economy. Increased requirements apply for cracking.

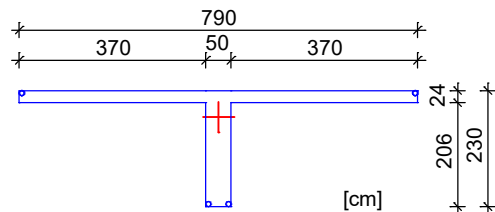


Static system and dimensions [m]

### Material

Concrete	SC45/55
Reinforcing steel	BSt 500, axis distance from edge 5 cm

### Section

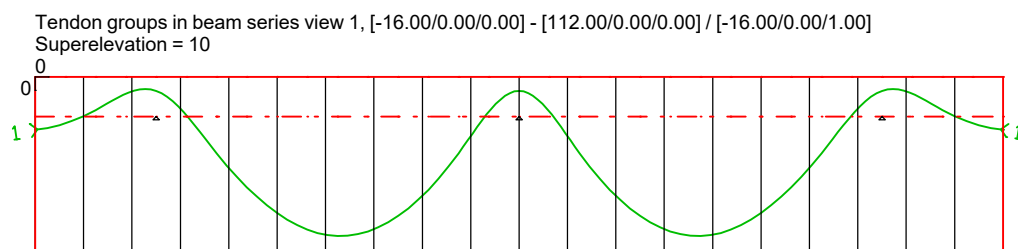


### Prestressing steel and prestressing system

Prestressing steel quality	St 1520/1770
Certification of the prestressing system	SIA 262
Number of tendons in the bundle	4
Section surface $A_p$	1800 mm <sup>2</sup>
E-modulus of the prestressing steel	195000 MN/m <sup>2</sup>
0.1% strain limit (yield strength) of the prestressing steel $f_{p0.1k}$	1520 MN/m <sup>2</sup>
Tensile strength of the prestressing steel $f_{pk}$	1770 MN/m <sup>2</sup>
Permissible prestressing force of a tendon $P_{m0}$	2230.2 kN
Prestressing loss from relaxation of prestressed steel	4.5 %
Friction coefficients when prestressing and releasing $\mu$	0.2
Unintentional deviation angle of a tendon $\beta'$	0.3 °/m
Slippage at prestressing anchor	6 mm
Duct diameter $d_h$	82 mm



The tendon guide is shown in the next figure. 4 bundled tendons are arranged such that they stretch across the entire girder length and are prestressed at both girder ends. The prestressing system, prestressing procedure and prestressing curve for a tendon group are also shown.



Tendon group ordinates  $z_v$  [cm] at the base points

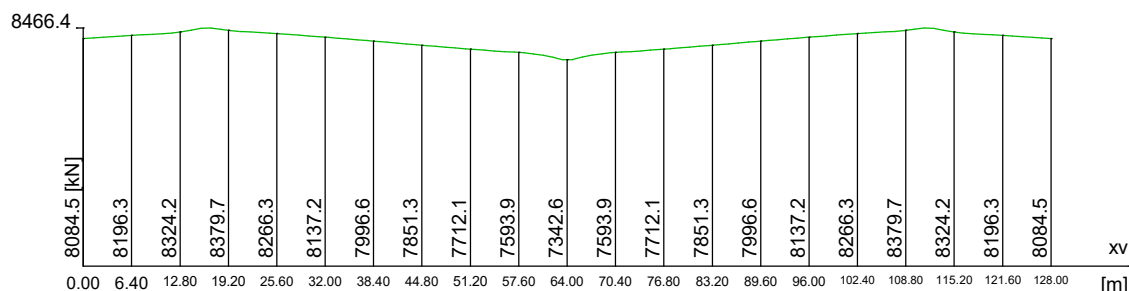
xv	0.00	6.40	12.80	19.20	25.60	32.00	38.40	44.80	51.20	57.60	64.00	70.40	76.80	83.20	89.60	96.00	102.40	108.80	115.20	121.60	128.00
1	70.0	52.0	19.2	42.0	120.2	180.1	208.8	201.5	157.4	78.3	18.5	78.3	157.4	201.5	208.8	180.1	120.2	42.0	19.2	52.0	70.0

Force function of tendon group 1 (4 tendon(s),  $l = 128.41$  m)

Prestressing system 1 - Example. Certification according to SIA 262.  
 $P_{m0} = 2230.2$  kN,  $A_p = 1800.0$  mm<sup>2</sup>,  $\mu_a = 0.20$ , Angle  $\beta' = 0.30$  °/m  
 E-Modulus = 195000 MN/m<sup>2</sup>,  $A_h = 5281.0$  mm<sup>2</sup>,  $\mu_n = 0.20$ , Slippage = 6.00 mm

Prestressing procedure 1 - Example

Pre. anchor : Start End  
 Normal. force : 1.000 1.000  
 Pre. force [kN]: 2230.2 2230.2  
 Extension [mm]: 674.9 70.8



Tendon guide and prestressing curve in the longitudinal section (4 tendons).

**Loads**

- Load case 1            Dead load
- Load case 2            Additional dead load:  $q=11.06$  kN/m
- Load case 3            Snow load:  $q=7.90$  kN/m
- Load case 10           Prestressing
- Load case 15           Creep-generating permanent load: Dead load, additional dead load and prestressing
- Load case 20           Creep and shrinkage  
 Coefficients:  $\varphi_{t\infty} = 2.55$ ;  $\rho = 0.8$ ;  $\epsilon_{t\infty} = -24.8 \cdot 10^{-5}$   
 Creep-generating permanent load case: 15  
 The redistribution of internal forces between concrete and prestressing steel are taken into account.

**SIA 262 actions**

**Standard design group**

**G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 0.8

Load cases

- 1    Dead load

**G - Additional dead load**

Gamma.sup / gamma.inf = 1.35 / 0.8

Load cases

- 2    Additional dead load

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Buildings  
 Snow loads  
 Psi.0 / Psi.1 / Psi.2 = 1 / 1 / 1

Load cases 1. Variant, inclusive

- 3    Snow load

**P - Prestressing**

Gamma.sup / gamma.inf = 1 / 1

Load cases internal prestressing

- 10   Prestressing

**CSR1 - Creep, shrinkage, relaxation**

Prestressing loss from relaxation of prestressed steel: 4.5 %.

Load cases

20 Creep, shrinkage

**1. Permanent and temporary situation**

Construction stage - UngROUTed

G Dead load  
P Prestressing

**2. Permanent and temporary situation - t0**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
QS Snow and ice load

**3. Permanent and temporary situation - too**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
CSR1 Creep, shrinkage, relaxation  
QS Snow and ice load

**1. Rare (characteristic) situation**

Construction stage - UngROUTed

G Dead load  
P Prestressing

**2. Rare (characteristic) situation - t0**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
QS Snow and ice load

**3. Rare (characteristic) situation - too**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
CSR1 Creep, shrinkage, relaxation  
QS Snow and ice load

**1. Frequent situation**

Construction stage

G Dead load  
P Prestressing

**2. Frequent situation - t0**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
QS Snow and ice load

**3. Frequent situation - too**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
CSR1 Creep, shrinkage, relaxation  
QS Snow and ice load

**1. Quasi-continuous situation**

Construction stage

G Dead load  
P Prestressing

**2. Quasi-continuous situation - t0**

Final state

G Dead load  
G Additional dead load  
P Prestressing  
QS Snow and ice load

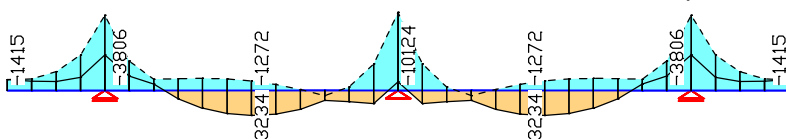
**3. Quasi-continuous situation - too**

Final state

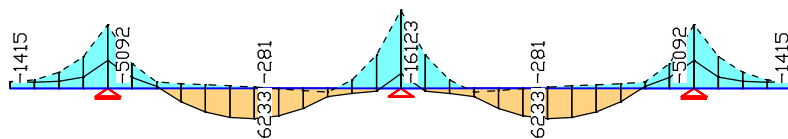
G Dead load  
G Additional dead load  
P Prestressing  
CSR1 Creep, shrinkage, relaxation  
QS Snow and ice load

In this example all possible combinations of load cases are generated and designed. This method is selected in the calculation settings and can be very slow when applied for a large number of load cases.

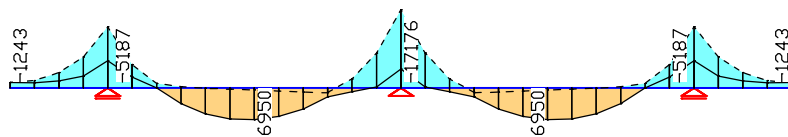
Below you will find an example of the curve of bending moment  $M_y$  for design situations in the ultimate limit states.



Bending moment  $M_y$  of the 1. permanent and temporary situation [kNm]



Bending moment  $M_y$  of the 2. permanent and temporary situation [kNm]



Bending moment  $M_y$  of the 3. permanent and temporary situation [kNm]

**Design according to SIA 262 (2017)**

**Design overview**

Se.	Prestress of component	Requi- rement	Reinforc. M R B Q T	Fatig. S P C	Crack width	Comp.- stress
1	Subsequent bond	raised	x + x x .	. . .	x	x

- (M) Nominal reinforcement against brittle failure (robustness reinforcement).
- (R) Crack reinforcement from restraint (x), required reinf. due to load (+).
- (B) Flexural reinforcement at ultimate limit state.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state.
- (T) Torsional reinforcement at ultimate limit state.
- (S) Reinforcing steel at fatigue check.
- (P) Prestressing steel at fatigue check.
- (C) Concrete at fatigue check.

**Settings for flexural and shear reinforcement**

M,N	Design mode for bend and longitudinal force: (ST) Standard, (SY) Symmetrical, (CM) Compression member.									
Red.	Reduction factor of prestress for determining the tensile zone for distribution of reinf. against brittle failure for area elements.									
Fac. as	Factor for bending reinf. of slabs in secondary dir. per 5.5.3.2.									
eta.t	Coefficient for concrete strength as per 4.2.1.3.									
fsk	Quality of stirrups [MN/m <sup>2</sup> ].									
Alpha	Compression field angle [°].									
kc	Reduction factor for concrete strength as per 4.2.1.7.									
Slab	Beams are designed like slabs.									
eps.v	Base value of strain as per 4.3.3.2.2 [o/oo].									
Dmax	Max. grain of concrete as per Eq. (37) [mm].									
Long.	Longitudinal reinforcement is graded as per 4.3.3.2.3.									
Se.	Concrete	Des.	Red.	Fac.	eta.t	fsk	Alpha	kc	Des. Base like value	Long. Dmax reinf. graded
1	SC45/55	ST	.	.	1.00	500	45.00	0.55	.	.

**Shear sections**

bw.nom	Nominal width of the prestressed section according to 4.3.3.3.5.									
h.nom	Nominal height of the prestressed section according to 4.3.3.3.5.									
kb, kd	Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d according to 4.3.3.4.2.									
z1, z2	Dimensions of the ideal hollow section for torsion as per 4.3.5.1.									
tk	Thickness of the ideal hollow section.									
Se.	Width [m]	Eff. width	Height[m]	Eff.height	Torsion section [m]					
1	0.500	0.450	2.300	2.250	0.90	2.200	0.400	0.100		

**Settings for crack control**

ds	Max. given bar diameter of the reinforcing steel as per Fig. 31									
fctd	Design value of concrete tensile strength.									
kt	Reduction factor for concrete tensile strength.									
Se.	Type of restraint	ds [mm]	kt	fctd	Regard pre-str. steel					
1	.	10	1.00	kt*fctm	.					

**Settings for the check of concrete compressive stresses**

Sigma.c	Concrete compressive stress in the serviceability limit state. (QC) Quasi-continuous combination.									
fck(t)	Compressive strength of concrete at the time t of prestressing.									
Sigma.c(t)	Concrete compressive stress at the time t of prestressing.									
Se.	per.sigma.c (QC)	fck(t) [MN/m <sup>2</sup> ]	per.sigma.c(t)							
1	0.45 fck	45.00	0.45 fck(t)							

**Stress calculation for beams**

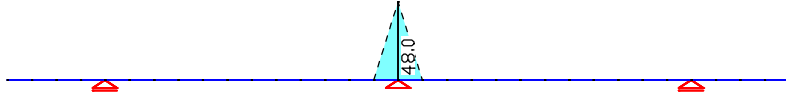
While calculating the reinforcement for robustness, the gross section is used.

At crack width limitation, the bending stress is calculated using the gross section.

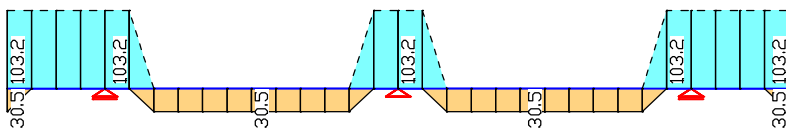
At the check of decompression and compressive stress of concrete, calculation of bending stress is carried out

- without internal tendons: at gross section,
- with internal tendons without bond: at net section,
- with internal tendons with bond before grouting: at net section, otherwise at ideally section.

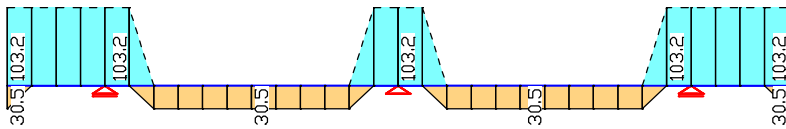
The following illustrations show the curve of the required bending and shear reinforcement.



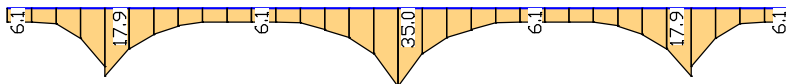
Longitudinal reinforcement  $A_s$  from the design in the ultimate limit states [ $cm^2$ ] (upper reinforcement with dashed lines).



Minimum reinforcement  $A_s$  for ensuring robustness (ductility) [ $cm^2$ ] (upper reinforcement with dashed lines).



Enclosing reinforcement  $A_s$  from the checks [ $cm^2$ ] (upper reinforcement with dashed lines).



(Minimum) lateral force reinforcement  $A_{sb,z}$  in the ultimate limit states [ $cm^2/m$ ].

The following pages contain excerpts from the detailed check log for beam 16 at location 2 (middle column).

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. against brittle failure acc. to Chapter 4.4.2 (Charact. C.).
  - fctd Design value of concrete tensile strength as per Chapter 4.4.1 [ $MN/m^2$ ]
  - zs,t/b Lever arm of inner strengths top/bottom with  $zs=0,9*d$  [m]
  - S.s,adm Admissible steel stress as per Fig. 31 [ $MN/m^2$ ]
  - max Sc Maximum concrete edge stress from Charact. C. [ $MN/m^2$ ] without the statically determined part of prestressing
- (R) Required reinforcement as per 4.4.2 for crack width limitation
  - Increase of reinforcement due to crack width check is marked by "!".
  - ds Maximal given steel diameter [mm]
- (B) Design of reinforcement at ultimate limit state
  - In case of dominant bending, compression reinforcement is marked with "\*\*".
  - fck Concrete strength for design of reinforcement [ $MN/m^2$ ]
  - N0, M0 Statically determined forces of tendons with bond [kN, kNm]
  - fp0.1k Charact. value of the 0.1% strain limit of the prestr. steel [ $MN/m^2$ ]
  - fpk Charact. value of the tensile strength of the prestr. steel [ $MN/m^2$ ]

**Location 2**

Beam 16,  $x = 4.00$  m (Beam length 4.00 m)  
 Cross-section 1: Polygon - SC45/55, 1 tendon group with bond  
 Steel 1; Design mode: Standard  
 (M) fctd=3.8; zs,o/u=2.025/2.025; Sigma.s,adm=435  
 (R) fctd=3.8; ds=10  
 (B) fck=45

Section properties	A [ $m^2$ ]	ys [m]	zs [m]	Iy [ $m^4$ ]	Iz [ $m^4$ ]	Iyz [ $m^4$ ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.958	3.950	0.521	1.2597	9.8822	0.0000

## Tendon groups with bond

No.	E-Modul [MN/m <sup>2</sup> ]	fp0.1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1520	1770	3.950	0.185	7200	82	7342.65	0.00

## 1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted

Relevant concrete internal forces from 1 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
1	-7342.59	-4388.77	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L10

## 2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted

No set of internal forces in this situation was relevant.

## 3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.64	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR): Nx0=-6635.00 kN; My0=2255.90; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces

Set	Concrete section			Bond section		
	Nx [kN]	My [kNm]	Mz [kNm]	Nx [kN]	My [kNm]	Mz [kNm]
2	-6520.60	-9688.93	0.00	114.41	-11944.83	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	L1+L2+0.96*L10+L20+L3

## 1. Frequent combination (TC.1): G.1+P, Construction stage grouted

No set of internal forces in this situation was relevant.

## 2. Frequent combination (TC.2): G.1+G.2+P+QS, Final state grouted

Relevant concrete internal forces from 2 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
1	-7342.59	-6866.21	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L2+L10

## 3. Frequent combination (TC.3): G.1+G.2+P+CSR1+QS, Final state grouted

No set of internal forces in this situation was relevant.

## 1. Quasi-continuous combination (QC.1): G.1+P, Construction stage grouted

No set of internal forces in this situation was relevant.

## 2. Quasi-continuous combination (QC.2): G.1+G.2+P+QS, Final state grouted

No set of internal forces in this situation was relevant.

## 3. Quasi-continuous combination (QC.3): G.1+G.2+P+CSR1+QS, Final state grouted

No set of internal forces in this situation was relevant.

## 1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted

Relevant concrete internal forces from 2 sets of internal forces

Set	Nx [kN]	My [kNm]	Mz [kNm]
2	-7342.59	-1111.65	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	0.80*L1+L10

## 2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted

No set of internal forces in this situation was relevant.

## 3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.64	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR): Nx0=-6635.00 kN; My0=2255.90; Mz0=0.00 kNm

Relevant values from 8 sets of internal forces

Set	Concrete section			Bond section		
	Nx [kN]	My [kNm]	Mz [kNm]	Nx [kN]	My [kNm]	Mz [kNm]
2	-6520.60	-17175.80	0.00	114.41	-19431.70	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	1.35*L1+1.35*L2+0.96*L10+L20+1.50*L3

**Design of longitudinal reinforcement**

Reinforcement	Nx	My	Mz	max Sc	Ap'	req.As	Situation
Lay. Type	[kN]	[kNm]	[kNm]	[MN/m <sup>2</sup> ]	[cm <sup>2</sup> ]	[cm <sup>2</sup> ]	
1 M	114.41	-11944.83	0.00	5.03	.	51.62	CC.3,2
R	-7342.59	-6866.21	0.00	.	.	0.00	TC.2,1
B	-6520.60	-17175.80	0.00	.	.	24.01	PC.3,2
2 M	114.41	-11944.83	0.00	5.03	.	51.62	CC.3,2
R	-7342.59	-6866.21	0.00	.	.	0.00	TC.2,1
B	-6520.60	-17175.80	0.00	.	.	24.01	PC.3,2
3 M	0.06	-6885.27	0.00	.	.	0.00	CC.1,1
R	0.00	0.00	0.00	0.00	.	0.00	-, -
B	-7342.59	-1111.65	0.00	.	.	0.00	PC.1,2
4 M	0.06	-6885.27	0.00	.	.	0.00	CC.1,1
R	0.00	0.00	0.00	0.00	.	0.00	-, -
B	-7342.59	-1111.65	0.00	.	.	0.00	PC.1,2

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to 5.5.2.2 is considered.

bw	Effective width for calculation of shear stresses from Qz [m]
bw.nom	Nominal value of the width when deducting the duct diameter [m]
bn	Statically effective width for shear design using Qy [m]
kb	Factor to calculate the inner lever arm from bn
h	Effective height for calculation of shear stresses from Qy and Mx [m]
h.nom	Nominal value of the height when deducting the duct diameter [m]
d	Statically effective height for shear design using Qz [m]
kd	Factor to calculate the inner lever arm from d
fsk	Strength of stirrup reinforcement [MN/m <sup>2</sup> ]
kc	Reduction coefficient to determine the concrete strength
Qy, Qz	Lateral forces for design in y- and z-direction [kN]
VRdc	Lateral force resistance of the compression field [kN]
z	Inner lever arm z=kb*bn resp. z=kd*d
Alpha	Compression field angle [°]
Asb.y,z	Req. stirrup reinforcement from aus Qy, Qz [cm <sup>2</sup> /m]
Asl.y,z	Req. longitudinal reinforcement from Qy, Qz [cm <sup>2</sup> ]

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - SC45/55, 1 tendon group with bond  
 bw/bw.nom/bn/kb=0.5/0.5/0.45/0.9; h/h.nom/d/kd=2.3/2.3/2.25/0.9  
 fsk=500; kc=0.55

**1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted**

No set of internal forces in this situation was relevant.

**2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Relevant concrete internal forces from 8 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]	Mx[kNm]	Qy[kN]	Qz[kN]
2	-6520.60	-17175.80	0.00	0.00	0.00	-3078.27

Load case combinations for the relevant sets of internal forces

Set	Combination
2	1.35*L1+1.35*L2+0.96*L10+L20+1.50*L3

**Check of the shear reinforcement and the compressive struts**

Qy	: 0.00 kN	Alpha	: 45.00 °
Qy/VyRdc	: 0.00	z	: 0.41 m
Situation	: -, -	req.Asb.y	: 0.00 cm <sup>2</sup> /m
		req.As1.y	: 0.00 cm <sup>2</sup>
Qz	: -3078.27 kN	Alpha	: 45.00 °
Qz/VzRdc	: 0.42	z	: 2.02 m
Situation	: PC.3,2	req.Asb.z	: 34.96 cm <sup>2</sup> /m
		req.As1.z	: 35.40 cm <sup>2</sup>

**Check of crack widths**

The check is led by limiting the steel stress.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

(TC) Frequent, (QC) Quasi-continuous combination

ds	Maximal given steel diameter [mm]
fctm	Mean value of the concrete tensile strength [MN/m <sup>2</sup> ]
Sigma.c	Maximal concrete edge stress in state I [MN/m <sup>2</sup> ]
Sigma.s	Reinf. steel stress in state II [MN/m <sup>2</sup> ]
per.	Permissible steel stress as per Table 17 and Fig. 31 [MN/m <sup>2</sup> ]

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - SC45/55, 1 tendon group with bond  
 Requirement: raised; ds=10; fctm=3.8

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross	2.926	3.950	0.525	1.2560	9.8822	0.0000
net	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.958	3.950	0.521	1.2597	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	fp0,1k [MN/m <sup>2</sup> ]	fpk [MN/m <sup>2</sup> ]	y [m]	z [m]	Ap [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1520	1770	3.950	0.185	7200	82	7342.65	0.00

**1. Frequent combination (TC.1): G.1+P, Construction stage grouted**

No set of internal forces in this situation was relevant.

**2. Frequent combination (TC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Frequent combination (TC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.64	-.-	-.-	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR) : Nx0=-6635.00 kN; My0=2255.90; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6520.60	-9688.93	0.00	114.41	-11944.83	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	L1+L2+0.96*L10+L20+L3

**1. Quasi-continuous combination (QC.1): G.1+P, Construction stage grouted**

No set of internal forces in this situation was relevant.

**2. Quasi-continuous combination (QC.2): G.1+G.2+P+QS, Final state grouted**

No set of internal forces in this situation was relevant.

**3. Quasi-continuous combination (QC.3): G.1+G.2+P+CSR1+QS, Final state grouted**

Loss of prestress by CSR in tendon groups

No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]	No.	CSR[%]
1	9.64	-.-	-.-	-.-	-.-	-.-	-.-	-.-	-.-

Stat. determ. part (P+CSR) : Nx0=-6635.00 kN; My0=2255.90; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces

Set	Concrete section			Bond section		
	Nx[kN]	My[kNm]	Mz[kNm]	Nx[kN]	My[kNm]	Mz[kNm]
2	-6520.60	-9688.93	0.00	114.41	-11944.83	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
2	L1+L2+0.96*L10+L20+L3

**Check of crack width for reinf. layer 2 (top) - Frequent combination**

Nx [kN]	: -6520.60	Sigma.c [MN/m <sup>2</sup> ]:	1.82
My [kNm]	: -9688.93	As [cm <sup>2</sup> ]:	51.62
Mz [kNm]	: 0.00	Sigma.s [MN/m <sup>2</sup> ]:	63.71 per. 355.00
Situation	: TC.3,2		

**Check of crack width for reinf. layer 2 (top) - Quasi-continuous combination (w.nom = 0.5 mm)**

Nx [kN]	: -6520.60	Sigma.c [MN/m <sup>2</sup> ]:	1.82
My [kNm]	: -9688.93	As [cm <sup>2</sup> ]:	51.62
Mz [kNm]	: 0.00	Sigma.s [MN/m <sup>2</sup> ]:	63.71 per. 435.00
Situation	: QC.3,2		

**Check of concrete compressive stress**

For the check, a non-cracked concrete section is assumed.

fck	Characteristic compressive concrete strength [MN/m <sup>2</sup> ]
fck(t)	Average compressive strength of concrete at time t of the beginning of prestressing (Situation G1+P) as per 4.1.5.2.4 [MN/m <sup>2</sup> ]
Sigma.x,min	Total maximal longitudinal compressive stress [MN/m <sup>2</sup> ]
Sigma.x,per	= 0.60*fck for Q.-cont. C. (QC) as per 4.1.5.2.4 = 0.45*fck for Q.-cont. C. (QC) as per 3.1.2.6.3
top, bottom	Position of the edge point: above, below of centre

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
Cross-section 1: Polygon - SC45/55, 1 tendon group with bond  
0.45\*fck=20.25; 0.45\*fck(t)=20.25

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.905	3.950	0.527	1.2535	9.8822	0.0000
ideally:	2.958	3.950	0.521	1.2597	9.8822	0.0000

**1. Quasi-continuous combination (QC.1): G.1+P, Construction stage grouted**

Relevant concrete internal forces from 1 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
1	-7342.59	-4388.77	0.00

Load case combinations for the relevant sets of internal forces

Set	Combination
1	L1+L10

2. Quasi-continuous combination (QC.2): G.1+G.2+P+QS, Final state grouted

No set of internal forces in this situation was relevant.

3. Quasi-continuous combination (QC.3): G.1+G.2+P+CSR1+QS, Final state grouted

Relevant concrete internal forces from 2 sets of internal forces

Set	Nx[kN]	My[kNm]	Mz[kNm]
2	-6520.60	-9688.93	0.00

Load case combinations for the relevant sets of internal forces

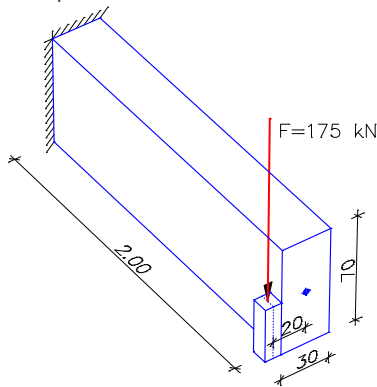
Set	Combination
2	L1+L2+0.96*L10+L20+L3

Check of compressive stress in concrete for the Quasi-continuous combination

Side	Se.- Pnt.	Sigma.x,min [MN/m²]	Sigma.x,per [MN/m²]	Period	Situation
top	1	-0.67	-20.25	Constr.	QC.1,1
bottom	7	-15.89	-20.25	Final	QC.3,2

## Torsional Beam

The depicted cantilever is subjected to an eccentrically acting load F = 175 kN. The required shear, torsion longitudinal and stirrup reinforcements are listed in the following log.



System drawing

SIA 262 actions

Standard design group

G - Dead load

Gamma.sup / gamma.inf = 1.35 / 0.8

Load cases

1	Load
---	------

1. Permanent and temporary situation

Final state

G - Dead load

Settings for flexural and shear reinforcement

M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 Red. Reduction factor of prestress for determining the tensile zone for distribution of reinf. against brittle failure for area elements.  
 Fac. as Factor for bending reinf. of slabs in secondary dir. per 5.5.3.2.  
 eta.t Coefficient for concrete strength as per 4.2.1.3.  
 fsk Quality of stirrups [MN/m²].  
 Alpha Compression field angle [°].  
 kc Reduction factor for concrete strength as per 4.2.1.7.  
 Slab Beams are designed like slabs.  
 eps.v Base value of strain as per 4.3.3.2.2 [o/oo].  
 Dmax Max. grain of concrete as per Eq. (37) [mm].  
 Long. Longitudinal reinforcement is graded as per 4.3.3.2.3.

Se.	Concrete	Des.	Red.	Fac.	eta.t	fsk	Alpha	kc	Des. Base	Long.
		M,N	for pre-str.	as		[MN/m²]	[°]		like value	Dmax reinf. graded
1	SC35/45		.	.	.	500	45.00	0.55	.	.

Shear sections

bw.nom Nominal width of the prestressed section according to 4.3.3.3.5.  
 h.nom Nominal height of the prestressed section according to 4.3.3.3.5.  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d according to 4.3.3.4.2.  
 z1, z2 Dimensions of the ideal hollow section for torsion as per 4.3.5.1.  
 tk Thickness of the ideal hollow section.

Se.	Width [m]	Eff. width	Height[m]	Eff.height	Torsion section [m]
	bw bw.nom	bn [m]	kb h h.nom	d [m]	kd z1 z2 tk
1	0.300 .	0.245	0.90 0.700 .	0.645 0.90	0.595 0.195 0.105



**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to 5.5.2.2 is considered.

bw Effective width for calculation of shear stresses from Qz and Mx [m]  
 bn Statically effective width for shear design using Qy [m]  
 kb Factor to calculate the inner lever arm from bn  
 h Effective height for calculation of shear stresses from Qy and Mx [m]  
 d Statically effective height for shear design using Qz [m]  
 kd Factor to calculate the inner lever arm from d  
 z1, z2 Height and width of the core section Ak for torsion [m]  
 tk Wall thickness of the torsion section [m]  
 fsk Strength of stirrup reinforcement [MN/m<sup>2</sup>]  
 kc Reduction coefficient to determine the concrete strength  
 Qy, Qz Lateral forces for design in y- and z-direction [kN]  
 VRdc Lateral force resistance without lateral reinforcement [kN]  
 VRdc Lateral force resistance of the compression field [kN]  
 z Inner lever arm z=kb\*bn resp. z=kd\*d  
 Alpha Compression filed angle [°]  
 Asb.y,z Req. stirrup reinforcement from aus Qy, Qz [cm<sup>2</sup>/m]  
 Asl.y,z Req. longitudinal reinforcement from Qy, Qz [cm<sup>2</sup>]  
 Mx Torsional moment for design [kNm]  
 TRdc Torsional resistance of the compression field [kNm]  
 Asb.T Req. stirrup reinforcement from torsion [cm<sup>2</sup>/m]  
 Asl.T Req. longitudinal reinforcement from torsion [cm<sup>2</sup>]

**Beam 1****Location 1**

Beam 1, x = 0.00 m (Beam length 2.00 m)  
 Cross-section 1: Polygon - SC35/45  
 bw/bn/kb=0.3/0.245/0.9; h/d/kd=0.7/0.645/0.9  
 fsk=500; kc=0.55  
 Torsion section z1/z2=0.595/0.195; tk=0.105

**1. Permanent and temporary comb. (PC.1): G, Final state****Concrete internal forces**

	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
Nx- :	0.00	-280.00	0.00	28.00	0.00	140.00
Nx+ :	0.00	-280.00	0.00	28.00	0.00	140.00
My- :	0.00	-472.50	0.00	47.25	0.00	236.25
My+ :	0.00	-280.00	0.00	28.00	0.00	140.00
Mz- :	0.00	-280.00	0.00	28.00	0.00	140.00
Mz+ :	0.00	-280.00	0.00	28.00	0.00	140.00
Mx- :	0.00	-280.00	0.00	28.00	0.00	140.00
Mx+ :	0.00	-472.50	0.00	47.25	0.00	236.25
Qy- :	0.00	-280.00	0.00	28.00	0.00	140.00
Qy+ :	0.00	-280.00	0.00	28.00	0.00	140.00
Qz- :	0.00	-280.00	0.00	28.00	0.00	140.00
Qz+ :	0.00	-472.50	0.00	47.25	0.00	236.25

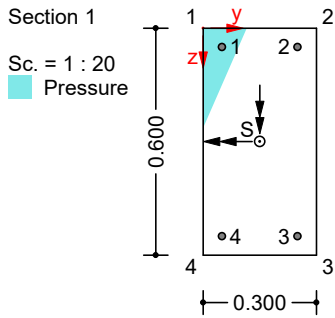
**Check of the shear reinforcement and the compressive struts**

Qy	: 0.00 kN	Alpha	: 45.00 °
Qy/VyRdc	: 0.00	z	: 0.22 m
Situation	: -, -	req.Asb.y	: 0.00 cm <sup>2</sup> /m
		req.As1.y	: 0.00 cm <sup>2</sup>
Qz	: 236.25 kN	Alpha	: 45.00 °
Qz/VzRdc	: 0.22	z	: 0.58 m
Situation	: PC.1, Qz+	req.Asb.z	: 9.36 cm <sup>2</sup> /m
		req.As1.z	: 2.72 cm <sup>2</sup>
Mx	: 47.25 kNm	Alpha	: 45.00 °
Q/VRdc+Mx/TRdc	: 0.54	req.Asb.T	: 4.68 cm <sup>2</sup> /m
Situation	: PC.1, Qz+	req.As1.T	: 7.40 cm <sup>2</sup>

# Single Design

A single rectangular section is designed for bending and normal force.

## Pos. 1 - Reinforced concrete design per SIA 262



Action  $N = 10.00 \text{ kN}; M_y = 67.50; M_z = 27.00 \text{ kNm}$   
 Resistance  $N = 10.00 \text{ kN}; M_y = 67.50; M_z = 27.00 \text{ kNm}$   
 Force system  $y_s / z_s = 0.150 / 0.300 \text{ m}$   
 Strength SC20/25;  $\gamma_c = 1.50; \gamma_s = 1.15$   
 Design mode Standard  
 Reinforcement  $3.63 \text{ cm}^2; 0.20 \%; \text{Concrete area} = 1800.00 \text{ cm}^2$

Concrete section					Inner Forces				
Point	y [m]	z [m]	eps[‰]	sigma[MPa]		y [m]	z [m]	F [kN]	
1	0.000	0.000	-3.00	-13.33	Compr.	0.034	0.078	-150.45	
	0.115	0.000	0.00	0.00	Tension	0.210	0.512	160.45	
2	0.300	0.000	4.80	0.00	Lev. arm	0.175	0.435		
3	0.300	0.600	11.64	0.00					
	0.000	0.600	3.84	0.00					
4	0.000	0.263	0.00	0.00					

Reinforcement									
Point	y [m]	z [m]	d1 [m]	Es, $\beta_s$ [MPa]	Zv0 [kN]	eps[‰]	sigma[MPa]	As [cm <sup>2</sup> ]	
1	0.050	0.050	0.050	205000 500	0.0	-1.13	-231.68	0.00	
2	0.250	0.050	0.050	205000 500	0.0	4.07	437.15	0.28	
3	0.250	0.550	0.050	205000 500	0.0	9.77	444.08	2.61	
4	0.050	0.550	0.050	205000 500	0.0	4.57	437.76	0.74	

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# DIN 4227 Checks

## Basics

The checks according to DIN 4227 can be used for prestressed beam, shell and solid structures **with subsequent bond** and are carried out subsequent to static analysis. The description of the load combinations is based on the existing internal forces from load cases and load case combinations. The definition of the prestressing as well as the actions from creep and shrinkage are described in the *Prestressed Concrete* section.

For beams and design objects, all checks are carried out at the polygon section. For general notes on using design objects, refer to the relevant chapter in the manual.

## Input

### Settings

This dialog is used to select the desired prestressed concrete checks and specify the corresponding combination types. The labels  $G1$ ,  $G2$ ,  $V$ ,  $P$ ,  $T$ ,  $DS$ ,  $KS$  as well as the operators "+" and "\*" can be used in the description as a load abbreviation for the check load cases. Safety factors must be placed before the load abbreviation.

Groups can be formed using parentheses ( ).

#### Stresses

Carry out stress check for the list of the combination rules.

#### Failure safety

Carry out failure safety check for the following combination rule.

Default combination:  $1.75*(G1+G2+P)+V+KS+DS+T$

#### Crack width limitation

Determine the reinforcement for limiting the crack width as a maximum of two combinations.

Default combinations:

$G1+G2+P+0.9*(V+KS)+DS+T$  and  $G1+G2+P+1.1*(V+KS)+DS+T$

**Shear**

Carry out the following shear checks.

**Serviceability state**

Combination rule for the shear check in the serviceability state.

Default combination:  $G1+G2+P+V+KS+DS+T$

**Limited prestressing**

Use the limits of the permissible principal tensile stresses according to DIN 4227-1, Table 9, Rows 48-49.

**Full prestressing**

Use the limits of the permissible principal tensile stresses according to Table 9, Rows 46-47.

**Ultimate state**

Combination rule for the shear check in the failure state.

Default combination:  $1.75*(G1+G2+P)+V+KS+DS+T$

**Always print reinforcement**

The necessary shear reinforcement according to Section 12.4.1 is always calculated and output regardless of the check limit.

**Check Zone a like Zone b**

The shear check according to *Zone a* is carried out as in *Zone b* according to Section 12.3.2 and 12.4.2 (4).

**Listing**

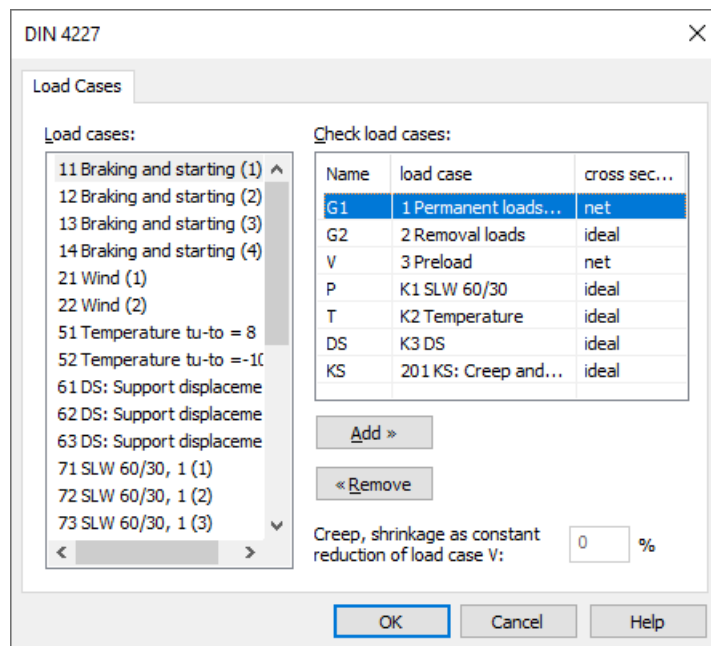
Set the scope of the log.

**Load cases...**

Dialog for describing the check load cases.

**Definition of Design Load Cases**

This dialog is used to assign the load cases or load case combinations calculated in the FEM module to standard load case types according to DIN 4227.



### Check load cases

- *G1*: Permanent loads, for instance dead load (combination or load case).
- *G2*: Additional loads, for instance roadway pavement (combination or load case).
- *V*: Prestressing (load case).
- *P*: Traffic (combination or load case).
- *T*: Temperature (combination or load case).
- *DS*: Support displacement (combination or load case).
- *KS*: Creep and shrinkage (load case). The input requires prior determination of redistribution internal forces with the FEM module in a load case of the *Creep and shrinkage* load type.

### Section

For the checks of beams and design objects, the load cases can be assigned the following section properties using the shortcut menu:

- *gross*: Gross section properties without consideration of ducts or tendon groups.
- *net*: Net section properties with consideration of the duct diameters at the intersection points of the splines through the section polygon at the result location.
- *ideal*: Ideal section properties corresponding to the net section properties additionally taking into account the tendon groups in the center of their ducts.

### Insert

Assign the selected load case to the selected check load case.

### Remove

Delete the selected check load case.

### Creep, shrinkage as constant reduction of load case V

If the calculation of a *Creep and shrinkage* load case is not desired, the stress loss from creep and shrinkage can alternatively be specified as a percentage reduction with respect to the *V* load case [%].

## Analysis Settings

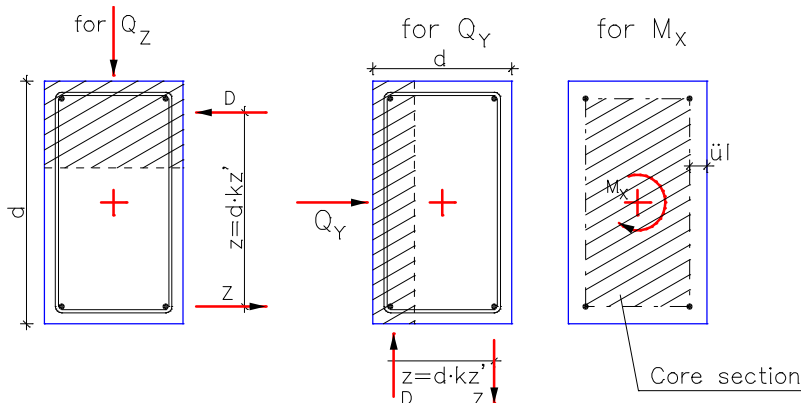
The analysis settings for the prestressed concrete checks are defined in the *DIN 4227 Checks* folder of the database. They are organized according to section type and are valid for stress check, failure safety check, crack check and shear check. The reinforcing steel is graphically defined together with the polygon section.

Settings for DIN 4227 checks [BMVSPG]															
	Section	Full polyg	Concrete	Cracking		$\beta_s$	Lateral force				Torsion			perm. Stress	
				$\xi_{I1}$	$d_s$ [mm]		y direct. d [m]	z direct. kz' d [m]	y di. d [m]	z di. d [m]	$\bar{u}_l$ [m]	S1, Tau (T,Q+T)	S2		
1	11	10	B45	0.46	25	500	1.00	0.75	1.90	0.75	1.00	1.90	0.100	1	1
2	12	0	B45	0.30	16	500	0.00	0.00	0.00	0.68	0.00	0.00	0.000	0	0
3															

### Input for sections of beams and design objects

<i>Section</i>	Section that is to be checked.
<i>Full polygon</i>	If no effective width is defined for the section, the full polygon given is used to determine the stresses resulting from longitudinal force for <i>V</i> and <i>KS</i> as well as to calculate the shear stress from lateral force and torsion in state I. When specifying the value "0", the section polygon is taken. The definition of the polygons is carried out in the section dialog.
<i>Concrete</i>	The concrete quality for the checks can be specified such that it differs from the material description of the FEM analysis. The concrete classes for normal and lightweight concrete according to DIN 4227 Part 1 and 4 are permissible. When specifying the value "0", the concrete quality from the material description is used.
<i><math>\xi_{I1}</math></i>	Bond coefficient $\xi_{I1}$ for taking into account the contribution of the prestressing steel according to DIN 4227-1/A1, Section 6.7.3.
<i><math>d_s</math></i>	Selected bar diameter according to DIN 4227-1/A1, Section 6.7.3, Table 6.
<i><math>\beta_s</math></i>	Yield strength of the shear stirrups [MN/m <sup>2</sup> ]. When entering "0", the calculation is performed with $\beta_s = 500$ MN/m <sup>2</sup> .
<i>Lateral force d</i>	Dimensions of the equivalent rectangle for the shear check in state II.

<i>Lateral force <math>kz'</math></i>	Coefficient for calculation of the lever arm $z$ of the inner forces for the shear checks in state II with $kz' = z/d$ (see below for an explanation)
<i>Torsion <math>d</math></i>	Dimensions of the equivalent hollow section for calculation of the torsion stresses in zone b and the torsion reinforcement.
<i><math>\ddot{u}l</math></i>	Distance of the core section from the section edge [m]. This corresponds to half of the wall thickness of the equivalent box girder section. When entering "0", no torsion reinforcement is calculated.
<i>S1, Tau</i>	Permissible principal tensile and shear stresses according to DIN 4227-1, Table 9. "0": Lines 53, 54, 59, 60 are decisive (box section) "1": Lines 52, 55, 58, 61 are decisive (full section)
<i>S2</i>	Permissible principal compressive stress according to DIN 4227-1, Table 9. "0": Line 62 is decisive (web) "1": Line 63 is decisive (chord)



The illustration displays the lever arms of the inner forces and the core section. The coefficient  $kz'$  relates to  $d$  and should not be confused with the coefficient  $k_z$  for the shear design according to DIN 1045.

If  $kz'$  is set to "0" for the y or z direction, the value is determined from the failure safety check and limited as follows:  
 $0.1 \leq kz' \leq 1$ .

### Input for area sections

<i>Section</i>	Section that is to be checked.
<i>Concrete</i>	The concrete quality for the checks can be specified such that it differs from the material description of the FEM analysis. The concrete classes for normal and lightweight concrete according to DIN 4227 Part 1 and 4 are permissible. When specifying the value "0", the concrete quality from the material description is used.
<i><math>\xi_1</math></i>	For area sections the bond action of the prestressing steel cannot be taken into account. The bond coefficient $\xi_1$ therefore has no meaning.
<i>ds</i>	Selected bar diameter according to DIN 4227-1/A1, Section 6.7.3, Table 6.
<i><math>\beta_s</math></i>	Yield strength of the shear stirrups [MN/m <sup>2</sup> ]. When entering "0", the calculation is performed with $\beta_s = 500$ MN/m <sup>2</sup> .
<i><math>kz'</math></i>	Coefficient for calculating the lever arm $z$ of the inner forces with $kz' = z/dz$ . Thereby $dz$ is the element thickness defined in the section dialog. If $kz'$ is set to "0", the value is determined from the failure safety check and limited as follows: $0.1 \leq kz' \leq 1$ .

## Recalculation of railroad bridges according to DIN 4227 and Ril 805

During recalculation of railroad bridges, Ril 805 of Deutsche Bahn AG as of March 1, 2021, allows reduction of safety factors for actions according to chapter M 805.0102. When using DIN 4227, this results in necessary modifications of checks as per Technical Guideline TM 057/2004/011 of DB Netz AG.

Recalculation of railroad bridges according to DIN 4227 and Ril 805 [RIL805]	
Bending design, modify stress-strain curves:	<input type="checkbox"/>
Shear checks, modify stirrup reinf. strength $\beta_s$ and table values:	<input type="checkbox"/>
Reduction coeff. $\gamma_{M,c}$ acc. to TM 057/2004/011, chap. 3.4.3:	<input type="text" value="1.20"/>
Reduction coeff. $\gamma_{M,B}$ acc. to Ril 805.0202, table 1:	<input type="text" value="1.10"/>
Safety factor $\gamma_{M,s}$ acc. to Ril 805.0202, chap. 2 (1):	<input type="text" value="1.20"/>
Safety factor $\gamma_{M,c}$ acc. to Ril 805.0202, chap. 2 (1):	<input type="text" value="1.50"/>

The options are defined in the DIN 4227 Checks folder of the database.

Depending on the selection, the following modifications apply:

- Adjustment of the stress/strain curves with the factors according to M 805.0202 for concrete  $\gamma_{M,c}$ , reinforcing steel and prestressing steel  $\gamma_{M,s}$  as well as the building state  $\gamma_B$  during bending design.
- Adjustment of limit stresses according to DIN 4227, Ch. 12.3.1 (2), with the coefficient  $\gamma_c$  for classification in zone a and zone b.
- Reduction of the table values for the calculation of the compressive strut angle with the coefficient  $\gamma_c$  for the verification of the principal compressive stresses in zone a.
- Adjustment of the limit values for the principle tensile stresses (condition I) or shear stresses (condition II) with the coefficient  $\gamma_c$ , above which shear reinforcement must be verified.
- Reduction of the table values for the calculation of the compressive strut angle with the coefficient  $\gamma_c$  for the design of the shear reinforcement.
- Reduction of the strength of the shear stirrups with the factors for reinforcing steel  $\gamma_{M,s}$  and the building state  $\gamma_B$ .

## Single Design

The single design function allows you to analyze individual section polygons separately from the whole system using predefined internal forces. The following data needs to be entered in the *Single Design* table, which is located in the *DIN 4227 Checks* folder of the database.

<i>Section</i>	Number of the section to be designed.
<i>Br</i>	Concrete calculatory strength in [MN/m <sup>2</sup> ].
<i>Nx, My, Mz</i>	Design internal forces without safety factors.
<i>Mode BU</i>	Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
<i>Mode BS</i>	Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
<i>Mode BD</i>	For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to DIN 1045, Section 25.2.2.1 (1).
<i>Mode DZ</i>	Determine strain state for existing reinforcing steel layers.
<i>Mode IN</i>	Design disabled.
<i>Mode LF</i>	Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
<i>Mode MY</i>	Determination of the maximum bearable bending moment $M_y$ . The moment $M_y$ is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.

The calculation is carried out from the opened input table via the *Single Design* or *Page Preview* menu item.



## Checks

### Stress Checks

#### Beams and design objects

The stress calculation is performed on the section polygon taking into account 3D bending with normal force. Gross and net or ideal section properties are taken into account according to user specifications. First, the stresses are calculated at all polygon points and the extremal values are saved or printed out. The maximum prestressing steel stresses are also determined. As the tendons do not yet lie in the bond during prestressing, a change of the prestressing steel stresses as a result of  $G1$  is not applied (prestressing with subsequent bond).

If a beam section under bending load does not fully contribute, the areas, which do not contribute, are to be selected in the section dialog. Alternatively, the section can be reduced to the effective areas and a full polygon can be defined for the stress determination (see *Analysis settings*). In both cases the normal stresses from bending are calculated at the reduced section and the normal stresses from the normal forces at the full section.

#### Area elements

Here the stress calculation is performed at the rectangle section with a width of 1 m. Only the gross section properties are taken into account as single tendons cannot be unambiguously allocated to the section in shell structures. Due to the two-axis stress state, in addition to the components  $\sigma_{x'}$ ,  $\sigma_y$  and  $\tau_{xy}$  the principal stresses  $\sigma_1$  and  $\sigma_2$  on the top and bottom sides are also calculated.

## Failure Safety Check

#### Beams and design objects

The failure safety check is carried out for the user-defined combination rule under failure loads. The tendon groups in the bond are accounted for with their prestrain on the resistance side, as specified in DIN 4227.

In the design the internal forces of the concrete section are reduced by the statically determined shares from prestressing as well as creep and shrinkage (V+KS). So only the strained portions from V+KS and the external loads are contained in the remaining internal forces for the composite section. If necessary, the mild-steel reinforcement positioned by the user will be increased until the composite internal forces can be absorbed. For reinforcing steel layers under compression, the value of  $\beta_s$  is reduced by factor 1.75/2.1 according to DIN 4227 Part 1, Section 11.2.2.

The position of the tendon groups in the section, the statically determined portions from V+KS and the internal forces of the concrete section and the composite section are logged in detail:

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	1,574	-0,000	0,000	0,6008	0,5178	0,0234
net :	1,558	-0,002	-0,002	0,6004	0,5174	0,0230
ideally:	1,638	0,007	0,006	0,6024	0,5195	0,0251

Tendon groups with bond

No.	E-Modul [MN/m <sup>2</sup> ]	$\beta_s$ [MN/m <sup>2</sup> ]	$\beta_z$ [MN/m <sup>2</sup> ]	y [m]	z [m]	Av [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	205000	1420	1570	0,167	0,159	12800	50	11160,76	3,62

1. Failure Safety Combination (FC.1): 1,75\*(G1+G2+P)+V+KS+DS+T

Loss of prestress by KS in tendon groups

No.	KS [%]	No.	KS [%]	No.	KS [%]	No.	KS [%]
1	12,01		-.-		-.-		-.-

Stat. determ. part (V+KS): Nx0=-9801,10 kN; My0=-1560,46; Mz0=-1636,78 kNm

	Concrete section			Bond section		
	Nx [kN]	My [kNm]	Mz [kNm]	Nx [kN]	My [kNm]	Mz [kNm]
Nx-	-9633,50	-3519,18	-1,75	167,59	-1958,73	1635,03
Nx+	-9633,02	3219,15	-2,37	168,07	4779,61	1634,41
My-	-9633,49	-3580,61	-1,75	167,61	-2020,15	1635,04
My+	-9633,02	3219,15	-2,37	168,07	4779,61	1634,41
Mz-	-9633,08	2052,45	-2,38	168,02	3612,91	1634,40
Mz+	-9633,49	-3580,61	-1,75	167,61	-2020,15	1635,04

Labels used in the log:

ys, zs	Location of the centroid in the section.
y, z	Location of the tendon group in the section.
Av	Section area $A_v$ of the tendon group.
ZV0	Prestressing force $Z_{v0}$ .

<i>Inclin.</i>	Inclination $\alpha$ of the tendon group in respect to the beam axis.
<i>KS</i>	Percent of prestressing force lost due to creep and shrinkage.
<i>N<sub>x0</sub></i>	$N_{x0} = Z_{V0} \cdot (1 - KS / 100) \cdot \cos \alpha$
<i>M<sub>y0</sub></i>	$M_{y0} = N_{x0} \cdot (z - z_s)$
<i>M<sub>z0</sub></i>	$M_{z0} = N_{x0} \cdot (y - y_s)$

### Area elements

The failure safety check is performed in a similar manner. As a separation into statically determined and undetermined shares of the internal forces from prestressing is not possible for shell structures, the prestressing is taken into account fully on the action side and, as a result, on the resistance side only reinforcing steel and concrete are considered. Additionally, the design is performed taking into account the deviation of reinforcement and crack direction according to the same criteria as for the bending design according to DIN 1045. When determining the required reinforced concrete reinforcement for the structure, the stress reserves in the tendons, which cannot be taken into account in the check for the reasons given above, can be considered in the calculation. When doing so, in addition to the prestrain the distribution and height level of the tendons must be taken into account.

In either case, the determination of the design internal forces is carried out according to the design guidelines as described in DIN 1045.

### Crack Width Limitation

The crack check is performed according to DIN 4227-1/A1, Section 6.7.3. Here the crack reinforcement is determined in the areas where, under the rare action combination, the magnitude of the concrete compressive stress at the component edge is less than 1 MN/m<sup>2</sup>. For area elements the analysis is performed in the reinforcement direction on the top and bottom sides. The crack reinforcement is calculated according to the following equation:

$$\mu_s = A_s / A_{bZ} = 0.8 \cdot k \cdot k_c \cdot \beta_{bZ} / \sigma_s - \xi_1 \cdot \mu_z$$

with

$\mu_s$  Reinforcing steel reinforcement content relating to the section under tension.

$A_{bZ}$  Tensile zone directly before crack formation.

$A_s$  Reinforcing steel reinforcement in the tensile zone.

$k$  Coefficient that takes into account the secondary crack formation for thick components:

$$k = 1.00 \text{ for section heights } d \leq 0.30 \text{ m}$$

$$k = 0.65 \text{ for section heights } d \geq 0.80 \text{ m}$$

The section height is the thickness for area sections, for beam sections it is the height of the enclosing rectangle. Intermediate values for  $k$  are determined by linear interpolation.

$k_c$  Coefficient for consideration of the effect of the stress distribution within the tensile zone  $A_{bZ}$  before the initial crack formation as well as the change of the inner lever arm during the transition to state II:

with

$$k_c = 0.4 \cdot \left[ 1 + \frac{\sigma_{bv}}{k_1 \cdot \beta_{bZ} \cdot d_0 / d'} \right] \leq 1$$

$d_0$  Section height as defined for coefficient  $k$ .

$$d' = d_0 \text{ for } d_0 < 1 \text{ m}$$

$$d' = 1 \text{ m for } d_0 \geq 1 \text{ m}$$

$$k_1 = 1.5 \text{ for compressive normal force.}$$

$$k_1 = 2/3 \cdot d' / d_0 \text{ for tensile normal force.}$$

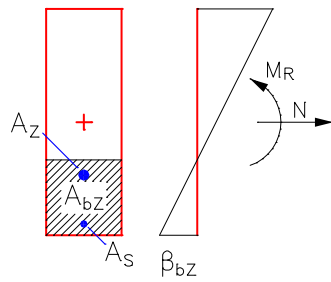
$\sigma_{bv}$  Central concrete stress share as a result of external normal force and 0.9 times the normal force from prestressing.

$\beta_{bZ}$  Central tensile strength of the concrete.

$\sigma_s$  Steel stress according to Table 6, taking into account the increase factor  $\sqrt{\beta_{bZ} / 2.7}$  for  $\beta_{bZ} > 2.7 \text{ N/mm}^2$ .

$\xi_1$  Bond coefficient for taking into account the contribution of the prestressing steel.

$\mu_z$  The prestressing steel reinforcement content corresponding to the section under tension  $A_z / A_{bZ}$ .



The determination of the tensile zone  $A_{bZ}$  is carried out under the assumption that directly before crack formation under normal force  $N$  resulting from permanent loads ( $G1$ ,  $G2$ ), prestressing ( $P$ ) as well as creep and shrinkage ( $KS$ ) through action of the crack moment  $M_R$ , the edge stress assumes the value  $\beta_{bZ}$ .

To determine the normal force  $N$ , the load factors of the rare action combination are taken into account. The same holds true for the calculation of the central concrete compressive stress  $\sigma_{bv}$  for determining the coefficient  $k_c$ . The direction of the crack moment is calculated at the beam section from the rare action combination; for area elements  $M_R$  acts in the direction of the reinforcement.

The bond coefficient  $\xi_1$  is used to influence the contribution of the prestressing steel. For area elements the tendon groups cannot be taken into account in the check. The program therefore sets  $\xi_1 = 0$  in the calculation.

Since normally two action combinations are defined with factors 0.9 and 1.1 for prestressing, the crack reinforcement results from the maximum of both combinations.

## Shear Check

The shear check according to DIN 4227-1, Section 12, is performed differently for the decisive internal force combinations for beam and shell structures.

### Beams and design objects

The following checks are performed:

- Diagonal principal tensile stresses according to state I in the serviceability state
- Diagonal principal compressive stresses according to the truss analogy at the failure state
- Design of the shear reinforcement from lateral force
- Determination of the torsion reinforcement at the equivalent hollow section

In case full polygons are defined in the analysis settings for the sections to be checked, their shear characteristics form the basis for the calculation, if not already effective widths have been selected in the section polygon.

#### Diagonal principal tensile stresses at serviceability state

A complete section analysis, which is automatically carried out as part of the checks, is necessary to determine the diagonal principal tensile stresses. The shear and section characteristics derived from this are used to determine the diagonal principal tensile stresses according to state I in the serviceability state at all result locations. The extreme values at the section edge are saved and logged. Normal stresses from diagonal bending with normal force as well as shear stresses from lateral force and torsion are taken into account.

The principal tensile stresses  $\sigma_1$  are limited according to Table 9, Lines 46 to 49. The check is limited to the area of the longitudinal compressive stresses according to Section 12.2 (1).

#### Diagonal principal compressive stresses at computed failure state

The check of the diagonal principal compressive stresses is performed as a function of the maximum edge tensile stress according to *Zone a* or *Zone b*.

#### Lateral Force, Zone a:

The main difficulty in determining the diagonal principal compressive stresses according to *Zone a* is the determination of the distribution of the average shear stresses as a result of the lateral force for freely selectable sections. For this purpose intersections perpendicular to the corresponding lateral force direction are created through all polygon points, side middle points and at the level of the section centroid. An additional intersection condensation is created by the program if required.

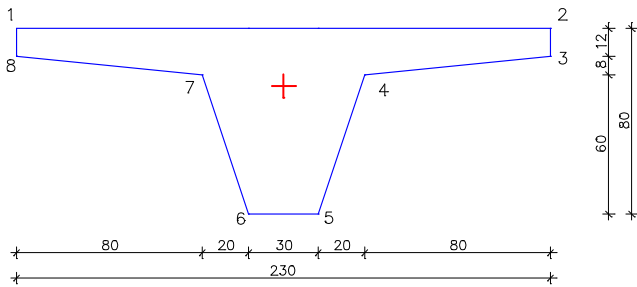
Along each intersection the distribution of the shear stresses  $\tau$  is calculated according to state I with the help of the boundary element method. From this the average shear stress  $\tau_m$  of a section results through integration and averaging over the corresponding section width. The following illustrations demonstrate this procedure using a continuous T-beam. The angle  $\vartheta$  of the struts in relation to the section normal is assumed as follows according to Equation (11):

$$\tan \vartheta = (1 - \Delta\tau / \tau_{Qu}) \tan \vartheta_1 \geq 0.4 \text{ with:}$$

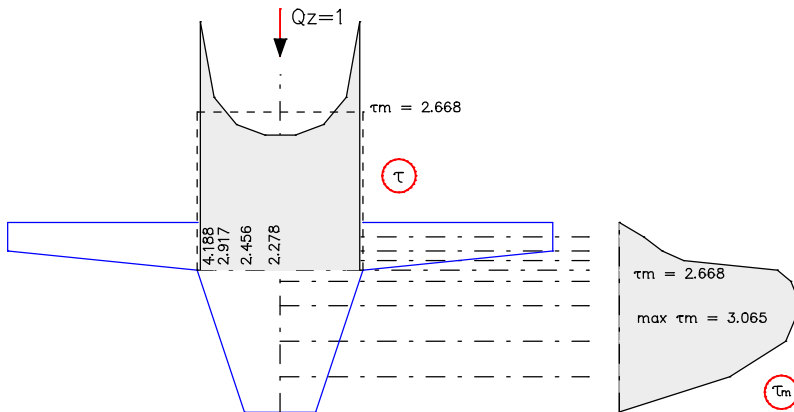
$\tau_{Qu}$  Maximum value of the existing shear stress  $\tau_m$  in the section as a result of lateral force according to state I.

$\Delta\tau$  60% of the shear stress value according to Table 9, Row 50.

$\vartheta_1$  Angle of the principal compressive stresses in state I in the centroid line of the girder or at the location of the greatest shear stress.



Section polygon [cm]

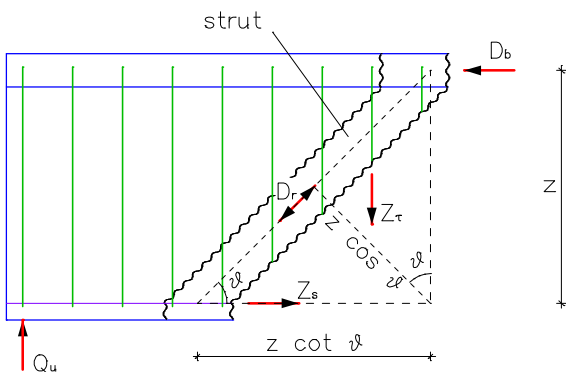


Intersections and shear stress distribution at the section polygon

The calculation of the diagonal principal compressive stresses from lateral force is carried out according to the truss analogy, i.e., loss of the diagonal principal tensile stresses given vertical shear stirrups. Taking into consideration the specifications of the following illustration and formulation of equilibrium, this results in:

$$Z_\tau = Z'_\tau z \cot \vartheta$$

$$D_\tau = D'_\tau z \cos \vartheta$$



Explanation of the truss analogy

$$\text{Intersection } \parallel \quad D_\tau \Rightarrow Z'_\tau = \frac{\tau_{Qu} \cdot b}{\cot \vartheta} \Rightarrow a_{S,bü}$$

$$\text{Intersection } \perp \quad D_{\tau} \Rightarrow D'_{\tau} = \frac{\tau_{Qu} \cdot b}{\cot \vartheta \cdot \sin \vartheta} \Rightarrow \sigma_{2,Qu}^{\parallel} = -\frac{2\tau_{Qu}}{\sin 2\vartheta}$$

$Z'_{\tau}$  and  $D'_{\tau}$  are the stirrup forces or concrete strut forces corresponding to the influence width. As a simplification, the shear distribution area  $b \cdot z$  was used here. The principal compressive stresses  $\sigma_2$  are limited according to user specification as per Table 9.

#### Lateral Force, Zone b:

In zone b the section is in the cracked state II by definition, meaning the shear cracks result from bending cracks. The decisive shear stress is determined by calculation according to DIN 4227.

$$\tau_{QR} = Q_u / (b \cdot z)$$

with

$z$  Lever arm of the inner forces in state II.

$b$  Width of the equivalent rectangle section.

The angle  $\vartheta$  of the struts in relation to the section normal is assumed as follows according to DIN 4227, Eq. 12:

$$\tan \vartheta = (1 - \Delta\tau / \tau_{QR}) \geq 0.4$$

with

$\tau_{QR}$  Calculated value of the shear stress as a result of lateral force according to state II.

$\Delta\tau$  60% of the shear stress value according to Table 9, Row 50.

The calculated value  $\tau_{QR}$  is limited according to Table 9, Row 56.

The diagonal principal compressive stresses result in the same way as *Zone a* and are also limited according to user specification as per Table 9.

#### Torsion:

The shear stresses from torsion are determined at the equivalent hollow section according to Figure 9.

$$\tau_{Tu} = \frac{M_{Tu}}{2d_1 A_k}$$

with

$d_1$  Wall thickness of the equivalent hollow section.

$A_k$  Core section area.

Assuming a constant strut angle of  $45^\circ$  and vertical stirrup orientation, the principal compressive stresses from torsion are:

$$\sigma_{2,Tu}^{\parallel} = -2\tau_{Tu}$$

These are superposed with the effect from lateral force and limited according to user specification as per Table 9.

#### Shear reinforcement from lateral force

The design of the shear reinforcement is carried out with the help of the corresponding stirrup tensile force  $Z'_{\tau}$  determined above and the yield strength  $\sigma_{s,u}$  of the shear stirrup and results in:

$$req a_{S,bü} = Z'_{\tau} / \sigma_{s,u}$$

As long as no other specification was made, shear reinforcement from lateral force is only added if the principal tensile stress (*Zone a*) resp. the shear stress (*Zone b*) exceeds the limit values according to user specification as per Table 9.

#### Torsion longitudinal and stirrup reinforcement

The required torsion longitudinal and stirrup reinforcement is calculated with respect to the unit of length as follows:

$$req a_{S,bü} = req a_{S,L} = \tau_{Tu} \cdot d_1 / \sigma_{s,u}$$

## Area elements

### Diagonal principal tensile stresses

The diagonal principal tensile stresses in the serviceability state are not analyzed for area elements.

### Shear stresses zone b

The check of the shear stresses is always performed in *Zone b* at the level of the zero line. The calculated value of the shear stress  $\tau_{QR}$  is determined in the same way as beams, however using the maximum lateral force

$$q_{\max} = \sqrt{q_x^2 + q_y^2}$$

of a check point. This is limited according to Table 9, Row 57.

### Shear reinforcement from lateral force

The shear design is performed according to the truss analogy for the maximum lateral force  $q_{\max}$  using the same procedure as for beams. This is only applied if the limit values according to Table 9, Row 51 are exceeded, where the reduction as per 12.4.1 (7) is not used. Alternatively, outputting the shear reinforcement independent of the limit value can be specified in the settings.

## Results

The reinforcement resulting from the failure safety check, crack width limitation and shear check is saved for graphical representation. In addition, results from the stress check can be displayed for all combinations defined in the settings.

### Stresses

$\sigma_x, \sigma_y$	Extremal normal stresses from bending and normal force [MN/m <sup>2</sup> ].
$\sigma_1, \sigma_2$	Extremal principal tensile and compressive stresses [MN/m <sup>2</sup> ].
$\tau$	Extremal shear stresses from lateral force and torsion [MN/m <sup>2</sup> ].
$\sigma_{xt} \dots \tau_t$	Stresses at the upper section edge [MN/m <sup>2</sup> ].
$\sigma_{xb} \dots \tau_b$	Stresses at the lower section edge [MN/m <sup>2</sup> ].

### Internal forces

The internal forces of the failure safety, crack and shear checks are listed in the detailed log.

### Bending Reinforcement

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.

### Shear reinforcement from lateral force

$a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area elements.
$A_{sb,y}$	Stirrup reinforcement of beams [cm <sup>2</sup> /m] from $Q_y$ .
$A_{sb,z}$	Stirrup reinforcement of beams [cm <sup>2</sup> /m] from $Q_z$ .
$z_y, z_z$	Inner lever arm [m] for lateral force $Q_y$ and $Q_z$ .

### Torsion reinforcement

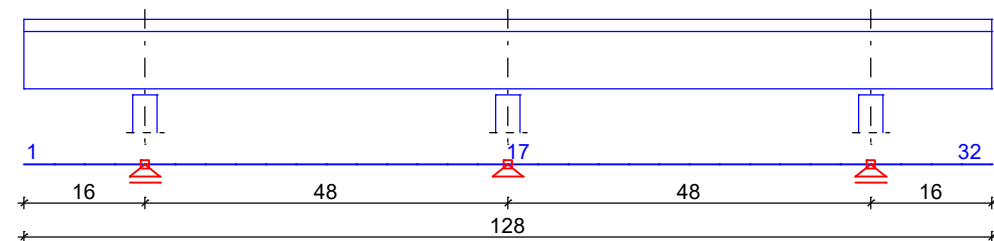
$A_{sb,T}$	Torsion reinforcement of beams [cm <sup>2</sup> /m].
$A_{sl,T}$	Longitudinal torsion reinforcement of beams [cm <sup>2</sup> ].

## Examples

### Prestressed Roof Construction

This example involves the wide-spanned roof construction of an entrance hall that is represented as a continuous girder over two spans with a double-sided cantilever. A T-beam is selected as the section. The figure below shows the system in longitudinal and lateral section view.

Limited prestressing with subsequent bond is applied to the roof construction in the longitudinal direction. Prestressing in the lateral direction is not applied for reasons of economy.

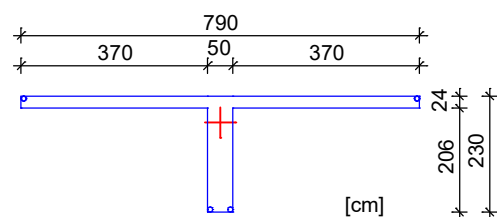


Static system and dimensions [m]

#### Material

Concrete *B55*  
Reinforcing steel *BSt 500/550*, axis distance from edge 5 cm

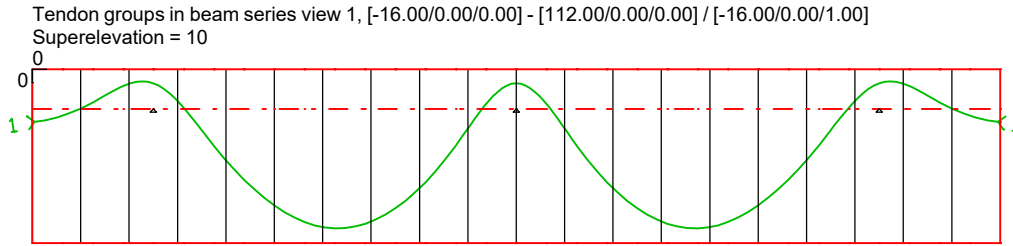
#### Section



#### Prestressing steel and prestressing system

Prestressing steel quality	St 1570/1770
Certification of the prestressing system	DIN 4227
Number of tendons in the bundle	4
Section surface $A_p$	1800 mm <sup>2</sup>
E-modulus of the prestressing steel	195000 MN/m <sup>2</sup>
Strain limit (yield strength) of the prestressing steel $\beta_s$	1570 MN/m <sup>2</sup>
Tensile strength of the prestressing steel $\beta_z$	1770 MN/m <sup>2</sup>
Permissible prestressing force of a tendon $P_{m0}$	1752 kN
Friction coefficients when prestressing and releasing $\mu$	0.21
Unintentional deviation angle of a tendon $\beta'$	0.0 °/m
Slippage at prestressing anchor	3 mm
Duct diameter $d_h$	80 mm

The tendon guide is shown in the next figure. 4 bundled tendons are arranged such that they stretch across the entire girder length and are prestressed at both girder ends. The prestressing system, prestressing procedure and prestressing curve for a tendon group are also shown.



Tendon group ordinates  $z_v$  [cm] at the base points

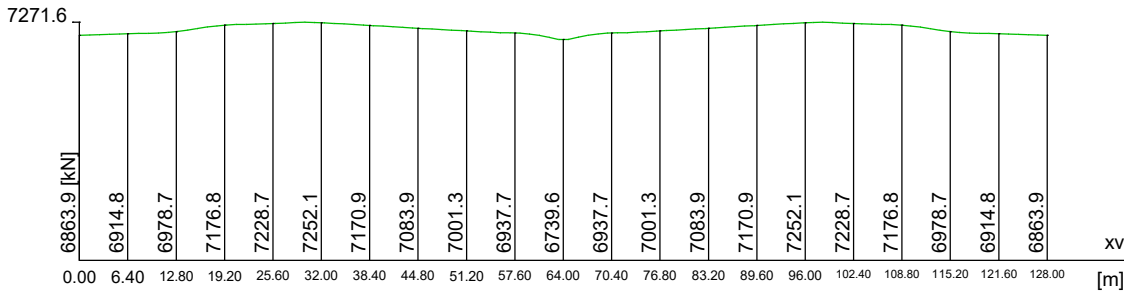
xv	0.00	6.40	12.80	19.20	25.60	32.00	38.40	44.80	51.20	57.60	64.00	70.40	76.80	83.20	89.60	96.00	102.40	108.80	115.20	121.60	128.00
1	70.0	52.0	19.2	42.0	120.2	180.1	208.8	201.5	157.4	78.3	18.5	78.3	157.4	201.5	208.8	180.1	120.2	42.0	19.2	52.0	70.0

Force function of tendon group 1 (4 tendon(s),  $l = 128.41$  m)

Prestressing system 1 - Example. Certification according to DIN 4227.  
 $P_{m0} = 1752.3$  kN,  $A_p = 1800.0$  mm<sup>2</sup>,  $\mu_a = 0.21$ , Angle  $\beta' = 0.00$  °/m  
 E-Modulus = 195000 MN/m<sup>2</sup>,  $A_h = 5026.5$  mm<sup>2</sup>,  $\mu_n = 0.21$ , Slippage = 3.00 mm

Prestressing procedure 1 - Example

Pre. anchor	:	Start		End
Normal. force	:	1.100	1.050	1.100
Pre. force [kN]	:	1927.5	1756.3	1927.5
Extension [mm]	:	617.7	-6.3	46.4



Tendon guide and prestressing curve in the longitudinal section

**Loads**

- Load case 1 Dead load ( $G_1$ )
- Load case 2 Additional dead load:  $q = 11.06$  kN/m ( $G_2$ )
- Load case 3 Snow load:  $p = 7.90$  kN/m ( $P$ )
- Load case 10 Prestressing ( $V$ )
- Load case 15 Creep-generating permanent load: ( $G_1 + V + G_2$ )
- Load case 20 Creep and shrinkage ( $KS$ )  
 Coefficients:  $\phi_{t\infty} = 2.55$ ,  $\rho = 0.8$ ,  $\epsilon_{t\infty} = -24.8 \cdot 10^{-5}$   
 creep-generating continuous load case: 15  
 The redistribution of internal forces between concrete and prestressing steel are taken into account.

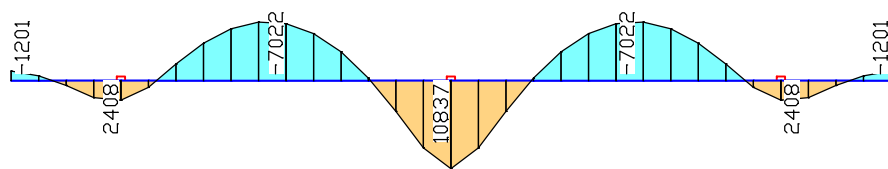
**Settings for DIN 4227 checks**

Sect ion	Full polyg	Concr ete	Cracking $\xi_1$	ds [mm]	$\beta_s$	Lateral force				Torsion			perm. Stress		
						y direct. d [m]	z direct. kz'	d [m]	kz'	y di. d [m]	z di. d [m]	$\ddot{u}$ [m]	S1, Tau (T,Q+T)	S2	
1	1	0	B55	0.30	12	500	0.50	0.80	2.30	0.00	0.50	2.30	0.000	1	0

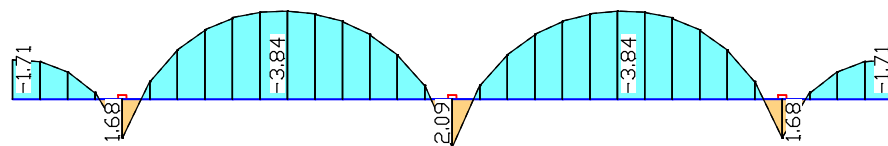
- $\xi_1$  Coefficient for beam sections DIN 4227-1/A1:1995-12, Sect. 6.7.3
- ds Selected bar diameter DIN 4227-1/A1:1995-12, Tab. 6
- d Width of substitute section, shear and/or torsion rectangle
- kz' Coefficient for the inner lever arm referring to d, "0": from failure safety check ( $kz' = z/d$ )
- $\ddot{u}$  Distance of core section from cross-section edge, "0": no torsional reinforcement
- S1, Tau "0": Tab. 9, lines 53, 54, 59,60 (box section); "1": Tab. 9, lines 52, 55, 58, 61 (full section)
- S2 "0": Tab. 9, line 62 (web); "1": Tab. 9, line 63 (chort)

In the following the curve of the bending moments and the normal stresses are shown as an example:

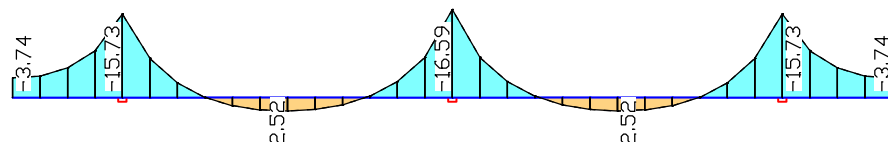




$M_y$  as a result of prestressing [kNm]

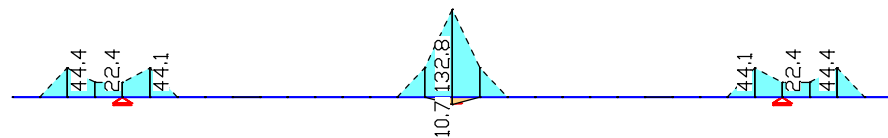


Normal stresses [MN/m<sup>2</sup>] from combination (G1+G2+V+KS+P) on the top side

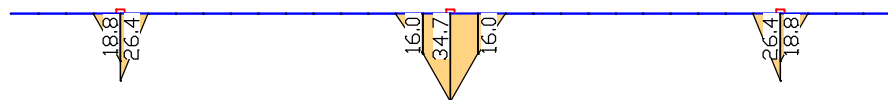


Normal stresses [MN/m<sup>2</sup>] from combination (G1+G2+V+KS+P) on the bottom side

Stress checks, failure safety, crack width limitation and shear checks are carried out according to the corresponding specifications. The following illustration shows the curve of the required bending and shear reinforcement.



Bending reinforcement [cm<sup>2</sup>] from failure safety check and crack width limitation (upper reinforcement with dotted lines).



Stirrup reinforcement  $A_{sz}$  from shear check [cm<sup>2</sup>/m].

The following pages contain excerpts from the check log for beam 16 at location 2 (middle column).

**Prestressed concrete design per DIN 4227-1/A1:1995-12**

The design occurs for components with partial prestress and supplemental bond (Part 1+4).

**All checks are made for the extreme values of actions.**

For the flexural design of prestressed beam sections with bond, the internal forces of the bond section and the tendon groups with their prestressing are considered.

With this, the situations before pressure grouting are excluded.

**Design settings**

Se.	Reinforcem.				Concrete stress			P. steel stress
	R	B	Q	T	Long.	tens.	compr.	
1	x	x	x	.	x	x	x	x

- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement from design at failure state.
- (Q) Lateral force reinforcement from design at failure state.
- (T) Torsional reinforcement from design at failure state.

**Settings for flexural and shear reinforcement**

$\beta_s$  Quality of stirrups [MN/m<sup>2</sup>].  
 Zone The shear check for zone a is performed like zone b as per Ch. 12.3.2.  
 Asb The shear reinforcement as per Ch. 12.4.1 is always outputted.

Se.	Concr.	Design mode for M, N	$\beta_s$ [MN/m <sup>2</sup> ]	Zone a like b	Always output Asb
1	B55	Standard	500	.	.

**Shear cross-sections**

dy, dz Width and height of the equivalent cross-section.  
 kz' Coeff. for calculating the inner lever arm z:  $kz' = z / d$ .  
 (\*) z = Distance between concrete compr. force and steel tens. force as maximum from flexural design at check location with  $0.1 \cdot d \leq z \leq 1$ .

Se.	Lateral force				Torsion		
	dy[m]	kz.y'	dz[m]	kz.z'	dy[m]	dz[m]	ül[m]
1	0.500	0.80	2.300	(*)	.	.	.

**Permissible concrete stresses as per DIN 4227-1, Table 9, Lines 46 to 63**

Q,T,Q+T Action: Lateral force, torsion, lateral force plus torsion.  
 S1 Diagonal principal tensile stress at serviceability state.  
 S2 Diagonal principal compressive stress at ultimate state.  
 Tau Shear stress at ultimate state.

Se.	Concr.	Serviceab. st.			Ultimate state			Compon.	per. S2 [MN/m <sup>2</sup> ]	
		Q,T	Q+T	partial prestr.	per. S1, Tau	Base val.	Tau			
1	B55	3.0	.	Full Se	2.2	.	9.0	.	Web	25.0

**Setting for crack width limitation**

βbZ Central concrete tensile strength as per Ch. 6.7.3, Eq. (1).  
 Xi1 Bond coeff. for considering the contribution of prestressing steel.  
 ds Diameter of the reinforcing bars.

Se.	βbZ [MN/m <sup>2</sup> ]	Xi1	ds [mm]
1	3.1	0.30	12

**Section geometry and reinforcing steel for beams**

Se.	Point	Concrete		Reinforcing steel					
		y [m]	z [m]	Es, βs	βs	βz	Prestr. system	Prestr. proce.	
1	1	0.000	0.000	210000	500	0.050	0.050	0.050	0.00
	2	3.700	0.000	210000	500	7.850	0.050	0.050	0.00
	3	4.200	0.000	210000	500	4.150	2.250	0.050	0.00
	4	7.900	0.000	210000	500	3.750	2.250	0.050	0.00
	5	7.900	0.240						
	6	4.200	0.240						
	7	4.200	2.300						
	8	3.700	2.300						
	9	3.700	0.240						
	10	-0.000	0.240						

**Tendon groups**

Type B Tendon group for prestressing of beams.  
 βs, βz Yield strength, tensile strength of prestr. steel as per DIN 4227.

T.group	Tendons	E-Modul	βs	βz	Prestr. system	Prestr. proce.				
No.	Type	Number[cm <sup>2</sup> ]	[MN/m <sup>2</sup> ]	[MN/m <sup>2</sup> ]	No.	Certific.				
1	B	4	18.00	195000	1570	1770	1	DIN 4227	1	.

**Actions and action combinations**

	LC/C	Cross-section
G1 (Permanent loads)	: 1	net
G2 (Additional loads)	: 2	ideall
V (Prestressing)	: 10	net
P (Traffic loads)	: 3	ideall
KS (Creep & shrinkage)	: 20	ideall
Failure safety check	: 1.75*(G1+G2+P)+V+KS	
Shear check at ultimate state	: 1.75*(G1+G2+P)+V+KS	
Shear check at serviceab. state	: G1+G2+P+V+KS	
Crack width limitation	: G1+G2+P+0.90*(V+KS)	
"	: G1+G2+P+1.10*(V+KS)	
Stress check	: G1+V	
"	: G1+V+G2+P	
"	: G1+V+G2+P+KS	
"	: G1+V+G2+0.7*P+KS	

**DESIGN FOR BEAMS**

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (R) Nominal reinforcement as per 6.7.3, A1:1995-12 for crack width limitation
- ds Maximal given steel diameter [mm]
- k Coefficient for consideration of non-linear distributed tensile stress
- βbZ Eff. central tensile concrete strength as per Ch. 6.7.3(1) [MN/m<sup>2</sup>]
- kc Coefficient to consider stress distribution in tensile zone according to Eq. (1), A1:1995-12
- Av Full section area for stress calc. from longitudinal force [m<sup>2</sup>]
- Az' Part of prestr. steel area Xi1\*Az which was used to reduce req.As
- Xi1 Bond coefficient for prestressing steel as per Eq. (1), A1:1995-12
- max Sx Maximal concrete edge stress from action combination [MN/m<sup>2</sup>]
- (B) Failure safety check
- In case of dominant bending, compression reinforcement is marked with "\*\*\*".
- βR Concrete strength as per Figure 6 [MN/m<sup>2</sup>]
- NO, MO Statically determined forces of tendons with bond [kN, kNm]
- βs, βz Yield strength, tens. strength of prestr. steel per DIN 4227 [MN/m<sup>2</sup>]

**Beam 16**

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - B55, 1 tendon group with bond  
 Steel 1; Design mode: Standard  
 (R) ds=12; k=0.65; βbZ=3.61561; Xi1=0.3  
 (B) βR=33

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.906	3.950	0.527	1.2537	9.8822	0.0000
ideally:	2.955	3.950	0.522	1.2593	9.8822	0.0000

Tendon groups with bond

No.	E-Modul [MN/m <sup>2</sup> ]	βs [MN/m <sup>2</sup> ]	βz [MN/m <sup>2</sup> ]	y [m]	z [m]	Az [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1570	1770	3.950	0.185	7200	80	6739.60	0.00

1. Crack Safety Combination (CC.1): G1+G2+P+0.90\*(V+KS)

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]
Nx-	-5662.71	-11248.46	0.00

2. Crack Safety Combination (CC.2): G1+G2+P+1.10\*(V+KS)

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]
Nx-	-6921.09	-9163.08	0.00

1. Failure Safety Combination (FC.1): 1.75\*(G1+G2+P)+V+KS

Loss of prestress by KS in tendon groups

No.	KS [%]	No.	KS [%]	No.	KS [%]	No.	KS [%]
1	4.92	-	-	-	-	-	-

Stat. determ. part (V+KS): Nx0=-6408.25 kN; My0=2178.80; Mz0=0.00 kNm

	Concrete section			Bond section		
	Nx [kN]	My [kNm]	Mz [kNm]	Nx [kN]	My [kNm]	Mz [kNm]
Nx-	-6291.90	-25680.25	0.00	116.34	-27859.06	0.00

Design of longitudinal reinforcement

Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	max Sc [MN/m <sup>2</sup> ]	kc	Az' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	Situation
1	R	-5662.71	-11248.46	0.00	2.76	0.34	10.80	11.47	CC.1,Nx-
	B	-6291.90	-25680.25	0.00	.	.	.	66.38	FC.1,Nx-
2	R	-5662.71	-11248.46	0.00	2.76	0.34	10.80	11.47	CC.1,Nx-
	B	-6291.90	-25680.25	0.00	.	.	.	66.38	FC.1,Nx-
3	R	-5662.71	-11248.46	0.00	2.76	.	.	0.00	CC.1,Nx-
	B	-6291.90	-25680.25	0.00	.	.	.	5.37*	FC.1,Nx-
4	R	-5662.71	-11248.46	0.00	2.76	.	.	0.00	CC.1,Nx-
	B	-6291.90	-25680.25	0.00	.	.	.	5.37*	FC.1,Nx-

Design of shear reinforcement, compressive and shear stresses at failure state

The percentage of nominal reinforcement acc. to Ch. 6.7 is not considered.

βs	Quality of stirrups [MN/m <sup>2</sup> ].
kz'	Coeff. for calculating the inner lever arm z: kz' = z / d
z	Inner lever arm [m]
z(..)	Max. lever arm z from failure safety check [m]
dy, dz	Width and height of the equivalent cross-section [m]
Qy, Qz	Lateral forces for design in y- and z-direction [kN]
Theta.Q	Inclination of the concrete compressive struts [°] as per Eq. (11) in zone a (State I) und Eq. (12) in zone b (State II)
Tau.Q	Shear stress from lateral force [MN/m <sup>2</sup> ], mean over the inters. line at the polygon in zone a (State I), calc. value Tau.r in zone b (State II)
Sigma1	Diagonal principal tensile stress, zone a [MN/m <sup>2</sup> ]
y, z, w	Position and width of the intersection line at the polygon, zone a [m]
~, req.	Tensile/shear stress per Table 9, line 50-55, for req. Asb in zone a/b
Sigma2	Diagonal principal compr. stress at failure state, zone a and b [MN/m <sup>2</sup> ]
Asb.y,z	Required stirrup reinforcement from lateral force Qy resp. Qz [cm <sup>2</sup> /m]

Beam 16

Location 2

Beam 16, x = 4.00 m (Beam length 4.00 m)  
 Cross-section 1: Polygon - B55, 1 tendon group with bond  
 βs=500  
 Lat. force: dy/dz=0.5/2.3; kz.y'=0.8; Lever arm z.z from failure s. check

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.906	3.950	0.527	1.2537	9.8822	0.0000
ideally:	2.955	3.950	0.522	1.2593	9.8822	0.0000

1. Shear Combination Failure State (SU.1): 1.75\*(G1+G2+P)+V+KS

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
Qy-	-6291.90	-25680.25	0.00	0.00	0.00	-4012.37

Max. edge tensile stress as per Ch. 12.3.1(2) = 8.58 MN/m<sup>2</sup> => Zone b

Diagonal principal compressive and shear stresses at failure state, zone b

Action	z [m]	Theta [°]	Tau.r, per [MN/m <sup>2</sup> ]	req. [MN/m <sup>2</sup> ]	Sigma2, per [MN/m <sup>2</sup> ]	Situation
max						
Qy	0.40	21.8	0.00	9.0	-0.00	SU.1,Qy-
Qz	1.68(1)	35.9	4.79	9.0	-10.08	SU.1,Qy-

Design of shear reinforcement in zone b

Action	z [m]	Theta.Q [°]	Tau.Q [MN/m <sup>2</sup> ]	Tau.T [MN/m <sup>2</sup> ]	Asb.y [cm <sup>2</sup> /m]	Asb.z [cm <sup>2</sup> /m]	Asl.T [cm <sup>2</sup> ]	Situation
max								
Qy	0.40	21.8	0.00	.	.	.	.	SU.1,Qy-
Qz	1.68(1)	35.9	4.79	.	34.68	.	.	SU.1,Qy-

Lever arm Situation from failure safety check

(1) : FC.1,Nx- (Max. from all internal forces sets)

**Check of concrete stress and prestressing steel stress**

For the check, a non-cracked concrete section is assumed.  
The permissibility of stresses as per Table 9 is not verified.

Sx Concrete stress from bending and longitudinal force [MN/m<sup>2</sup>]  
Sz Prestressing steel stress [MN/m<sup>2</sup>]

**Beam 16**

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
Cross-section 1: Polygon - B55, 1 tendon group with bond

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.906	3.950	0.527	1.2537	9.8822	0.0000
ideally:	2.955	3.950	0.522	1.2593	9.8822	0.0000

**Tendon groups with bond**

No.	E-Modul [MN/m <sup>2</sup> ]	βs [MN/m <sup>2</sup> ]	βz [MN/m <sup>2</sup> ]	y [m]	z [m]	Az [mm <sup>2</sup> ]	Duct d [mm]	Prestress [kN]	Inclin. [°]
1	195000	1570	1770	3.950	0.185	7200	80	6739.60	0.00

**1. Stress Combination: G1+V**

	max Sx [MN/m <sup>2</sup> ]	Se.- pnt.	min Sx [MN/m <sup>2</sup> ]	Se.- pkt.	max Sz [MN/m <sup>2</sup> ]	Tendon no.
top	0.01		0.01			
bottom	-10.17		-10.17			
Extremum	0.01	4	-10.17	8	936.06	1

**2. Stress Combination: G1+V+G2+P**

	max Sx [MN/m <sup>2</sup> ]	Se.- pnt.	min Sx [MN/m <sup>2</sup> ]	Se.- pkt.	max Sz [MN/m <sup>2</sup> ]	Tendon no.
top	1.77		1.77			
bottom	-16.16		-16.16			
Extremum	1.77	4	-16.16	8	941.73	1

**3. Stress Combination: G1+V+G2+P+KS**

	max Sx [MN/m <sup>2</sup> ]	Se.- pnt.	min Sx [MN/m <sup>2</sup> ]	Se.- pkt.	max Sz [MN/m <sup>2</sup> ]	Tendon no.
top	2.09		2.09			
bottom	-16.59		-16.59			
Extremum	2.09	4	-16.59	8	895.71	1

**4. Stress Combination: G1+V+G2+0.7\*P+KS**

	max Sx [MN/m <sup>2</sup> ]	Se.- pnt.	min Sx [MN/m <sup>2</sup> ]	Se.- pkt.	max Sz [MN/m <sup>2</sup> ]	Tendon no.
top	1.87		1.87			
bottom	-15.84		-15.84			
Extremum	1.87	4	-15.84	8	895.00	1

**Check of diagonal principal tensile stresses at serviceability state**

The check is limited to the area of longitudinal compressive stress.

Sigmal,max Maximum principal tensile stress at state I [MN/m<sup>2</sup>]  
 Sigmal,per Permissible principle tensile stress as per Table 9 [MN/m<sup>2</sup>]  
 y, z Edge point of the section with Sigmal,max [m]  
 Sigma.x Corresponding stress due to bend and longitudinal force [MN/m<sup>2</sup>]  
 Tau.xy, Tau.xz Corresponding edge shear stresses from lateral force and torsion in y- and z-direction [MN/m<sup>2</sup>]  
 Thetal Angle between Sigmal and the longitudinal stress Sigma.x [°]

**Beam 16**

**Location 2**

Beam 16, x = 4.00 m (Beam length 4.00 m)  
Cross-section 1: Polygon - B55, 1 tendon group with bond

Section properties	A [m <sup>2</sup> ]	ys [m]	zs [m]	Iy [m <sup>4</sup> ]	Iz [m <sup>4</sup> ]	Iyz [m <sup>4</sup> ]
gross :	2.926	3.950	0.525	1.2560	9.8822	0.0000
net :	2.906	3.950	0.527	1.2537	9.8822	0.0000
ideally:	2.955	3.950	0.522	1.2593	9.8822	0.0000

**1. Shear Combination Serviceab. State (SS.1): G1+G2+P+V+KS**

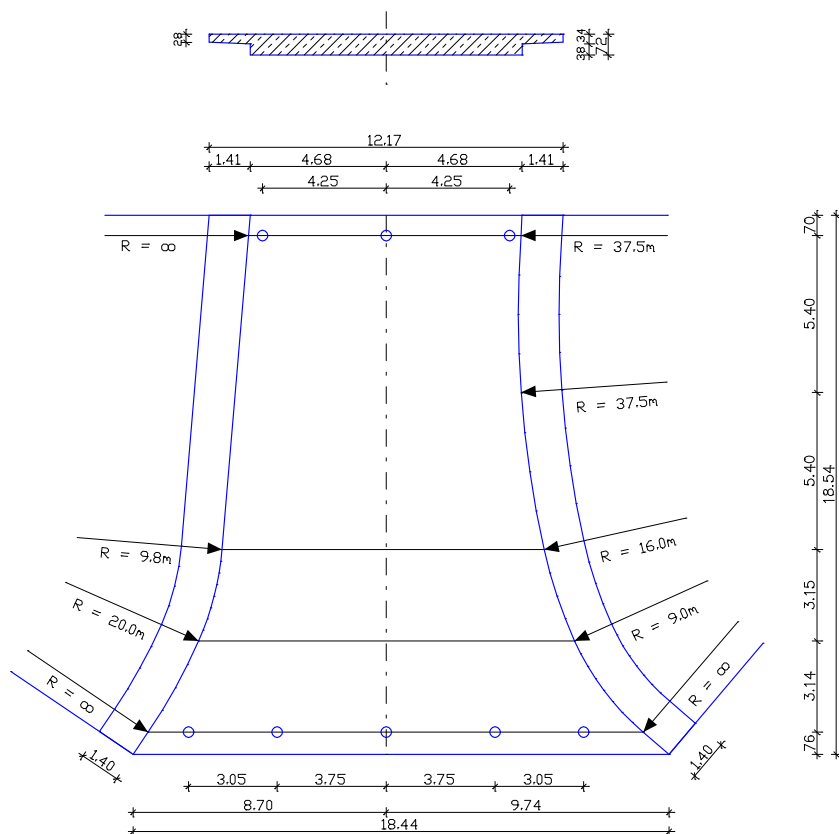
Concrete internal forces	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
Qy-	-6291.90	-10205.77	0.00	0.00	0.00	-2216.23

**Check of diagonal principal tensile stress in the zone of long. compr. stress**

Tau from	Sigmal max	Sigmal per.	Thetal [°]	Tau.xy [MN/m <sup>2</sup> ]	Tau.xz [MN/m <sup>2</sup> ]	Sigma.x [MN/m <sup>2</sup> ]	Section point y [m]	Section point z [m]	Sit.
Qy :	0.00	3.00	0.0	0.00	0.00	-0.43	4.200	0.309	Qy-
Qz :	0.00	3.00	0.4	0.11	0.00	-16.56	4.033	2.300	Qy-
Mx :	2.54	3.00	42.8		2.74	-0.43	4.200	0.309	Qy-
Qy+Mx :	0.00	3.00	0.0	0.00	0.00	-0.43	4.200	0.309	Qy-
Qz+Mx :	0.00	3.50	0.4	0.11	0.00	-16.56	4.033	2.300	Qy-
	2.54	3.50	42.8		2.74	-0.43	4.200	0.309	Qy-

## Prestressed Slab Bridge

The following example analysis demonstrates the application of a 3D tendon guide using a slab bridge that is curved in the ground plan.



Static system and dimensions

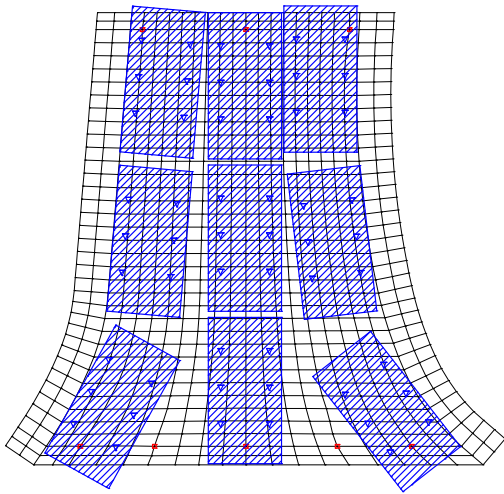
### Material

Concrete	B45
Reinforcing steel	BSt 500/550, overlap 5 cm
Prestressing steel	ST 1570/1770

### Loads

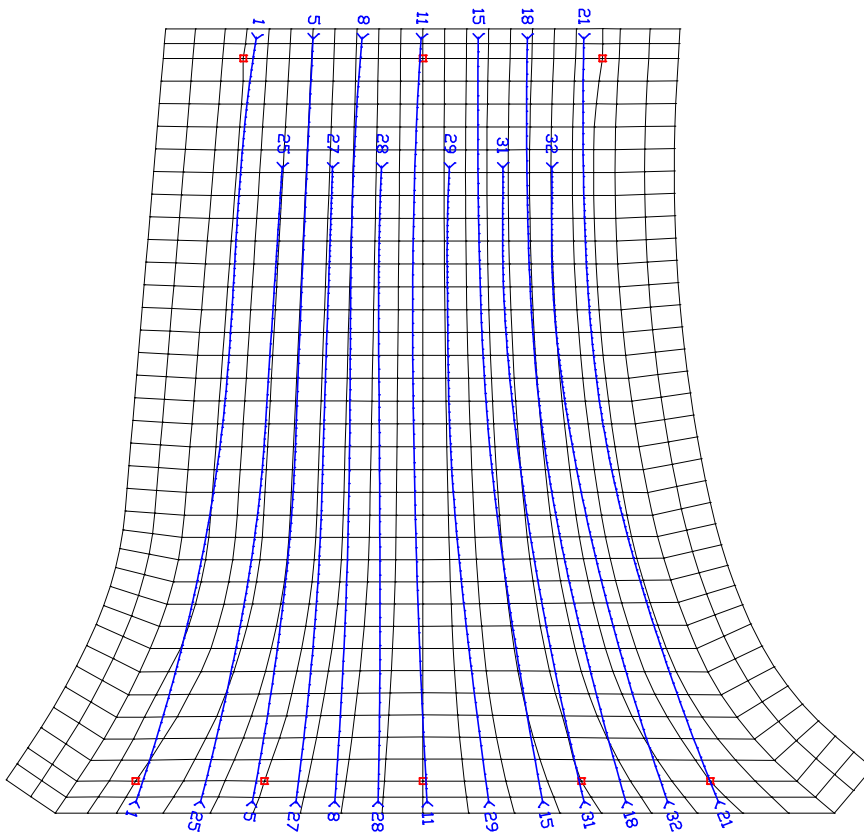
- Load case 1 Dead load, roadway pavement 2 kN/m<sup>2</sup>, edge load 5 kN/m (*G1*).
- Load case 2..10 SLW 60 (heavy truck) in different load positions according to the illustration (*P*).
- Load case 11 Principal traffic lane 5 kN/m<sup>2</sup>, secondary lane 3 kN/m<sup>2</sup> (*P*).
- Load case 12 Prestressing (*V*).
- Load case 13 Creep-generating permanent load: *G1+V*
- Load case 14 Creep and shrinkage (*KS*).  
Coefficients:  $\varphi_{t\infty} = 2.40$ ,  $\rho = 0.8$ ,  $\epsilon_{t\infty} = -30 \cdot 10^{-5}$   
creep-generating continuous load case: 13  
The redistribution of internal forces between concrete and prestressing steel are taken into account.

To keep the example analysis simple, only 9 heavy truck positions were analyzed. The extremal traffic loads *P* for the prestressed concrete check are obtained by combining the loads from load cases 2 to 11.



Analyzed heavy truck positions

The following figure shows the element mesh with tendons in the ground plan. Subsequently, the relevant input data for the FEM analysis are provided.



**Material properties**

No.	Material type	EModule [MN/m <sup>2</sup> ]	GModule	Poisson ratio	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]
1	B45	37000	15000	0.20	1.000e-05	25.000
2	B45	37000	15000	0.20	1.000e-05	25.000
3	B45	37000	15000	0.20	1.000e-05	25.000

**Creep and shrinkage properties**

Material	phi.t	rho	epsilon.s
1	2.400	0.800	-30.00E-05
2	2.400	0.800	-30.00E-05
3	2.400	0.800	-30.00E-05

**Section properties**

No. 1 FL d=0.72  
 No. 2 FL d=0.34  
 No. 3 FL d=0.28

**Prestressing properties**

Tendon group No.	count of tendons	system no.	procedure no.	type	max. P [kN]	min R [m]	length [m]
1	4	1	1	Area	6349.4	31.9	18.4
5	3	1	1	Area	4799.4	38.4	18.2
8	3	1	1	Area	4773.0	58.3	18.1
11	4	1	1	Area	6372.9	75.6	18.1
15	3	1	1	Area	4786.2	47.7	18.2
18	3	1	1	Area	4793.9	32.5	18.3
21	4	1	1	Area	6403.6	25.1	18.6
25	2	1	1	Area	3128.3	19.7	15.2
27	1	1	1	Area	1565.2	19.6	15.1
28	1	1	1	Area	1567.5	23.2	15.1
29	2	1	1	Area	3130.0	21.3	15.1
31	1	1	1	Area	1560.1	17.8	15.2
32	1	1	1	Area	1559.9	17.5	15.4
Total:	32						219.1

**Prestressing systems (values for a single tendon)**

No.	Area [mm <sup>2</sup> ]	E-Module [MN/m <sup>2</sup> ]	fp01k	fpk	Pm0 [kN]	μa	μn	β' [°/m]	slip [mm]	dh [mm]	licensed by
1	1680.0	195000	1570	1770	1635	0.20	0.20	0.30	3.00	80	DIN 4227

**Prestressing procedures**

No.	Normalized force at beginning				Normalized force at end				Kappa
	Tens.-1	Rel.-1	Tens.-2	Rel.-2	Tens.-1	Rel.-1	Tens.-2	Rel.-2	
1	1.05	0.00	0.00	0.00	1.02	0.00	0.00	0.00	-.-

**Load case combination 1, Traffic - variable inclusive action**

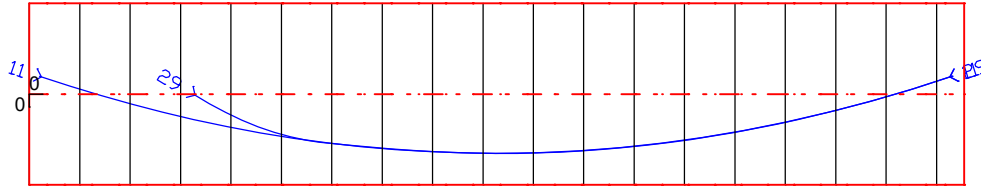
Load case	Factor
11 Traffic on Head- and secondary	1.000

**Load case combination 1, Traffic - 1. variable exclusive action**

Load case	Factor
2 Traffic SLW 1	1.000
3 Traffic SLW 2	1.000
4 Traffic SLW 3	1.000
5 Traffic SLW 4	1.000
6 Traffic SLW 5	1.000
7 Traffic SLW 6	1.000
8 Traffic SLW 7	1.000
9 Traffic SLW 8	1.000
10 Traffic SLW 9	1.000

The tendon curve of two tendon groups is shown in the slab longitudinal section together with ordinate table and prestressing curve for tendon group 11.

Tendon groups in sectional view 1, [10.20/-5.00/0.00] - [10.29/13.54/0.00] / [10.20/-5.00/1.00]  
 Superelevation = 5



Tendon group ordinates  $z_v$  [cm] at the base points

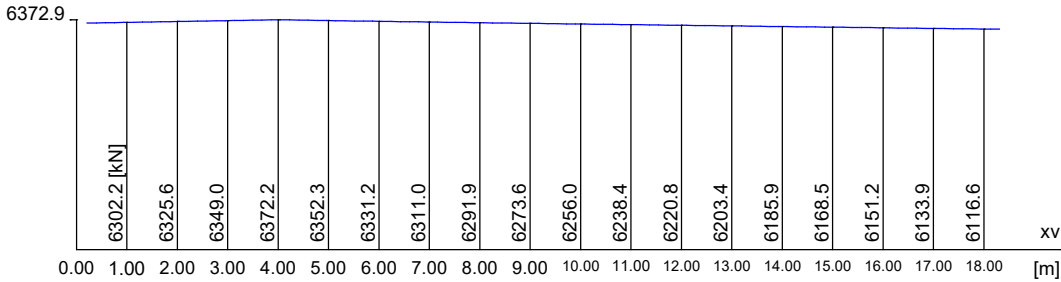
xv	0.00	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00	15.00	16.00	17.00	18.00
11		-1.9	3.9	8.9	13.2	16.8	19.6	21.6	22.9	23.5	23.3	22.4	20.7	18.3	15.1	11.2	6.5	1.1	-5.1
29					8.3	16.0	19.6	21.4	22.8	23.4	23.4	22.5	20.8	18.3	15.1	11.2	6.5	1.1	-5.1

Force function of tendon group 11 (4 tendon(s),  $l = 18.12$  m)

Prestressing system 1 - Example. Certification according to DIN 4227.  
 $P_{m0} = 1635.0$  kN,  $A_p = 1680.0$  mm<sup>2</sup>,  $\mu_a = 0.20$ , Angle  $\beta' = 0.30$  °/m  
 $E$ -Modulus = 195000 MN/m<sup>2</sup>,  $A_h = 5026.5$  mm<sup>2</sup>,  $\mu_n = 0.20$ , Slippage = 3.00 mm

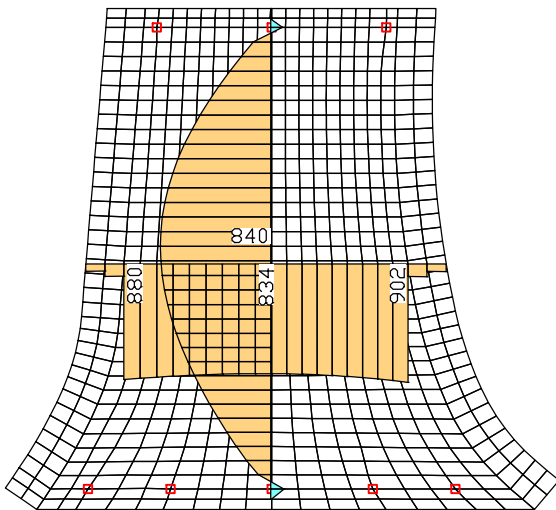
Prestressing procedure 1 - Example

Pre. anchor : Start End  
 Normal. force : 1.050 1.020  
 Pre. force [kN]: 1716.8 1667.7  
 Extension [mm]: 92.2 0.3

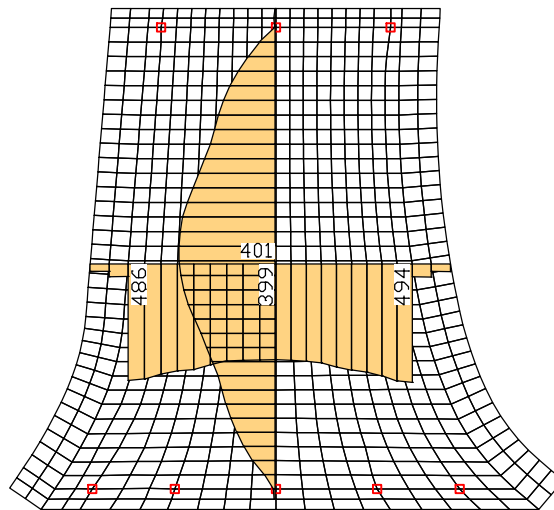


Tendon guide and prestressing curve in the longitudinal section

The other illustrations show internal forces and stresses for the selected loads. The illustration of the internal forces from prestressing shows that in the middle area of the slab the prestressing forces are uniformly distributed.

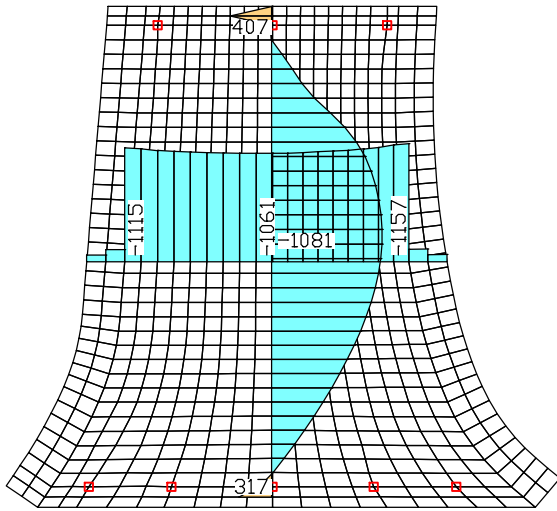


$m_y$  [kNm/m] from permanent load (LC 1)

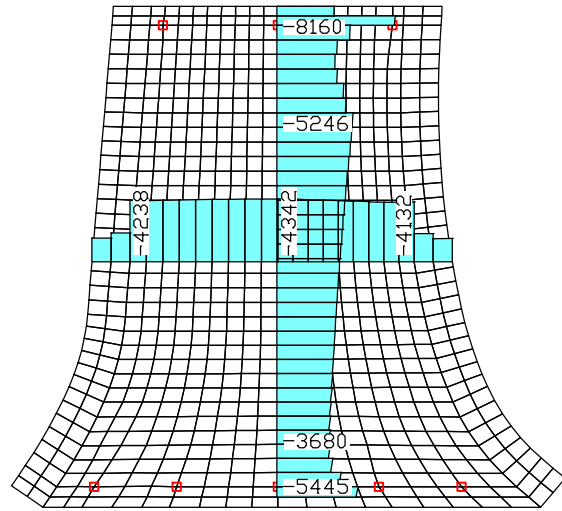


max  $m_y$  [kNm/m] from traffic (LCC 1)

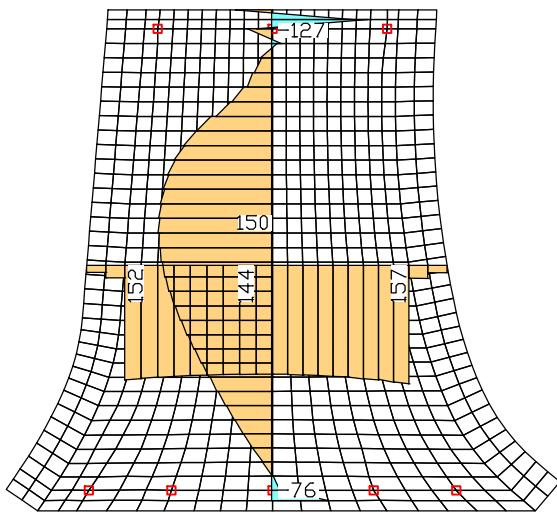




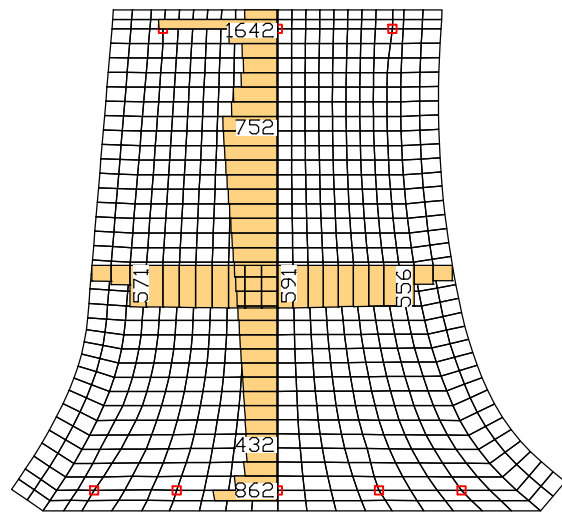
$m_y$  [kNm/m] from prestressing (LC 12)



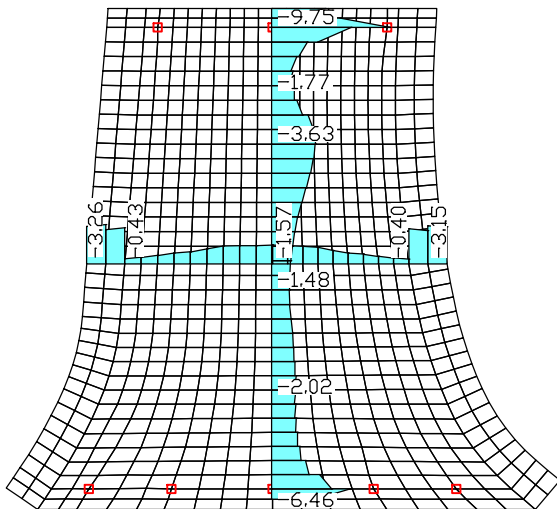
$n_y$  [kN/m] from prestressing (LC 12)



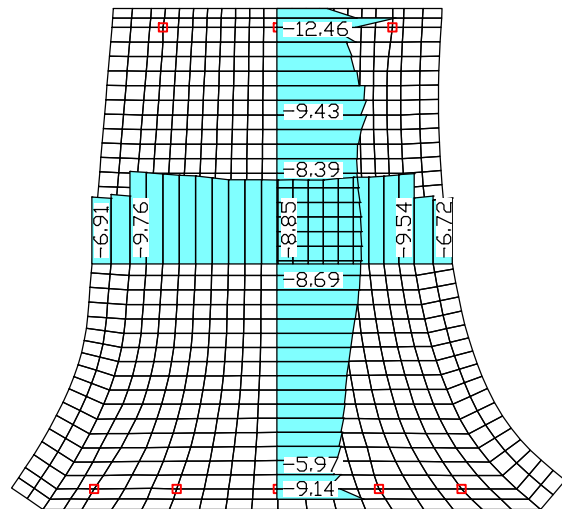
$m_y$  [kNm/m] from KS (LC 14)



$n_y$  [kNm/m] from KS (LC 14)



$\max \sigma_y$  [MN/m<sup>2</sup>] from  $G1+V+KS+P$   
on the bottom side



$\min \sigma_y$  [MN/m<sup>2</sup>] from  $G1+V+KS+P$   
on the top side

As you can see, the slab is fully prestressed in the y direction (slab longitudinal direction). In the lateral direction, small tensile stresses result that can be absorbed by the reinforced concrete reinforcement (see *checks*) or by lateral prestressing. The slab's lateral direction is not analyzed in this example. The additional specifications for the prestressed concrete checks according to DIN 4227 are listed in the following.

**Prestressing settings**

**Design load cases**

Name	Load case	Section
G1	1 Dead load	gross
V	12 Prestressing	gross
P	K1 Traffic	gross
KS	14 Creep, shrinkage	gross

**Stress design**

No.	Combination
1	G1+V
2	G1+V+KS
3	G1+V+KS+0,5*P
4	G1+V+KS+P

**Failure safety**

Combination:  $1,75*(G1+P)+V+KS$

**Crack width limitation**

- 1. combination:  $G1+P+1,1*(V+KS)$
- 2. combination:  $G1+P+0,9*(V+KS)$

**Shear design serviceability state**

Combination:  $G1+P+V+KS$   
 Admissible main stresses per table 9, line 46-49: limited prestressing

**Shear design limit state**

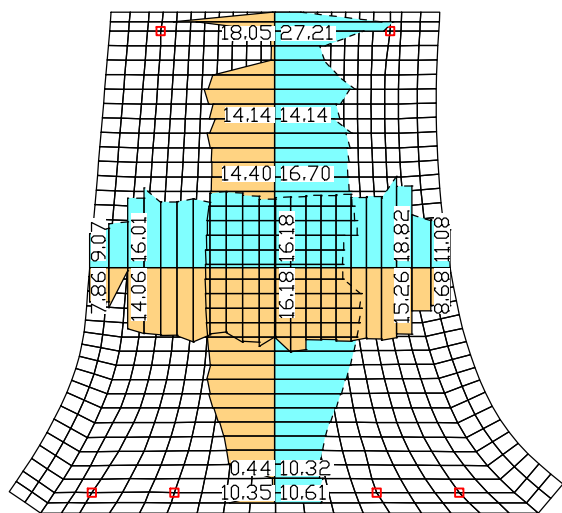
Combination:  $1,75*(G1+P)+V+KS$

**Settings for DIN 4227 checks**

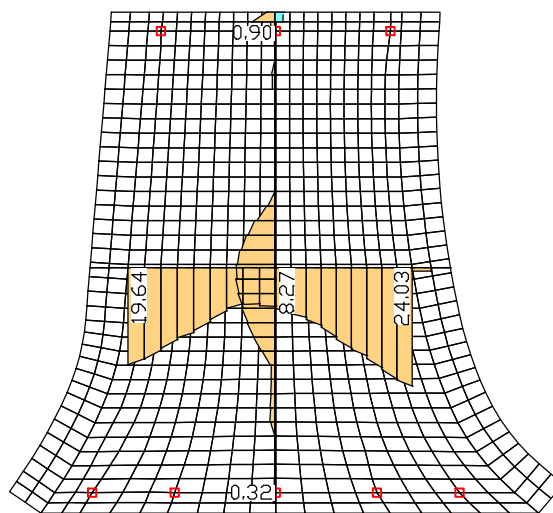
	Sect ion	Full polyg	Concr ete	Cracking		Lateral force				Torsion			perm. Stress		
				Xi1	ds [mm]	βs	y direct. d [m]	kz'	z direct. d [m]	kz'	y di. d [m]	z di. d [m]	ül [m]	S1, Tau (T,Q+T)	S2
1	1	0	B45	0.30	16	500	0.00	0.00	0.00	0.79	0.00	0.00	0.000	0	0
2	2	0	B45	0.30	16	500	0.00	0.00	0.00	0.79	0.00	0.00	0.000	0	0
3	3	0	B45	0.30	16	500	0.00	0.00	0.00	0.79	0.00	0.00	0.000	0	0

- Xi1 Coefficient for beam sections DIN 4227-1/A1:1995-12, Sect. 6.7.3
- ds Selected bar diameter DIN 4227-1/A1:1995-12, Tab. 6
- d Width of substitute section, shear and/or torsion rectangle
- kz' Coefficient for the inner lever arm referring to d, "0": from failure safety check ( $kz'=z/d$ )
- ül Distance of core section from cross-section edge, "0": no torsional reinforcement
- S1,Tau "0": Tab. 9, lines 53, 54, 59,60 (box section); "1": Tab. 9, lines 52, 55, 58, 61 (full section)
- S2 "0": Tab. 9, line 62 (web); "1": Tab. 9, line 63 (chort)

The upper and lower reinforced concrete reinforcement that results from the maximum of the failure safety and crack safety checks is shown in the following two steps.

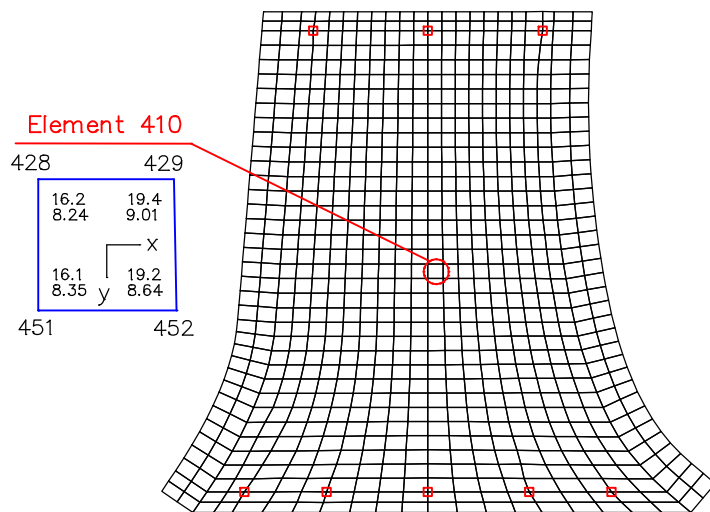


Reinforcement  $a_{sx}$  [cm<sup>2</sup>/m] from failure safety and crack check



Reinforcement  $a_{sy}$  [cm<sup>2</sup>/m] from failure safety and crack check

An additional excerpt from the check log is provided for an element from the middle of the bridge.



Element 410, lower bending reinforcement [cm<sup>2</sup>/m] from failure safety and crack check.

**Prestressed concrete design per DIN 4227-1/A1:1995-12**

The design occurs for components with partial prestress and supplemental bond (Part 1+4).

**All checks are made for the extreme values of actions.**

For the flexural design of area sections, the stress reserves of tendon groups with bond are ignored.

**Design settings**

Se.	Reinforcem.	Concrete stress	P. steel
	R B Q T	Long. tens. compr.	stress
1	x x x .	x . .	.

- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement form design at failure state.
- (Q) Lateral force reinforcement from design at failure state.
- (T) Torsional reinforcement from design at failure state.

**Settings for flexural and shear reinforcement**

$\beta_s$	Quality of stirrups [MN/m <sup>2</sup> ].				
Zone	The shear check for zone a is performed like zone b as per Ch. 12.3.2.				
Asb	The shear reinforcement as per Ch. 12.4.1 is always outputted.				
Se.	Concr.	Design mode	$\beta_s$	Zone a	Always
		for M, N	[MN/m <sup>2</sup> ]	like b	output Asb
1	B45	Standard	500	.	.

**Shear cross-sections**

dy, dz Width and height of the equivalent cross-section.  
 kz' Coeff. for calculating the inner lever arm z:  $kz' = z / d$ .  
 Se. Lateral force Torsion  
 dy[m] kz.y' dz[m] kz.z' dy[m] dz[m] ul[m]  
 1 1.000 . 0.720 0.79 . . .

**Permissible concrete stresses as per DIN 4227-1, Table 9, Lines 46 to 63**

Q,T,Q+T Action: Lateral force, torsion, lateral force plus torsion.  
 S1 Diagonal principal tensile stress at serviceability state.  
 S2 Diagonal principal compressive stress at ultimate state.  
 Tau Shear stress at ultimate state.  
 Serviceab. st. .... Ultimate state .....  
 partial prestr. per. S1, Tau Base val. Tau per. S2  
 per.S1 [MN/m<sup>2</sup>] Compon. [MN/m<sup>2</sup>] [MN/m<sup>2</sup>] Compon. [MN/m<sup>2</sup>]  
 Se. Concr. Q,T Q+T Q T Q+T Q T Q+T Q,T,Q+T  
 1 B45 2.6 . 1.2 . . 4.8 . . .

**Setting for crack width limitation**

BbZ Central concrete tensile strength as per Ch. 6.7.3, Eq. (1).  
 Xi1 Bond coeff. for considering the contribution of prestressing steel.  
 ds Diameter of the reinforcing bars.  
 Se. BbZ Xi1 ds  
 [MN/m<sup>2</sup>] [mm]  
 1 2.7 0.30 16

**Reinforcing steel of area elements**

Se. Lay. Qua. E-Modul dt x db x asx dt y db y asy as  
 [MN/m<sup>2</sup>] [m] [cm<sup>2</sup>/m] [m] [m] [cm<sup>2</sup>/m] [cm<sup>2</sup>/m] fix  
 1 1 500M 210000 0.050 . 0.00 0.050 . 0.00 .  
 2 500M 210000 . 0.050 0.00 . 0.050 0.00 .

**DESIGN FOR AREA ELEMENTS**

**Design of longitudinal reinforcement**

(R) Nominal reinforcement as per 6.7.3, A1:1995-12 for crack width limitation  
 ds Maximal given steel diameter [mm]  
 k Coefficient for consideration of non-linear distributed tensile stress  
 BbZ Eff. central tensile concrete strength as per Ch. 6.7.3(1) [MN/m<sup>2</sup>]  
 kc Coefficient to consider stress distribution in tensile zone according to Eq. (1), A1:1995-12  
 max Sx Maximal concrete edge stress from action combination [MN/m<sup>2</sup>]  
 (B) Failure safety check  
 In case of dominant bending, compression reinforcement is marked with "\*\*\*".  
 BR Concrete strength as per Figure 6 [MN/m<sup>2</sup>]  
 Angle Angle of reinforcement asy [°]  
 msd Design moment [kNm/m]  
 nsd Longitudinal force for design [kN/m]

**Element 410**

**Location 2**

Element 410, Shell, 4 result loaction(s)  
 Cross-section 1: d=0.72 - B45  
 Steel 1; Design mode: Standard  
 (R) ds=16; k=0.706; BbZ=3.16287  
 (B) BR=27; Angle=0

**1. Crack Safety Combination (CC.1): G1+P+1.1\*(V+KS)**

Concrete internal forces  
 nx [kN/m] ny [kN/m] nxy [kN/m] mx [kNm/m] my [kNm/m] mxy [kNm/m]  
 nx- : 366.16 -4123.47 -197.97 -29.97 -178.14 -6.72  
 nx+ : 366.16 -4123.47 -197.97 -29.97 -178.14 -6.72  
 ny- : 366.16 -4123.47 -197.97 -29.97 -178.14 -6.72  
 ny+ : 366.16 -4123.47 -197.97 -29.97 -178.14 -6.72  
 nxy- : 366.16 -4123.47 -197.97 -29.97 -178.14 -6.72  
 nxy+ : 366.16 -4123.47 -197.97 -29.97 -178.14 -6.72  
 mx- : 366.16 -4123.47 -197.97 -40.07 -23.65 -7.94  
 mx+ : 366.16 -4123.47 -197.97 43.44 226.98 -2.08  
 my- : 366.16 -4123.47 -197.97 -29.97 -178.14 -6.72  
 my+ : 366.16 -4123.47 -197.97 43.44 226.98 -2.08  
 mxy- : 366.16 -4123.47 -197.97 -28.87 -128.06 -14.11  
 mxy+ : 366.16 -4123.47 -197.97 -14.12 76.83 4.65

Relevant internal forces for design  
 x-direction y-direction  
 Set, var. nsd [kN/m] msd [kNm/m] nsd [kN/m] msd [kNm/m]  
 nx- (3): 564.13 -36.69 -3925.50 -184.87  
 mx- (3): 564.13 -48.01 -3925.50 -31.59

2. Crack Safety Combination (CC.2): G1+P+0.9\*(V+KS)

Concrete internal forces

	nx [kN/m]	ny [kN/m]	nxy [kN/m]	mx [kNm/m]	my [kNm/m]	mxy [kNm/m]
nx-	299.58	-3373.75	-161.98	-16.97	6.72	-3.81
nx+	299.58	-3373.75	-161.98	-16.97	6.72	-3.81
ny-	299.58	-3373.75	-161.98	-16.97	6.72	-3.81
ny+	299.58	-3373.75	-161.98	-16.97	6.72	-3.81
nxy-	299.58	-3373.75	-161.98	-16.97	6.72	-3.81
nxy+	299.58	-3373.75	-161.98	-16.97	6.72	-3.81
mx-	299.58	-3373.75	-161.98	-27.07	161.21	-5.03
mx+	299.58	-3373.75	-161.98	56.44	411.84	0.84
my-	299.58	-3373.75	-161.98	-16.97	6.72	-3.81
my+	299.58	-3373.75	-161.98	56.44	411.84	0.84
mxy-	299.58	-3373.75	-161.98	-15.87	56.81	-11.20
mxy+	299.58	-3373.75	-161.98	-1.12	261.70	7.56

Relevant internal forces for design

	x-direction		y-direction	
Set, var.	nsd [kN/m]	msd [kNm/m]	nsd [kN/m]	msd [kNm/m]
mx+ (1):	461.56	57.27	-3211.77	412.68

1. Failure Safety Combination (FC.1): 1.75\*(G1+P)+V+KS

Concrete internal forces

	nx [kN/m]	ny [kN/m]	nxy [kN/m]	mx [kNm/m]	my [kNm/m]	mxy [kNm/m]
nx-	332.87	-3748.61	-179.97	7.68	543.25	1.70
nx+	332.87	-3748.61	-179.97	7.68	543.25	1.70
ny-	332.87	-3748.61	-179.97	7.68	543.25	1.70
ny+	332.87	-3748.61	-179.97	7.68	543.25	1.70
nxy-	332.87	-3748.61	-179.97	7.68	543.25	1.70
nxy+	332.87	-3748.61	-179.97	7.68	543.25	1.70
mx-	332.87	-3748.61	-179.97	-9.99	813.61	-0.44
mx+	332.87	-3748.61	-179.97	136.15	1252.21	9.83
my-	332.87	-3748.61	-179.97	7.68	543.25	1.70
my+	332.87	-3748.61	-179.97	136.15	1252.21	9.83
mxy-	332.87	-3748.61	-179.97	9.61	630.90	-11.23
mxy+	332.87	-3748.61	-179.97	35.42	989.46	21.59

Relevant internal forces for design

	x-direction		y-direction	
Set, var.	nsd [kN/m]	msd [kNm/m]	nsd [kN/m]	msd [kNm/m]
nx- (1):	512.85	9.38	-3568.63	544.95
mx- (3):	512.85	-10.43	-3568.63	813.17
mx+ (1):	512.85	145.98	-3568.63	1262.04

Design of longitudinal reinforcement

Lay., Type	Reinf. for x-direction				Reinf. for y-direction			
	max Sc [MN/m <sup>2</sup> ]	kc	req.asx [cm <sup>2</sup> /m]	Situation	max Sc [MN/m <sup>2</sup> ]	kc	req.asy [cm <sup>2</sup> /m]	Situation
1 R	1.34	0.55	16.30	CC.1,mx-(3)	.	.	0.00	CC.1,nx-(3)
B	.	.	5.46	FC.1,mx-(3)	.	.	0.00	FC.1,nx-(1)
2 R	1.30	0.52	19.44	CC.2,mx+(1)	0.32	0.02	0.29	CC.2,mx+(1)
B	.	.	9.84	FC.1,mx+(1)	.	.	9.01	FC.1,mx+(1)

Design of shear reinforcement and shear stress at failure state

The percentage of nominal reinforcement acc. to Ch. 6.7 is not considered.

d	Element thickness [m]
βs	Quality of stirrups [MN/m <sup>2</sup> ]
kz'	Coeff. for calculating the inner lever arm z: kz' = z / d
qx,qr	Slab lateral forces in x,y-direction, qr=(qx <sup>2</sup> +qy <sup>2</sup> ) <sup>1/2</sup> [kN/m]
tau.r	Calculated value of shear stress from qr [MN/m <sup>2</sup> ]
tau.57	Maximal permissible shear stress as per Table 9, row 57 [MN/m <sup>2</sup> ]
tau.51	Limit value for req.asb per Table 9, row 51, w/o red. per 12.4.1(7)
Theta	Angle between the compressive struts and the sectional normal [°]

Element 410

Location 2

Element 410, Shell, 4 result loaction(s)  
 Cross-section 1: d=0.72 - B45  
 βs=500; kz'=0.79

1. Shear Combination Failure State (SU.1): 1.75\*(G1+P)+V+KS

Concrete internal forces

	qx [kN/m]	qy [kN/m]
qx-	-6.31	-9.37
qx+	41.76	-5.63
qy-	33.90	-40.25
qy+	12.06	14.06

Shear reinforcement an shear stress in zone B

qr [kN/m]	tau.r [MN/m <sup>2</sup> ]	tau.57 [MN/m <sup>2</sup> ]	tau.51 [MN/m <sup>2</sup> ]	Theta [°]	z [m]	req.asb [cm <sup>2</sup> /m <sup>2</sup> ]	Situation
11.30	0.02	4.8	1.2	21.8	0.57	.	SU.1,qx-

Check of concrete stress

For the check, a non-cracked concrete section is assumed.  
 The permissibility of stresses as per Table 9 is not verified.

S1,2	Principal stresses from bending and longitudinal force [MN/m <sup>2</sup> ]
Phi	Angle of principal stress relative to the system of internal forces [°]

**Element 410****Location 2**

Element 410, Shell, 4 result location(s)  
Cross-section 1: d=0.72 - B45

**1. Stress Combination: G1+V**

	max S1 [MN/m <sup>2</sup> ]	cor.S2 [MN/m <sup>2</sup> ]	cor.Phi [°]	min S2 [MN/m <sup>2</sup> ]	cor.S1 [MN/m <sup>2</sup> ]	cor.Phi [°]
top	0.94	-3.36	-2.64	-3.36	0.94	87.36
bottom	0.16	-8.71	-2.41	-8.71	0.16	87.59

**2. Stress Combination: G1+V+KS**

	max S1 [MN/m <sup>2</sup> ]	cor.S2 [MN/m <sup>2</sup> ]	cor.Phi [°]	min S2 [MN/m <sup>2</sup> ]	cor.S1 [MN/m <sup>2</sup> ]	cor.Phi [°]
top	0.74	-4.22	-2.18	-4.22	0.74	87.82
bottom	0.21	-6.21	-2.78	-6.21	0.21	87.22

**3. Stress Combination: G1+V+KS+0.5\*P**

	max S1 [MN/m <sup>2</sup> ]	cor.S2 [MN/m <sup>2</sup> ]	cor.Phi [°]	min S2 [MN/m <sup>2</sup> ]	cor.S1 [MN/m <sup>2</sup> ]	cor.Phi [°]
top	0.80	-5.11	-1.76	-6.57	0.32	88.20
bottom	0.63	-3.87	-3.62	-6.21	0.21	87.22

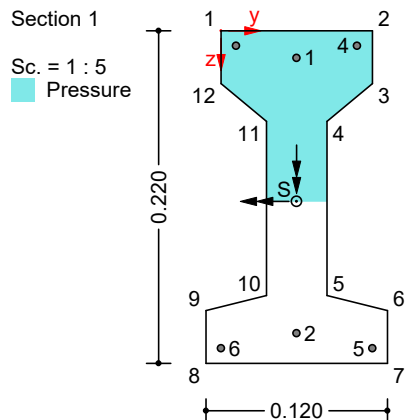
**4. Stress Combination: G1+V+KS+P**

	max S1 [MN/m <sup>2</sup> ]	cor.S2 [MN/m <sup>2</sup> ]	cor.Phi [°]	min S2 [MN/m <sup>2</sup> ]	cor.S1 [MN/m <sup>2</sup> ]	cor.Phi [°]
top	0.86	-6.01	-1.46	-8.91	-0.11	88.42
bottom	1.07	-1.54	-5.70	-6.21	0.21	87.22

# Single Design

In the following example a failure safety check is performed on a prestressed concrete section using the single design. In this case the tendon groups are defined as prestressed concrete steel layers 1 and 2. Internal forces without safety factors are applied for the design.

## Pos. 1 - Reinforced concrete design per DIN 4227



Action N = 0.00 kN; My = 25.00; Mz = 0.00 kNm  
 Resistance N = 0.00 kN; My = 25.00; Mz = -0.00 kNm  
 Force system ys / zs = 0.050 / 0.113 m  
 Strength βR = 33.00 MPa; gamma = 1.75  
 Design mode Standard  
 Reinforcement 4.14 cm<sup>2</sup>; 2.79 %; Concrete area = 148.50 cm<sup>2</sup>  
 Prestress P0 N0 = -129.60 kN; My0 = -9.10; Mz0 = -0.00 kNm  
 Resistance + P0 N+N0 = -129.60 kN; My+My0 = 15.90; Mz+Mz0 = -0.00 kNm

Concrete section Point	y [m]	z [m]	eps[‰]	sigma[MPa]	Inner Forces	y [m]	z [m]	F [kN]
1	0.000	0.000	-3.50	-33.00	Compr.	0.050	0.033	-146.74
2	0.100	0.000	-3.50	-33.00	Tension	0.050	0.203	146.74
3	0.100	0.035	-2.42	-33.00	Lev. arm	0.000	0.170	
4	0.070	0.060	-1.64	-31.95				
	0.070	0.113	0.00	0.00				
5	0.070	0.175	1.92	0.00				
6	0.110	0.185	2.23	0.00				
7	0.110	0.220	3.31	0.00				
8	-0.010	0.220	3.31	0.00				
9	-0.010	0.185	2.23	0.00				
10	0.030	0.175	1.92	0.00				
	0.030	0.113	0.00	0.00				
11	0.030	0.060	-1.64	-31.95				
12	0.000	0.035	-2.42	-33.00				

Reinforcement Point	y [m]	z [m]	d1 [m]	Es, βs [MPa]	Zv0 [kN]	eps[‰]	sigma[MPa]	As [cm <sup>2</sup> ]
1	0.050	0.018	0.018	205000 1420	12.0	-2.94	-303.27	0.40
2	0.050	0.200	0.020	205000 1420	117.6	2.69	1420.00	1.20
3	0.010	0.010	0.010	210000 500	0.0	-3.19	-416.67	0.41
4	0.090	0.010	0.010	210000 500	0.0	-3.19	-416.67	0.41
5	0.100	0.210	0.010	210000 500	0.0	3.00	500.00	0.86
6	0.000	0.210	0.010	210000 500	0.0	3.00	500.00	0.86

## References

DIN 4227 Part 1, Issue July 1988

Prestressed concrete – Part 1: Structural members made of normal-weight concrete, with limited concrete tensile stresses or without concrete tensile stresses.

Publisher: DIN Deutsches Institut für Normung e.V.

Beuth Verlag, Berlin 1988.

DIN 4227-1/A1, Issue December 1995

Prestressed concrete – Part 1: Structural members made of normal-weight concrete, with limited concrete tensile stresses or without concrete tensile stresses; amendment A1:1995-07.

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Betonkalender 1992, Ernst & Sohn Verlag, Berlin 1992.

Rossner, W.; Graubner C.-A.

Spannbetonbauwerke, Teil 1 (Prestressed Concrete Structures, Part 1),

Bemessungsbeispiele nach DIN 4227 (Design Examples According to DIN 4227).

Ernst & Sohn Verlag, Berlin 1992.



# DIN Technical Report Bridge Checks

## Basics

The bridge checks as described in DIN Technical Report 101 and 102 (abbreviated in the following as TR 101 and TR 102 or simply I and II) are designed to supplement the 3D FEM analysis. They can be used for bridges and other engineering constructions in which actions from street or railway traffic have to be taken into account. Permitted structure models include beam, area and solid structures. In the calculation settings you can select which version of the standard is decisive:

- Edition 2003-03
- Edition 2009-03

Differing components can be combined in a structure model:

- Non-prestressed components
- Prestressed components with subsequent bond
- Prestressed components without bond
- Components with external prestressing
- Mixed-construction components

The checks are carried out subsequent to the static calculation. According to TR 101, the calculated load cases and load model variants must be assigned to action groups. The design situations for the construction stages and final states are then selected in the desired check combinations. Based on these design situations the program will take into account the preset safety factors and combination coefficients to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements.

The dialogs for structure type and check selection as well as for load model variants and actions are grouped in section *DIN Technical Report bridge checks* of the database. Detailed check specifications and reinforcement data are entered during section definition.

As an addition to TR 102, Section 3.1.4, all elements with materials *C12/15* to *C100/115*, *LC12/13* to *LC60/66* and the user-defined material *CX* are checked. For classes *LC12/13* to *LC60/66*, the guidelines of DIN 1045-1 are applicable; for strength classes above *C55/67* and *LC55/60*, guideline 5.3.3 (9) of DIN 1045-1 is used.

For beams and design objects, all checks are carried out at the polygon section. For general notes on using design objects, refer to the relevant chapter in the manual.

In the *DIN 1045-1 Design* folder of the database you can also perform a single design for user-defined polygon sections or composite sections.

# Input

## Type of Structure

The dialog for selecting the structure type is opened from the database or the *Settings* in the *Analysis* menu. You can choose either the road or railroad traffic route. The construction type is currently limited to concrete.

## Load Model 1 for Road Bridges

Load model 1 consists of two parts according to TR 101, Section 4.3.2:

- a) Double-axle (TS tandem system)
- b) Uniformly distributed area load (UDL)

These loads should be applied in both the longitudinal and lateral directions of the bridge in the least favorable position. In the lateral direction, the load positions are determined by dividing the roadway into computational lanes. Since the decisive lane division is not always known in advance, you can define different load position variants.

Load model 1 can be edited from the *DIN Technical Reports Bridge Checks / Load model 1* folder in the database. Select *New* from the context menu of load model 1 to create a new variant of load positions and open the corresponding input dialog. The adjusted base values of the tandem system ( $\alpha_{Qi} \cdot Q_{ik}$ ) and the load ordinates of the UDL loads ( $\alpha_{qi} \cdot q_{ik}$ ) can be set by clicking *Properties* in the context menu of load model 1.

Location	Tandem system TS alpha.Qi * Qik [kN]	UDL system alpha.qi * qik [kN/m²]
Lane 1	240	9
Lane 1	160	2.5
Lane 3	0	2.5
Other lanes		2.5
Remaining area (qrk)		2.5

During FEM calculation, all vertical load portions of load model 1 are applied as area loads on area or solid elements in the global z-direction. They are calculated in separate load cases to allow for later determination of the extremal reactions. With the option *Use projective loads* the loads are projected perpendicular to their surfaces in local t-direction (see *Free area load - polygon*). With projected loads, it is usually useful to arrange the load above the elements and orient its local t-direction according to the global z-direction. Only elements are loaded, which are not hidden by other elements. This also applies to a partial overlap.

Load model 1:

The video "[https://download.infograph.de/video\\_de/LM%201%20eingegeben.mp4](https://download.infograph.de/video_de/LM%201%20eingegeben.mp4)" (German language) shows the definition of load model 1.

### Off

Exit the load model 1 screen.

### TS New

Enter a new position of the tandem system (see also tandem system). The positions of the tandem system are mutually exclusive.

### UDL New

Consecutive input of rectangular or triangular load areas of the UDL load. The partial areas of the UDL load can act simultaneously. The input for UDL2 (lane 2) is preset. You can select a different lane or remaining area from the context menu.

UDL (2) ✕

Location:  
Lane 1 ▼

Point	x [m]	y [m]	z [m]
1	0	0.5	0
2	14.6	0.5	0
3	14.6	3.5	0
4	0	3.5	0

### UDL Gen

Generate new load areas of the UDL load. The rectangular generation area is separated into the same number of load areas on the opposite edges.

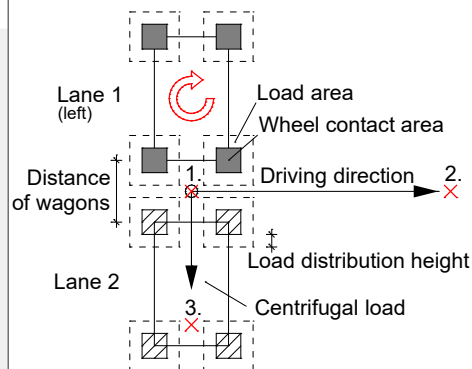
## Tandem System of Load Model 1

Tandem system (1) ✕

Distance of wagons:  m    Lanes:     Centrifugal load:  kN

Load distribution height:  m

Point	x [m]	y [m]	z [m]
1	1.305	3.5	0
2	2.305	3.5	0
3	1.305	4.5	0



Input dialog and schematic diagram of the tandem system in load model 1 according to TR 101.

### Distance of wagons

Distance of the centroid of the wheel-ground contact area (see figure).

### Lanes

Lane arrangement from left to right.

### Centrifugal load

Load amount  $Q_t$  of this tandem system position. The point load acts at point 1 perpendicular to the direction of traffic. Its eccentric location is not considered. In the *Use projective loads* mode, the centrifugal load is an area load that is distributed evenly over the projected load areas.

### Load distribution height

Yields the load areas of the tandem system in conjunction with the wheel-ground contact area (see figure).

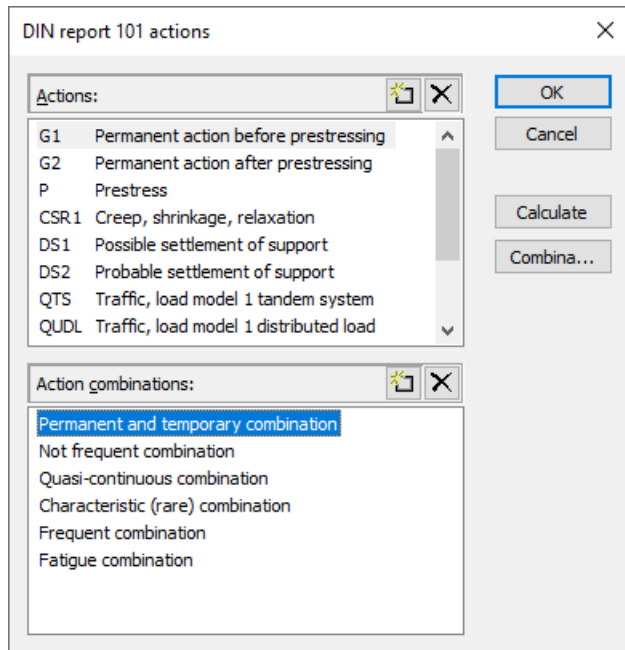
### Point

1. Reference point of the tandem system
2. Point of the local x axis (direction of traffic)
3. Point in the local xy plane

## Actions and Action Combinations

The design values of the load are calculated based on the internal forces of individual load cases and load case combinations. For this the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the decisive action combinations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Actions

List of defined actions. The following actions can be defined:

- G1 to G9: Permanent actions
- P: Prestressing
- QTS, UDL: Traffic, load model 1 TS and UDL
- QI: Traffic, braking and starting
- Qt: Traffic, centrifugal load
- T: Temperature
- W: Wind
- DS1: Possible support displacement
- DS2: Probable support displacement
- CSR1, CSR2: Creep, shrinkage and relaxation at various times
- A: Accidental action
- E: Earthquake
- Q1 to Q9: Other variable action
- Qfat: Cyclic fatigue action

The actions DS1 and DS2 and also the actions Q, T and W are only taken into account in the design situations if they have an unfavorable effect.

The actions QTS, QUDL, QI and Qt are only available when selecting the *Road* traffic route.

### Action combinations

List of defined action combinations.



Insert a new action or action combination.



Delete the selected action or action combination.

### Calculate

Calculate the defined action combinations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all combinations in the database. This allows you to evaluate the results without having to execute the checking module.

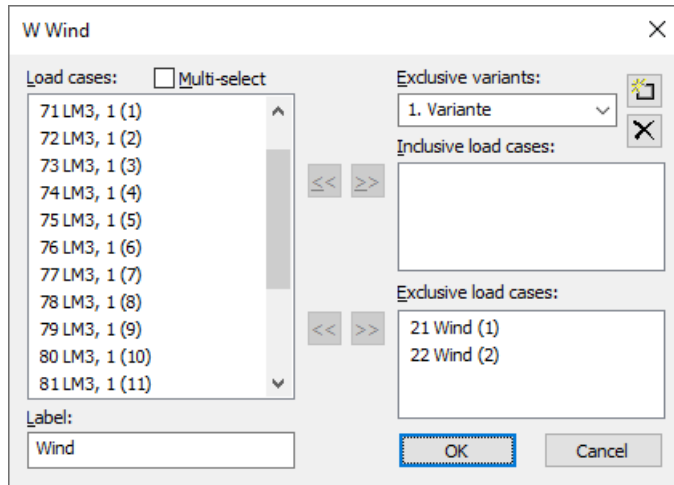
Each time you execute the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

The following table demonstrates how the combinations are used in the various checks of Technical Report 102.

Situation	Ultimate limit state	TR 102	Serviceability limit state	TR 102
Permanent & temp. Accidental Earthquake	Longitudinal reinf.	4.3.1		
	Lateral reinf.	4.3.2		
	Torsional reinf.	4.3.3		
Characteristic (rare)			Crack reinforcement	4.4.2.2
			Crack width Class A	4.4.2.4
			Concrete compr. stress	4.4.1.1
			Concrete tensile stress	4.4.0.3
Frequent	Robustness reinf.	4.3.1.3	Concrete compr. stress	4.4.1.2
			Reinforcing steel stress	4.4.1.3
			Decompression Class A	4.4.2.1
			Crack width Class B	4.4.2.4
Not frequent	Fatigue, simplified	4.3.7.4 4.3.7.5(101)	Principal tensile stress	4.4.0.3
			Decompression Class B	4.4.2.1
			Crack width Class C, D	4.4.2.4
Quasi-continuous			Concrete compr. stress	4.4.1.2
			Prestressing steel stress	4.4.1.4
			Decompression Class C	4.4.2.1
			Decompression Class D	ARS 6/09
			Crack width Class E	4.4.2.4
			Deformations	4.4.3
Fatigue	Fatigue reinf. steel	A.106.3.2		
	Fatigue prestr. steel	4.3.7.5		
	Fatigue concrete	4.3.7.5		

## Definition of an Action

The load cases are assigned to the actions after an action is selected or a new one is entered. Depending on the action type, you have access to a variety of dialogs of which one example is shown below. The available dialog options are explained at the end.



### Load cases

Select load cases or load case combinations to add them to the current action.



### Multi-select

Load cases and combinations can be added to the actions more than once.

### Label

User-defined label for the action.

### Exclusive variables

An exclusive variant consists of both inclusive and exclusive portions. The variants are mutually exclusive. The buttons  or  are used to add or delete action variants.

### Inclusive load cases

Selected load cases and combinations that can have a simultaneous effect.

### Exclusive load cases

Selected load cases and combinations that are mutually exclusive.

### Use moving loads

The actions of load model 1 can be recorded using either *moving loads* or *normal* load cases. The *moving loads* are described in a separate dialog (see *Load model 1*). When using *normal* load cases, it is assumed for the sake of simplicity that the vertical ( $Q_{TS}$ ) and horizontal ( $Q_t$ ) actions of the tandem system act independently. If the individual vehicles of a tandem system are described in different *normal* load cases, they must be assigned to the corresponding lane. In the combination, a maximum of one of its load cases is taken into account on each lane. This means that the tandem system consists of a maximum of three simultaneously acting load cases.

### Prestressing loss from relaxation of prestressing steel

The prestressing loss is defined as a constant percentage reduction of prestress.

### CS as constant reduction of prestress

As an alternative to defining CS load cases, you can allow for the effect of creep and shrinkage by defining a constant percentage reduction of prestress.

### Internal prestressing

Selected load cases that describe internal prestressing. The reactions of the individual load cases are added together.

## External prestressing

Selected load cases that describe external prestressing. The reactions of the individual load cases are added together.

## Definition of an Action Combination

Depending on which check is selected, different action combinations are necessary. You can enter them using the following dialog.

Variant	State	Actions
1)	Constr. - Ungr.	G1 + P
2)	Final	G1 + G2 + P + CSR1 + DS1 + QK

Action	Gamma.sup	Gamma.inf
G1	1.35	1
G2	1.35	1
P, CSR1	1	1
DS1, DS2	1	
QTS, QUDL, QI	1.5	

Variant	QTS	QUDL	QI	T	W
a)	1.0	1.0	0	0	0.5
b)	0.75	0.4	0	0	1.0
c)	0.75	0.4	0	1.0	0
d)	0.75	0.4	1.0	0.8	0
e)	0.75	0.4	1.0	0	0.5

### Situations

List of design or check situations. Each situation can be valid for either the construction stage or the final state. For prestressed components with subsequent bond the tendons can be set ungrouted. The *QK* action indicates variable actions based on the table of combination values. The buttons or allow you to add or delete situations. By double-clicking on a situation it can be modified subsequently.

### Partial safety factors

Table of partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$  for the actions. The values are suggested based on TR 101, Table C.1 (road bridges).

### Combination Values

Table of the combination coefficients for the variable actions. The values are suggested analogously to TR 101, Table C.2 (road bridges). The buttons or allow you to add or delete combination variants. For the calculation only the variants listed here are taken into account.

### Standard

Recommended values are assigned to the safety and combination coefficients. All relevant actions of the final state are selected for the design situation.

## Section Input

The section inputs contain all of the specific settings made for the checks in the ultimate limit and serviceability state. In addition to these specifications, the selected material properties and the properties of the reinforcing steel are also relevant for the design. An overview of the design specifications can be accessed in the *DIN Technical Reports Bridge Checks* section of the database.

## Checks

The following dialog is used to define which ultimate limit state and serviceability checks are available for the section. The analysis settings allow to override this selection for the entire structure.

### Prestress of component

The type of prestressing can be selected for each section separately:

- *not prestressed*
- *subsequent bond*
- *without bond*
- *external*
- *mixed construction*

### Requirement class

The check conditions for the decompression and crack width checks are defined in DIN TR 102, Table 4.118, using the requirement classes A through E (II-4.4.0.3 (103)). The minimum requirement class is determined based on the prestressing type according to ARS 11/2003 or 6/2009 for road bridges and Ril 804.4201 for railroad bridges.

### Prestressed box section according to ARS

This button allows you to select a special rule for prestressed box sections in requirement class D for road bridges according to ARS 11/2003 and 6/2009. In this case – and contrary to Table II-4.118 – the decompression check must be carried out for the quasi-continuous action combination.

### Robustness

This check determines the minimum reinforcement against failure without warning as per II-4.3.1.3, which is also referred to as the robustness reinforcement.



## Base Values

The base values apply for all checks in the ultimate limit and serviceability states.

Properties for element 405 - DIN TR 102 - Base values

Section: Form, Shear stresses, Material, Bedding, DIN TR 102, Checks, Base values, Shear section, Concrete stress, Crack width, Fatigue, Variation coefficients, Recalculation Guideline, Thermal analysis, General, Eccentricity

Number: 11 - Mi, Section Type: Polygon, Material Type: C40/50, New, Copy, Delete

Label: Main girder, Properties concern further elements.

Design mode for bend and longitudinal force: Standard, Alternative concrete: Null, Reduction factor of prestr. for robustness:

Lateral force and torsion

Quality of stirrups: 500A, 500, cot Theta, Method: 1.75, Standard, Eff. height d [m]: , Factor for min rho\_w: 1.6

Asl acc. to Fig. 4.12 [cm<sup>2</sup>]: 0, Asl extension to [cm<sup>2</sup>]: , Laying meas. cv,l [m]: 0.1, Lever arm from bend., Separate check for x- and y-direction, Design like slabs

OK, Cancel, Help

### Design mode

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined ratios between the reinforcement layers will not be affected.
- *Compression member*: For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to TR 102, Section 5.4.1.2.1.

### Alternative concrete

This value is necessary to perform a design according to the standard if the material type *Beton* is selected.

### Reduction factor of prestr. for robustness

The regulations of the Section 4.3.1.3 (108) are decisive for the arrangement of the robustness reinforcement. Thus for the determination of the tensile zone the statically determined effect of prestressing is not taken into account. Because this cannot be determined for area elements the prestress can alternatively be reduced by a reduction factor. The specification of an appropriate value is subject to the discretion of the user.

### Effective height

Effective static height for the shear design of area elements [m].

### cot Theta, Method

$\cot \Theta$  defines the concrete strut angle according to TR 102, Section 4.3.2.4.4 (1)\*P. The program will suggest a value of 1 (45° strut angle). Alternatively, you can select any value between 4/7 and 3. Entering a higher number will normally result in a lower necessary lateral force reinforcement  $A_{sw}$ , a lower absorbable lateral force  $V_{Rd,max}$  and a larger displacement  $a_1$  according to II-5.4.2.1.3 (3)\*. In the case of longitudinal tensile load,  $\cot \Theta$  should not fall below the value 1 according to edition 2009. Four calculation methods can be chosen for the check:

- *Standard*: The input value is limited to the range permitted in accordance with TR 102, Eq. (4.122) for lateral force, torsion and combined loads (method with load-dependent strut angle).
- *Constant*: The check is carried out using the chosen value for  $\cot \Theta$  without further limitations (cf. interpretation No. 24 of NABau for DIN 1045-1).
- *Std./45°*: For lateral force  $\cot \Theta$  is limited according to TR 102, Eq. (4.122). For torsion a constant strut angle of 45° is assumed for simplification according to TR 102, Chapter 4.3.3.2.2 (3)\*P.
- *Std./30°*: For lateral force  $\cot \Theta$  is limited according to TR 102, Eq. (4.122). For torsion a constant strut angle of 30° is assumed.

The actual effective angle of the concrete struts is logged for each check location.

**Bending reinforcement  $A_{sl}$  according to Fig. 4.12**

The bending tensile reinforcement to be accounted for as per TR 102, Section 4.3.2.3, Fig. 4.12 [cm<sup>2</sup>].

**Automatic extension of  $A_{sl}$  to**

You can optionally specify a maximum value for area elements and the program will automatically increase the above input value until that maximum value is reached in order to avoid stirrup reinforcement [cm<sup>2</sup>].

**Quality of stirrups**

- 420S: Reinforcing rod with  $f_{yk} = 420$  MN/m<sup>2</sup>.
- 500A: Reinforcing rod with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- 500M: Reinforcing meshes with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- General: Freely definable steel quality [MN/m<sup>2</sup>].

**Factor for min  $\rho_w$** 

The minimum reinforcement level  $\min \rho_w$  complies with II-5.4.2.2 (4)\*P and is defined using a factor related to the base values  $\rho$  found in Tab. 5.7. The program will suggest a factor of 1 for beams and design objects and a factor of 0.6 for area elements as per II-5.4.3.3 (2)\*P. The factor can be any number between 0 and 1.6, which is the nominal value for structured sections with prestressed tension chord.

**Design like slabs**

Beams or design objects are treated like slabs, which means that a minimum lateral force reinforcement will not be determined as per II-5.4.3.3 (2)\*P if no lateral force reinforcement is required for computation.

**Laying measure  $c_{v,l}$** 

In Section II-4.3.2.4.2 (2)\* (Edition 2009) and Interpretation No. 24 of the NABau, the internal lever arm  $z$  is limited to the maximum value derived from  $z = d - c_{v,l} - 30$  mm and  $z = d - 2c_{v,l}$ . Note that  $c_{v,l}$  is the laying measure of the longitudinal reinforcement in the concrete compressive zone. For  $c_{v,l}$  the program will suggest the shortest axis distance of the longitudinal reinforcement from the section edge  $d_1$ .

**Separate check for x and y direction**

For two-axes stressed slabs, the lateral force check can be performed separately in the x and y stress directions as described in Section II-4.3.2.2 (5)\* (Edition 2009) and Interpretation No. 76 of the NABau. The user is responsible for properly aligning the reinforcement directions.

**Lever arm from bending design**

According to Section II-4.3.2.4.2 (2)\* the lever arm  $z$  for lateral force design of area elements is normally assumed to be  $0.9 \cdot d$ . Alternatively, the lever arm from bending design can be used. The program determines the maximum lever arm at each check location for both reinforcement directions depending on the design situation and limits the value to the range of  $0.1 \cdot d \leq z \leq \max(d - 2c_{v,l}; d - c_{v,l} - 30 \text{ mm})$ . When the check is performed for the resulting force  $q_r$  the minimum from  $z_x$  and  $z_y$  is used.

## Shear Section

For polygon sections, additional section dimensions are required for the lateral force and torsion design according to TR 102. These are explained in the following. In case of sections with internal prestressing or with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.

Properties for element 405 - DIN TR 102 - Shear section

Section

- Form
- Shear stresses
- Material
  - Default values
  - Creep coefficients
- Bedding
- DIN TR 102
  - Checks
  - Base values
  - Shear section
  - Concrete stress
  - Crack width
  - Fatigue
  - Variation coefficients
- Recalculation Guideline
  - Thermal analysis
- General
  - Eccentricity

Number: 11 - M; Section Type: Polygon Material Type: C40/50

Label: Main girder

Height [m]: 1.9 Nom. height: 1.9

Eff. width [m]: 0.9 Factor kb: 0.9

Lever arm: kb\*eff. width Qy

Width [m]: 1 Nom. width [m]: 0.82

Eff. height [m]: 1.8 Factor kd: 0.9

Lever arm: kd\*eff. height Qz

teff [m]: 0.2  Box section

Core section Ak = z1 \* z2

z1 [m]: 1.7 z2 [m]: 0.8 Mx

OK Cancel Help

### Width

Section width for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Height

Section height for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Effective height

Effective static height for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Effective width

Effective static width for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Nom. width, nom. height

The nominal width or height of internally prestressed components as per TR 102, Section 4.3.2.2 (8)P for including the duct diameter in the calculation of the design value of the lateral load-bearing capacity  $V_{Rd,max}$ :

### Factor kb, Factor kd

Factor for calculating the inner lever arm  $z$  from the effective width  $b_n$  or effective height  $d$  in the lateral load-bearing capacity check for  $Q_y$  or  $Q_z$ .

**Lever arm**

The lever arm  $z$  can be defined in the following ways:

- $kd \cdot \text{effective height}$ ,  $kb \cdot \text{effective width}$ : The entered values are used for calculation.
- *from bending design*: During the bending design the program determines at each check location the largest lever arm for every design situation. It is defined as the distance between the centroids of the concrete compressive and steel tensile forces.
- *RCG Eq. 12.16*: During the bending design a lever arm weighted by the force increase in the tendons is determined according to Eq. (12.16) of the recalculation guideline for road bridges (RCG). At the check location, the maximum from the sets of internal forces with prestressing steel within the tensile zone at cracked state is decisive for each design situation. If the prestressing steel is positioned within the compressive zone for all sets of internal forces, the lever arm according to Equation (12.16) is determined by  $z = z_s$ .
- $0.9 \cdot dp$ : For the  $Q_z$  check the lever arm is determined with the effective heights  $d_p$  of the tendons with bond. Thereby  $d_p$  is assumed to be the average value weighted by the tendon areas at the check location (cf. Rossner/Graubner 2005, p. 252).

The decisive lever arm is limited to the range  $0.1 \cdot d \leq z \leq \max(d - 2c_{v,I}; d - c_{v,I} - 30 \text{ mm})$  according to TR 102, Section 4.3.2.4.2 (2)\*, and documented in the detailed listing.

**Core section  $A_k = z_1 \cdot z_2$**

Dimensions of the core section for calculating the torsion reinforcement [m].

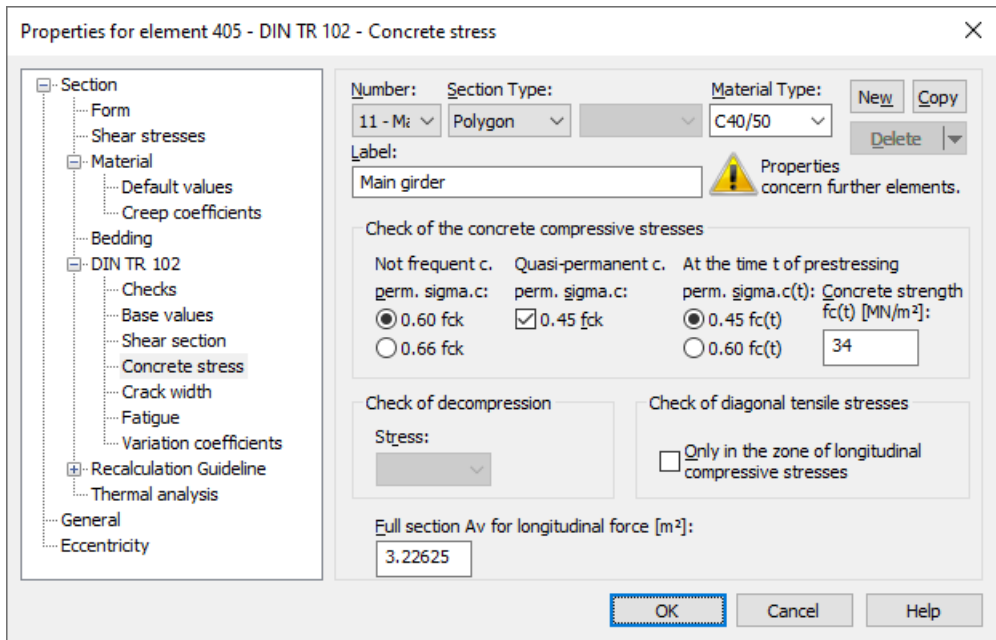
**teff**

The effective wall thickness of the torsion section according to TR 102, Fig. 4.15 [m].

**Box section**

Data for determining the factor  $\alpha_{c,red}$  in TR 102; Eq. (4.40), the torsion section utilization according to Eq. (4.47a) or (4.47b) and for checking the principal compressive stresses as per II-4.3.3.2.2 (2).

**Concrete Stress**



**perm. sigma.c**

The concrete compressive stress  $\sigma_c$  must be limited to  $0.60 f_{ck}$  under the infrequent combination in the construction stages and final states according to II-4.4.1.2 (103)P. The limit may be increased by 10% if the concrete compressive zone is helically reinforced (e.g., by lateral reinforcement). If serviceability is significantly influenced by creep, the limit  $0.45 f_{ck}$  should also be maintained under the quasi-continuous combination according to II-4.4.1.2 (104)\*P.

**perm. sigma.c(t)**

Permissible concrete stress  $\sigma_{c(t)}$  at time  $t$  when prestressing is introduced. In this case  $f_{c(t)}$  denotes the average value of the concrete compressive strength at time  $t$  when prestressing is introduced according to II-4.4.1.2 (102)P. If the compressive stress exceed the value  $0.45 f_{c(t)}$ , the nonlinearity of the creep should be taken into account according to the standard. The program assumes that prestressing is introduced in design situation G1+P.

**fc(t)**

Concrete strength at time  $t$  when prestressing is introduced as per II-4.4.1.2 (102)P [MN/m<sup>2</sup>].

**Decompression, Stress**

Decisive stress for the decompression check for area elements ( $\sigma_1, \sigma_x, \sigma_y$ ).

**Diagonal tensile stresses only in the zone of longitudinal compressive stresses**

With this option you can apply the usage guidelines according to II-4.4.0.3 (6)P of the 2009 Edition. These guidelines allow you to limit the check to the area of longitudinal compressive stresses for prestressed railway bridges as long as no tensile-stressed chords are connected. The user is responsible for checking the usage requirements.

**Av**

Area of the full section for calculating the normal stress from the longitudinal force according to II-4.4.1.1 (109) [m<sup>2</sup>], if the section dimensions have been reduced to the effective width. If alternatively the section has been defined as full polygon with specification of the effective width, the input of  $A_v$  is disabled (see chapter *Structure description - Polygon section*).

**Crack Width**

These specifications apply to the minimum reinforcement calculation and the crack width check.

**wk,per**

Calculated value of the crack width as per II-4.4.0.3 (103), Table 4.118 [mm]. The program will suggest a tabular value based on the selected requirement class and the prestressing of the component. This value can be modified after the input field is enabled.

**max. ds**

Largest available bar diameter of the reinforcing steel reinforcement according to II-4.4.2.2 (6)\* [mm].

**Coefficient xi1**

The bond coefficient  $\xi_1$  according to TR 102, Eq. (4.197) defines the extent to which prestressing steel as per II-4.4.2.2 (7)\* can be added to the minimum crack reinforcement. It is also used in the calculation of the effective reinforcement level according to Eq. (4.200a) and thus the direct calculation of the crack width. Data input is blocked for area elements since prestressing steel is normally not taken into account here.

**Determ. of the tensile zone**

You can specify the tensile zone where the minimum crack reinforcement described in II-4.4.2.2 will be placed by selecting either an action combination (AC) or a restraint (bending, central tension).

**Thick component**

Based on section II-4.4.2.2 (9) of edition 2009, the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined to Equation (4.197a). Therewith a reduction compared to the calculation with Equation (4.194) can be achieved.

**Coefficient k**

Coefficient used to take into account nonlinear distributed concrete tensile stress in the section according to II-4.4.2.2 (5)\*.

**Concrete age**

The age of the concrete is used to determine the effective concrete tensile strength  $f_{ct,eff}$  as per II-4.4.2.2 (5)\*. This is done separately for the minimum reinforcement calculation and the crack width calculation.

**Check method**

The crack width check can be performed either through direct calculation according to Section II-4.4.2.4 or by simply limiting the bar distances based on the information provided in Table II-4.121. According to *Zilch and Rogge (2002, p. 277)*, the simplified method only yields definitive results for single-layer tensile reinforcement with  $d_1 = 4$  cm. The user is responsible for the evaluation of these requirements.

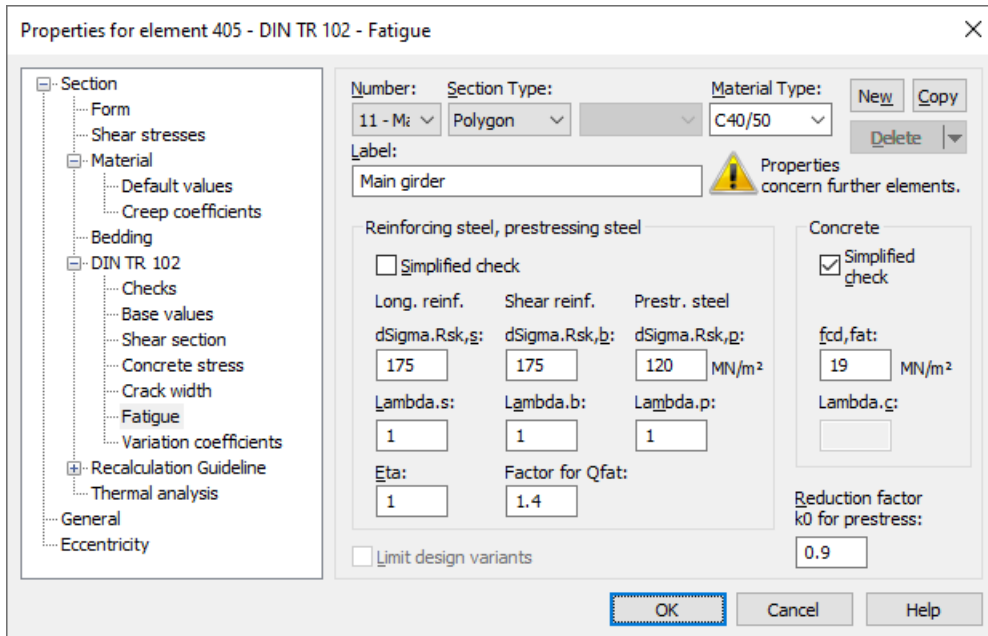
**sr,max**

When calculating the crack width, by default the crack distance is determined according to TR 102 Eq. (4.203). Alternatively, you can preset an upper limit for  $s_{r,max}$  [mm] so that, for example, the special conditions of Equation (4.204) are taken into account.

**max. s**

Largest existing bar distance of the reinforcing steel reinforcement for the simplified crack width check [mm].

**Fatigue**



**dSigma.Rsk,s, dSigma.Rsk,b**

The permissible characteristic stress range  $\Delta\sigma_{Rsk}(N^*)$  of the longitudinal reinforcement and shear reinforcement at  $N^*$  load cycles according to the Wöhler curves specified in II-4.3.7.8 [MN/m²]. The value found in Table 4.117, Row 1, is suggested in the dialog. For the shear reinforcement, the mandrel diameter is taken to be  $d_{br} = 4 d_s$ .

**dSigma.Rsk,p**

The permissible characteristic stress range  $\Delta\sigma_{Rsk}(N^*)$  of the prestressing steel at  $N^*$  load cycles according to the Wöhler curves specified in II-4.3.7.7 [MN/m<sup>2</sup>]. The value found in Table 4.116, Row 4, is suggested in the dialog.

**Lambda.s, Lambda.b, Lambda.p**

Correction coefficient  $\lambda_s$  for determining the damage-equivalent stress range  $\Delta\sigma_{s,eq}$  from the stress range  $\Delta\sigma_s$  of the steel according to II-A.106.2 (102)P and II-A.106.3.1 (101)P for longitudinal reinforcement, shear reinforcement and prestressing steel.

**Lambda.c**

Correction coefficient  $\lambda_c$  for determining the damage-equivalent concrete stress according to II-A.106.3.2 (102) for railroad bridges.

**Eta**

Increase factor  $\eta$  for the reinforcing steel stress of the longitudinal reinforcement. This factor is used to take into account the varying bonding behavior of reinforcing and prestressing steel as per II-4.3.7.3 (3)P.

**fcd,fat**

Concrete compressive strength before onset of cyclic load according to II-4.3.7.4 (101)P and (102)P [MN/m<sup>2</sup>]. In general, the following applies:

$$f_{cd,fat} = \beta_{cc}(t_0) \cdot f_{cd} \cdot \left(1 - \frac{f_{ck}}{250}\right)$$

with

$$\beta_{cc}(t_0) = e^{0.2(1-\sqrt{28/t_0})}, \quad t_0 = \text{time of the initial stressing of the concrete.}$$

$f_{cd,fat}$  for  $t_0 = 28$  and  $f_{cd} = 0.85 \cdot f_{ck} / \gamma_{c,fat}$  is suggested in the dialog.

**k0**

The statically determined share of prestressing must be reduced in the case of beams and design objects. A base value of 0.9 as specified in II-4.3.7.2 (103) P is suggested in the dialog. For prestressing tendon couplers a further reduction of the base value is required.

**Simplified check**

The simplified check according to II-4.3.7.2 (103)P bases on the frequent action combination including the traffic loads used for the serviceability checks. The method for concrete is defined in II-4.3.7.4, the permissible stress ranges for steel are suggested according to II-4.3.7.5 (101) in the dialog. For shear reinforcement this value is reduced analogous to Table 4.117.

**Limit design variants**

For area elements, the variants for determining the stress range can be limited to the corresponding sets of design internal forces. For more information see section '*Check Against Fatigue > Special Characteristic of Shell Structures*'.

**Factor for Qfat**

Increase factor for the cyclic fatigue action defined as  $Q_{fat}$  which is taken into account during the calculation of the damage equivalent stress range  $\Delta\sigma_{s,eq}$  of the reinforcing and prestressing steel. With it, e.g. the rules for the fatigue check for road bridges according to II-A.106.2 (101)P can be applied.

## Variation Coefficients

Properties for element 405 - DIN TR 102 - Variation coefficients

Section: Form, Shear stresses, Material, Bedding, DIN TR 102, Recalculation Guideline, General, Eccentricity

Number: 11 - M<sub>i</sub> Section Type: Polygon Material Type: C40/50

Label: Main girder

Variation coefficients for the internal prestressing in the proofs of decompression, crack reinforcement and crack width.

	r.sup	r.inf
Construction stage:	1.1	0.95
Final state:	1.1	0.9

OK Cancel Help

The coefficients for taking the variation of prestressing force into account are defined in II-2.5.4.2 (4) for the final state. They vary depending on the prestressing type. Smaller variations can be assumed in the construction stage according to II-4.4.2.1 (107)P. In the action combinations, situations are declared as construction stage or finale state. The values for garland-shaped guided tendons with subsequent bond are suggested in the dialog.

The defined variation coefficients are taken into account for the effects from internal prestressing in the following checks:

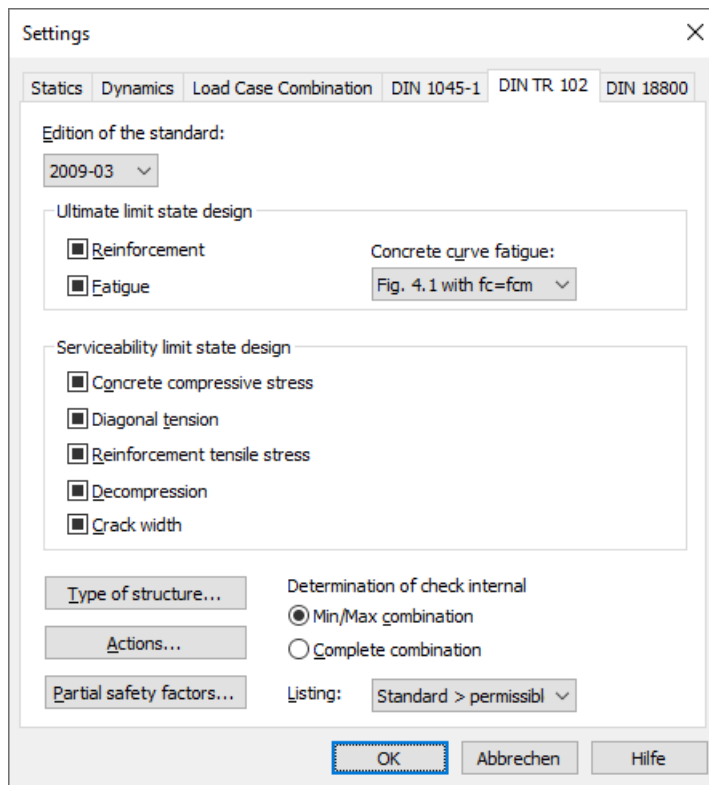
- Minimum reinforcement for crack width limitation
- Crack width check
- Decompression check
- Check of the diagonal principal tensile stresses

Regarding the effects from external prestressing, the variation coefficients correspond to  $r_{sup} = r_{inf} = 1$  according to II-2.5.4.2 (4).



## Analysis Settings

The *DIN TR 102* dialog page can be opened using the *Settings* function of the *Analysis* menu.



### Edition of the standard

The edition you select will be used for all subsequent calculations.

### Check selection

When selecting checks, the following cases are to be distinguished:

- or  The check is performed according to the settings in the section dialog (see *Section inputs*).
- or  The check is performed for all sections of the structure.
- or  The check is performed for no sections of the structure.

Corresponding section settings are bundled as follows:

Reinforcement	Bend and longitudinal force Lateral force Torsion Robustness
Crack width	Minimum crack reinforcement Calculation of the crack width

An overview of the checks can be accessed using the *Design Settings* function in the *DIN Technical Report Bridge Checks* folder of the database.

### Concrete curve fatigue

For the fatigue checks, the curve to determine the concrete compressive stresses can be selected.

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *EN 1992-2 Design > Check internal forces*.

**Type of structure...**

Open the dialog for selecting the structure type.

**Actions...**

Open the dialog for describing actions.

**Partial safety factors...**

Open the dialog for modifying partial safety factors.

**Listing**

- *No*: No log is generated by the checking program.
- *Standard*: Log with tabular output of results.
- *Detailed*: Additional output of the decisive combination internal forces at the check locations.
- *Standard > permissible*: Standard log limited to check locations where the permissible limit values are exceeded.
- *Detailed > permissible*: Detailed log limited to check locations where the permissible limit values are exceeded.

## Single Design

The single design function allows you to analyze individual sections independently of the global system using predefined internal forces. The following data needs to be entered in the *Single Design* table, which is located in the *DIN Technical Report Bridge Checks* folder of the database.

**Section**

Number of the section to be designed. Both polygon and composite sections can be designed.

**Combination**

Design situation:

- *0*: Permanent and temporary design situation.
- *1*: Accidental design situation.

**Nsd, Mysd, Mzsd**

Internal forces being designed. The internal forces refer to the centroid in polygon sections or the section zero point in composite sections.

**Mode**

- *Standard*: Standard design mode for bending with normal force throughout the load range. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members a symmetrical design is carried out with allowance for the minimum reinforcement according to TR 102, Section 5.4.1.2.1 (1)\*P.
- *Strains*: Determine strain state for existing reinforcing steel layers.
- *Strains SLS*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. In the compression zone, a linear strain-stress curve of the concrete with the gradient  $\tan \alpha = E_{cm}$  is used.
- *Strains SLS2*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A nonlinear strain-stress curve of the concrete is used as shown in TR 102, Figure 4.1. Note that a horizontal progression is assumed for strains exceeding  $\epsilon_{c1}$ .
- *Load bearing capacity*: Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
- *Maximum bending moment My*: Determination of the maximum bearable bending moment  $M_y$ . The moment  $M_y$  is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.
- *Inactive*: Design disabled.

The calculation can be carried out while the input table is open using the *Single Design* or *Page Preview* menu item.

# Prestressed Structures

## Internal Prestressing

For internal prestressing, the tendon groups as well as the prestressing system and procedures are entered using the *Prestressing* function of the *Structure* menu. To include them in the FEM calculation, you then need to define a load case of the *Prestressing* load type. For more information, refer to the *Prestressed Concrete* section of the manual.

Prestressing with bond and prestressing without bond are differentiated in the section inputs and the specifications for the *Creep and shrinkage* load case. For prestressed components with subsequent bond the tendons can be set ungrouted for the respective situation in the action combination dialog.

## External Prestressing, Mixed Construction

External prestressing can be taken into account by entering the external forces directly in the program. For mixed construction, the additional tendons with bond must be entered as described above.

## Variation of Prestressing

The consideration of possible prestressing force variation is defined in II-2.5.4. For checks in the ultimate limit state, the following applies to the prestressing force design value according to II-2.5.4.2 (6) P:

$$P_d = \gamma_p \cdot P_{m,t}$$

with

$P_{m,t}$  Average value of prestressing force at time t including prestressing losses from friction, slippage, creep and shrinkage as well as relaxation.

$\gamma_p$  Partial safety factor of the prestressing force,  $\gamma_p = 1$  as specified in II-2.3.3.1 (101)P.

In the serviceability limit state, two characteristic values for the prestressing force are defined in II-2.5.4.2 (3)P:

$$P_{k,sup} = r_{sup} \cdot P_{m,t} \quad \text{Upper characteristic value.}$$

$$P_{k,inf} = r_{inf} \cdot P_{m,t} \quad \text{Lower characteristic value.}$$

The variation coefficients for internal prestressing are defined separately for construction stages and final states. They are used in the following checks:

- Minimum reinforcement for crack width limitation.
- Crack width check.
- Decompression check.
- Check of the diagonal principal tensile stresses.

Regarding the effects from external prestressing, the variation coefficients correspond to  $r_{sup} = r_{inf} = 1$  as specified in II-2.5.4.2 (4).

## Creep and Shrinkage

Similar to prestressing, creep and shrinkage are taken into account by defining the appropriate load case (*Creep and shrinkage* load type) in the FEM calculation. Besides the creep-generating permanent load case, you also need to specify whether the internal forces relocation between concrete and prestressing steel is to be taken into account. This option is only useful in the case of tendons with bond.

The decisive creep and shrinkage coefficients for calculating the *Creep and shrinkage* load case are entered in the Section dialog. Alternatively, you can also use this dialog to calculate the coefficients according to Book 525, Section 9.1.4.

The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1+\rho \cdot \varphi} (\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,s})$$

In this case:

- $\sigma_b(t)$  Concrete stress from creep and shrinkage at time  $t$ .
- $E_b$  E-modulus of the concrete.
- $\rho$  Relaxation coefficient according to Trost for time  $t$  (normally  $\rho = 0.80$ ).
- $\varphi$  Creep coefficient for time  $t$ .
- $\varepsilon_b(t)$  Concrete strain from creep and shrinkage at time  $t$ .
- $\varepsilon_{b,0}$  Concrete strain from creep-generating continuous load.
- $\varepsilon_{b,s}$  Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, yield the internal forces and deformations of the concrete. The resulting stress changes in the prestressing steel are also determined provided they are selected in the load case. Any influence from the relaxation of the prestressing steel will be ignored in this case. According to Zilch/Rogge (2002, p. 256), this influence can be calculated separately (see following section) and combined with the changes from creep and shrinkage for all time-dependent prestressing losses:

$$\Delta\sigma_{p,csr} = \Delta\sigma_{pr} + E_p \cdot \Delta\varepsilon_{cpt}$$

with

- $\Delta\sigma_{pr}$  Prestressing loss from relaxation of the prestressing steel.
- $\Delta\varepsilon_{cpt}$  Concrete strain change from creep and shrinkage.
- $E_p$  E-modulus of the prestressing steel.

## Relaxation of Prestressing Steel

According to TR 102, Section 4.2.3.5.5, the stress change  $\Delta\sigma_{pr}$  in the tendons at position x caused by relaxation must be taken into account in addition to the stress loss from concrete creep and shrinkage. The relaxation can be defined by a ratio of initial stress to characteristic tensile stress ( $\sigma_{p0}/f_{pk}$ ) from the building inspection certification with an initial stress of

$$\sigma_{p0} = \sigma_{pg0} - 0.3 \Delta\sigma_{p,csr} \quad (4.11)$$

with

$\Delta\sigma_{p,csr}$  Stress change in the tendons due to creep, shrinkage and relaxation at position x at time t .

$\sigma_{pg0}$  Initial stress in the tendons from prestressing and permanent actions.

Since the entire stress loss cannot be known in advance, the input value  $\Delta\sigma_{p,csr}$  for Eq. (4.11) must be estimated and, if necessary, iteratively corrected (cf. König et al. 2003, p. 38). Alternatively, you may ignore the second expression in Eq. (4.11) according to the Technical Report for the sake of simplicity and to be on the safe side.

The following table shows an example of stress loss due to relaxation.

Characteristic values of the stress losses $\Delta\sigma_{pr}$ in % of the initial tension $\sigma_{p0}$ for prestressing steel strand St 1570 / 1770 with very low relaxation							
$\sigma_{p0}/f_{pk}$	Time interval after prestressing in hours						
	1	10	200	1000	5000	$5 \cdot 10^5$	$10^6$
0.45							
0.50							
0.55						1.0	1.2
0.60					1.2	2.5	2.8
0.65				1.3	2.0	4.5	5.0
0.70			1.0	2.0	3.0	6.5	7.0
0.75		1.2	2.5	3.0	4.5	9.0	10.0
0.80	1.0	2.0	4.0	5.0	6.5	13.0	14.0

For tendons with DIN 4227 certification, the example of  $t = \infty$  with a permissible utilization of 0.55 according to DIN 4227-1, Tab. 9, Row 65, results in a stress loss of around 1%, which generally can be ignored.

Tendons with new certification may be utilized by up to 0.65 according to Technical Report 102, Section 4.4.1.4. This results in significantly higher stress losses that must be accounted for.

You can define the stress losses in the actions CSR1 and CSR2 of the *DIN Technical Report 101 Actions* dialog.

## Ultimate Limit State Design

The following checks are available:

- Bending with or without longitudinal force or longitudinal force only (TR 102, Section 4.3.1).
- Failure without warning (Section 4.3.1.3).
- Lateral force (Section 4.3.2).
- Torsion and combined stressing (Section 4.3.3).
- Punching shear (Section 4.3.4).
- Fatigue check (Section 4.3.7).

## Design Combinations

According to TR 101, Section 9.4.2, the following combinations are taken into account in the ultimate limit states:

- Combination for permanent and temporary design situations

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (9.10)$$

- Combination for accidental design situations

$$\sum_{j \geq 1} \gamma_{GAj} \cdot G_{kj} + \gamma_{PA} \cdot P_k + A_d + \psi_{11} \cdot Q_{k1} + \sum_{i > 1} \psi_{2i} \cdot Q_{ki} \quad (9.11)$$

- Combination for design situations resulting from earthquakes ( $A_E$ )

$$\sum_{j \geq 1} G_{kj} + P_k + \gamma_1 \cdot A_{Ed} + \sum_{i \geq 1} \psi_{2i} \cdot Q_{ki} \quad (9.12)$$

The weighting factor for the earthquake action is assumed as  $\gamma_1 = 1$  according to DIN 4149, Eq. (37).

For the check against fatigue two alternative action combinations can be used:

- Frequent combination for simplified checks according to TR 101, Section 9.5.2, Equation (9.17), in conjunction with TR 102, Sections 4.3.7.4 and 4.3.7.5 (101).

$$\sum_{j \geq 1} G_{kj} + P_k + \psi_{11} \cdot Q_{k1} + \sum_{i > 1} \psi_{2i} \cdot Q_{ki} \quad (9.17)$$

- Fatigue combination according to DIN EN 1992-1-1, Chapter 6.8.3, Equation (6.69), for checks with damage equivalent stress ranges based on TR 102, Section 4.3.7.5 (102) and Annex 106.

$$\left( \sum_{j \geq 1} G_{kj} + P_k + \psi_{11} \cdot Q_{k1} + \sum_{i > 1} \psi_{2i} \cdot Q_{ki} \right) + Q_{fat} \quad (6.69)$$

In this equation  $Q_{k,1}$  and  $Q_{k,i}$  are non-cyclic, non-permanent actions, whereas  $Q_{fat}$  defines the action of the relevant fatigue load.

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

## Partial Safety Factors for Construction Material

The partial safety factors for the construction material properties are preset with the values specified in TR 102, Section 2.3.3.2, Table 2.3. If necessary, they can be adjusted. In design situations resulting from earthquakes, the factors of the permanent and temporary design situation apply as specified in DIN 4149:2005-04, Chapter 8.1.3 (3). Guideline 5.3.3 (9) of DIN 1045-1 applies for strength classes C55/67 and LC55/60 or higher.

## Stress-Strain-Curves

The following characteristics are used for section design:

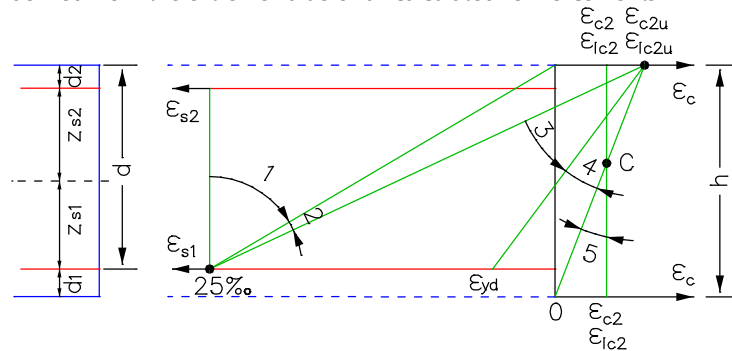
- *Concrete*: Parabola-rectangle diagram according to TR 102, Fig. 4.2.
- *Reinforcing steel*: Stress-strain curve according to Fig. 4.5b) with rising upper branch.
- *Prestressing steel*: Stress-strain curve according to Fig. 4.6b) with horizontal upper branch as per 4.2.3.3.3 (6)\*.

For the fatigue checks, the user defines one of the following curves to determine the concrete compressive stresses:

- Stress-strain curve according to Fig. 4.1 with  $f_c = f_{cm}$ .
- Parabola-rectangle diagram according to Fig. 4.2.
- Linear curve with the gradient  $\arctan E_{cm}$ .

## Design for Bending With or Without Longitudinal Force and Longitudinal Force Only

The design for normal force and bending moment is carried out according to TR 102, Section 4.3.1. The reinforcement required for each internal force combination at the reinforced concrete section is determined iteratively based on the formulation of equilibrium conditions as well as the limit strain curve depicted in the illustration below. The final result is derived from the extreme value of all calculated reinforcements.



Strain areas for the design

You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes.

### Mode Standard

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tensile section to the greatest degree possible. Given ratios between certain reinforcement layers in the tension or compression zone are maintained as far as possible. For reasons of economy and to limit the compression zone height according to TR 102, Section 2.5.3.4.2 (5), the compression reinforcement in strain area 3 will be determined such that the following applies for the steel strain  $\varepsilon_{s1}$ :

$$\varepsilon_{s1} \geq \max\left(\varepsilon_{yd}, \frac{\varepsilon_{c2u}}{x/d} - \varepsilon_{c2u}\right) \quad [‰]$$

with

$\varepsilon_{yd}$  The steel strain associated with the design value for strength  $f_{yd}$ .

$x/d$  Corresponding compression zone height.

≤ 0.45 for concrete strength classes up to C50/60.

≤ 0.35 for concrete strength class C55/67 or higher and for lightweight concrete according to DIN 1045-1, Section 8.2 (3).

The procedure in strain areas 4 and 5 is the same as with symmetrical design. The requirements for the minimum reinforcement for the less stressed direction of slabs and walls according to Chapter 5.4.3.2.1 (2)\*P resp. 5.4.7.3 (2)\*P are not taken into account during bending design.

### Mode Symmetrical

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected.

**Mode Compression member**

The design is performed symmetrically. In addition, the minimum reinforcement required by II-5.4.1.2.1 (1)\*P is determined:

$$A_{s,\min} = 0.15 |N_{Ed}| / f_{yd} \geq 0.003 A_c$$

with

$N_{Ed}$  Design value of the longitudinal force to be absorbed.

$f_{yd}$  Design value for the reinforcing steel strength at the yield strength.

Concrete compression according to II-4.3.1.2 (1)P(X) cannot be checked.

**Inclusion of tendons with bond**

When designing beams and design objects, the internal forces of the concrete section is reduced by the statically determined portions which result from prestressing minus the losses from creep, shrinkage and prestressing steel relaxation (CSR). Situations prior to the grouting of the tendons are excluded. So only the restraint portions from  $P+CSR$  and the external loads are contained in the remaining internal forces for the composite section. If necessary, the reinforcing steel positioned by the user will be increased until the composite internal forces can be absorbed.

The position of the tendon groups in the section, the prestressing losses from CSR, the statically determined portions and the internal forces of the concrete section and the composite section are written to the detailed log.

As a separation into statically determined and undetermined shares of the internal forces from prestressing is not possible for shell structures, the prestressing is taken into account fully on the action side when designing the longitudinal reinforcement. As a result, on the resistance side only mild steel and concrete are considered whereas the strain reserves of the tendons with bond are not used.

**Minimum Reinforcement Against Failure Without Warning**

According to TR 102, Section 4.3.1.3, brittle failure of the section that occurs without warning must be prevented. In most cases, the minimum reinforcement for crack width limitation as per Section II-4.4.2 is enough to prevent such failure according to II-4.3.1.3 (3). If necessary, the failure can be prevented by applying robustness reinforcement as per II-4.3.1.3 (105)P, Item (b):

$$A_s = M_{r,ep} / (f_{yk} \cdot z_s)$$

with

$M_{r,ep}$  Crack moment by which a tensile stress of  $f_{ctk;0.05} = 0.7 \cdot f_{ctm}$  occurs without the effect of prestressing at the section edge.

$z_s$  Lever arm of internal forces.

The robustness reinforcement  $A_s$  should be applied to areas where tensile stresses occur in the concrete under the infrequent action combination. According to II-4.3.1.3 (108), the statically undetermined prestressing effect should be taken into account in this combination rather than the statically determined prestressing effect.

The program determines all stresses at the gross section. The statically determined prestressing effect can only be subtracted for beams and design objects. For area elements the prestress is alternatively reduced by a user-defined reduction factor. The crack moment results in  $M_{r,ep} = W_y \cdot f_{ctk;0.05}$ , the lever arm  $z_s$  of the internal forces is assumed to be  $0.9 \cdot d$  for the sake of simplicity. The calculated reinforcement is evenly distributed to the reinforcement layers in the tensile zone. In the design mode *symmetrical* reinforcement is also applied to the remaining layers. This will not affect the predefined relationships between the individual reinforcement layers. For sections with mode *compression member* the robustness reinforcement is not checked because minimum reinforcement is already determined during the design for bending with longitudinal force.

**Minimum Surface Reinforcement for Prestressed Components**

A minimum surface reinforcement must always be specified for prestressed components in accordance with TR 102, Section 5.4.0 (Edition 2009), and Ref. No. 002 of the Knowledge Base of the Building and Civil Engineering Standards Committee (NABau), regardless of the guidelines for robustness or crack reinforcement. The reinforcement determined according to Table 5.7 and 5.7.a can be entered into the program by specifying a base reinforcement in the reinforcing steel description.



## Design for Lateral Force

Lateral force design involves determining the lateral force reinforcement and includes a concrete strut check as per TR 102, Section 4.3.2. The following special conditions apply:

- The angle of the lateral force reinforcement is assumed to be 90°.
- In the calculation, the value for  $\cot \Theta$  is limited to the range permitted in accordance with Eq. (4.122) (method with variable or load-dependent strut angle), unless the check with a constant value is selected in the section dialog. The actual effective angle of the concrete struts is logged for each check location. Edition 2009: For perpendicular stirrups or longitudinal tensile load,  $\cot \Theta$  should not fall below the limit value of 1.0 as per II-4.3.2.4.4 (1)\*P. This is guaranteed by the program provided the user does not specify a smaller value.
- The minimum reinforcement is maintained in the calculated stirrup reinforcement as described in II-5.4.2.2 (3)\*P and (4)\*P while the reinforcement level  $\rho$  specified in Table II-5.7 is weighted with a user-defined factor. For areas, the minimum reinforcement will only be determined if a lateral force reinforcement is required for computation (cf. Building and Civil Engineering Standards Committee (NABau) No. 131).
- For beams and design objects, the shear design is performed separately for the  $Q_y$  and  $Q_z$  lateral forces.
- Slab and shell elements are designed for lateral force  $q_r = \sqrt{q_x^2 + q_y^2}$ . Depending on which has a negative effect, either the principal compressive force or principal tensile force is used for the associated longitudinal force. If selected, the check will be carried out separately for the reinforcement directions x and y in accordance with Section II-4.3.2.2 (5) \* (Edition 2009). In this case, the normal force in reinforcement direction is used for the associated longitudinal force. If lateral force reinforcement is necessary, it must be added from both directions.
- There is **no** reduction of the action from loads near supports as specified in II-4.3.2.2, Sections (10) and (11).
- For beams and design objects, the decisive values of the equivalent rectangle are determined by the user independently of the normal section geometry. The coefficients for calculating the inner lever arm  $z$  based on the effective width and effective height must also be specified. Alternatively, the lever arm from bending design can be used.
- For area elements, the calculation is normally performed with the lever arm  $z = 0.9 d$ . If selected, the lever arm from bending design is used alternatively.
- As described in II-4.3.2.4.2 (2)\*, the internal lever arm is limited to the maximum value derived from  $z = d - c_{v,1} - 30$  mm and  $z = d - 2c_{v,1}$  (Edition 2009 and Interpretation No. 24 of the NABau). Note that  $c_{v,1}$  is the laying measure of the longitudinal reinforcement in the concrete compressive zone. If  $c_{v,1}$  is not specified, the program will use the shortest axis distance of the longitudinal reinforcement from the section edge  $d_1$  in its place.
- For sections with internal prestressing, the design value of lateral load-bearing capacity  $V_{Rd,max}$  is determined using the nominal value  $b_{w,nom}$  of the section width as per II-4.3.2.2 (8)\*P.
- Edition 2009: The lateral load-bearing capacity  $V_{Rd,max}$  is only checked for lateral forces  $V_{Ed} > V_{Rd,ct}$  as per II-4.3.2.2 (4)P.
- The necessity of a lateral force reinforcement is checked according to II-4.3.2.3. As in the previous case, **no** reduction of the action from loads near supports occurs.

The formulas from TR 102 that are used are listed below.

### Components without computationally necessary lateral force reinforcement

#### Edition 2003

$$V_{Rd,ct} = [0.10 \cdot \kappa \cdot \eta_1 \cdot (100 \rho_1 \cdot f_{ck})^{1/3} - 0.12 \cdot \sigma_{cp}] \cdot b_w \cdot d \quad (4.118a:2003)$$

with a minimum value of

$$V_{Rd,ct,min} = [\eta_1 \cdot v_{min} - 0.12 \sigma_{cd}] \cdot b_w \cdot d$$

$$v_{min} = 0.035 \cdot \kappa^{3/2} \cdot f_{ck}^{1/2} \quad (4.118b:2003)$$

In the accidental design situation, the following applies as per II-4.3.2.3 (101)

$$V_{Rd,ct} = [0.115 \cdot \kappa \cdot \eta_1 \cdot (100 \rho_1 \cdot f_{ck})^{1/3} - 0.12 \cdot \sigma_{cp}] \cdot b_w \cdot d \quad (4.118c:2003)$$

with a minimum value of

$$V_{Rd,ct,min} = [\eta_1 \cdot v_{min} - 0.12 \sigma_{cd}] \cdot b_w \cdot d$$

$$v_{min} = 0.040 \cdot \kappa^{3/2} \cdot f_{ck}^{1/2} \quad (4.118d:2003)$$

**Edition 2009**

$$V_{Rd,ct} = [0.15/\gamma_c \cdot \kappa \cdot \eta_1 \cdot (100 \rho_1 \cdot f_{ck})^{1/3} - 0.12 \cdot \sigma_{cp}] \cdot b_w \cdot d \quad (4.118a:2009)$$

with at least

$$V_{Rd,ct,min} = [\eta_1 \cdot v_{min} - 0.12 \sigma_{cd}] \cdot b_w \cdot d$$

$$v_{min} = \frac{\kappa_1}{\gamma_c} \cdot \sqrt{\kappa^3 \cdot f_{ck}} \quad (4.118b:2009)$$

where

$\gamma_c$  is the partial safety factor for reinforced concrete as per II-2.3.3.2 (1)P, Table 2.3.

$\kappa_1 = 0.0525$  for  $d \leq 600$  mm

$= 0.0375$  for  $d \geq 800$  mm

For  $600 \text{ mm} < d < 800$  mm,  $\kappa_1$  can be interpolated linearly.

$$\kappa = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

$V_{Rd,ct}$  is the design value of the absorbable lateral force in a component without lateral force reinforcement.

$\eta_1 = 1.0$  for normal concrete; for lightweight concrete according to DIN 1045-1, Table 10.

$\rho_1$  is the longitudinal reinforcement level with

$$\rho_1 = \frac{A_{sl}}{b_w \cdot d} \leq 0.02$$

$A_{sl}$  is the area of the tensile reinforcement that extends beyond the section being considered by at least measure  $d$  and is effectively fixed into place there (see Fig. II-4.12). For prestressing with immediate bond, the entire prestressing steel area can be taken into account for  $A_{sl}$ .

$b_w$  is the smallest section width in the tensile zone of the section in mm.

$d$  is the effective static height of the bending reinforcement in the viewed section in mm.

$f_{ck}$  is the characteristic value of the concrete compressive strength in N/mm<sup>2</sup>.

$\sigma_{cd}$  is the design value of the concrete longitudinal stress at the level of the section's centroid with

$$\sigma_{cd} = N_{Ed} / A_c \text{ in N/mm}^2.$$

$N_{Ed}$  is the design value of the longitudinal force in the section as a result of external actions or prestressing ( $N_{Ed} < 0$  as the longitudinal compressive force).

**Components with computationally necessary lateral force reinforcement**

$$V_{Rd,sy} = \frac{A_{sw}}{s_w} \cdot f_{yd} \cdot z \cdot \cot \Theta \quad (4.27)$$

Where

$V_{Rd,sy}$  is the design value of the absorbable lateral force that is limited by the load-bearing capacity of the lateral force reinforcement.

$A_{sw}$  is the section area of the lateral force reinforcement.

$s_w$  is the distance of the reinforcement perpendicular to the component axis measured in the direction of the component axis.

$z$  is the inner lever arm.

$\cot \Theta$  is the strut angle of the truss:

$$4/7 \leq \cot \Theta \leq \frac{1.2 - 1.4 \sigma_{cd} / f_{cd}}{1 - V_{Rd,c} / V_{Ed}} \leq 7/4 \quad (4.122)$$

(see ARS 11/2003 (11) resp. Ril 804.4201 (10))

Edition 2009:  $\cot \Theta < 1$  should only be used as an exception. In the case of longitudinal tensile stress this lower limit applies basically (ref. also Book 525, Corr. 1:2005-05). The program takes the limit into account as long as the user does not enter a smaller value.

$$V_{Rd,c} = \beta_{ct} \cdot 0.10 \cdot \eta_1 \cdot f_{ck}^{1/3} \left(1 + 1.2 \frac{\sigma_{cd}}{f_{cd}}\right) \cdot b_w \cdot z \quad (4.123:2003)$$

$$V_{Rd,c} = c_j \cdot 0.48 \cdot \eta_1 \cdot f_{ck}^{1/3} \left(1 + 1.2 \frac{\sigma_{cd}}{f_{cd}}\right) \cdot b_w \cdot z \quad (4.123:2009)$$

Where

$$\beta_{ct} = 2.4$$

$$c_j = 0.5$$

$\eta_1 = 1.0$  for normal concrete; for lightweight concrete according to DIN 1045-1, Table 10.

$\sigma_{cd}$  is the design value of the concrete longitudinal stress at the level of the section's centroid with  $\sigma_{cd} = N_{Ed} / A_c$  in N/mm<sup>2</sup>.

$N_{Ed}$  is the design value of the longitudinal force in the section as a result of external actions or prestressing ( $N_{Ed}$  as longitudinal compressive force).

$V_{Ed}$  is the design value of the acting lateral force.

$$V_{Rd,max} = \frac{b_w \cdot z \cdot \alpha_c \cdot f_{cd}}{\cot \Theta + \tan \Theta} \quad (4.26)$$

With

$V_{Rd,max}$  Design value of the absorbable lateral force that is limited by the strut strength. The stresses in the struts must be limited to the value

$$\sigma_c \leq \alpha_c \cdot f_{cd} \quad (4.21)$$

$\alpha_c$  Reduction factor for the strut strength.

$\alpha_c = 0.75 \eta_1$  with  $\eta_1 = 1.0$  for normal concrete; for lightweight concrete according to DIN 1045-1, Tab. 10.

The amount of the tensile force in the longitudinal reinforcement due to lateral force according to Fig. 4.13 is  $\Delta F_{sd} = 0.5 \cdot |V_{Ed}| \cdot (\cot \Theta - \cot \alpha)$ .

### Lateral force reinforcement

$$\rho_w = \frac{A_{sw}}{s_w \cdot b_w \cdot \sin \alpha} \quad (5.16)$$

Where

$\rho_w$  is the reinforcement level of the lateral force reinforcement.

$\alpha$  is the angle between the lateral force reinforcement and the beam axis.

$\min \rho_w$  is minimum value of  $\rho_w$  according to II-5.4.2.2 (4)\*P

In general:  $\rho_w = 1.0 \rho$

Slabs:  $\rho_w = 0.6 \rho$

Structured sections with prestressed tension chord:  $\rho_w = 1.6 \rho$

$\rho$  is the value according to TR 102, Table 5.7.

## Design for Torsion and Combined Loads

The design for torsion is carried out in accordance with TR 102, Section 4.3.3. This design involves determining the diagonal tensile reinforcement and the longitudinal reinforcement and includes a concrete strut check under maximum torsion stress and a concrete strut check under simultaneously acting lateral force. The strut angle is determined according to Equation (4.122) with the lateral force according to Equation (4.242). Alternatively a strut angle of 45° or 30° for torsion (TR 102, Chapter 4.3.3.2.2 (3)\*P) or a constant value  $\cot \Theta$  for lateral force and torsion (cf. interpretation No. 24 of NABau for DIN 1045-1) can be chosen in the section dialog.

For hollow box girders, an additional check is carried out as per 4.3.3.2.2 (2) to ensure that the principal compressive stress  $\sigma_2$  from bending, longitudinal force and torsion in state I does not exceed the value  $f_{cd} = \alpha_c \cdot f_{ck} / \gamma_c$ . For this purpose the longitudinal compressive stress  $\sigma_{Ed}$  is calculated with a distance of  $t_{eff}/2$  from the cross-section edge and the shear stress from torsion is assumed as  $\tau_{Ed} = T_{Ed} / (2 \cdot A_k \cdot t_{eff})$ .

The equivalent section on which this design is based is defined by the user independently of the normal section geometry.

Relevant formulas from TR 102:

$$V_{Ed,T} = \frac{T_{Ed} \cdot z}{2A_k} \quad (4.142)$$

where

$V_{Ed,T}$  is the shear force in a wall of the check section as a result of a torsion moment.

$A_k$  is the area enclosed by the center lines of the walls.

$z$  is the height of the wall, which is defined by the distance of the intersection points of the wall center line to the center lines of the adjacent walls.

$$V_{Ed,T+V} = V_{Ed,T} + \frac{V_{Ed} \cdot t_{eff}}{b_w} \quad (4.242)$$

where

$V_{Ed}$  is the design value of the acting lateral force according to II-4.3.2.

$t_{eff}$  is the effective thickness of a wall;  $t_{eff}$  is twice the distance from the center line to the exterior but not greater than the thickness of the existing wall (see Fig. 4.15).

$$T_{Rd,sy} = \frac{A_{sw}}{s_w} \cdot f_{yd} \cdot 2A_k \cdot \cot \Theta \quad (4.43)$$

$$T_{Rd,sy} = \frac{A_{sl}}{u_k} \cdot f_{yd} \cdot 2A_k \cdot \tan \Theta \quad (4.44)$$

where

$T_{Rd,sy}$  is the design value of the absorbable torsion moment of the section.

$A_{sw}$  is the section area of the torsion reinforcement perpendicular to the component axis.

$s_w$  is the distance of the torsion reinforcement measured in the direction of the component axis.

$A_{sl}$  is the section area of the torsion longitudinal reinforcement.

$u_k$  is the perimeter of area  $A_k$ .

$\Theta$  is the strut angle of the truss.

$$T_{Rd,max} = \frac{\alpha_{c,red} \cdot f_{cd} \cdot 2A_k \cdot t_{eff}}{\cot \Theta + \tan \Theta} \quad (4.40)$$

where

$T_{Rd,max}$  is the design value of the maximum absorbable torsion moment of the section.

$\alpha_{c,red} = 0.7 \alpha_c$  in general (with  $\alpha_c = 0.75 \eta_1$  with  $\eta_1 = 1.0$  for normal concrete; for lightweight concrete according to DIN 1045-1, Table 10).

$\alpha_{c,red} = \alpha_c$  for box sections with reinforcement at the inner and outer sides of the walls.

$$\left[ \frac{T_{Ed}}{T_{Rd,max}} \right]^2 + \left[ \frac{V_{Ed}}{V_{Rd,max}} \right]^2 \leq 1 \quad \text{for compact sections} \quad (4.47a)$$

$$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed}}{V_{Rd,max}} \leq 1 \quad \text{for box sections} \quad (4.47b)$$

where

$V_{Rd,max}$  is the design value of the absorbable lateral force according to Eq. (4.26).

For an approximately rectangular full section no shear and torsion reinforcement beside the minimum reinforcement according to Section 5.4.2.2, Table 5.7, is required, if the following conditions are met:

$$T_{Ed} \leq V_{Ed} \cdot \frac{b_w}{4.5} \quad (4.48)$$

$$V_{Ed} \left[ 1 + \frac{4.5 T_{Ed}}{V_{Ed} \cdot b_w} \right] \leq V_{Rd,ct} \quad (4.49)$$

### Note

In accordance with DIN TR 102, Chapter 4.3.3.1 (7)\*P, the torsion longitudinal reinforcement in compression chords can be reduced according to the existing compression forces. In tension chords it is to be added to the remaining longitudinal reinforcement. With the help of the graphical representation of the prestressing steel stresses for the permanent and temporary combination, the region and the amount of prestressing steel that can be taken into account for the torsion longitudinal reinforcement according to Chapter 4.3.3.1 (8)P can be evaluated.

## Punching Shear

The punching shear check based on DIN Technical Report 102, Section 4.3.4, is essentially the same as the check based on DIN 1045-1. For more information on this, refer to section *DIN 1045-1* of the manual. This check is accessed from the *Structure* menu.

## Check Against Fatigue

The user can select two alternative methods for design:

- Simplified check for the frequent action combination according to TR 102, Sections 4.3.7.4 and 4.3.7.5 (101), taking the relevant traffic loads at serviceability limit state into account.
- Check with damage equivalent stress ranges for the fatigue combination according to TR 102, Section 4.3.7.5 (102) and Annex 106, considering the specific fatigue load  $Q_{fat}$  analogous to DIN EN 1992-1-1:2005, Chapter 6.8.3.

For road bridges,  $Q_{fat}$  is the fatigue load model 3 according to TR 101, Section 4.6.

The increase factors according to I-4.6.1 (7) must be taken into account when entering the load ordinates while the factors according to II-A.106.2 (101)P are defined in the section dialog.

According to TR 101, Section 6.9 in conjunction with TR 102, Annex A.106.3, the traffic model 71 in the infrequent action combination including the dynamic factor as per TR 101 plays the determinant role in calculating the stress range for railroad bridges.

According to Section II-4.3.7.2 (103)P, the 0.9-fold statically determined portion of the prestressing force is to be used in the check. For construction joints with tendon couplers, this value is to be further reduced by an appropriate factor. The decisive reduction factor is defined in the Section dialog.

The curve to determine the concrete compressive stresses in state II is selected in the settings dialog.

## Fatigue of Longitudinal Reinforcement, Shear Reinforcement and Prestressing Steel

The fatigue check is carried out according to Section II-4.3.7.5. The steel stresses are calculated for longitudinal reinforcement from bending and longitudinal force as well as for prestressing steel in beams and design objects under the assumption of a cracked concrete section. For shear and longitudinal reinforcement from lateral force and torsion, the stresses are calculated as per II-4.3.7.3 (4)\*P based on a truss model with the strut angle  $\tan \Theta_{fat} = \sqrt{\tan \Theta}$  according to Eq. (4.187). Where  $\Theta$  is the angle between the concrete compression struts and the beam axis used in the corresponding ultimate limit state design. The prestressing steel stresses in area elements are determined at the uncracked concrete section. Tendons without bond and external tendons are not checked.

**Simplified check**

According to Section II-4.3.7.5 (101), adequate fatigue resistance may be assumed if the stress range under the frequent action combination does not exceed 70 MN/m<sup>2</sup> for unwelded reinforcing bars.

**Check with damage equivalent stress ranges**

The check is considered completed if the following applies for reinforcing steel and prestressing steel:

$$\gamma_{F,fat} \cdot \gamma_{Ed,fat} \cdot \Delta\sigma_{s,equ} \leq \Delta\sigma_{Rsk}(N^*) / \gamma_{s,fat} \quad (4.191)$$

with

$$\gamma_{F,fat} = \gamma_{Ed,fat} = 1$$

$\gamma_{s,fat}$  = 1.15 for reinforcing steel and prestressing steel with new certification.

= 1.25 for prestressing steel with DIN 4227-1 certification according to ARS 11/03 (13).

$\Delta\sigma_{Rsk}(N^*)$  Permissible characteristic stress range at  $N^*$  load cycles based on the Wöhler curves according to II-4.3.7.7 for prestressing steel or II-4.3.7.8 for reinforcing steel.

$\Delta\sigma_{s,equ}$  Damage equivalent stress range with  $\Delta\sigma_{s,equ} = \lambda_s \cdot \Delta\sigma_s$  according to Eq. (A.106.1) or Eq. (A.106.6).

$\lambda_s$  Correction coefficient according to Annex II-A.106.2 or II-A.106.3.

$\Delta\sigma_s$  Calculated stress range of the cyclic fatigue action  $Q_{fat}$  for longitudinal reinforcement from bending and longitudinal force including increase factor  $\eta$  as per 4.3.7.3 (3)\*P Eq. (4.193) to take into account the varying bond behavior of reinforcing and prestressing steel.

The values for  $\Delta\sigma_{Rsk}(N^*)$ ,  $\eta$  and  $\lambda_s$  are specified by the user in the Section dialog.

**Calculation method**

The maximum from the robustness, crack and bending reinforcement is assumed as the existing bending reinforcement. If as a result the load from the fatigue combination in state II cannot be absorbed, the design will be repeated using the existing reinforcement and the check internal forces.

The maximum stress range per steel layer that results from the strain state in state II or the truss model is determined separately for each check situation. Multiplying the coefficients  $\eta$  and  $\lambda_s$  yields the damage equivalent stress range  $\Delta\sigma_{s,equ}$ . If for longitudinal and shear reinforcement this range exceeds the permitted stress range, the necessary reinforcement will be iteratively increased until the check succeeds for all situations. In the design modes *Symmetrical* and *Compression member*, the longitudinal reinforcement is applied at all predefined locations. This will not affect the predefined relationships between the individual reinforcement layers.

The decisive reinforcement used for the check, which may have been increased, is recorded in the check log and saved for graphical representation.

**Fatigue of Concrete Under Compressive Stress**

The fatigue check for concrete subject to compressive stress is performed for bending and longitudinal force at the cracked section. This check takes into account the final longitudinal reinforcement and may include an increase applied during the fatigue check for reinforcing steel. In the case of road bridges, the check specified in II-4.3.7.1 (102), Item (g), is not necessary if the concrete compressive stress as per II-4.4.1.2 (103)P is limited to  $0.6 \cdot f_{ck}$  in the serviceability state. The struts of components subject to lateral force stress are not analyzed.

**Simplified check**

The check according to II-4.3.7.4 (101)P is considered successful if compliance with TR 102 Eq. (4.188) or DIN 1045-1 Eq. (123) is maintained.

$$\frac{\max|\sigma_{cd}|}{f_{cd,fat}} \leq 0.5 + 0.45 \cdot \frac{\min|\sigma_{cd}|}{f_{cd,fat}} \leq 0.9 \quad \text{for concrete up to C50/60 or LC50/55} \quad (4.188)$$

$$\frac{\max|\sigma_{cd}|}{f_{cd,fat}} \leq 0.5 + 0.45 \cdot \frac{\min|\sigma_{cd}|}{f_{cd,fat}} \leq 0.8 \quad \text{for concrete from C55/67 or LC55/60} \quad (123)$$

with

$\max|\sigma_{cd}|$ ,  $\min|\sigma_{cd}|$  Design values of the maximum and minimum concrete compressive stress under the frequent action combination. In the case of tensile stresses,  $\min|\sigma_{cd}|$  is to be assumed as zero.

$f_{cd,fat}$  Design value of the concrete compressive strength before cyclic load is applied. You can specify this value in the Section dialog.

### Check with damage equivalent concrete compressive stresses

The check for railroad bridges according to TR 102, Section A.106.3.2 is proved, if following equation is fulfilled:

$$E_{cd,max,equ} + 0.43\sqrt{1-R_{equ}} \leq 1 \quad (\text{A.106.12})$$

with

$$R_{equ} = \min |\sigma_{cd,equ}| / \max |\sigma_{cd,equ}|$$

$$E_{cd,max,equ} = \gamma_{Ed,fat} \cdot \max |\sigma_{cd,equ}| / f_{cd,fat}$$

In that  $\max |\sigma_{cd,equ}|$  and  $\min |\sigma_{cd,equ}|$  are the upper and lower compressive stress of the damage equivalent stress range for  $N = 10^6$  cycles.

The upper and lower stress of the damage equivalent stress range shall be calculated with the equation (A.106.13).

$$\max |\sigma_{cd,equ}| = |\sigma_{c,perm}| + \lambda_c (\max |\sigma_{c,71}| - |\sigma_{c,perm}|)$$

$$\min |\sigma_{cd,equ}| = |\sigma_{c,perm}| - \lambda_c (|\sigma_{c,perm}| - \min |\sigma_{c,71}|)$$
(A.106.13)

with

$|\sigma_{c,perm}|$  Absolute value of the compressive stress under the fatigue combination without load model 71.

$\max |\sigma_{c,71}|$ ,  $\min |\sigma_{c,71}|$  Absolute minimum and maximum compressive stress under the fatigue combination with load model 71 and the dynamic coefficient  $\Phi_2$  according to DIN TR 101.

$\lambda_c$  Correction coefficient for the calculation of the stresses caused by load model 71.

### Special Characteristic of Shell Structures

In shell structures the strain state at the cracked concrete section under general stress cannot be determined unambiguously. The design is therefore carried out separately for the reinforcement directions x and y with the design internal forces from Wolfensberger/Thürlimann or Rüschi as described above. The reinforcement calculated in this manner yields a reliable load-bearing capacity.

When calculating the stress range for reinforcing steel and concrete, this method can lead to unrealistic results in the case of torsional or shear stresses as shown in the following example:

Assume two identical sets of slab internal forces:

Set	mx [kNm/m]	my [kNm/m]	mxy [kNm/m]
1	300	200	100
2	300	200	100

According to Wolfensberger/Thürlimann, this results in design variants for the x direction:

Set	Variant	m [kNm/m]
1	1	$mx +  mxy  = 400$
	2	$mx -  mxy  = 200$
2	1	$mx +  mxy  = 400$
	2	$mx -  mxy  = 200$

The torsional moments generate a variation of the design moments and thus a calculatory stress range. This may lead to a necessary reinforcement increase in the fatigue check due to apparent overstressing. For design normal forces, this applies for the shear forces in a corresponding manner.

Selecting **Limit design variants** in the Section dialog allows you to avoid the described effect. In this case only the corresponding variants are compared when determining the stress range, i.e. only the first and second variants of both sets in this example. Assuming constant stress, the stress range is thus correctly determined to be zero.

**This alternative, however, does not ensure that all conceivable stress fluctuations are analyzed. You should therefore be particularly careful when assessing the results. For this purpose the detailed log indicates the main variants and design internal forces used for the check.**

When determining the design internal forces according to Rüschi for inclined reinforcement, the described relationships apply accordingly.

If the check is performed with the action combination according to TR 102, Section 4.3.7.5 (102) and Annex 106, the design internal forces are determined from the sum of non-cyclic and cyclic internal forces of the area elements.

## Serviceability Limit State Design

The following checks are performed:

- Limiting the concrete compressive stresses (TR 102, Section 4.4.1.2).
- Limiting the reinforcing steel stresses (Section 4.4.1.3).
- Limiting the prestressing steel stresses (Section 4.4.1.4).
- Check of decompression (Section 4.4.2.1).
- Minimum reinforcement for crack width limitation (Section 4.4.2.2).
- Calculation of crack width (Section 4.4.2.3 and 4.4.2.4).
- Limiting diagonal principal tensile stresses (Section 4.4.0.3, Edition 2009).
- Limiting deformations (Section 4.4.3).

## Design Combinations

According to TR 101, Section 9.5.2, the following combinations are taken into account in the serviceability limit states:

- Combination for characteristic (rare) situations

$$\sum_{j \geq 1} G_{kj} + R_k + Q_{k1} + \sum_{i > 1} \psi_{0i} \cdot Q_{ki} \quad (9.16)$$

- Combination for frequent situations

$$\sum_{j \geq 1} G_{kj} + R_k + \psi_{11} \cdot Q_{k1} + \sum_{i > 1} \psi_{2i} \cdot Q_{ki} \quad (9.17)$$

- Combination for quasi-continuous situations

$$\sum_{j \geq 1} G_{kj} + R_k + \sum_{i \geq 1} \psi_{2i} \cdot Q_{ki} \quad (9.18)$$

- Combination for infrequent situations

$$\sum_{j \geq 1} G_{kj} + R_k + \psi'_{11} \cdot Q_{k1} + \sum_{i > 1} \psi_{1i} \cdot Q_{ki} \quad (9.19)$$

For each combination you can define different design situations for the construction stages and final states. If necessary, the combination required by the check will automatically be determined from the section specifications. Each check is carried out for all the situations of a combination.

## Stress Determination

At the uncracked concrete section, the program assumes an elastic behavior of the concrete and steel under tensile and compressive stress according to TR 102, Section 4.4.1.1 (6). As for cracked concrete sections, the concrete compressive stresses defined by Reg. No. 098 of the NABau Knowledge Base (Building and Civil Engineering Standards Committee) are determined according to Figure 4.1 with  $f_c = f_{cm}$ . Note here that a horizontal curve is assumed for strains of  $\epsilon_{c1}$  or higher.

### Area elements

For area elements the concrete stresses are calculated at the gross section. The steel stress check is carried out for reinforcing steel by determining the strain state at the cracked concrete section and for the prestressing steel at the uncracked concrete section.

### Beams and design objects

The action combination stresses that can be determined without checks are always calculated at the gross section without considering the specified full section.

Conversely, in the checks the stresses are determined as follows and are graphically displayed or logged:

- When checking the crack reinforcement and crack width, the concrete stress is calculated at the gross section. The longitudinal force according to II-4.4.1.1 (109) is based on the area of the full section



- When checking the decompression, the concrete compressive stresses and the diagonal principal tensile stresses, the bending stress is calculated
  - without internal tendons at the gross section
  - with internal tendons without bond at the net section
  - with internal tendons with bond for situations before being grouted at the net section or otherwise at the ideal section
- The longitudinal force according to II-4.4.1.1 (109) is based on the area of the full section
- The reinforcing and prestressing steel stresses are checked by determining the strain state at the cracked concrete section

## Limiting the Concrete Compressive Stresses

The concrete compressive stress check is carried out according to TR 102, Section 4.4.1.2. Based on II-4.4.1.1 (5), the cracked state is assumed if the tensile stress calculated in the uncracked state under the rare action combination exceeds the value  $f_{ctm}$ .

The calculation in the cracked state is performed by determining the strain state with the final longitudinal reinforcement (maximum from robustness, crack and bending reinforcement including a possible increase from the fatigue check). For beams and design objects, the tendons with bond are taken into account on the resistance side provided that they are grouted in the check situation. For area elements, the compressive stress for both reinforcement directions is determined separately and the extreme value is checked since the general strain state cannot be determined unambiguously.

For beams and design objects, the stresses from longitudinal force according to II-4.4.1.1 (109) are calculated with the full section. In all other cases, the rules for stress analysis indicated above apply.

In prestressed concrete components as per II-4.4.1.2 (102)P, the maximum concrete compressive stress must be limited to  $0.60 f_{c(t)}$  when entering the average prestressing value. If the concrete compressive stress exceeds the value  $0.45 f_{c(t)}$ , the nonlinearity of the creep must be taken into account.  $f_{c(t)}$  indicates the average value of the concrete compressive strength at time  $t$  when the prestressing is entered.

The program assumes the time of introducing the prestressing to coincide with situation  $G1+P$ . If a situation  $G1+P$  is defined in the combination selected above, the concrete stress is checked against the limit value  $0.45 f_{c(t)}$  or  $0.60 f_{c(t)}$  for this situation depending on the user's specification. The value for  $f_{c(t)}$  is also defined in the dialog.

In the other construction stages and final states, the concrete compressive stress  $\sigma_c$  must be limited to  $0.60 f_{ck}$  under the rare combination according to II-4.4.1.2 (103)P. The limit may be increased by 10% if the concrete compressive zone is helically reinforced (e.g., by lateral reinforcement). If serviceability is significantly impacted by the effect of creep, the limit  $0.45 f_{ck}$  should also be maintained under the quasi-continuous combination as per II-4.4.1.2 (104)\*P. Both options are considered based on the user's specifications.

## Limiting the Reinforcing and Prestressing Steel Stresses

### Reinforcing steel

For reinforcing steel, the limitation of steel stress under the infrequent combination to  $0.80 f_{yk}$  is checked in accordance with II-4.4.1.3. In this check the reinforcement corresponds to the maximum value from the robustness, crack and bending reinforcement, including a possible increase as a result of the fatigue check. The determination of the strain state is performed at the cracked concrete section. Beam tendons with bond are taken into account on the resistance side provided they are grouted.

### Prestressing steel

For tendons with bond, the limitation of steel stress under the quasi-continuous combination is checked at the cracked concrete section for beams and design objects and at the uncracked concrete section for area elements. In either case the following limits apply:

- $0.65 f_{pk}$  as per II-4.4.1.4 for tendons with DIN 1045-1 and EC2 certification
- $0.75 \beta_s$  or  $0.55 \beta_z$  as per DIN 4227-1, Tab. 9, Row 65, for tendons with DIN 4227 certification

For situations prior to stressing and for tendons without bond, the stress  $\sigma_{pm0}$  is checked in accordance with TR 102, Eq. (4.6) or DIN 4227-1, Tab. 9, Row 65. External tendons are not checked.

## Check of Decompression

This check is carried out for prestressed components of requirement classes A-C at the uncracked concrete section with the combinations found in TR 102, Table 4.118.

For prestressed concrete bridges with box section in requirement class *D*, a decompression check must be carried out as per ARS 11/2003 (4) and ARS 6/2009 (4) for a special quasi-continuous action combination that you can include in the program by defining a corresponding situation.

For beams and design objects, the stresses from longitudinal force according to II-4.4.1.1 (109) are calculated with the full section. For area elements, the principal tensile stress  $\sigma_1$  or one of the longitudinal tensile stresses  $\sigma_x$  or  $\sigma_y$  is checked depending on the user's selection. The latter can be used to limit the check to the direction of the prestressing if the internal force systems are appropriately aligned (cf. Reg. No. 069 in the Knowledge Base of the Building and Civil Engineering Standards Committee, NABau). In all other cases, the rules for stress analysis indicated above apply.

The stress limits must be specified separately for different construction stages and structure types.

### Construction stage according to II-4.4.0.3 (102)P and II-4.4.2.1 (107)P

#### Edition 2003:

- Class A-C buildings: The tensile stresses  $\sigma_c \leq 0.85 f_{ctk;0.05}$  are permissible at the upper and lower section edges (cf. Ril 804.4201 (15) and Reg. No. 006 in the Knowledge Base of the Building and Civil Engineering Standards Committee, NABau).
- Class D box bridges: The tensile stresses  $\sigma_c \leq 0.85 f_{ctk;0.95}$  are permissible at the upper and lower section edges (cf. ARS 11/2003 (4)).

#### Edition 2009:

- For exclusively bondless prestressing, the tensile stresses on the edge closest to the tendon must be limited to the value found in Table 4.118a or to  $0.85 f_{ctk;0.05}$  in all other cases.
- Class D box bridges: The tensile stresses  $\sigma_c \leq 0.85 f_{ctk;0.95}$  are permissible at the upper and lower section edges (cf. ARS 6/2009 (4)).

### Final state according to II-4.4.2.1 (106)P

Tensile stresses in class A-C buildings and class D box bridges are not permitted to occur at the edge that is directly adjacent to the tendon. The program determines the above section edge as follows:

- Beams and design objects: If the edge point next to the tendon is above the centroid, the stress on the upper side of the section will be checked. If not, the lower side of the section will be checked.
- Area elements: The check will be carried out for the upper or lower section edge if the tendon next to the check point is located above or below the centroid level of the element in question. Tendons outside of the element are taken into account at a distance of up to five times the section height.

If the tendon guide is ambiguous, the check will be carried out for both sides.

## Concrete Tensile Stresses in Bridge Transverse Direction

If prestressing occurs in the bridge longitudinal direction and the bridge lateral direction is designed as a non-prestressed construction, a check must be carried out as per TR 102, Section 4.4.0.3 (4)P, to ensure that the concrete tensile stresses in the bridge lateral direction determined in state I under the rare action combination do not exceed the values found in Table 4.118a.

If necessary, the check can be carried out in the graphical user interface by controlling the edge stresses from the rare combination.

## Minimum Reinforcement for Crack Width Limitation

The minimum reinforcement for crack width limitation is defined in TR 102, Section 4.4.2.2. According to 4.4.2.2 (5)\*, the minimum reinforcement is to be applied in areas where tensile forces are expected. Areas under tension can be defined in the section dialog by choosing either an action combination or a restraint (bending, central tension). Reinforcing steel layers that are not under tension are also provided with reinforcement in the *symmetrical* and *compression member* design modes. This will not affect the predefined relationships between the individual reinforcement layers.

For profiled sections, each subsection (web or flange) should be checked individually in accordance with Section 4.4.2.2 (4) \*P. This cannot be done if any polygonal section geometries are taken into consideration. For this reason, the program always determines the minimum reinforcement based on the entire section. For rectangular full sections, Equation (4.195) is used. In all other cases, Equation (4.195a) applies.

**Determining the minimum reinforcement**

The minimum reinforcement  $A_s$  is determined based on Equation (4.194) of TR 102:

$$A_s = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} / \sigma_s \quad (4.194)$$

where

$k_c$  is the coefficient for consideration of stress distribution prior to crack formation.

For rectangular sections and webs of T-beams and box girders:

$$k_c = 0.4 (1 + \sigma_c / (k_1 \cdot f_{ct,eff})) \leq 1 \quad (4.195)$$

For tension flanges of T-beams and box girders:

$$k_c = 0.9 \cdot F_{cr} / A_{ct} / f_{ct,eff} \geq 0.5 \text{ (cf. Zilch/Rogge 2002, p. 277)} \quad (4.195a)$$

with tensile force  $F_{cr}$  in the tension chord in state I immediately before crack formation with edge stress  $f_{ct,eff}$ . The tensile force is calculated by integrating the tensile stresses over the area  $A_{ct}$ .

$\sigma_c$  is the concrete stress at the level of the centroidal axis of the section or subsection, which, in an uncracked state, is subject to the action combination on the entire section that leads to the initial crack formation. ( $\sigma_c < 0$  for compressive stress).

$k_1 = 1.5 h/h'$  for compressive normal force,  
 $= 2/3$  for tensile normal force.

$h$  is the height of the section or subsection.

$h' = \min(h; 1 \text{ m})$ .

$k$  is the coefficient for taking into account nonlinearly distributed tensile stresses entered by the user.

$A_{ct}$  is the area of the concrete tensile zone during initial crack formation in state I. To determine the value, the program scales the bending moment of the action combination until the maximum edge stress in state I corresponds to the value  $f_{ct,eff}$ .

$f_{ct,eff}$  is the effective concrete tensile strength in relation to age of concrete as per II-4.4.2.2 (5)\*:

$f_{ct,eff} = 0.5 f_{ctm}$  at an age of 3-5 days,

$f_{ct,eff} = f_{ctm}$  at an age of 6-28 days,

$f_{ct,eff} = f_{ctm}$  but not less than 3 MN/m<sup>2</sup>, if older than 28 days.

$\sigma_s$  is the maximum permitted stress in the reinforcing steel reinforcement in relation to the limit diameter of the reinforcing steel.

The largest existing bar diameter  $d_s$  is specified in the section dialog. Equation (4.196) provides a modified limiting diameter  $d_s^*$  to be used as an input value for Table II-4.120:

$$d_s = d_s^* \cdot k_c \cdot k \cdot h_t / (4(h-d)) \cdot f_{ct,eff} / f_{ct0} \geq d_s^* \cdot f_{ct,eff} / f_{ct0} \quad (4.196)$$

where

$d_s^*$  is the limit diameter of the reinforcement according to Table II-4.120.

$h$  is the component height.

$d$  is the effective static height.

$h_t$  is the height of the tensile zone in the section or subsection before initial crack formation.

$f_{ct0}$  is the tensile strength of the concrete from which the values in Table II-4.120 are derived ( $f_{ct0} = 3.0 \text{ MN/m}^2$ ).

According to Zilch/Rogge (2002, p. 277), the expression  $k_c \cdot k \cdot h_t / (4(h-d))$  is generalized to  $0,6 \cdot k_c \cdot k \cdot A_{ct} / A_{c,eff}$  with effective tensile zone  $A_{c,eff}$  as shown in Fig. II-4.137. The modified limit diameter  $d_s^*$  and the permissible crack width  $w_k$  are used based on Table II-4.120 to determine the permissible reinforcing steel stress  $\sigma_s$  for Eq. (4.194).

If the crack width check is to be carried out at the same time, the program will determine whether the specified crack width according to Section II-4.4.2.4 is maintained by inserting the calculated minimum reinforcement. If necessary, the minimum reinforcement is increased iteratively until the check limit is reached. The increased reinforcement is indicated by an exclamation mark "!" in the log.

The guideline defined in ARS 11/2003 (14) or Section II-4.4.2.2 (106) of Edition 2009, which specifies that minimum crack

reinforcement must always be applied for construction joints, can be taken into account by manually specifying a base reinforcement. The same applies for Section (107).

Edition 2009: Based on Section II-4.4.2.2 (9), the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined to Equation (4.197a), but the value may not fall below the value in Equation (4.197b). It is not necessary to insert more reinforcing steel as results from Equation (4.194). The rules specified before will be used, if the option is selected by the user, whereas the possibility of lower reinforcement for slowly hardening concrete according to Section (10) will not be used.

### Special characteristic of prestressed concrete structures

According to Chapter II-4.4.2.2 (7)\*, for a 300 mm square section around a tendon with immediate or subsequent bond, the minimum reinforcement required for this region may be reduced by  $\xi_1 \cdot A_p$ .

Where

$A_p$  is the section area of the prestressing steel in the tendon.

$\xi_1$  is the ratio of the prestressing and reinforcing steel bond strengths.

For beams and design objects, the tendons with bond can be added using the  $\xi_1$  value specified in the section dialog as long as they are grouted in the check situation. Note that prestressed steel cannot be taken into account for area elements.

According to Paragraph (3)\* of Section II-4.4.2.2, the minimum reinforcement for prestressed components with bond is not necessary in areas in which compressive concrete stresses larger than 1 MN/m<sup>2</sup> occur at the section edge under the rare (characteristic) action combination and the characteristic prestress values. This condition is automatically checked by the program.

To delimit the areas where no minimum reinforcement is required, the concrete compressive stresses in state I are calculated at the gross cross-section with the mean characteristic prestress. The affected structural areas can be evaluated in the graphical stress representation for the characteristic combination.

## Calculation of the Crack Width

The crack width check is performed by means of direct calculation as per TR 102, Section 4.4.2.4, with the action combination that is based on the requirement class specified in Tab. II-4.118. Enter the limit diameter and the age of the concrete in the Section dialog to determine the effective tensile strength.

The stresses from longitudinal force according to II-4.4.1.1 (109) are calculated with the full section.

Depending on concrete edge stress  $\sigma_c$  in state I, the following crack states must be differentiated (cf. Book 525, p. 191 and Guide to DIN TR 102, p. 24):

$\sigma_c \leq f_{ct,eff}$  Stage of single crack formation

$\sigma_c > f_{ct,eff}$  Stage of completed crack formation

with

$f_{ct,eff}$  Effective concrete tensile strength in relation to age of concrete as per II-4.4.2.2 (5)\*.

Edition 2009: In this case a minimum concrete tensile strength is not included.

By limiting the maximum crack distance and the difference of the strains, the formulas in TR 102, Section 4.4.2.4, of the standard as specified in Book 525, p. 104, can be used for both the single crack formation and the completed crack formation stages. This is why the program checks the crack width for all cases where  $\sigma_c > 0$ .

The program performs the check according to the following steps:

- Determine strain state II under the check combination defined by the requirement class with the stress-strain curve according to TR 102, Fig. 4.1. For beams and design objects, all tendons in a bond are considered on the resistance side.
- Define effective tensile zone  $A_{c,eff}$  (see next section), determine reinforcing steel and prestressing steel layers within  $A_{c,eff}$ .

- Calculate reinforcement level:

$$eff \rho = (A_s + \xi_1^2 \cdot A_p) / A_{c,eff} \quad (4.200a)$$

$$\rho_{tot} = (A_s + A_p) / A_{c,eff} \quad (4.200b)$$

with

$\xi_1$  Bond coefficient according to user specification.

- Determine individually for each reinforcing steel layer:

#### Difference of the average strain for concrete and reinforcing steel

$$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - 0.4 f_{ct,eff} / eff \rho (1 + \alpha_E \cdot eff \rho)] / E_s \geq 0.6 \sigma_s / E_s \quad (4.202)$$

with

$$\alpha_E = E_s / E_{cm}$$

$$\sigma_s = \sigma_2 + 0.4 f_{ct,eff} (1/eff \rho - 1/\rho_{tot}) \quad (4.199)$$

$\sigma_s$  is limited to  $f_{yk}$  in the program

$\sigma_2$  Reinforcing steel stress from strain state II .

$f_{ct,eff}$  Effective concrete tensile strength at specified age of concrete.

#### Maximum crack spacing

$$s_{r,max} = d_s / (3.6 eff \rho) \leq \sigma_s \cdot d_s / (3.6 f_{ct,eff}) \quad (4.203)$$

If an upper limit for the crack distance based on Equation (4.203) was specified in the section dialog, then the special conditions of Equation (4.204) can be taken into account.

#### Calculated crack width

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (4.201)$$

The layer with the largest calculated crack width is shown in the log.

- For sections under tension, the check is performed separately for each of the two effective tensile zones. The maximum value is shown in the log.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the crack width. For that purpose a design is carried out using the decisive check combination for calculating the crack width. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The crack width is checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check) and saved for graphical representation together with the decisive reinforcing steel stress.

The rules according to ARS 11/2003 (14) resp. II-4.4.2.2 (106), (107) and II-4.4.2.3 (5)\*P for construction joints are not included.

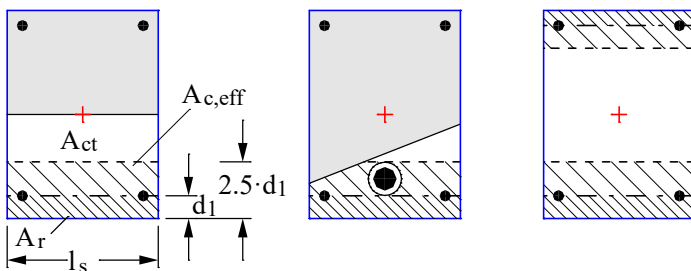
## Determining the Effective Area $A_{c,eff}$

According to DIN TR 102, Figure 4.137, the effective tensile zone  $A_{c,eff}$  defines the area of a rectangular, uniaxially stressed concrete section in which the model assumptions specified in Book 466 are applicable (cf. Book 525, Explanation of DIN 1045-1, Figure 53). Although the program can apply this model to any section and stress situation, the user has the responsibility and discretion to do so.

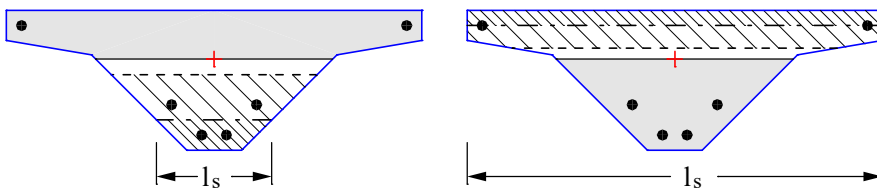
When determining  $A_{c,eff}$ , the following steps are performed by the program:

- Determine tensile zone  $A_{ct}$  in state I: when calculating the minimum reinforcement, the stress that led to the initial crack is used; when calculating the crack width, the check combination based on the requirement class is used. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Define the centroid line of the reinforcement as a regression line through the reinforcing steel layers in the tensile zone. For area elements, a horizontal line through the centroid of the reinforcement layers under tension is assumed.
- Determine the truncated residual area  $A_r$  to the edge and the sum of section lengths  $l_s$ . The average edge distance is then assumed as  $d_1 = A_r / l_s$ , but not less than the smallest edge distance of the reinforcing steel layers in the tensile zone.
- Shift the centroid line in parallel by  $1.5 \cdot d_1$ . For area elements,  $2.5 \cdot d_1 \leq (h-x) / 2$  is maintained ( $x$  = compression zone height in state I).
- The resulting polygon is intersected with the tensile zone and then defines the effective tensile zone  $A_{c,eff}$ .
- If all the reinforcing steel layers of the section are under tension, then two zones will be determined; one for the layers above the centroid and the other for layers below the centroid. The area of each zone is limited to  $A_c / 2$ .
- Edition 2009: If the minimum reinforcement for thicker components under central restraint is selected in the section dialog, the height of  $A_{c,eff}$  is  $h_{eff} \geq 2.5 d_1$  according to Figure 4.137 d).

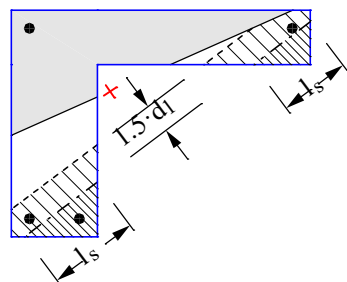
The following illustrations show the effective tensile zones determined by the program in typical situations. The last case (edge beam) deviates from the model assumptions in Book 466 to such a degree that it is questionable as to whether it should be used.



Effective tensile zones at a rectangular section under uniaxial bending, normal force with double bending and central tension



Effective tensile zones at a bridge section under uniaxial bending



Effective tensile zone at an edge beam under uniaxial bending

## Crack Width Check by Limitation of the Bar Distances

As an alternative to the direct crack width calculation described in Section 4.4.2.4, you can choose the basic check according to TR 102, Section 4.4.2.3 (2)\* through limitation of the bar distances as shown in Table 4.121 in the selection dialog.

The program performs the check according to the following steps:

- Determine strain state II under the check combination defined by the requirement class with the stress-strain curve according to TR 102, Fig. 4.1. For beams and design objects, all tendons in a bond are considered on the resistance side.
- Determine the reinforcing steel stress  $\sigma_s$  for each reinforcement layer based on Equation (4.199).
- Compare the value entered in the dialog (**max. s**) with the table value (**perm. s**), which is derived from the calculated steel stress  $\sigma_s$  and the permissible crack width  $w_k$ . In the log, the location with the largest quotient (**max. s / perm. s**) is checked.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the permissible bar distances. For this purpose, a design is carried out with the decisive action combination for the check. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The bar distances are then checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check).

### Note

According to Section 4.4.2.3 (2)\*, the simplified check can only be used in the event of crack formation resulting from mostly direct actions (restraint). According to *Zilch and Rogge (2002, p. 277)*, this method only yields definitive results for single-layer tensile reinforcement with  $d_1 = 4$  cm. The user is responsible for the evaluation of these requirements.

## Limiting Diagonal Principal Tensile Stresses

For prestressed road bridges with thin flanges, the formation of shear cracks must be limited in accordance with II-4.4.0.3 (5)P, Edition 2009, or ARS 11/2003 (16). It is to be ensured that the diagonal principal tensile stresses affected by lateral force and torsion do not exceed the values  $f_{ctk;0.05}$ . The check is to be carried out in state I for the frequent combination.

For prestressed railway bridges, the principal tensile stresses as per II-4.4.03 (6)P, Edition 2009, or Ril 804.4201 (18), must be limited as well. Note, however, that you can limit the check to the area of longitudinal compressive stresses as long as no tensile-stressed chords are connected. You can choose this option in the section dialog. The user is responsible for verifying the usage requirements.

### Beams and design objects

The program analyzes the section to determine all shear parameters from lateral force and torsion for the section edge. Calculation points are all points of the cross-section polygon, the edge centers and a variable count of additional edge points depending on the edge length. The shear stresses from lateral force are determined on the basis of either the theory of thick-walled profiles or an average across the section width, depending on which option is selected in the Section dialog. The diagonal principal tensile stress can thus be calculated for each section point:

$$\sigma_1 = \frac{\sigma_x}{2} + \frac{1}{2} \sqrt{\sigma_x^2 + 4\tau_{xy}^2 + 4\tau_{xz}^2}$$

with

$\sigma_x$  Longitudinal stress from bending and longitudinal force in accordance with the rules for stress analysis indicated above.

$\tau_{xy}$  Edge shear stress in the y direction from lateral force and torsion.

$\tau_{xz}$  Edge shear stress in the z direction from lateral force and torsion.

The angle between  $\sigma_1$  and  $\sigma_x$  is derived with:

$$\tan 2\vartheta_1 = \frac{-2\tau}{\sigma_x}$$

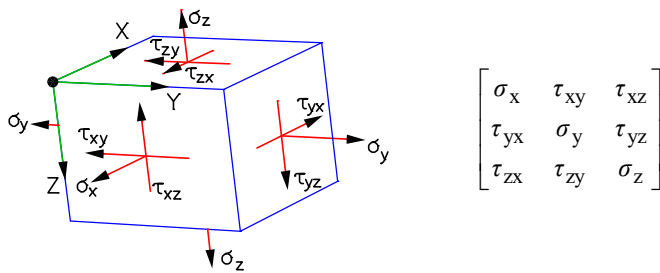
with

$$\tau = \sqrt{\tau_{xy}^2 + \tau_{xz}^2} \text{ as the resulting edge shear stress.}$$

The maximum value for  $\sigma_1$  is listed in the log together with the relevant section point.

**Area elements**

For area elements the check is carried out for principal tensile stresses at the gross section. The 3D stress state is described by the following stress tensor:



with

$\sigma_x, \sigma_y$  Longitudinal stress from bending and normal force.

$\sigma_z = 0$ .

$\tau_{xy} = \tau_{yx}$  Shear stress from torsional moment and shear force with a linear curve along the section height.

$\tau_{xz} = \tau_{zx}$  Shear stress from lateral force  $q_x$  with a parabolic curve along the section height and the maximum value  $1.5 \cdot q_x / h$  in the centroid level.

$\tau_{yz} = \tau_{zy}$  Shear stress from lateral force  $q_y$  with a parabolic curve along the section height and the maximum value  $1.5 \cdot q_y / h$  in the centroid level.

The maximum principal stress  $\sigma_1$  with its height level  $z$  in relation to the upper section edge is calculated by determining the stress tensor for each layer and solving the eigenvalue problem for the three principal stresses. These are shown together with the associated stress components in the results log.



## Limiting Deformations

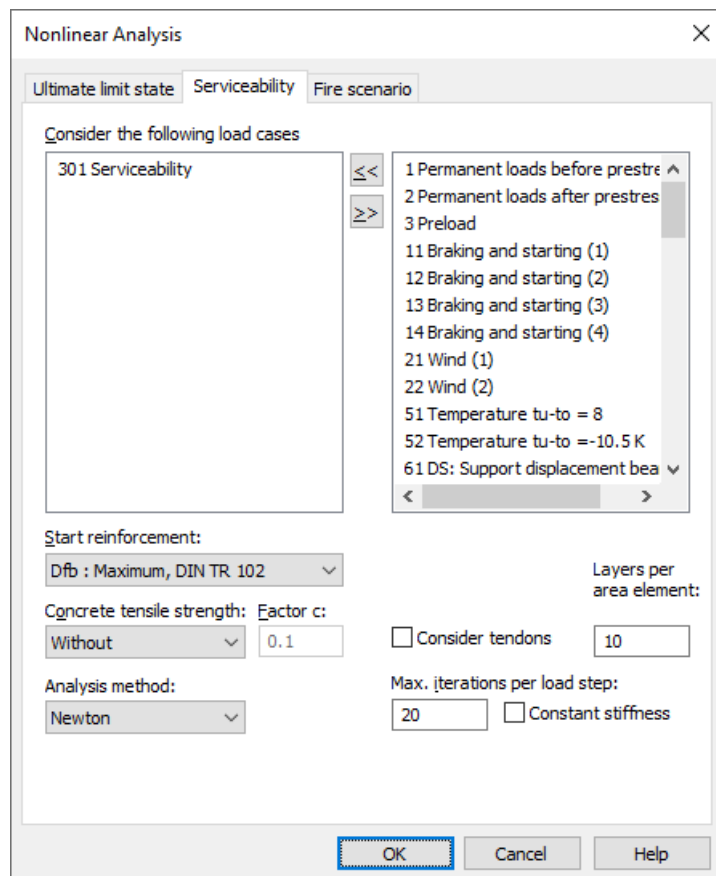
According to TR 102, Section 4.4.3.1 (1)\*P, the deformations of a component or structure may not impair its proper functioning or appearance. The deformation limit values for railroad bridges are specified in TR 101, Section G3. With respect to other bridges, the appropriate limit values must be agreed to with the builder.

Bending is to be defined under the quasi-continuous action combination based on II-4.3.3.1 (106). The applied calculation methods should correspond to the 'actual relationships and boundary conditions of the structure' according to II-4.4.3.2 (103)P. Book 525, Section 11.3.2, makes references to various bibliographic sources.

The InfoCAD program system allows you to perform a realistic check as part of a nonlinear system analysis for beam and shell structures that takes geometric and physical nonlinearities into account. The resistance of the tendons with bond is currently not included in the calculation.

Editing is performed in the following steps:

- Define the check situation with the *Load group* function in the Load dialog by grouping the decisive individual load cases. The variable loads must first be weighted with the combination coefficients  $\psi_2$  for the quasi-continuous combination.
- Select the check load cases in the *Nonlinear Analysis / Serviceability* dialog of the analysis settings for the FEM analysis.
- Set the reinforcement determined in the ultimate limit state in the *Start reinforcement* selection field (maximum from bending, robustness, crack check and fatigue).
- Perform the FEM analysis to determine the deformations in state II.
- Check the system deformations displayed graphically or in tabular form.



For a detailed description of the nonlinear system analysis, refer to the relevant chapter of the manual.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The resulting bending, robustness and crack reinforcement, the decisive maximum value and the stirrup and torsion reinforcement are provided for the graphical representation as well.

The log shows the design internal forces and necessary reinforcements, checked stresses or crack widths at each result location. If the permissible limit values are exceeded, they are reported as warnings and indicated at the check location. The detailed log also lists the decisive combination internal forces of all design situations.

### Stresses for beams and design objects

$\sigma_x$	Longitudinal stresses in the decompression and concrete compressive stress checks [MN/m <sup>2</sup> ].
$\sigma_1, \sigma_x, \tau_{\max}$	Diagonal principal tensile stresses and related longitudinal and shear stresses [MN/m <sup>2</sup> ].
$\sigma_s, \Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd}, \Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete under longitudinal compression [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,y}, \Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from $Q_y$ and $Q_z$ [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,T}, \Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and longitudinal torsion reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{\text{perm}}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{\text{perm}}$	Stress range utilization.

### Stresses for area elements

$\sigma_x, \sigma_y, \sigma_1$	Longitudinal stress in x or y direction or principal tensile stresses in the decompression check (depending on user specification) [MN/m <sup>2</sup> ].
$\sigma_2$	Principal compressive stresses [MN/m <sup>2</sup> ].
$\sigma_1, \sigma_x, \sigma_y$	Diagonal principal tensile stresses and related longitudinal and shear
$\tau_{xy}, \tau_{xz}, \tau_{yz}$	stresses [MN/m <sup>2</sup> ].
$\sigma_{sx}, \Delta\sigma_{sx}$	Stresses and stress ranges for reinforcing steel in the x direction [MN/m <sup>2</sup> ].
$\sigma_{sy}, \Delta\sigma_{sy}$	Stresses and stress ranges for reinforcing steel in the y direction [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd,x}, \Delta\sigma_{cd,x}$	Stresses and stress ranges in the concrete fatigue check under longitudinal compression in the x-
$\sigma_{cd,y}, \Delta\sigma_{cd,y}$	and y-direction [MN/m <sup>2</sup> ].
$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{\text{perm}}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{\text{perm}}$	Stress range utilization.

### Bending reinforcement

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams and design objects.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.

### Reinforcement from lateral force

$a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area elements from $q_x$ .
$a_{sbx}, a_{sby}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area elements from $q_x$ and $q_y$ .
$A_{sb,y}, A_{sb,z}$	Stirrup reinforcement of beams and design objects [cm <sup>2</sup> /m] from $Q_y$ and $Q_z$ .
$A_{sl}$ for $a_{sb}=0$	Longitudinal reinforcement [cm <sup>2</sup> ] of area elements.
$z_y, z_z$	Inner lever arm [m] for lateral force $Q_y$ and $Q_z$ .

$\Delta F_{sd,y}, \Delta F_{sd,z}$  The amount of tensile force [kN] in the longitudinal reinforcement as a result of lateral force  $Q_y$  and  $Q_z$  according to Fig. 4.13.

### Torsion reinforcement

$A_{sb,T}$  Torsional stirrup reinforcement of beams and design objects [cm<sup>2</sup>/m] from  $M_x$ .

$A_{sl,T}$  Torsional longitudinal reinforcement of beams and design objects [cm<sup>2</sup>] from  $M_x$ .

### Design values

$V_{Rd,ct}, v_{Rd,ct}$  Absorbable design lateral force without shear reinforcement [kN or kN/m].

$v_{Rd,max}$  Absorbable design lateral force of concrete struts for areas [kN/m].

$V_{Rd,max}$  Absorbable design lateral force of concrete struts for beams and design objects [kN].

### Crack width

$w_{k,top}, w_{k,bottom}$  Computed crack width at the top and bottom of the cross-section separately for the x and y reinforcement direction for area elements.

$w_k / w_{per}$  Crack width utilization.

# Examples

## Road Bridge in Solid Construction

In this example the features and capabilities of the *DIN Technical Report Bridge Checks* program module are demonstrated based on a simple bridge slab.

It was chosen inspired by part 1 of the book

Bauer, Thomas / Müller, Michael, Straßenbrücken in Massivbauweise nach DIN-Fachbericht (Road Bridges in Solid Construction According to DIN Technical Report)

**Beispiele prüffähiger Standsicherheitsnachweise (Examples of Verifiable Stability Safety Checks).**

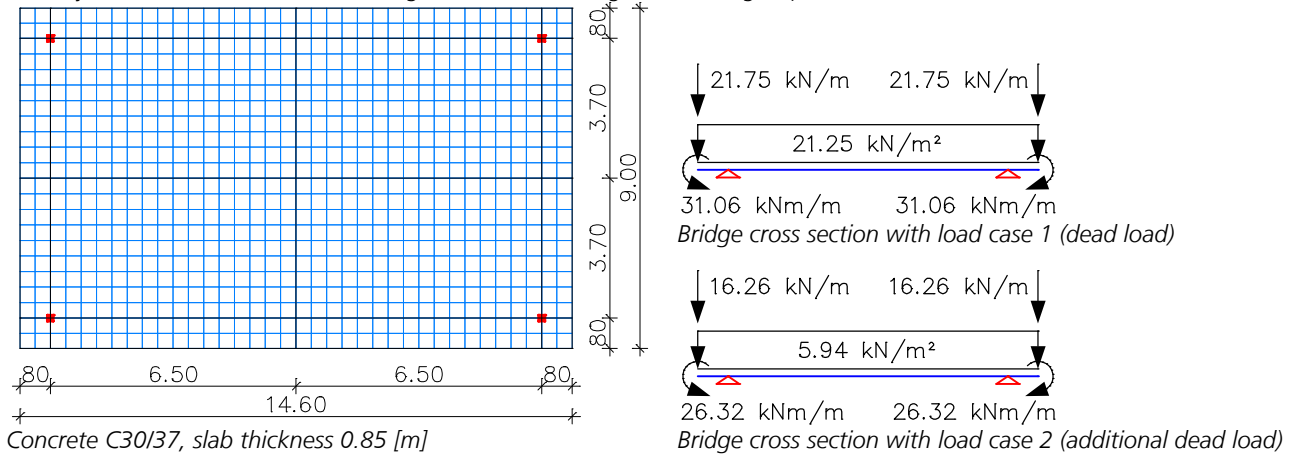
Stahlbeton- und Spannbetonüberbau nach DIN-Fachbericht 101 und 102 (Reinforced Concrete and Prestressed Concrete Superstructures According to DIN Technical Reports 101 and 102).  
2nd Revised Edition. Bauwerk Verlag GmbH, Berlin 2003.

The DIN Technical Reports are divided into the following sections in the program:

- Section-dependent check specifications
- Selection of the structure type
- Load model 1
- Combination of actions
- Performing checks

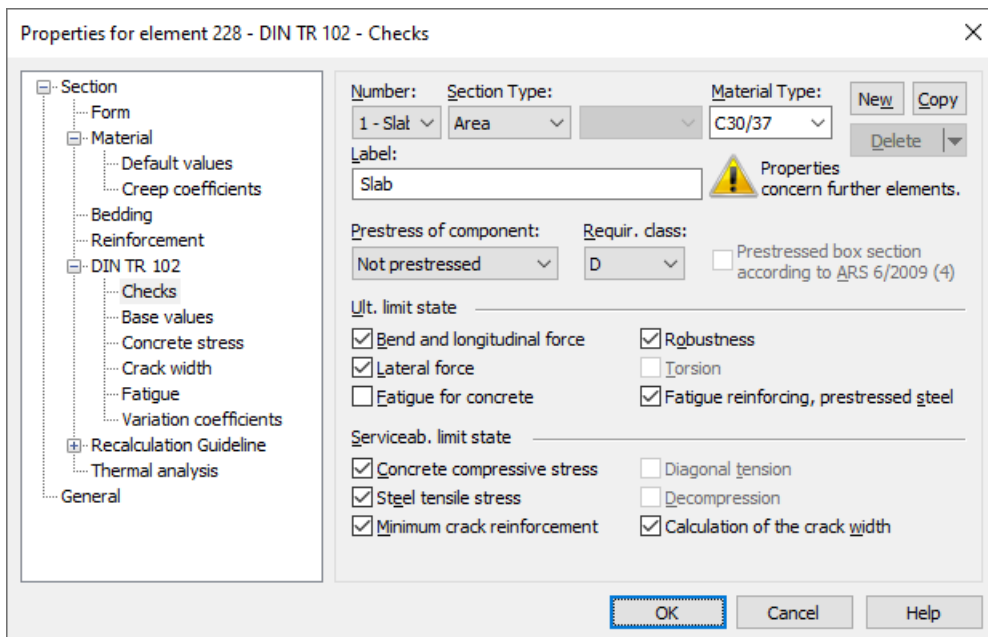
### The Task

The objective is to calculate a road bridge that is to be designed as a single-span, reinforced concrete slab.

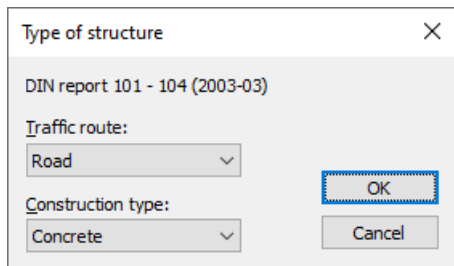


The depicted FEM system is generated using slab elements. The supports are to be entered without restraint. The load cases for dead load, additional loads and all loads that do not belong to load model 1 according to DIN Technical Report 101 are specified as usual.

After that complete the section-dependent check specifications:



## Type of structure



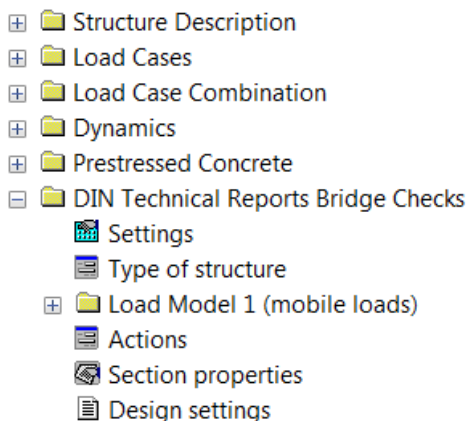
Open the *DIN Technical Report* folder in the database and select the traffic route and the construction type under *Type of structure*.

## Load model 1

Load model 1 consists of two parts:

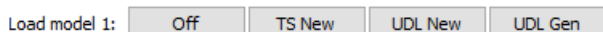
- (a) Twin axle (TS tandem system)
- (b) Uniformly distributed area loads UDL

These loads should be applied in both the longitudinal and lateral directions of the bridge in the least favorable position. In the lateral direction, the load positions are determined by dividing the roadway into computational lanes. Since the decisive lane division is not always known in advance, you can define different load position variants.

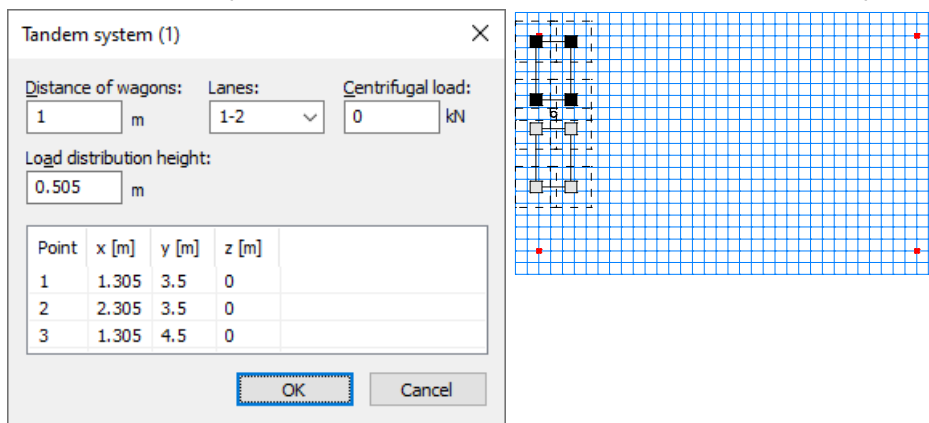


In the *DIN Technical Report* folder right-click *Load model 1* and then select *New* from the context menu.

This creates the 1st variant of the load model and opens an input dialog.

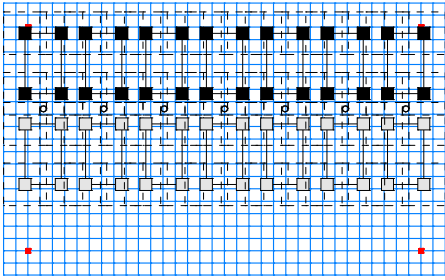


Click *TS New* to specify the centroid and the direction of traffic for the 1st tandem system.



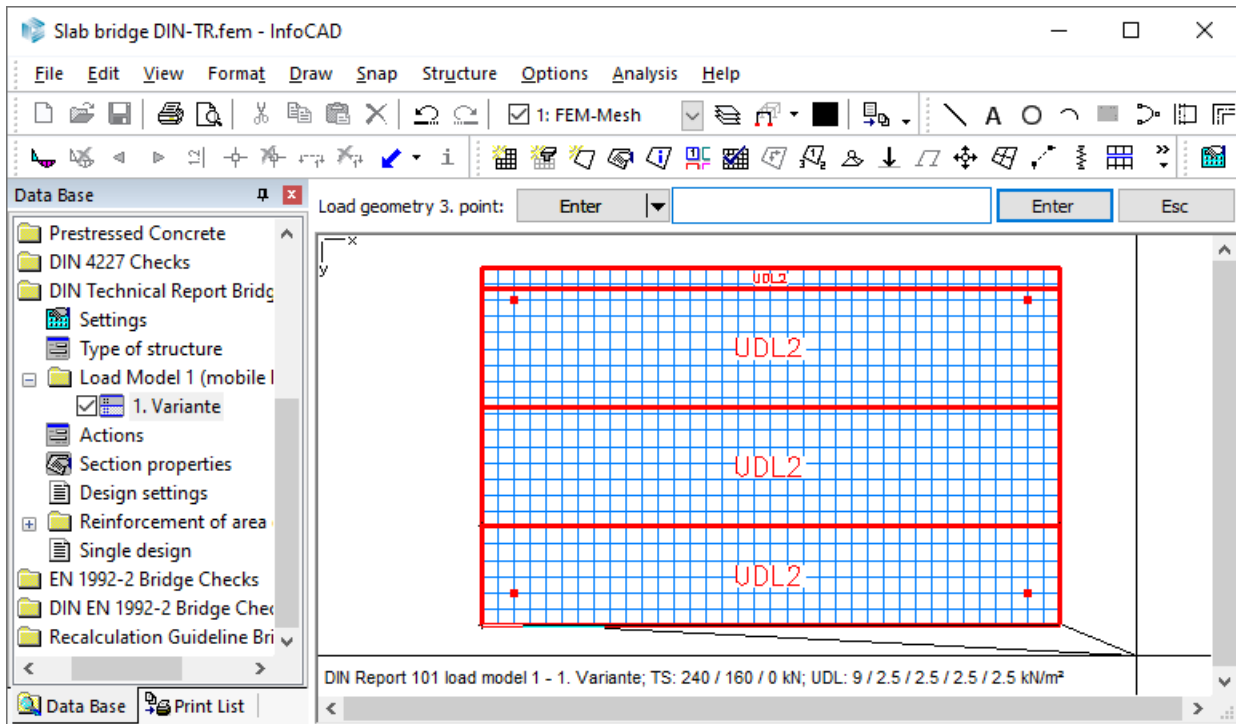
The properties of the tandem system position are set in the dialog.

The Lane 1 is defined left of the traffic direction and indicated by a dark hatching pattern. The area resulting from the load distribution height is shown in dashes.

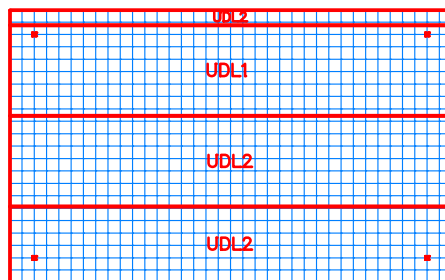
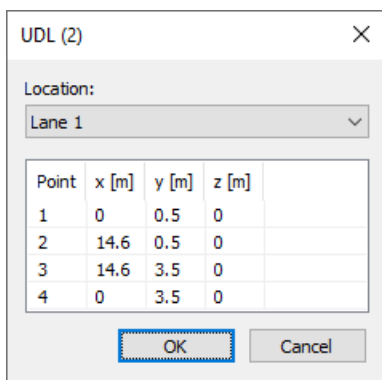


Once the tandem system is selected, you can either copy or generate the desired load positions. The usual snap functions such as *Mid-point* or *Endpoint* can be used for this purpose.

Now click *UDL New* to define the load areas of the UDL system in consecutive order.



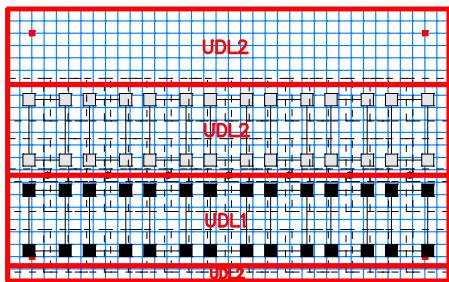
Initially, all the load areas are accepted as UDL2 (lanes 2). Double-click the second UDL area and specify *Lane 1* for this.



Completed UDL areas (TS hidden)

This done, the 1st variant of the load model has been defined. To enter the 2nd variant, right-click *Load model 1* in the database again and select *New* from the context menu.

All of the TS and UDL positions can be copied to the 2nd variant via the clipboard and then modified (in this case: rotated by 180°).

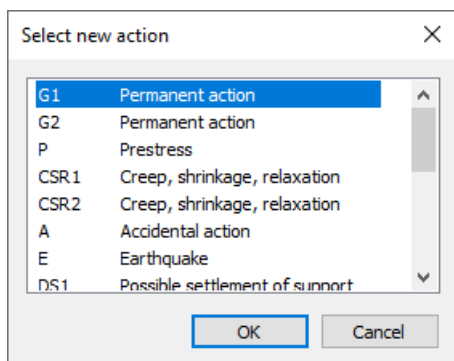


Load model 1 is now fully described and can be calculated. This is done in conjunction with the load cases as part of the FEM calculation. The results of all load positions are individually saved and grouped in the

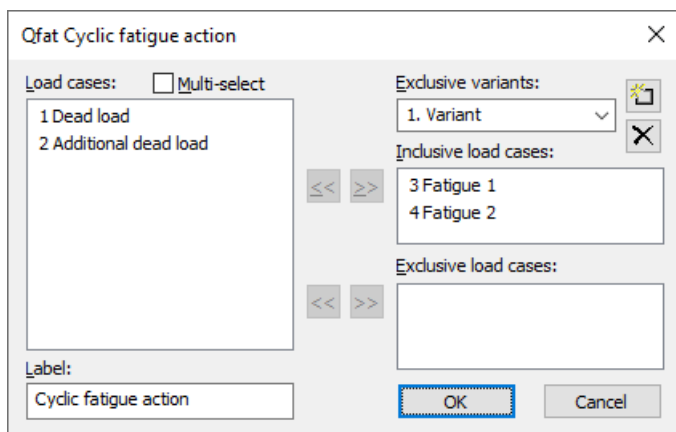
 *Load model 1 (Mobile loads)* folder.

### Combination of actions

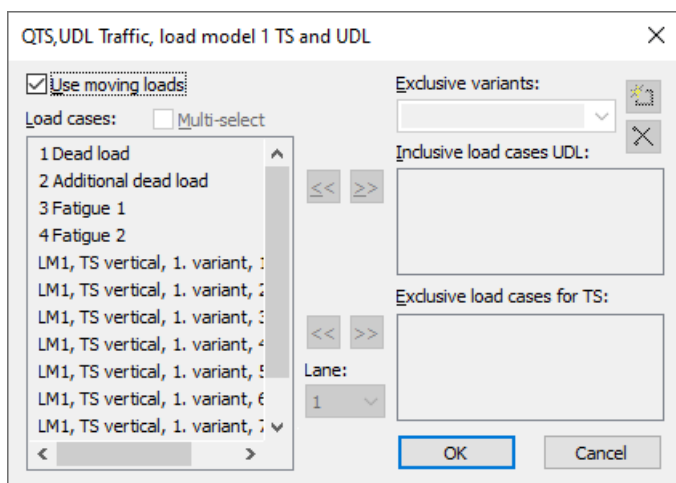
To describe the actions and their combinations, click the *Actions* option in the database and then select *Insert*.



The actions to be considered are selected in the dialog. You can now assign load cases to them.



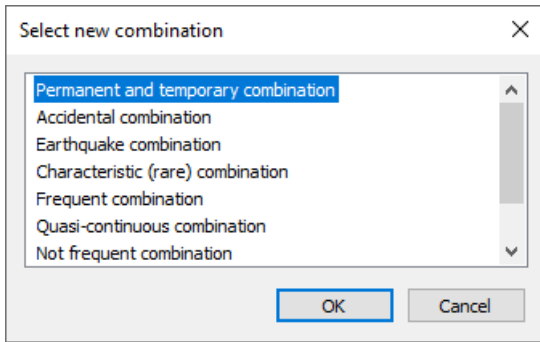
For variable actions, you can add multiple load cases to form inclusive and exclusive groups. If required, you can also define multiple mutually exclusive variants.



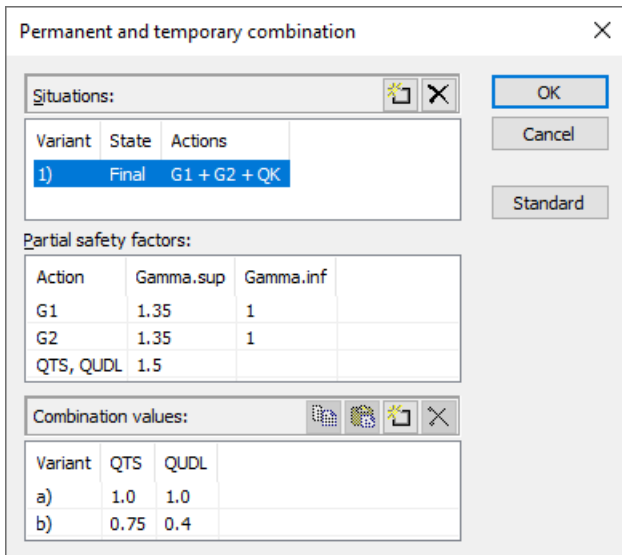
The variable actions QTS and QUDL are taken directly from load model 1.

The *Use moving loads* option must be enabled for this to work.

Alternatively, you can also use freely-definable load cases or combinations.

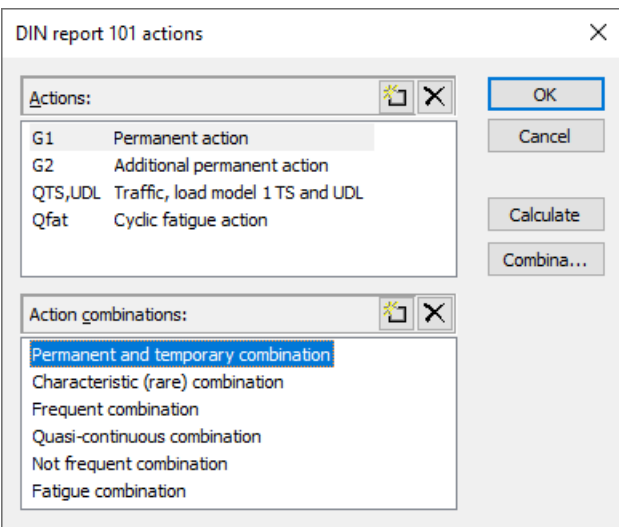


Once the respective loads have been assigned to the actions, you can add the combinations required for the checks.



The following variants are derived using the safety factors and combination coefficients according to Technical Report 101/102.

You can accept the suggestion by clicking OK in order to continue selecting more combinations.



To determine the extremal internal forces from the defined combinations, click *Calculate*.

The program combines the results of the load cases, combinations and load models that are used and then makes them available in the usual manner:

Area element internal forces

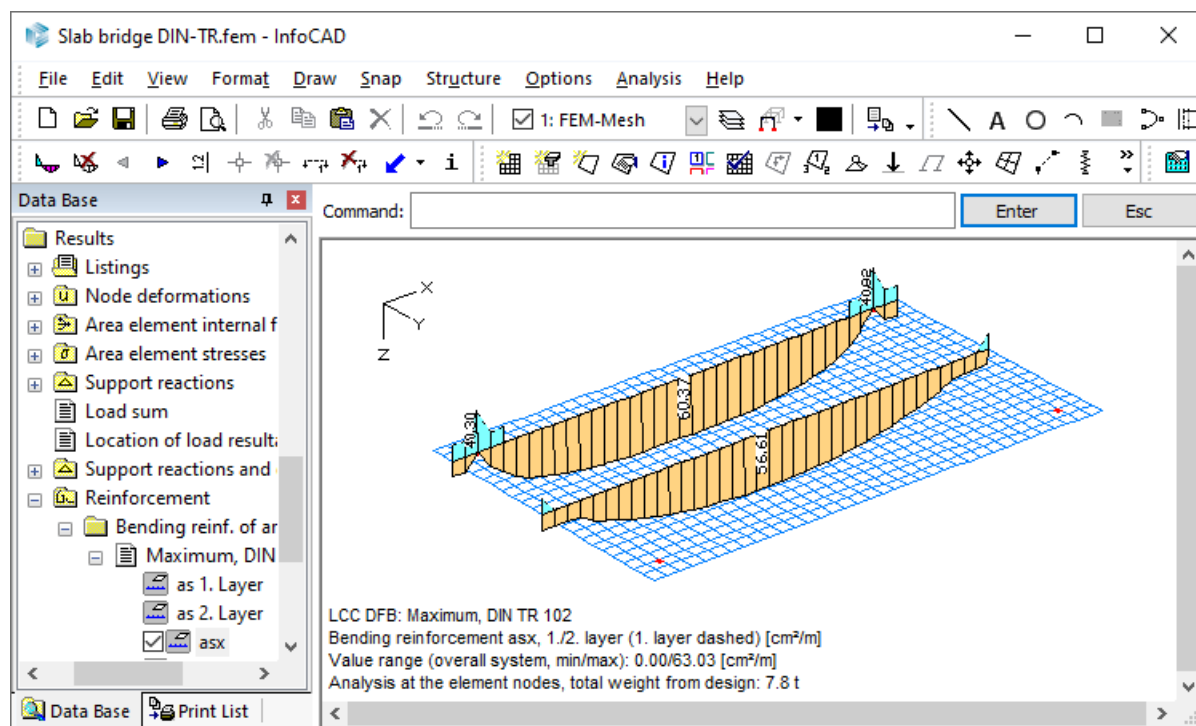
- 1. Permanent and temporary combination, DIN TR 102
- 1. Characteristic (rare) combination, DIN TR 102
- 1. Frequent combination, DIN TR 102
- 1. Quasi-continuous combination, DIN TR 102
- 1. Not frequent combination, DIN TR 102
- 1. Fatigue situation, basic part, DIN TR 102
- 1. Fatigue situation, cyclic part, DIN TR 102

The calculation of combinations is optional here since it will be performed by the checking program in any case.



## Performing checks

The *DIN Technical Report Bridge Checks* checking program is started from the *Analysis* menu. Afterwards, the results can be obtained from the data base.



Maximum value of the upper and lower longitudinal reinforcement from bending with normal force, crack check and fatigue.

The summary log for a single element in the midspan is provided below:

### Design according to DIN Technical Report 101 and 102 (2009-03)

The actions are combined according to DIN Technical Report 101 using the individual partial safety and combination factors.

All checks are made for the extreme values of actions.

#### Type of structure

Traffic route: Road  
Construction type: Concrete

#### Design overview

Se.	Class of component	Prestress	Box se.	Reinforc.	Fatig.	Crack-	De-	Stress checks
			ARS	M R B Q T	S P C	width	comp.	Comp. Tens. S P
1	D	Not prestressed	.	x x x x .	x . .	x	.	x . x .

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state.
- (T) Torsional reinforcement at ultimate limit state.
- (S) Reinforcing steel at stress and fatigue check.
- (P) Prestressing steel at stress and fatigue check.
- (C) Concrete at fatigue check.

#### Settings for flexural and shear reinforcement

- M,N Design mode for bend and longitudinal force:  
(ST) Standard, (SY) Symmetrical, (CM) Compression member.
- fyk Quality of stirrups.
- Theta Angle of concrete truss. Program-sided, the given value of cot Theta is limited to the value range according to Eq. (II-4.122).
- Slabs Beams are designed like slabs.
- As1 Given reinforcement according to picture 4.12, increase to maximum.
- rhov Minimum reinf. min rhov = Factor \* rho with rho according to table 5.7.
- x,y Separate lateral force design for reinforcement directions x and y.
- cv1 Laying measure of the long. reinforcement to limit the lever arm z.
- Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den-	Dsn.	fyk	cot	Dsn.	As1	Fac.	Dsn.	cv1	Red.
		sity		[MPa]	Theta	like	[cm²]	max	rhov	x,y	pre-
		[kg/m³]	M,N			slabs	given			[mm]	str.
1	C30/37	.	ST	500	1.75	.	0.00	0.00	0.60	.	60

**Shear sections**

bw.nom Nominal width of the prestressed section acc. to II-4.3.2.2 (8)\*P.  
 h.nom Nominal height of the prestressed section acc. to II-4.3.2.2 (8)\*P.  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d. z is limited according to II-4.3.2.4.2(2)\*.  
 z1, z2 Height and width of the core section for torsion.  
 teff Thickness of the torsion box.  
 B. Box section; Load-bearing capacity is determined acc. to Eq. (4.47b).

Se.	Width [m]		Eff. width		Height[m]		Eff.height		Torsion section [m]		
	bw	bw.nom	bn [m]	kb	h	h.nom	d [m]	kd	z1	z2	teff B.
1	1.000	.	.	.	0.850	.	0.780	0.90	.	.	.

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel.  
 max.s Maximal given bar spacing of the reinforcing steel.  
 Xi1 Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 sr,max Upper limit for the crack spacing from equ. (4.203).  
 Method Direct calculation of the crack width as per chapt. 4.4.2.4 or check by limiting the bar spacing according to table 4.121.  
 TM Thick member according to chapt. 4.4.2.2(9) to determine As,min.

Se.	wk,per [mm]	ds [mm]	max.s [mm]	Coeff.	sr,max	Concr. age	Method	Combination	TM
				Xi1 k	[mm]	As,min	wk crack w.	for As,min	
1	0.20	28	.	0.50	.	> 28d	> 28d	Calcul.	Charact.

**Settings for the check of concrete stresses**

Av Full section area for stress calculation from longitudinal force.  
 fc(t) Compressive strength of concrete at the time t of prestressing.  
 Sigma.c(t) Concrete compressive stress at the time t of prestressing.  
 Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.x Concrete stress from bending and normal force in beam direction.  
 Sigma.1 Diagonal principal tensile stress for area elements.  
 Sigma.x,y Concrete stresses for area elements in the internal forces system.  
 P.Tens. Check principal tens. stress only in the zone of long. compression.  
 (NC), (QC) Not frequent, Quasi-continuous combination.

Se.	Av [m²]	fc(t) [MN/m²]	per.sigma.c(t) (NK, QK)	per.sigma.c (NK) (QK)	Dekompression stress	P.Tens. only f. compr.zone
1	.	.	.	0.60 fck	0.45 fck	.

**Settings for fatigue check**

fcd,fat Compr. strength of concrete before beginning of cyclic actions.  
 dSigma.Rsk,s Permissible charact. stress fatigue range of longitudinal reinf.  
 dSigma.Rsk,b Permissible charact. stress fatigue range of shear reinf.  
 dSigma.Rsk,p Permissible charact. stress fatigue range of prestr. steel.  
 Lambda.s,b,p Corresp. correction coeff. for damage equivalent fatigue range.  
 Lambda.c Correction coeff. for damage equivalent concrete stresses.  
 Eta Increase factor for reinf. steel stress acc. to II-4.3.7.3 (3)P.  
 k0 Reduction factor for the statically determined part of prestressing acc. to II-4.3.7.2 (103)P.  
 Factor Qfat Factor for Qfat to calculate the damage equ. fatigue range for reinf. steel and prestressing steel acc. to Annex II-A.106.  
 Ch. SPC Check for (S) reinforcing steel, (P) prestr. steel, (C) concrete.  
 (x) Check with damage equivalent fatigue ranges - fatigue comb.  
 (+) Simplified check acc. to II-4.3.7.4/5 - frequent combination.

Se.	Ch. SPC	fcd,fat	dSigma [MN/m²]			Lambda			Eta	Factor	k0
		[MN/m²]	Rsk,s	Rsk,b	Rsk,p	s	b	p	c	Qfat	.
1	x..	.	175.00	175.00	.	1.26	1.26	.	.	1.40	*)

\*) For this section, the calculation of the fatigue range is reduced to corresponding variants of design forces.

**Stress calculation for area elements**

Stresses of concrete are calculated at gross section.  
 The design of reinf. steel stresses is examined in the condition of cracked concrete section.

**Partial safety factors for material at ultimate limit state**

	concrete (gamma.c)	Reinf.steel (gamma.s)	Prestr.steel (gamma.s)
Permanent and temporary comb., earthquakes	1.50	1.15	1.15
Accidental combination	1.30	1.00	1.00
Fatigue check	1.50	1.15	1.15

For concrete above the quality classes C55/67 and LC55/60, rule 5.3.3 (9) of DIN 1045-1 (2001, 2008) is applied.

**Reinforcing steel of area elements**

Se.	Lay.	Qua.	E-Modul [MN/m²]	dt x [m]	db x [m]	asx [cm²/m]	dt y [m]	db y [m]	asy [cm²/m]	as fix
1	1	500M	200000	0.060	.	0.00	0.080	.	0.00	.
	2	500M	200000	.	0.060	0.00	.	0.080	0.00	.

**DESIGN FOR AREA ELEMENTS**

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. for robustness as per II-4.3.1.3 (105)P (N.frequ. C.)
- (R) Nominal/requ. reinforcement as per II-4.4.2.2/4 for crack width limitation  
Increase of reinforcement due to crack width check is marked by "!".
- (B) Design of reinforcement at ultimate limit state  
In case of dominant bending, compression reinforcement is marked with "\*".  
For the less stressed direction of slabs and walls, the minimum reinforcement as per II-5.4.3.2.1 (2)\*P, 5.4.7.3 (2)\*P is not determined.

Element No.	Se.	Lo.	Lay.	Reinforcem. Type	Reinf. for x-direction			Reinf. for y-direction		
					nsd [kN/m]	msd [kNm/m]	req.asx [cm <sup>2</sup> /m]	nsd [kN/m]	msd [kNm/m]	req.asy [cm <sup>2</sup> /m]
124	1	1	1	M	0.00	760.28	0.00	0.00	-46.54	7.05
				R	0.00	760.28	0.00	0.00	-47.71	15.63
				B	0.00	1681.10	0.00	0.00	-65.28	1.87
				M	0.00	1114.50	6.88	0.00	12.78	7.05
				R	0.00	1053.99	60.35!	0.00	21.95	15.63
				B	0.00	1681.10	52.53	0.00	44.86	1.28
124	1	2	1	M	0.00	769.11	0.00	0.00	-39.99	7.05
				R	0.00	769.11	0.00	0.00	-41.00	15.63
				B	0.00	1679.93	0.00	0.00	-56.11	1.61
				M	0.00	1113.04	6.88	0.00	4.63	7.05
				R	0.00	1052.71	60.29!	0.00	13.24	15.63
				B	0.00	1679.93	52.49	0.00	34.76	0.99
124	1	3	1	M	0.00	757.32	0.00	0.00	-24.47	7.05
				R	0.00	757.32	0.00	0.00	-24.59	15.63
				B	0.00	1653.88	0.00	0.00	-34.84	1.00
				M	0.00	1095.71	6.88	0.00	27.55	7.05
				R	0.00	1036.42	59.73!	0.00	39.02	15.63
				B	0.00	1653.88	51.55	0.00	70.72	2.03
124	1	4	1	M	0.00	748.50	0.00	0.00	-30.37	7.05
				R	0.00	748.50	0.00	0.00	-30.49	15.63
				B	0.00	1655.08	0.00	0.00	-41.29	1.18
				M	0.00	1097.19	6.88	0.00	35.65	7.05
				R	0.00	1037.72	59.79!	0.00	47.67	15.63
				B	0.00	1655.08	51.60	0.00	82.24	2.36

**Design of shear reinforcement**

The percentage of nominal reinforcement as per II-5.4.2.2 (4)P is considered

- vRd Absorbable lateral force of comp. struts per II-4.3.2.4.4 (2)\*P [kN/m]
- Angle Angle cot Theta between the compressive strut and the element plane
- Asl Req. longitudinal reinf. as per Fig. II-4.12 for req. asb [cm<sup>2</sup>]
- qr Lateral force for design =  $(q_x^2 + q_y^2)^{1/2}$  [kN/m]
- req.asb Req. stirrup reinforcement [cm<sup>2</sup>/m<sup>2</sup>]
- req.As1 Req. longitudinal reinf. as per Fig. II-4.12 [cm<sup>2</sup>] for req. asb

Element No.	Loc.	qx [kN/m]	qy [kN/m]	n [kN/m]	qr [kN/m]	qr/vRd	Angle [cm <sup>2</sup> /m <sup>2</sup> ]	req. asb	As1 f. asb=0	req. As1
								[cm <sup>2</sup> /m <sup>2</sup> ]	[cm <sup>2</sup> ]	[cm <sup>2</sup> ]
124	1	69.36	50.20	0.00	85.62	0.02	1.75	0.00	.	.
	2	69.36	51.88	0.00	86.62	0.02	1.75	0.00	.	.
	3	69.50	51.88	0.00	86.73	0.02	1.75	0.00	.	.
	4	69.50	50.20	0.00	85.74	0.02	1.75	0.00	.	.

**Fatigue check for longitudinal reinforcement**

For the check, a cracked concrete section is assumed.

- dSigma.s,eq Damage equivalent stress range [MN/m<sup>2</sup>]
- dSigma.s,zul = dSigma.Rsk,s / gamma.s,fat [MN/m<sup>2</sup>]
- (simplified) = dSigma.Rsk,s as per II-4.3.7.5 (101) [MN/m<sup>2</sup>]

Element No.	Lo.	Lay.	Check in x-direction			Check in y-direction				
			as [cm <sup>2</sup> /m]	req. [cm <sup>2</sup> /m]	dSigma.s [MN/m <sup>2</sup> ]	as [cm <sup>2</sup> /m]	req. [cm <sup>2</sup> /m]	dSigma.s [MN/m <sup>2</sup> ]		
124	1	1	0.00	.	.	152.17	15.63	.	27.52	152.17
		2	60.35	.	73.00	152.17	15.63	.	26.36	152.17
124	2	1	0.00	.	.	152.17	15.63	.	33.84	152.17
		2	60.29	.	77.80	152.17	15.63	.	22.58	152.17
124	3	1	0.00	.	.	152.17	15.63	.	26.75	152.17
		2	59.73	.	77.58	152.17	15.63	.	47.62	152.17
124	4	1	0.00	.	.	152.17	15.63	.	23.87	152.17
		2	59.79	.	72.54	152.17	15.63	.	51.08	152.17

**Fatigue check for shear reinforcement**

The stress calculation is based on a framework model.

- dSigma.s,eq Damage equivalent fatigue stress range [MN/m<sup>2</sup>]
- dSigma.s,per = dSigma.Rsk,b / gamma.s,fat [MN/m<sup>2</sup>]
- (simplified) = dSigma.Rsk,b as per II-4.3.7.5 (101) [MN/m<sup>2</sup>]

Element No.	Se.	Loc.	asb [cm <sup>2</sup> /m <sup>2</sup> ]		dSigma.s [MN/m <sup>2</sup> ]	
			giv.	req.	equ.	per.
124	1	1	0.00	.	.	152.17
		2	0.00	.	.	152.17
		3	0.00	.	.	152.17
		4	0.00	.	.	152.17

**Check of crack widths**

The check is led by direct calculation of the crack width.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

wk Calculated value of crack width as per II-4.4.2.4 [mm]  
 wk,per Permissible crack width as per specification [mm]  
 nsd, msd Longitudinal force and moment for design [kN/m, kNm/m]  
 (CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination  
 (NC) Not frequent combination

Element No.	Se.	C.	Loc.	Check in x-direction				Check in y-direction			
				nsd [kN/m]	msd [kNm/m]	wk [mm]	wk,per [mm]	nsd [kN/m]	msd [kNm/m]	wk [mm]	wk,per [mm]
124	1	TC	1	0.00	1053.99	0.20	0.20	0.00	-47.31	0.01	0.20
			2	0.00	1052.71	0.20	0.20	0.00	-40.97	0.01	0.20
			3	0.00	1036.42	0.20	0.20	0.00	-25.75	0.00	0.20
			4	0.00	1037.72	0.20	0.20	0.00	-31.82	0.01	0.20

**Check of concrete compressive stress**

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked. On cracked section, the stress min(Sigma.x, Sigma.y) as the extremal value of the two reinforcing directions is checked.

Sigma,min Total maximal longitudinal compressive stress [MN/m<sup>2</sup>]  
 Sigma,zul = 0.60\*fck resp. 0.66\*fck for N.frequ. C. (NC) as per 4.4.1.2 (103)P  
 = 0.45\*fck for Q.-cont. C. (QC) as per 4.4.1.2 (104)\*P  
 (t,b) Upper, lower edge of section

Element No.	Se.	Loc.	Stress	min Sigma [MN/m <sup>2</sup> ]	Sigma,per [MN/m <sup>2</sup> ]	Side		Period	Situation
						t	b		
124	1	1	Sigma.x(II)	-13.93	-18.00	x	.	Final	NC.1
			Sigma.x(II)	-10.94	-13.50	x	.	Final	QC.1
		2	Sigma.x(II)	-13.92	-18.00	x	.	Final	NC.1
			Sigma.x(II)	-10.91	-13.50	x	.	Final	QC.1
		3	Sigma.x(II)	-13.76	-18.00	x	.	Final	NC.1
			Sigma.x(II)	-10.78	-13.50	x	.	Final	QC.1
		4	Sigma.x(II)	-13.78	-18.00	x	.	Final	NC.1
			Sigma.x(II)	-10.81	-13.50	x	.	Final	QC.1

**Check of reinforcing steel stress for the Not frequent combination**

For the check, a cracked concrete section is assumed.

Sigma.s,per = 0.80 \* fyk (NC) as per II-4.4.1.3 (105)

Element No.	Se.	Lo.	Lay.	Check in x-direction			Check in y-direction		
				as [cm <sup>2</sup> /m]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]	as [cm <sup>2</sup> /m]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]
124	1	1	1	0.00	.	400.00	15.63	40.72	400.00
			2	60.35	257.59	400.00	15.63	11.18	400.00
124	1	2	1	0.00	.	400.00	15.63	34.99	400.00
			2	60.29	257.47	400.00	15.63	4.05	400.00
124	1	3	1	0.00	.	400.00	15.63	21.41	400.00
			2	59.73	255.72	400.00	15.63	24.10	400.00
124	1	4	1	0.00	.	400.00	15.63	26.57	400.00
			2	59.79	255.85	400.00	15.63	31.19	400.00

## Railroad Overpass With Prestressed Concrete Superstructure

This example was chosen on the basis of part 2 of the book

Müller, Michael / Bauer, Thomas

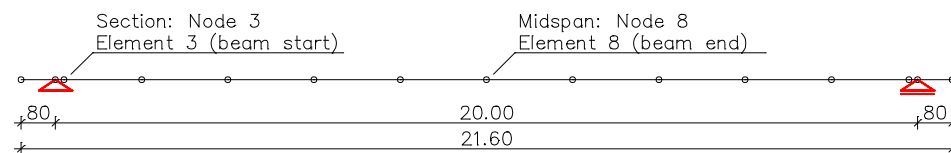
Eisenbahnbrückenbau nach DIN-Fachbericht (Railroad Bridge Construction According to DIN Technical Report)

**Beispiele prüffähiger Standsicherheitsnachweise (Examples of Verifiable Stability Safety Checks)**

Band 1: Stahlbeton- und Spannbetonüberbau nach DIN-Fachbericht 101 und 102. (Volume 1: Reinforced Concrete and Prestressed Concrete Superstructures According to DIN Technical Reports 101 and 102)

2nd Revised Edition. Bauwerk Verlag GmbH, Berlin 2003.

The construction is a prestressed beam in the longitudinal direction with subsequent bond in requirement class B. The lateral direction is not analyzed in this example. The support is free floating in the longitudinal direction and fixed in the lateral direction.



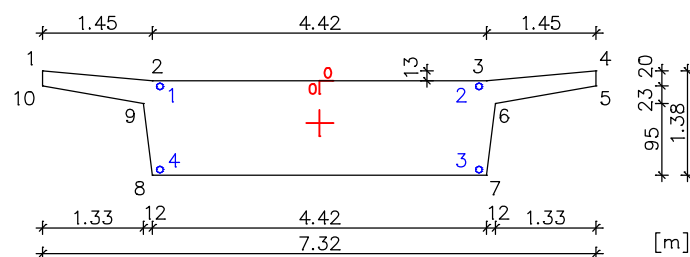
Static system and dimensions

### Material

Concrete C40/50

Reinforcing steel BSt 500/550, axis distance from edge 7.5 cm

### Section



According to the draft specification the superstructure must be designed for load model 71 with a classification coefficient of  $\alpha = 1.0$  and for load model SW2.

The main checks are carried out for the following times:

$t_0 = 10$  days: Time of prestressing

$t_1 = 100$  days: Time of traffic transfer

$t_\infty = 100$  years: Time after conclusion of creep and shrinkage

The following checks are carried out as a part of this example:

Ultimate limit state

- Bending with longitudinal force
- Lateral force
- Failure without warning (robustness)
- Fatigue of longitudinal reinforcement and prestressing steel
- Fatigue of shear reinforcement
- Fatigue of concrete under compressive stress

Serviceability limit state

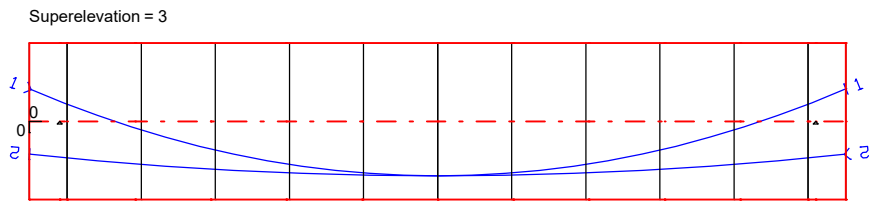
- Decompression
- Minimum reinforcement for the crack width limitation
- Crack width calculation
- Limiting the concrete compressive stresses
- Limiting the reinforcing and prestressing steel stresses
- Limiting diagonal principal tensile stresses

Because of their low influence on the selected checks, actions arising from centrifugal loads, lateral impacts, wind loads, the 'unloaded train' load model, starting and braking, derailment, actions on rails, traffic loads on service sidewalks and soil pressure are ignored.

**Prestressing steel and prestressing system**

Prestressing steel quality	St 1500/1770
Certification of the prestressing system	SUSPA EC 140
Number of tendons in the bundle	6
Section surface $A_p$	2660 mm <sup>2</sup>
E-modulus of the prestressing steel	190000 MN/m <sup>2</sup>
0.1% strain limit (yield strength) of the prestressing steel $f_{p0.1k}$	1500 MN/m <sup>2</sup>
Tensile strength of the prestressing steel $f_{pk}$	1770 MN/m <sup>2</sup>
Permissible prestressing force of a tendon $P_{m0}$	3391.5 kN
Friction coefficients when prestressing and releasing $\mu$	0.21
Unintentional deviation angle of a tendon $\beta'$	0.3 °/m
Slippage at prestressed tie bolt	6 mm
Duct diameter $d_h$	97 mm
Allowance value for ensuring an overstressing reserve $\kappa$	1.5
Variation factors of internal prestressing	
Construction stage ( $r_{sup} / r_{inf}$ )	1.1 / 0.95
Final state ( $r_{sup} / r_{inf}$ )	1.1 / 0.9

The tendon guide is shown in the next figure. 2 tendon groups with 6 bundled tendons each are arranged such that they stretch across the entire girder length and are alternately prestressed at both girder ends. The prestressing systems, prestressing procedures and prestressing curves for both tendon groups are also shown.

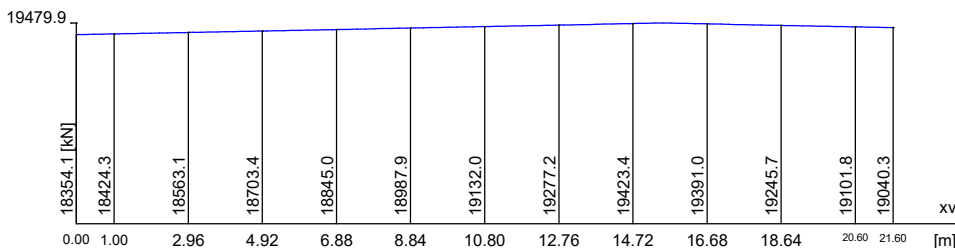


Tendon group ordinates  $z_v$  [cm] at the base points

xv	0.00	1.00	2.96	4.92	6.88	8.84	10.80	12.76	14.72	16.68	18.64	20.60	21.60
1	-28.6	-15.1	7.6	25.3	37.9	45.5	48.0	45.5	37.9	25.3	7.6	-15.1	-28.6
2	28.8	32.2	37.9	42.3	45.5	47.4	48.0	47.4	45.5	42.3	37.9	32.2	28.8

*Tendon guide in the longitudinal section*

Force function of tendon group 1 (6 tendon(s),  $l = 21.67$  m)  
 Prestressing system 1 - SUSPA EC 140. Certification according to EC2.  
 $P_{m0} = 3391.5$  kN,  $A_p = 2660.0$  mm<sup>2</sup>,  $\mu_a = 0.21$ , Angle  $\beta' = 0.30$  °/m  
 E-Modulus = 190000 MN/m<sup>2</sup>,  $A_h = 7389.8$  mm<sup>2</sup>,  $\mu_n = 0.21$ , Slippage = 6.00 mm  
 Prestressing procedure 1 - with reserve  
 Tensioning with  $P_{max}$  (DIN Report, DIN 1045-1, DIN EN 1992-1-1).  $\kappa = 1.5$ .  
 Pre. anchor : Start  
 Normal. force : 1.000  
 Pre. force [kN]: 3446.0  
 Extension [mm]: 141.8



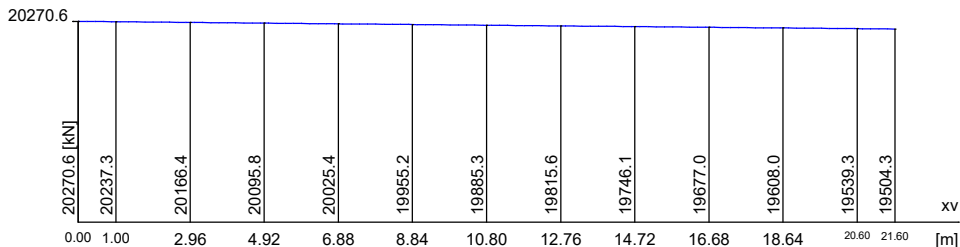
*Prestressing curve of tendon group 1 in the longitudinal section*

Force function of tendon group 2 (6 tendon(s), l = 21.60 m)

Prestressing system 1 - SUSPA EC 140. Certification according to EC2.  
 Pm0 = 3391.5 kN, Ap = 2660.0 mm<sup>2</sup>, μa = 0.21, Angle β = 0.30 °/m  
 E-Modulus= 190000 MN/m<sup>2</sup>, Ah = 7389.8 mm<sup>2</sup>, μn = 0.21, Slippage = 6.00 mm

Prestressing procedure 1 - with reserve  
 Tensioning with Pmax (DIN Report, DIN EN 1992-1-1). Kappa = 1.5.

Pre. anchor : Start  
 Normal force : 1.000  
 Pre. force [kN]: 3522.5  
 Extension [mm]: 147.7

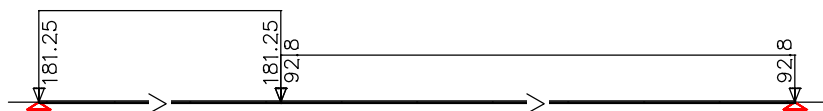


Prestressing curve of tendon group 2 in the longitudinal section

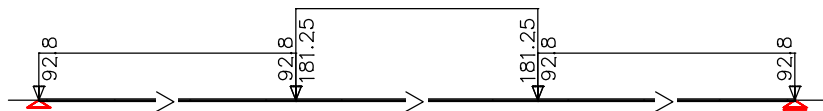
**Loads**

- Load case 1 Dead load (G1).
- Load case 2 Additional loads 103,7 kN/m (G2).
- Load case 3 Prestressing (P).
- Load case 4 Creep-generating permanent load (G1+G2+P)
- Load case 5 Creep and shrinkage (CS,  $t_{\infty}$ ). The specifications  
 RH=80%,  $t_0=10$  d,  $t_s=1$  d, cement hardening = normal  
 result in the following coefficients:  $\phi_{t_{\infty}} = 1.57$ ;  $\epsilon_{t_{\infty}} = -30.2 \cdot 10^{-5}$ ;  $\rho = 0.8$   
 Creep-generating permanent load case: 4,  
 The redistribution of internal forces between concrete and prestressing steel are taken into account.
- Load case 11-13 Positions of load model 71 (Q1)
- Load case 21 Load model SW/2 (Q2)
- Load case 31 Temperature  $\Delta x_{neg}$  (T)
- Load case 32 Temperature  $\Delta x_{pos}$  (T)

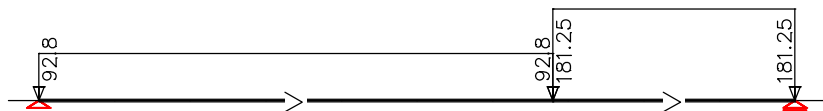
**Load model 71**



Load case 11: Load model 71, left  
 The dynamic coefficient  $\Phi$  of 1.16 is already considered in the load ordinates.

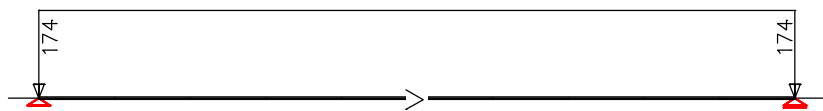


Load case 12: Load model 71, middle



Load case 13: Load model 71, right

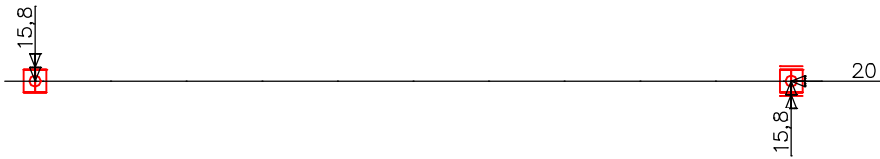
**Load model SW/2**



Load case 21: The dynamic coefficient  $\Phi$  of 1.16 is already considered in the load ordinates.

## Temperature

Due to the free-floating support, the temperature loads  $\Delta T_{Mz}$  and  $\Delta T_N$  that are to be applied result in support movements and therefore restoring forces according to Müller, pp. 71-72.



Load case 31: Restoring forces as external support reactions

The actions and action combinations defined for the checks are listed in the following.

### DIN Report 101 actions

#### G1 - Permanent action, dead load

1 G1

#### G2 - Permanent action, additional dead load

2 G2

#### P - Prestress, intern

3 P

#### CSR1 - Creep, shrinkage, relaxation, t1

CS as constant reduction of the prestressing with 4 %.  
Prestressing loss from relaxation of prestressed steel: 2.2 %.

#### CSR2 - Creep, shrinkage, relaxation, too

5 CS,too  
Prestressing loss from relaxation of prestressed steel: 6.2 %.

#### T - Temperature, 1. Variante, exclusive

31 Temperature dTN (dx,neg)  
32 Temperature dTN (dx,pos)

#### Q1 - Load group gr11-gr14 (1 track), 1. Variante, exclusive

11 Qvk LM 71,1  
12 Qvk LM 71,2  
13 Qvk LM 71,3

#### Q2 - Load group gr16-gr17 (1 track), 1. Variante, inclusive

21 Qvk LM SW/2

#### Qfat - Cyclic fatigue action, 1. Variante, exclusive

11 Qvk LM 71,1  
12 Qvk LM 71,2  
13 Qvk LM 71,3

### Permanent and temporary combination, situations

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) t1	Final	G1 + G2 + P + CSR1 + QK
3) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.



**Permanent and temporary combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1.35	1
G2	1.35	1
P, CSR1, CSR2	1	1
T	1.35	0
Q1	1.45	0
Q2	1.2	0

**Permanent and temporary combination, combination coefficients**

Variant	T	Q1	Q2
a) T	1	0.8	0
b) T	1	0	0.8
c) Q1	0.8	1	0
d) Q2	0.8	0	1

**Characteristic (rare) combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) t1	Final	G1 + G2 + P + CSR1 + QK
3) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Characteristic (rare) combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0

**Characteristic (rare) combination, combination coefficients**

Variant	T	Q1	Q2
a) T	1	0.8	0
b) T	1	0	0.8
c) Q1	0.8	1	0
d) Q2	0.8	0	1

**Frequent combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) t1	Final	G1 + G2 + P + CSR1 + QK
3) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Frequent combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0

**Frequent combination, combination coefficients**

Variant	T	Q1	Q2
a) T	0.6	0	0
b) Q1	0.5	0.8	0
c) Q2	0.5	0	0.8

**Quasi-continuous combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Quasi-continuous combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0

**Quasi-continuous combination, combination coefficients**

Variant	T	Q1	Q2
a)	0.5	0	0

**Not frequent combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) t1	Final	G1 + G2 + P + CSR1 + QK
3) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Not frequent combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0

**Not frequent combination, combination coefficients**

Variant	T	Q1	Q2
a) T	0.8	0.8	0
b) T	0.8	0	0.8
c) Q1	0.6	1	0
d) Q2	0.6	0	1

**Fatigue combination, situations**

Variant	State	Actions
1)	Final	G1 + G2 + P + CSR2 + QK + Qfat

QK means changeable actions in accordance with the table of the combination coefficients.

**Fatigue combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0
Qfat	1	0

**Fatigue combination, combination coefficients**

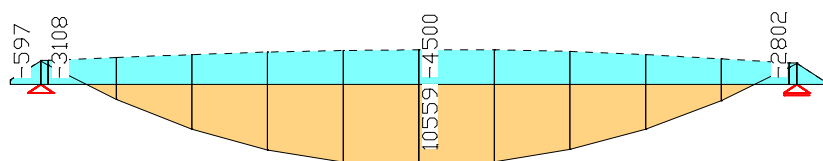
Variant	T	Q1	Q2
a)	0.6	0	0

Notes on the check situations:

- For the checks at time  $t_1$  the influence of the creep and shrinkage was estimated at 4% of the prestressing.
- For the decompression check in the construction stage, an action variant at time  $t_1$  (G1+P+CSR1) was included in the frequent combination.
- For the concrete compressive stress check at the time when prestressing is transferred, an action variant at time  $t_0$  (G1+P) was included in the infrequent combination.

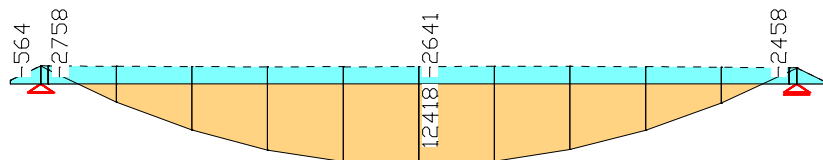
Below you will find an example of the curve of bending moment  $M_y$  for design situations in the ultimate limit states.

**2. Permanent and temporary combination –  $t_1$**



Bending moment  $M_y$  [kNm]

**3. Permanent and temporary combination - too**



Bending moment  $M_y$  [kNm]

**Design according to DIN Technical Report 101 and 102 (2009-03)**

**Design overview**

Se.	Class, Prestress of component	Box se. ARS	Reinforc. M	Fatig. R	Crack- width comp. S	De- comp. P	Stress checks Tens. S
1	B Subsequent bond	.	x	x	x	x	x x x

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state.
- (T) Torsional reinforcement at ultimate limit state.
- (S) Reinforcing steel at stress and fatigue check.
- (P) Prestressing steel at stress and fatigue check.
- (C) Concrete at fatigue check.

**Dispersion of prestressing**

The dispersion of prestressing is considered at the following checks:

- Check of decompression
- Nominal reinforcement for crack width limitation
- Check of crack width

All other checks are made using the mean value  $P_{m,t}$  of prestressing.

Se.	Prestressing of component	Const.period r.sup	Final state r.inf
1	Subsequent bond	1.10	0.95

**Settings for flexural and shear reinforcement**

- M,N Design mode for bend and longitudinal force: (ST) Standard, (SY) Symmetrical, (CM) Compression member.
- fyk Quality of stirrups.
- Theta Angle of concrete truss. Program-sided, the given value of cot Theta is limited to the value range according to Eq. (II-4.122).
- Slabs Beams are designed like slabs.
- Asl Given reinforcement according to picture 4.12, increase to maximum.
- rhov Minimum reinf. min rhov = Factor \* rho with rho according to table 5.7.
- x,y Separate lateral force design for reinforcement directions x and y.
- cvl Laying measure of the long. reinforcement to limit the lever arm z.
- Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den- sity [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] Fig. 4.12 given max	Fac. rhov	Dsn. cvl x,y [mm]	Red. pre- str.
1	C40/50	.	ST	500	1.75	.	64.35	1.60	75	.

**Shear sections**

bw.nom Nominal width of the prestressed section acc. to II-4.3.2.2 (8)\*P.  
 h.nom Nominal height of the prestressed section acc. to II-4.3.2.2 (8)\*P.  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d. z is limited according to II-4.3.2.4.2(2)\*.  
 z1, z2 Height and width of the core section for torsion.  
 teff Thickness of the torsion box.  
 B. Box section; Load-bearing capacity is determined acc. to Eq. (4.47b).

Se.	Width [m]	Eff. width	Height[m]	Eff.height	Torsion section [m]						
	bw	bw.nom	bn	kb	h	h.nom	d [m]	kd	z1	z2	teff
1	4.420	4.129	4.345	0.90	1.250	1.155	1.175	0.90	1.100	4.270	0.150

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel.  
 max.s Maximal given bar spacing of the reinforcing steel.  
 Xil Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 sr,max Upper limit for the crack spacing from equ. (4.203).  
 Method Direct calculation of the crack width as per chapt. 4.4.2.4 or check by limiting the bar spacing according to table 4.121.  
 TM Thick member according to chapt. 4.4.2.2(9) to determine As,min.

Se.	wk,per	ds	max.s	Coeff.	sr,max	Concr. age	Method for	Tensile zone	
	[mm]	[mm]	[mm]	Xil	k	As,min	wk	for As,min	
						3- 5d	> 28d	Calcul.	Charact.
1	0.20	16	.	0.00	0.50	.	.	.	

**Settings for the check of concrete stresses**

Av Full section area for stress calculation from longitudinal force.  
 fc(t) Compressive strength of concrete at the time t of prestressing.  
 Sigma.c(t) Concrete compressive stress at the time t of prestressing.  
 Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.x Concrete stress from bending and normal force in beam direction.  
 Sigma.l Diagonal principal tensile stress for area elements.  
 Sigma.x,y Concrete stresses for area elements in the internal forces system.  
 P.Tens. Check principal tens. stress only in the zone of long. compression.  
 (NC), (QC) Not frequent, Quasi-continuous combination.

Se.	Av	fc(t)	per.sigma.c(t)	per.sigma.c	Dekompression	P.Tens. only
	[m²]	[MN/m²]	(NK, QK)	(NK)	(QK)	stress
			fc(t)	fck	0.45 fck	Sigma.x
1	6.39	34.40	0.45	0.60	0.45	f. compr.zone

**Settings for fatigue check**

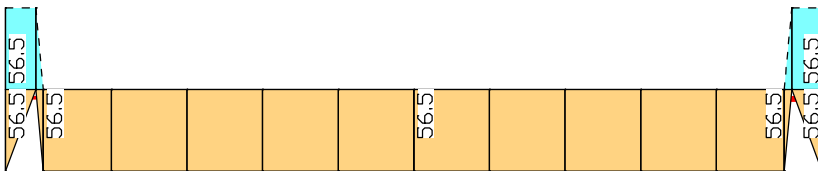
The concrete compressive stresses are calculated acc. to fig. 4.1 with fc=fcm.

fcd,fat Compr. strength of concrete before beginning of cyclic actions.  
 dSigma.Rsk,s Permissible charact. stress fatigue range of longitudinal reinf.  
 dSigma.Rsk,b Permissible charact. stress fatigue range of shear reinf.  
 dSigma.Rsk,p Permissible charact. stress fatigue range of prestr. steel.  
 Lambda.s,b,p Corresp. correction coeff. for damage equivalent fatigue range.  
 Lambda.c Correction coeff. for damage equivalent concrete stresses.  
 Eta Increase factor for reinf. steel stress acc. to II-4.3.7.3 (3)P.  
 k0 Reduction factor for the statically determined part of prestressing acc. to II-4.3.7.2 (103)P.  
 Factor Qfat Factor for Qfat to calculate the damage equ. fatigue range for reinf. steel and prestressing steel acc. to Annex II-A.106.  
 Ch. SPC Check for (S) reinforcing steel, (P) prestr. steel, (C) concrete.  
 (x) Check with damage equivalent fatigue ranges - fatigue comb.  
 (+) Simplified check acc. to II-4.3.7.4/5 - frequent combination.

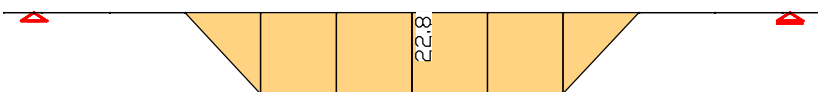
Se.	Ch.	fcd,fat	dSigma	Lambda	Eta	Factor	k0
	SPC	[MN/m²]	Rsk,s	Rsk,b	Rsk,p	s	s
			b	p	c	Qfat	
1	xxx	20.90	175.00	175.00	120.00	0.75	0.79

**Results**

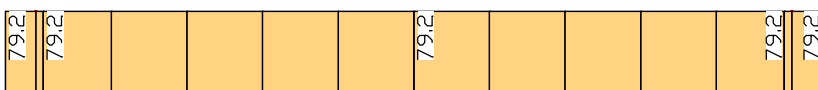
The following illustrations show the curve of the required longitudinal and shear reinforcement. With respect to design in the ultimate limit state, the strain reserves of the tendons are enough to perform the check without additional bending reinforcement. Hence only the robustness and crack reinforcements are necessary.



Minimum reinforcement  $A_s$  for ensuring robustness (ductility) [cm²] (upper reinforcement with dashed lines).



Reinforcement  $A_s$  for limiting the crack width [cm²]



Lateral force reinforcement  $A_{sbz}$  in the ultimate limit states [cm²/m]

Excerpts from the standard check log for beam 3 at location 1 (first cut, x = 0.2 m) and beam 8 at location 1 (midspan) are provided below.

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.  
 (M) Nominal reinf. for robustness as per II-4.3.1.3 (105)P (N.frequ. C.)  
 (R) Nominal/requ. reinforcement as per II-4.4.2.2/4 for crack width limitation  
 Increase of reinforcement due to crack width check is marked by "I".  
 Ap' Part of prestr. steel area Xil\*Ap which was used to reduce req.As  
 Xil Bond coefficient for prestressing steel as per Eq. (II-4.197)  
 (B) Design of reinforcement at ultimate limit state  
 In case of dominant bending, compression reinforcement is marked with "\*\*".

Beam No.	Se.	Lo.	Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	
3	1	1	1	M	-1.39	265.36	0.00	.	0.00	
				R	0.00	0.00	0.00	.	0.00	
			2	M	-32625.51	-2757.57	0.00	.	0.00	
				R	-1.39	265.36	0.00	.	0.00	
			3	M	72.95	787.67	0.00	.	28.25	
				R	0.00	0.00	0.00	.	0.00	
	8	1	1	1	M	0.00	7936.54	0.00	.	0.00
					R	-42919.03	-12664.60	0.00	.	0.00
				2	M	0.00	7936.54	0.00	.	0.00
					R	-42919.03	-12664.60	0.00	.	0.00
				3	M	12.00	21797.83	0.00	.	28.25
					R	-29442.16	7663.69	0.00	.	11.40
4	M	12.00	21797.83	0.00	.	28.25				
	R	-29442.16	7663.69	0.00	.	11.40				

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to II-5.4.2.2 (4)P is considered.

VRd, TRd Design value of maximum absorbable lateral force, torsional moment  
 Angle Angle cot Theta between the compressive strut and the beam axis  
 Asb,Asl.T Req. stirrup reinf. from lateral force and torsion, torsional reinf.  
 Asl Req. longitudinal reinf. acc. to Fig. II-4.12 for req. Asb.  
 Sigma2 Box section, max. principal compressive stress [MN/m<sup>2</sup>]  
 For the check of Sigma2/fcd, state II is marked with "\*\*\*"  
 fcd = alpha.cc \* fck / gamma.c, per. Sigma2 in state I [MN/m<sup>2</sup>]  
 = alpha.cw \* vl \* alpha.cc \* fck / gamma.c, per. Sigma2 in state II

Beam No.	Loc.	VRd	Angle	Asb.y [cm <sup>2</sup> /m]	Qz/VRd	Angle [cm <sup>2</sup> /m]	Asb.z [cm <sup>2</sup> /m]	Asl [cm <sup>2</sup> /m]	Q/VRd+ Mx/TRd	Asb.T, Asl.T [cm <sup>2</sup> /m, cm <sup>2</sup> ]	Sigma2 /fcd
3	1	0.00	1.75	0.00	0.09	1.75	79.21	64.35	.	.	.
8	1	0.00	1.75	0.00	0.00	1.75	79.21	64.35	.	.	.

**Fatigue check for longitudinal reinforcement and prestressing steel**

For the check, a cracked concrete section is assumed.

Type S Longitudinal reinforcement from N and M, layer number  
 Type P Prestressing steel, tendon number  
 dSigma.s, equ Damage equivalent stress range [MN/m<sup>2</sup>]  
 dSigma.s, zul = dSigma.Rsk,s / gamma.s, fat [MN/m<sup>2</sup>]  
 (simplified) = dSigma.Rsk,s as per II-4.3.7.5 (101) [MN/m<sup>2</sup>]

Beam No.	Se.	Loc.	Steel Type	No.	As [cm <sup>2</sup> ]	dSigma.s [MN/m <sup>2</sup> ]		
					giv. req.	equ. per.		
3	1	1	S	1	0.00	.	152.17	
				2	0.00	.	152.17	
			S	3	28.25	.	1.17	152.17
				4	28.25	.	1.17	152.17
			P	1	159.60	.	0.19	104.35
				2	159.60	.	0.40	104.35
8	1	1	S	1	0.00	.	152.17	
				2	0.00	.	152.17	
			S	3	28.25	.	28.09	152.17
				4	28.25	.	28.09	152.17
			P	1	159.60	.	14.24	104.35
				2	159.60	.	14.24	104.35

**Fatigue check for shear reinforcement**

The stress calculation is based on a framework model.

Type BY, BZ Shear reinforcement from Qy or Qz [cm<sup>2</sup>/m]  
 Asb giv. Given reinforcement from preceded dimensioning [cm<sup>2</sup>/m]  
 Asb req. Requ. reinf. for compl. with the permiss. stress range [cm<sup>2</sup>/m]  
 dSigma.s, equ Damage equivalent fatigue stress range [MN/m<sup>2</sup>]  
 dSigma.s, per = dSigma.Rsk,b / gamma.s, fat [MN/m<sup>2</sup>]  
 (simplified) = dSigma.Rsk,b as per II-4.3.7.5 (101) [MN/m<sup>2</sup>]

Beam No.	Se.	Loc.	Rei. Type	Asb [cm <sup>2</sup> /m]	dSigma.s [MN/m <sup>2</sup> ]	
				giv. req.	[MN/m <sup>2</sup> ] [MN/m <sup>2</sup> ]	
3	1	1	BY	0.00	.	152.17
			BZ	79.21	79.21	97.48
8	1	1	BY	0.00	.	152.17
			BZ	79.21	79.21	7.92

**Fatigue check for concrete under compressive stress**

For the check, a cracked concrete section is assumed.

Sigma.cd,min Absolute value of damage equiv. min. compressive stress [MN/m<sup>2</sup>]  
 Sigma.cd,max Absolute value of damage equiv. max. compressive stress [MN/m<sup>2</sup>]  
 Sigma.cd,per = fcd,fat \* (1 - 0.43 \* (1 - Sigma.cd,min/Sigma.cd,max)<sup>0.5</sup>) [MN/m<sup>2</sup>]  
 t, b Position of the edge point: above, below of centre

Beam	No.	Se.	Loc.	Sigma.cd,min [MN/m <sup>2</sup> ]	Sigma.cd,max [MN/m <sup>2</sup> ]	Sigma.cd,per [MN/m <sup>2</sup> ]	Se.- pnt.	Side t b
	3	1	1	5.96	6.11	19.51	8	. x
	8	1	1	4.02	7.70	14.69	4	x .

**Check of crack widths**

The check is led by direct calculation of the crack width.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

wk Calculated value of crack width as per II-4.4.2.4 [mm]  
 wk,per Permissible crack width as per specification [mm]  
 Sigma.c Maximal concrete edge stress in state I [MN/m<sup>2</sup>]  
 (CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination  
 (NC) Not frequent combination

Beam	No.	Se.	C.	Lo.	Reinf. Layer	Nx [kN]	My [kNm]	Mz [kNm]	Sigma.x [MN/m <sup>2</sup> ]	wk [mm]	wk,per [mm]
	3	1	NC	1	.	-42351.9	-3867.42	0.00	-3.77	0.00	0.20
	8	1	NC	1	4	-29446.1	7660.53	0.00	1.05	0.00	0.20

**Check of decompression**

For the check, a non-cracked concrete section is assumed.

(TC) Frequent combination, (QC) Quasi-continuous combination  
 (NC) Not frequent combination  
 (t,b) Position of the edge point: above, below of centre

Beam	No.	Se.	Loc.	max Sigma.x [MN/m <sup>2</sup> ]	per. Sigma.x [MN/m <sup>2</sup> ]	Se.- Pnt.	Side t b	Period	Situation
	3	1	1	-2.82	0.00	1	x .	Final	TC.3
	8	1	1	-0.47	0.00	7	. x	Final	TC.3

**Check of concrete compressive stress**

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked.

Sigma.x,min Total maximal longitudinal compressive stress [MN/m<sup>2</sup>]  
 Sigma.x,zul = 0.60\*fck resp. 0.66\*fck for N.frequ. C. (NC) as per 4.4.1.2 (103)P  
 = 0.45\*fck for Q.-cont. C. (QC) as per 4.4.1.2 (104)\*P  
 (t,b) Position of the edge point: above, below of centre

Beam	No.	Se.	Loc.	Sigma.x,min [MN/m <sup>2</sup> ]	Sigma.x,per [MN/m <sup>2</sup> ]	Se.- Pnt.	Side t b	Period	Situation
	3	1	1	(I) -8.62	-15.48	7	. x	Constr.	NC.1
				(I) -8.62	-15.48	7	. x	Constr.	QC.1
	8	1	1	(I) -14.34	-15.48	7	. x	Constr.	NC.1
				(I) -14.34	-15.48	7	. x	Constr.	QC.1

**Check of diagonal principal tensile stresses for the frequent combination**

Sigma1,max Maximum principal tensile stress at state I [MN/m<sup>2</sup>]  
 Sigma1,per = fctk;0.05 as per II-4.4.0.3 (5/6)P [MN/m<sup>2</sup>]  
 y, z Edge point of the section with max Sigma1 [m]  
 t, b Position of the section point: above, below of centre  
 Thetal Angle between Sigma1 and the longitudinal stress Sigma.x [°]

Beam	No.	Se.	Loc.	Sigma1,max [MN/m <sup>2</sup> ]	Sigma1,per1 [MN/m <sup>2</sup> ]	Thetal [°]	Section point y[m] z[m]	Side t b	State
	3	1	1	0.59	2.45	19.79	-2.329 0.300	x .	Final
	8	1	1	0.00	2.45	0.96	-2.329 0.300	x .	Final

**Check of steel stress**

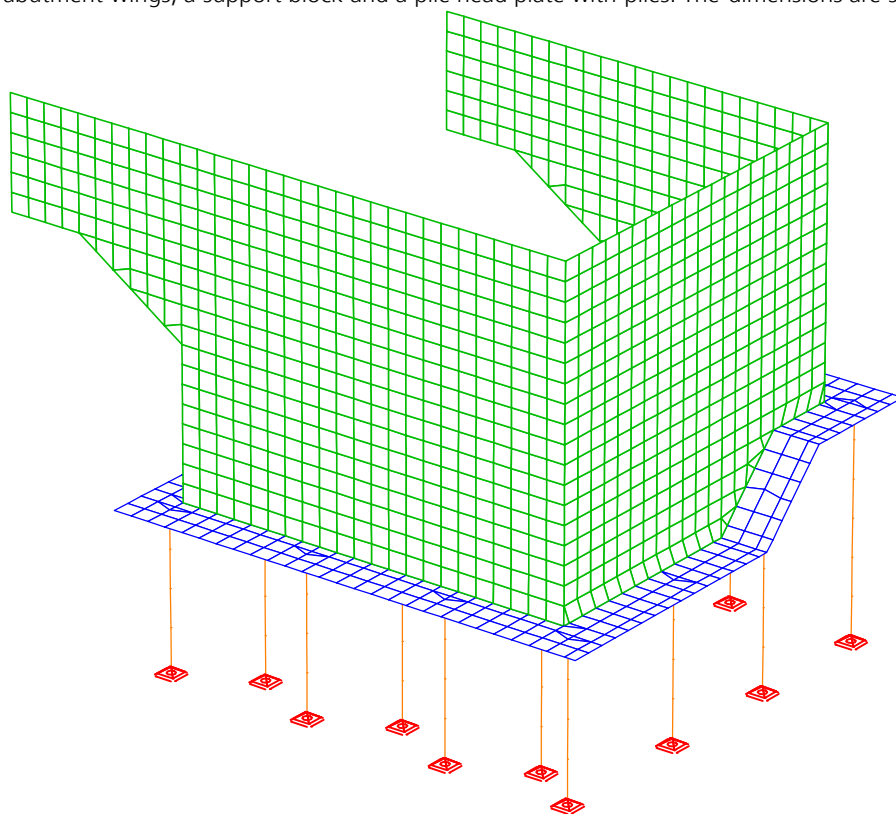
For the check, a cracked concrete section is assumed.  
 For tendon groups without bond and/or for situations before grouting, the prestressing steel stress is checked acc. to Eq. (4.6).

Type S Long. reinf. from N and M, layer number, N.frequ. C. (NC)  
 Type P Prestressing steel, Tendon number, Q.-cont. C. (QC)  
 Sigma.s,per = 0.80 \* fyk (NK) as per II-4.4.1.3 (105)  
 Sigma.p,per = 0.65 \* fpk (QK) as per II-4.4.1.4 (1)\*P

Beam	No.	Se.	Lo.	Steel Type	As [cm <sup>2</sup> ]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]	Situation
	3	1	1	S 1	0.00	.	400.00	NC.3
				S 2	0.00	.	400.00	NC.3
				S 3	28.25	-38.06	400.00	NC.3
				S 4	28.25	-38.06	400.00	NC.3
				P 1	159.60	1154.41	1275.00	QC.1
				P 2	159.60	1268.00	1275.00	QC.1
	8	1	1	S 1	0.00	.	400.00	NC.1
				S 2	0.00	.	400.00	NC.1
				S 3	28.25	-5.41	400.00	NC.3
				S 4	28.25	-5.40	400.00	NC.3
				P 1	159.60	1198.75	1275.00	QC.1
				P 2	159.60	1245.94	1275.00	QC.1

## Bridge Abutment

This example demonstrates how the DIN Technical Report applies to road bridge abutments. The construction consists of abutment wings, a support block and a pile head plate with piles. The dimensions are shown in the following figures.



*Static System*

### Material

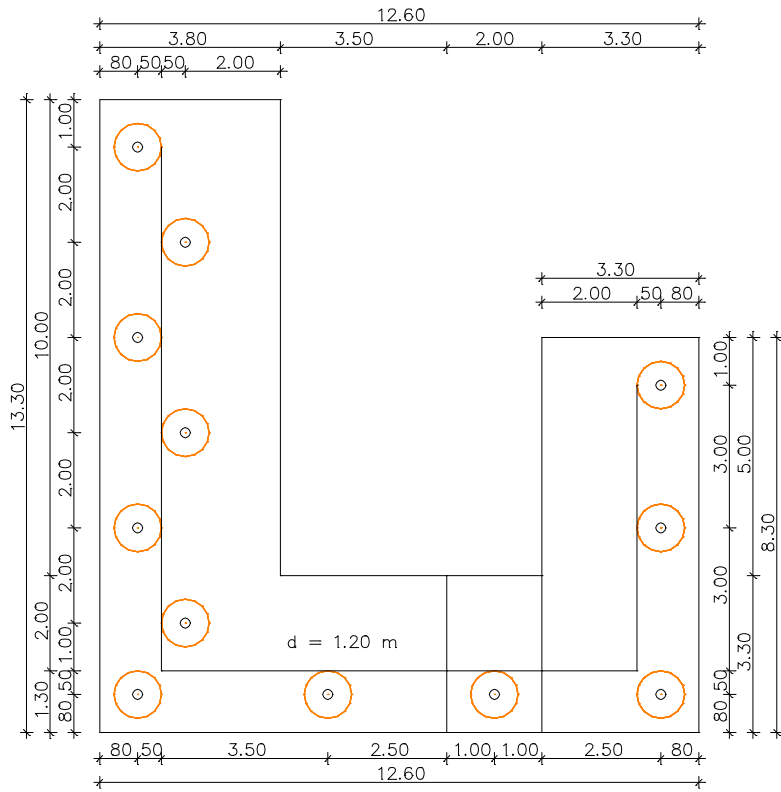
Concrete	C 30/37 (areas), C 35/45 (piles)
Reinforcing steel	BSt 500/550, axis distance from edge 8 cm
Bedding	50 MN/m <sup>3</sup> horizontal bedding module

### Section

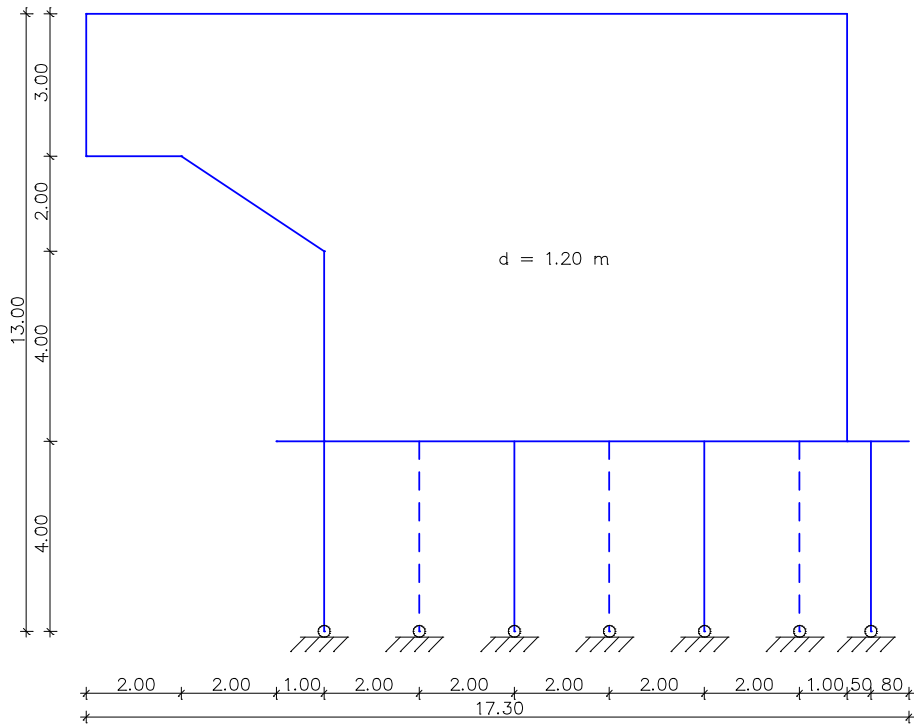
Wing walls	d = 1.2 m (section 1)
Support block	d = 1.5 m (section 2)
Pile head plate	d = 1.2 m (section 3)
Piles	∅ = 1.0 m (section 4)
Requirement class	D

The depicted FEM system is generated with shell elements in conjunction with beam elements. A 3D drawing consisting of edges and model surfaces formed the basis for creating the structure. It was important to position the model objects on the center lines or centroid levels of the structural components since they are used to describe the elements.

**Dimensions**

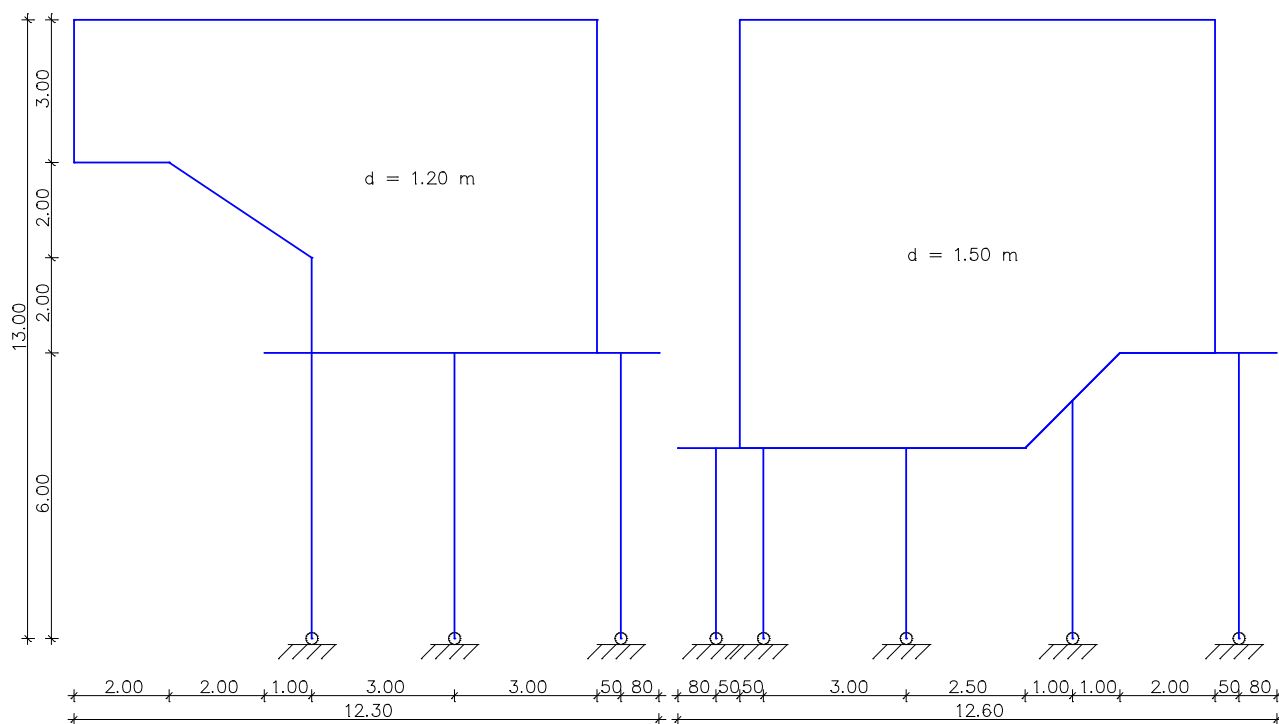


Top view of the pile head plate



View of the long wing wall





View of the short wing wall and the support block

### Checks

The following checks are carried out as a part of this example:

#### Ultimate limit state

- Bending with longitudinal force
- Lateral force
- Failure without warning (robustness)

#### Serviceability limit state

- Minimum reinforcement for the crack width limitation
- Crack width calculation
- Limiting the concrete compressive stresses
- Limiting the reinforcing steel stresses

Due to the intersection the checks are not carried out for the pile head plate. The required specifications are listed further below.

### Loads

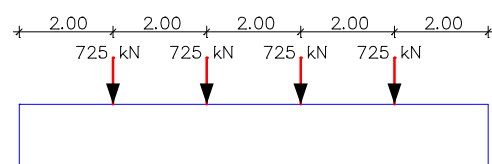
Due to the scope of the project, the actions were limited to the decisive loads.

- Load case 1 G: Dead load
- Load case 2 G: Soil pressure
- Load case 3 G: Superstructure
- Load case 4 Q: UDL (uniformly distributed load)
- Load case 5 Q: TS position 1
- Load case 6 Q: TS position 2

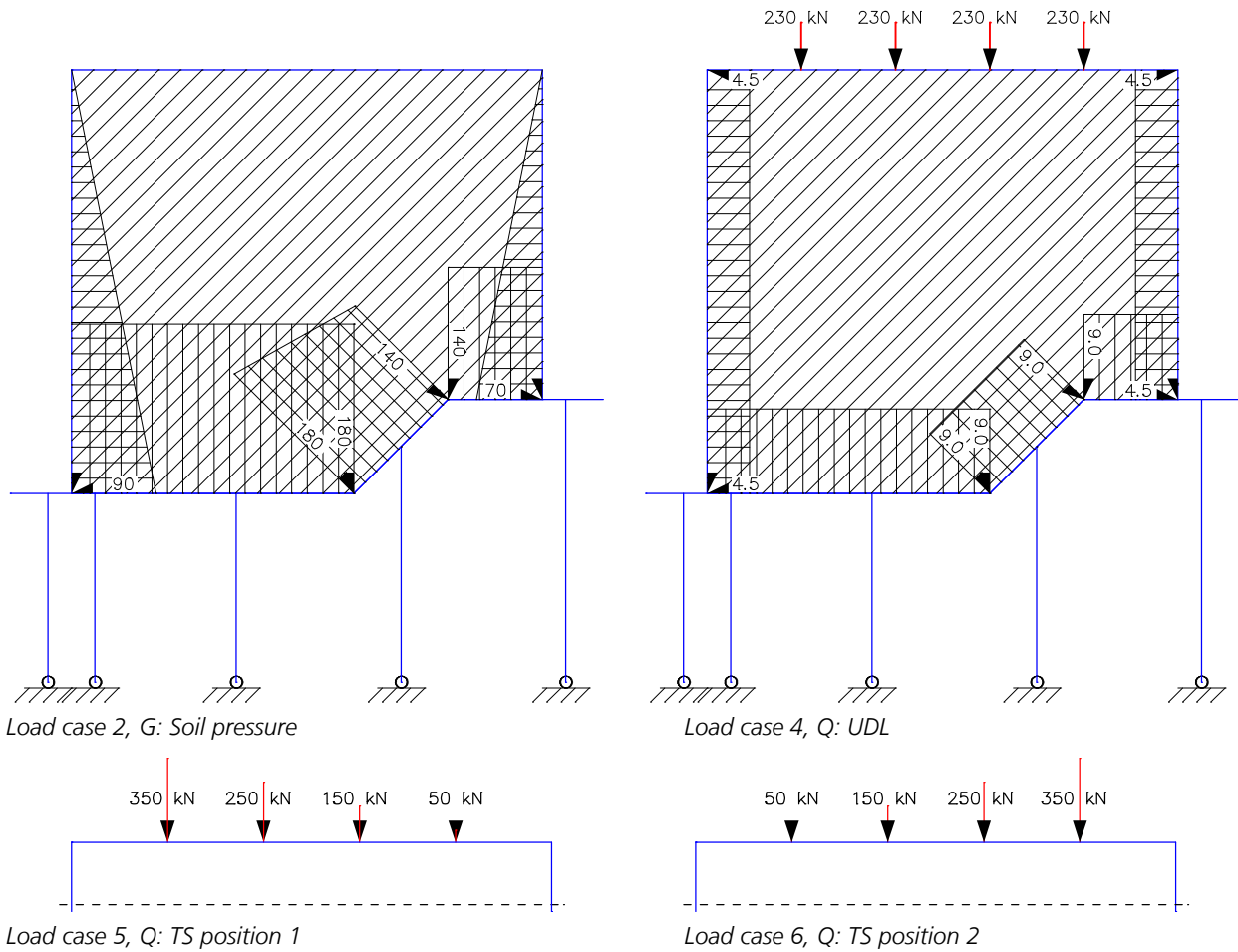
#### Load data load case 1: G: Dead load

No.	Dead load (EG) referring to material and cross-section properties weighting factor in direction		
	X [-]	Y [-]	Z [-]
1	0.0000	0.0000	1.0000

Load case 1, G: Dead load



Load case 3, G: Superstructure



The actions and action combinations defined for the checks are listed in the following.

**DIN Report 101 actions**

**G1 - Permanent action**

- 1 G: Dead load
- 2 G: Earth pressure

**G2 - Superstructure**

- 3 G: Superstructure

**QUDL - Traffic, load model 1 distributed load, 1. Variant, inklusive**

- 4 Q: Uniformly distributed load

**QTS - Traffic, load model 1 tandem system, 1. Variant, exklusive**

- 5 Q: TS position 1
- 6 Q: TS position 2

**Permanent and temporary combination, situations**

Variant	State	Actions
1)	Constr.	G1
2)	Final	G1 + G2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Permanent and temporary combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1.35	1
G2	1.35	1
QTS, QUDL	1.5	0

**Permanent and temporary combination, combination coefficients**

Variant	QTS	QUDL
a)	1	1

**Characteristic (rare) combination, situations**

Variant	State	Actions
1)	Final	G1 + G2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Characteristic (rare) combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
QTS, QUDL	1	0

**Characteristic (rare) combination, combination coefficients**

Variant	QTS	QUDL
a)	1	1

**Frequent combination, situations**

Variant	State	Actions
1)	Final	G1 + G2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Frequent combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
QTS, QUDL	1	0

**Frequent combination, combination coefficients**

Variant	QTS	QUDL
a)	0.75	0.4

**Not frequent combination, situations**

Variant	State	Actions
1)	Final	G1 + G2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Not frequent combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
QTS, QUDL	1	0

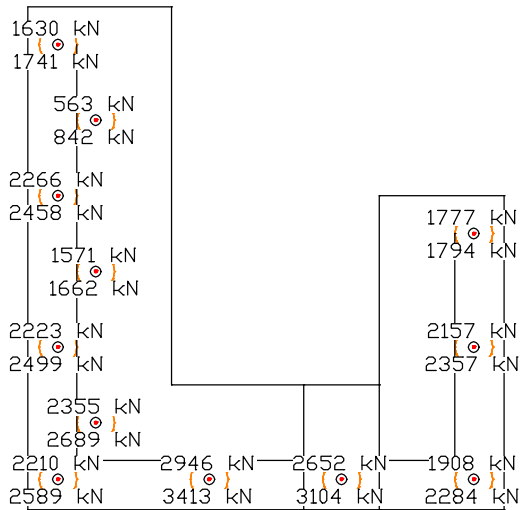
**Not frequent combination, combination coefficients**

Variant	QTS	QUDL
a)	0.8	0.8

Due to the low compressive forces and the high bending moments, the construction stage (G1) is decisive for load-bearing safety in several areas.

The following shows an example of the support reactions  $R_z$  for the characteristic (rare) combination.

1. Characteristic (rare) combination



Support reactions  $R_z$

Design according to DIN Technical Report 101 and 102 (2009-03)

Design overview

Se.	Class	Prestress of component	Box se. ARS	Reinforc. M R B Q T	Fatig. S P C	Crack- width	De- comp.	Stress checks Comp. Tens. S P
1	D	Not prestressed	.	x x x x .	. . . .	x	.	x . x .
2	D	Not prestressed	.	x x x x .	. . . .	x	.	x . x .
4	D	Not prestressed	.	. x x x .	. . . .	x	.	x . x .

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state.
- (T) Torsional reinforcement at ultimate limit state.
- (S) Reinforcing steel at stress and fatigue check.
- (P) Prestressing steel at stress and fatigue check.
- (C) Concrete at fatigue check.

Settings for flexural and shear reinforcement

- M,N Design mode for bend and longitudinal force: (ST) Standard, (SY) Symmetrical, (CM) Compression member.
- fyk Quality of stirrups.
- Theta Angle of concrete truss. Program-sided, the given value of cot Theta is limited to the value range according to Eq. (II-4.122).
- Slabs Beams are designed like slabs.
- Asl Given reinforcement according to picture 4.12, increase to maximum.
- Fac. Minimum reinf. min rhow = Factor \* rho with rho according to table 5.7.
- x,y Separate lateral force design for reinforcement directions x and y.
- cvl Laying measure of the long. reinforcement to limit the lever arm z.
- Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den- sity [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] Fig. 4.12	Fac. rhow	Dsn. cvl x,y [mm]	Red. pre- str.
1	C30/37	.	SY	500	1.75	.	0.00 0.00	0.60	. 80	.
2	C30/37	.	SY	500	1.75	.	0.00 0.00	0.60	. 80	.
4	C35/45	.	CM	500	1.75	.	0.00 .	1.00	. 80	.

Shear sections

- bw.nom Nominal width of the prestressed section acc. to II-4.3.2.2 (8)\*P.
- h.nom Nominal height of the prestressed section acc. to II-4.3.2.2 (8)\*P.
- kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d. z is limited according to II-4.3.2.4.2(2)\*.
- z1, z2 Height and width of the core section for torsion.
- teff Thickness of the torsion box.
- B. Box section; Load-bearing capacity is determined acc. to Eq. (4.47b).

Se.	Width [m]	Eff. width [m]	Height [m]	Eff.height [m]	Torsion section [m]					
bw	bw.nom	bn [m]	kb	h	h.nom	kd	z1	z2	teff	B.
1	1.000	.	.	1.200	.	1.120 0.90	.	.	.	.
2	1.000	.	.	1.500	.	1.420 0.90	.	.	.	.
4	0.707	.	0.627 0.90	0.707	.	0.627 0.90	0.547	0.547	0.160	.

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel.  
 max.s Maximal given bar spacing of the reinforcing steel.  
 Xil Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 sr,max Upper limit for the crack spacing from Eq. (4.203).  
 Method Direct calculation of the crack width as per Chapt. 4.4.2.4 or check by limiting the bar spacing according to Table 4.121.  
 TM Thick member according to Chapt. 4.4.2.2(9) to determine As,min.

Se.	wk,per	ds	max.s	Coeff.	sr,max	Concr.	age	Method for	Combination	TM
	[mm]	[mm]	[mm]	Xil	k	[mm]	As,min	wk	crack w.	for As,min
1	0.20	16	.	0.50	.	3- 5d	> 28d	Calcul.	as per class	.
2	0.20	16	.	0.50	.	3- 5d	> 28d	Calcul.	as per class	.
4	0.15	16	.	0.50	.	3- 5d	> 28d	Calcul.	as per class	.

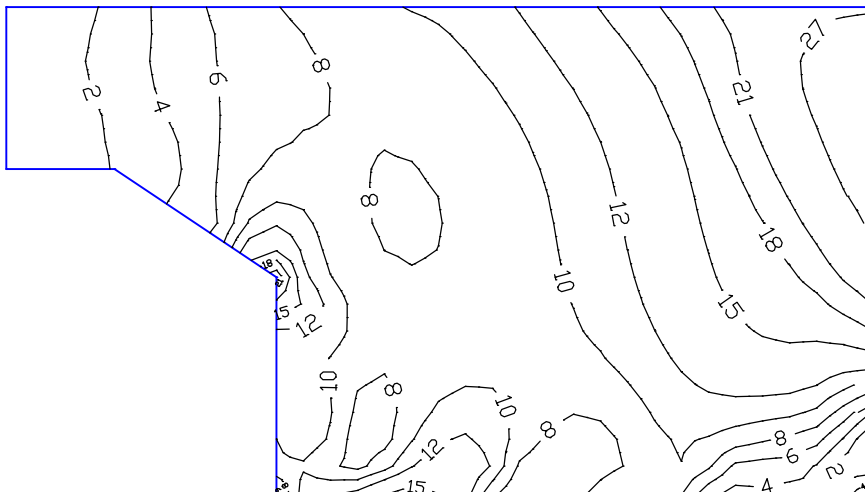
**Settings for the check of concrete stresses**

Av Full section area for stress calculation from longitudinal force.  
 fc(t) Compressive strength of concrete at the time t of prestressing.  
 Sigma.c(t) Concrete compressive stress at the time t of prestressing.  
 Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.x Concrete stress from bending and normal force in beam direction.  
 Sigma.l Diagonal principal tensile stress for area elements.  
 Sigma.x,y Concrete stresses for area elements in the internal forces system.  
 P.Tens. Check principal tens. stress only in the zone of long. compression.  
 (NC), (QC) Not frequent, Quasi-continuous combination.

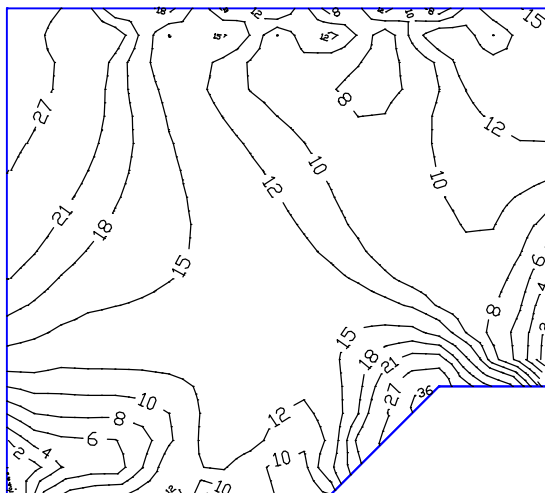
Se.	Av	fc(t)	per.sigma.c(t)	per.sigma.c	Dekompression	P.Tens. only
	[m <sup>2</sup> ]	[MN/m <sup>2</sup> ]	(NK, QK)	(NK)	stress	f. compr.zone
1	.	.	.	0.60 fck	.	.
2	.	.	.	0.60 fck	.	.
4	0.77	.	.	0.60 fck	.	.

**Results**

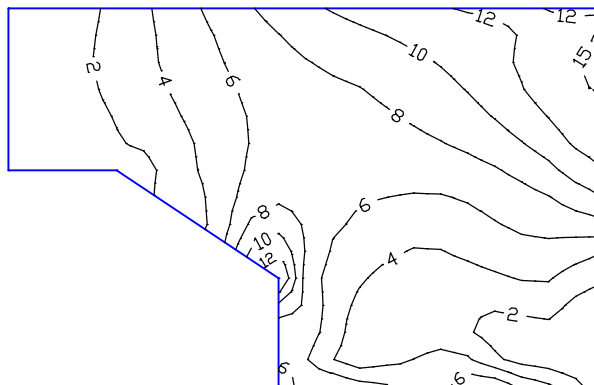
The following figures show the curve of the required longitudinal reinforcement.



Long wing wall

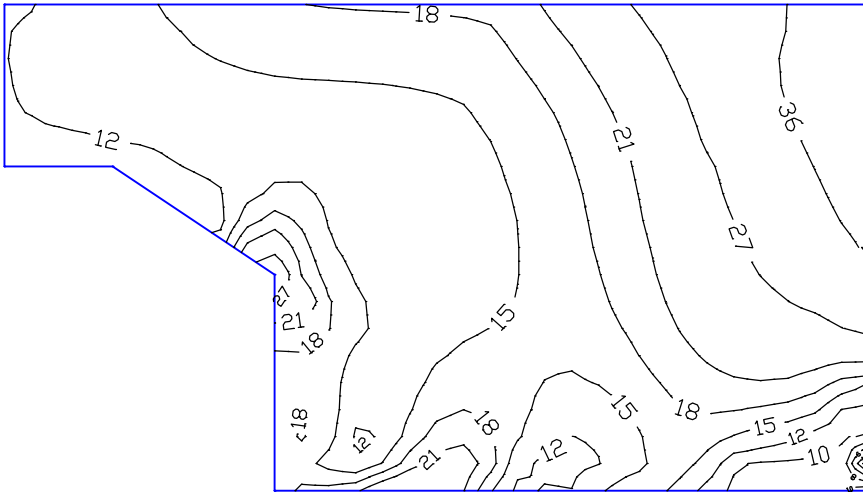


Support block

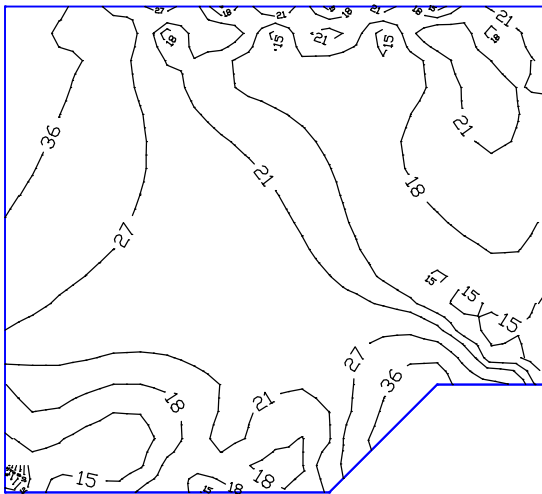


Short wing wall

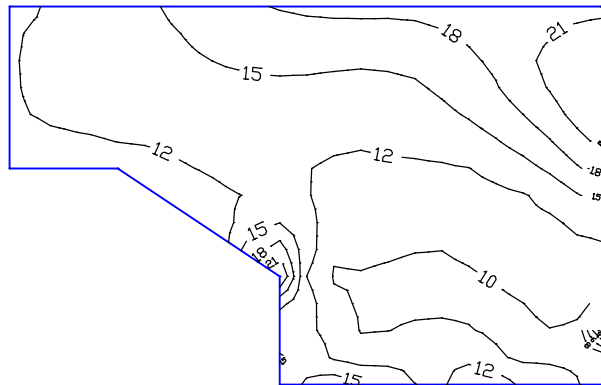
Required horizontal internal longitudinal reinforcement for the load-bearing capacity [cm<sup>2</sup>/m]



Long wing wall

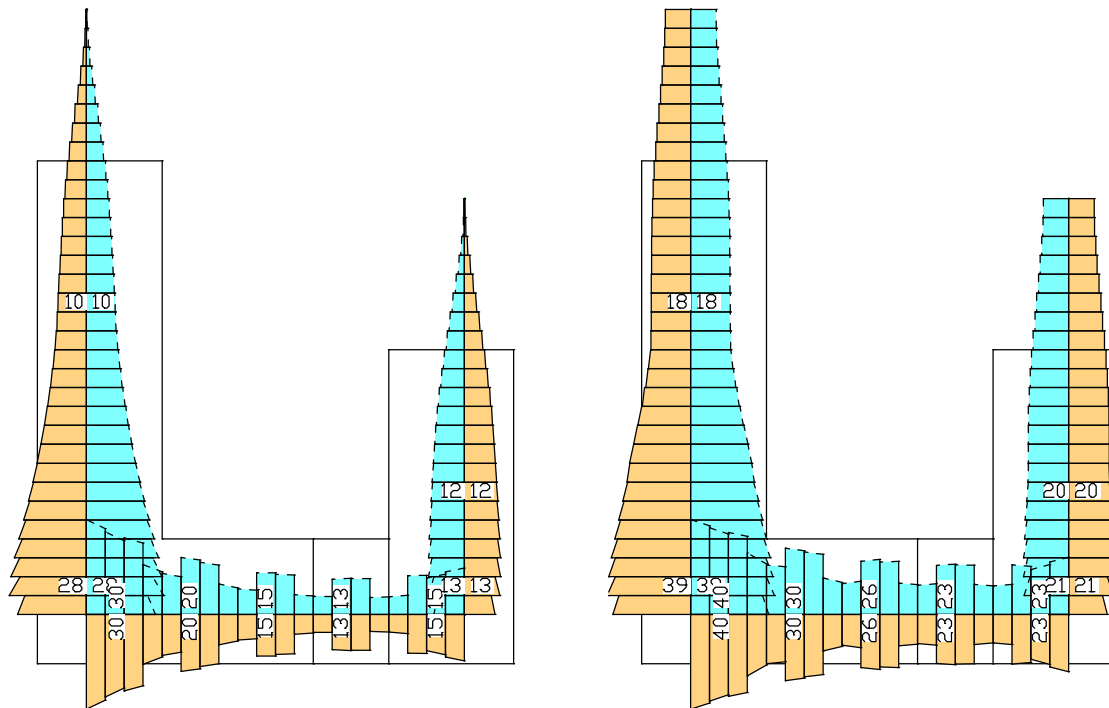


Support block



Short wing wall

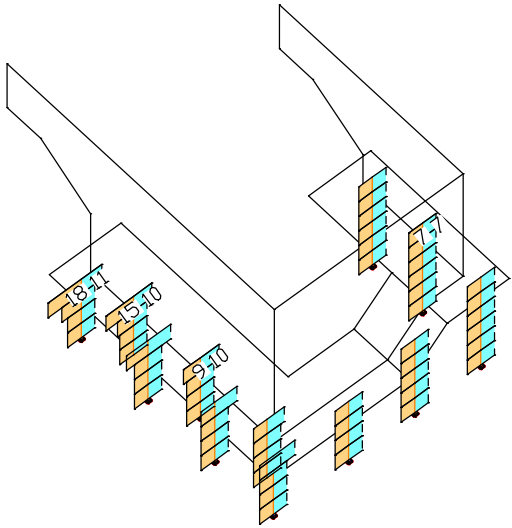
Maximum required horizontal internal longitudinal reinforcement [cm<sup>2</sup>/m]



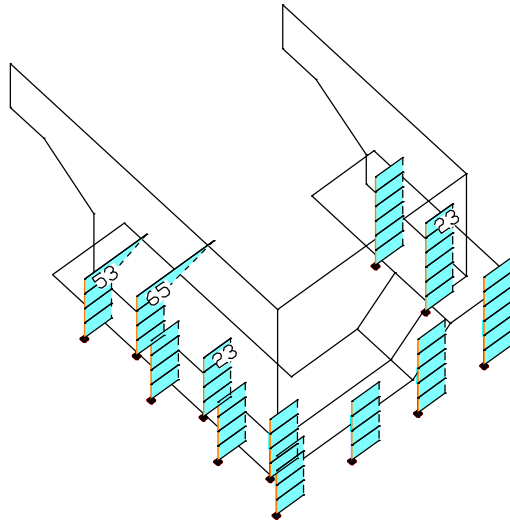
Reinforcement for the load-bearing capacity

Maximum required reinforcement

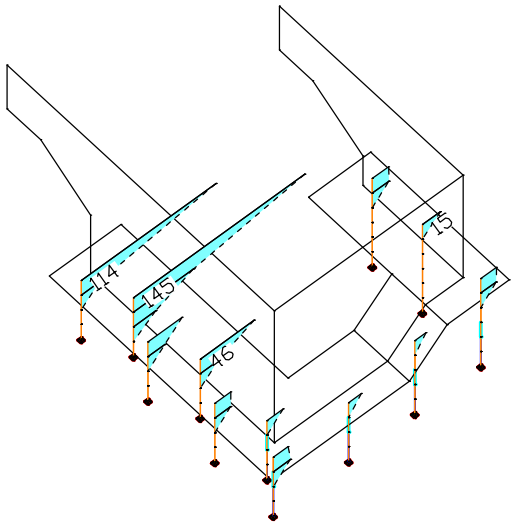
Horizontal longitudinal reinforcement along the upper edge [cm<sup>2</sup>/m]



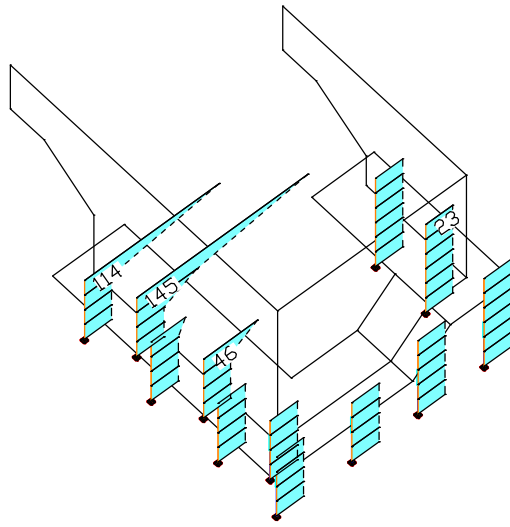
Lateral force reinforcement [cm<sup>2</sup>/m]



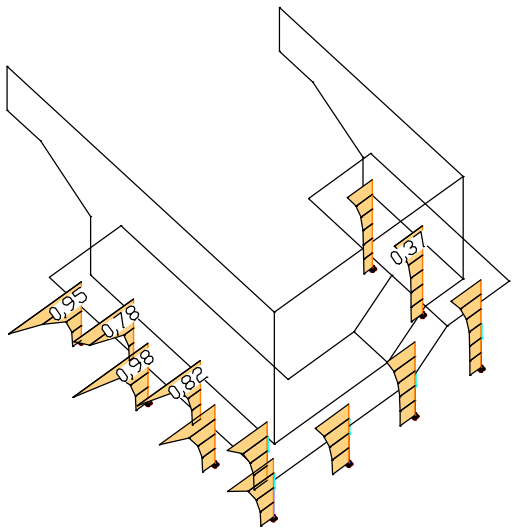
Load-bearing capacity reinforcement [cm<sup>2</sup>]



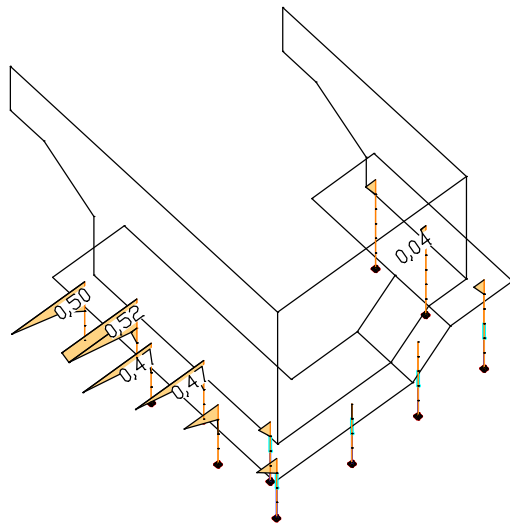
Crack check reinforcement [cm<sup>2</sup>]



Maximum required reinforcement [cm<sup>2</sup>]



Concrete compressive stresses  $\sigma_c / \sigma_{c,perm}$  [MN/m<sup>2</sup>]



Reinforcing steel stresses  $\sigma_s / \sigma_{s,perm}$  [MN/m<sup>2</sup>]

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# EN 1992-2 Bridge Checks

## Basics

The bridge checks as described in EN 1992-2 (Eurocode 2, Part 2) are designed to supplement the 3D FEM analysis. They can be used for bridges and other engineering constructions in which actions from street or railway traffic according to EN 1991-2 (Eurocode 1, Part 2) have to be taken into account. Permitted structure models include beam, area and solid structures. In detail the following standards are considered:

- EN 1992-2:2005/AC:2008 as the base document
- OENORM EN 1992-2:2012 with the National Annex Austria B 1992-2:2019-05
- DIN EN 1992-2:2010 with the National Annex Germany 2013-04.
- SS EN 1992-2:2005/AC:2008 with National Annex Sweden 2015-01 (EKS 9) in conjunction with SS EN 1992-1-1:2019 (EKS 11).
- BS EN 1992-2:2005/AC:2008 with the National Annex Great Britain 2005.

The desired rule is selected in the *Design Codes* dialog in the *Options* menu. The relevant entry, calculation and results dialogs appear depending on which rule is selected. When selecting the material the following alternatives are available:

- *C12/15-EN-D* to *C100/115-EN-D*, *LC12/13-EN-D* to *LC80/88-EN-D* and the user-defined material *CX-EN-D* for design in accordance with DIN EN 1992-1-1
- *C12/15-EN* to *C90/105-EN*, *LC12/13-EN* to *LC80/88-EN* and the user-defined material *CX-EN* for design in accordance with the other standards

Differing components can be combined in a structure model:

- Non-prestressed components
- Prestressed components with subsequent bond
- Prestressed components without bond
- Components with external prestressing
- Mixed-construction components

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases and variants of load models to the actions in accordance with EN 1991-2:2003. The program will take into account the preset safety factors and combination coefficients defined in EN 1990:2021 (Eurocode 0) for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements.

The actions and check selection dialogs can be opened from the analysis settings. Detailed check specifications and reinforcement data must be entered during section definition.

For beams and design objects, all checks are carried out at the polygon section. In addition, composite sections can be verified in the ultimate limit state. For general notes on using design objects, refer to the relevant chapter in the manual.

In the *EN 1992-2 Bridge Checks* folder of the database and the national variants folders, a single design can also be performed for the user-defined polygon and composite sections.

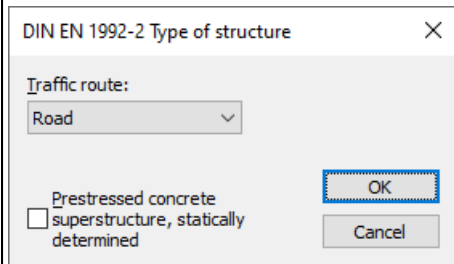
The EN 1992-2 guidelines are primarily cited for the following comments. Reference to the relevant national settings is only made if they contain different or complementary rules. The passages in question are marked by a vertical line left of the text.

# Input

## Type of Structure

DIN EN 1992-2:

The dialog for selecting the structure type is opened from the database or the *Settings* in the *Analysis* menu. You can choose either the road, railway or combined traffic route.



### Prestressed concrete superstructure

Selection of the check conditions for road bridges according to Table 7.101DE for in longitudinal direction statically determined members with tendons in bond.

## Load Model 1 for Road Bridges

Load model 1 consists of two parts according to EN 1991-2, Section 4.3.2:

- Double-axle (TS tandem system)
- Uniformly distributed area load (UDL)

These loads should be applied in both the longitudinal and lateral directions of the bridge in the least favorable position. In the lateral direction, the load positions are determined by dividing the roadway into computational lanes. Since the decisive lane division is not always known in advance, you can define different load position variants.

Load model 1 can be edited from the *EN 1992-2 Bridge Checks / Load model 1* folder in the database. Select *New* from the context menu of load model 1 to create a new variant of load positions and open the corresponding input dialog.

The adjusted base values of the tandem system ( $\alpha_{Qi} \cdot Q_{ik}$ ) and the load ordinates of the UDL loads ( $\alpha_{qi} \cdot q_{ik}$ ) can be set by clicking *Properties* in the context menu of load model 1.

Properties load model 1		
Location	Tandem system TS $\alpha_{Qi} \cdot Q_{ik}$ [kN]	UDL system $\alpha_{qi} \cdot q_{ik}$ [kN/m <sup>2</sup> ]
Lane 1	<input type="text" value="240"/>	<input type="text" value="9"/>
Lane 1	<input type="text" value="160"/>	<input type="text" value="2.5"/>
Lane 3	<input type="text" value="80"/>	<input type="text" value="2.5"/>
Other lanes		<input type="text" value="2.5"/>
Remaining area (qrk)		<input type="text" value="2.5"/>
<input type="checkbox"/> Tandem systems in cross section decoupled <input type="checkbox"/> Use projective loads		
		<input type="button" value="OK"/> <input type="button" value="Cancel"/>

During FEM calculation, all vertical load portions of load model 1 are applied as area loads on area or solid elements in the global z-direction. They are calculated in separate load cases to allow for later determination of the extremal reactions. With the option *Use projective loads* the loads are projected perpendicular to their surfaces in local t-direction (see *Free area load - polygon*). With projected loads, it is usually useful to arrange the load above the elements and orient its local t-direction according to the global z-direction. Only elements are loaded, which are not hidden by other elements. This also applies to a partial overlap.

Load model 1:

The video "[https://download.infograph.de/video\\_de/LM%201%20eingegeben.mp4](https://download.infograph.de/video_de/LM%201%20eingegeben.mp4)" (German language) shows the definition of load model 1.

### Off

Exit the load model 1 screen.

### TS New

Enter a new position of the tandem system (see also *Tandem system*). The positions of the tandem system are mutually exclusive.

### UDL New

Consecutive input of rectangular or triangular load areas of the UDL load. The partial areas of the UDL load can act simultaneously. The input for UDL2 (lane 2) is preset. You can select a different lane or remaining area from the context menu of the load area.

UDL (2) ✕

Location:  
Lane 1 ▼

Point	x [m]	y [m]	z [m]
1	0	0.5	0
2	14.6	0.5	0
3	14.6	3.5	0
4	0	3.5	0

### UDL Gen

Generate new load areas of the UDL load. The rectangular generation area is separated into the same number of load areas on the opposite edges.

## Tandem System of Load Model 1

Tandem system (1) ✕

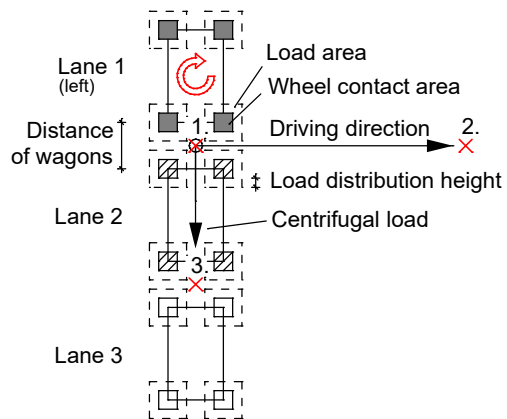
Distance of wagons:  m

Lanes:  ▼

Centrifugal load:  kN

Load distribution height:  m

Point	x [m]	y [m]	z [m]
1	1	2.2	0
2	4	2.2	0
3	1	3.2	0



Input dialog and schematic diagram of the tandem system in load model 1 according to EN 1991-2.

### Distance of wagons

Distance of the centroid of the wheel-ground contact area (see figure).

### Lanes

Lane arrangement from left to right.

### Centrifugal load

Load amount  $Q_t$  of this tandem system position. The point load acts at point 1 perpendicular to the direction of traffic. Its eccentric location is not considered. In the *Use projective loads* mode, the centrifugal load is an area load that is distributed evenly over the projected load areas.

### Load distribution height

Yields the load areas of the tandem system in conjunction with the wheel-ground contact area (see figure).

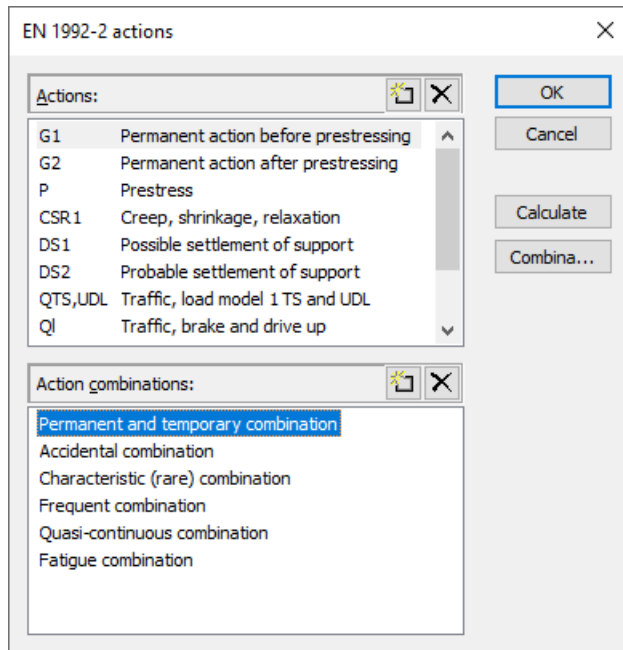
### Point

1. Reference point of the tandem system
2. Point of the local x axis (direction of traffic)
3. Point in the local xy plane

## Actions and Action Combinations

The design values of the load are calculated based on the internal forces of individual load cases and load case combinations. For this the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the decisive action combinations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Actions

List of defined actions. The following actions can be defined:

- *G1* to *G9*: Permanent actions
- *P*: Prestressing
- *QTS,UDL*: Traffic, load model 1 TS and UDL
- *Ql*: Traffic, braking and starting
- *Qt*: Traffic, centrifugal load
- *T*: Temperature
- *W*: Wind
- *DS1*: Possible support displacement
- *DS2*: Probable support displacement
- *CSR1, CSR2*: Creep, shrinkage and relaxation at various times
- *A*: Accidental action
- *E*: Earthquake
- *Q1* to *Q9*: Other variable action
- *Qfat*: Cyclic fatigue action

In accordance with EN 1990/A1, Table A.2.4(B), Note 2, the actions *DS1* and *DS2* and also the actions *Q*, *T* and *W* are only taken into account in the design situations if they have an unfavorable effect.

DIN EN 1992-2:

The actions *QTS*, *QUDL*, *Ql* and *Qt* are only available when selecting the *Road* traffic route.

### Action combinations

List of defined action combinations.



Insert a new action or action combination.



Delete the selected action or action combination.

## Calculate

Calculate the defined action combinations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all combinations in the database. This allows you to evaluate the results without having to execute the checking module.

Each time you execute the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

The following table demonstrates how the combinations are used in the various checks.

Situation	Ultimate limit state	EN 1992-1-1	EN 1992-2
Permanent & temp.	Longitudinal reinforcement	6.1	3.1.6
Accidental	Lateral reinforcement	6.2	
Earthquake	Torsion reinforcement	6.3	
Characteristic (rare)	Robustness reinforcement		6.1 (110)
Frequent	Fatigue simplified	6.8.6 (2)	
Fatigue	Concrete	6.8.7 (1)	NN.3.2
	Reinforcing steel	6.8.4	NN.2.1
	Prestressing steel	6.8.4	NN.3.1
Situation	Serviceability Limit State	EN 1992-1-1	EN 1992-2
Characteristic (rare)	Concrete compressive stress		7.2 (102)
	Reinforcing steel stress	7.2 (5)	
	Prestressing steel stress	7.2 (5)	
Frequent	Decompression class XC2-XS3		7.3.1
	Crack width, prestressing with bond		7.3.1
Quasi-continuous	Concrete compressive stress	7.2 (2)	
	Crack width, reinforced concrete and prestressing without bond		7.3.1
	Deformations	7.4	

DIN EN 1992-2:

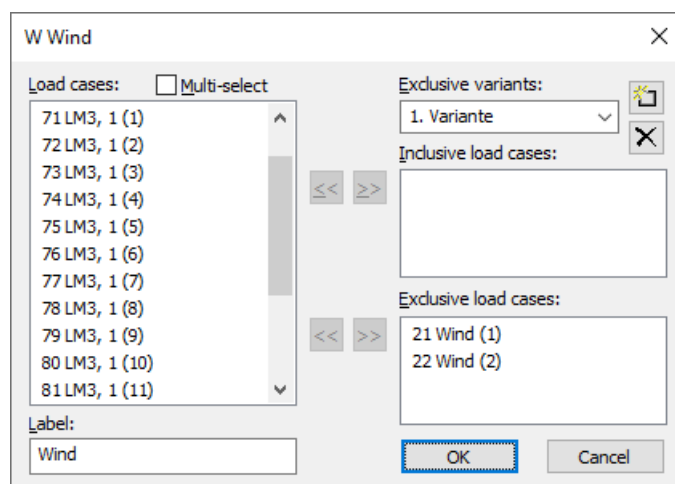
For the decompression check and the control of the permitted edge stresses and crack widths, special action combinations according to Tab. 7.101DE, footnote a and b (road bridges), and Tab. 7.102DE, footnote a and b (railroad bridges), are available.

OENORM EN 1992-2:

For the decompression check and the control of the crack widths, special action combinations according to Tab. 2AT, footnote c, are available.

## Definition of an Action

The load cases are assigned to the actions after an action is selected or a new one is entered. Depending on the action type, you have access to a variety of dialogs of which one example is shown below. The available dialog options are explained at the end.



### Load cases

Select load cases or load case combinations to add them to the current action.

**Multi-select**



Load cases and combinations can be added to the actions more than once.

**Label**

User-defined label for the action.

**Exclusive variants**

An exclusive variant consists of both inclusive and exclusive portions. The variants are mutually exclusive.

The buttons  or  are used to add or delete action variants.

**Inclusive load cases**

Selected load cases and combinations that can have a simultaneous effect.

**Exclusive load cases**

Selected load cases and combinations that are mutually exclusive.

**Use moving loads**

The actions of load model 1 can be recorded using either *moving loads* or *normal* load cases. The *moving loads* are described in a separate dialog (see *Load model 1*). When using *normal* load cases, it is assumed for the sake of simplicity that the vertical ( $Q_{TS}$ ) and horizontal ( $Q_L$ ) actions of the tandem system act independently. If the individual vehicles of a tandem system are described in different *normal* load cases, they must be assigned to the corresponding lane. In the combination, a maximum of one of its load cases is taken into account on each lane. This means that the tandem system consists of a maximum of three simultaneously acting load cases.

**Prestressing loss from relaxation of prestressing steel**

The prestressing loss is defined as a constant percentage reduction of prestress.

**CS as constant reduction of prestress**

As an alternative to defining CS load cases, you can allow for the effect of creep and shrinkage by defining a constant percentage reduction of prestress.

**Internal prestressing**

Selected load cases that describe internal prestressing. The reactions of the individual load cases are added together.

**External prestressing**

Selected load cases that describe external prestressing. The reactions of the individual load cases are added together.



## Definition of an Action Combination

Depending on which check is selected, different action combinations are necessary. You can enter them using the following dialog.

**Permanent and temporary combination**

Situations:

Variant	State	Actions
1)	Constr. - Ungr.	G1 + P
2)	Final	G1 + G2 + P + CSR1 + DS1 + QK

Partial safety factors:

Action	Gamma.sup	Gamma.inf
G1	1.35	1
G2	1.35	1
P, CSR1	1	1
DS1, DS2	1.2	
QTS, QUDL, QI	1.35	
T	1.5	
W	1.5	

Combination values:

Variant	QTS	QUDL	QI	T	W
a)	1.0	1.0	0	0	0.6
b)	1.0	1.0	0	0.6	0
c)	0.75	0.4	0	1.0	0
d)	0	0	0	0	1.0
e)	0.75	0.4	1.0	0.6	0

### Situations

List of design or check situations. Each situation can be valid for either the construction stage or the final state. For prestressed components with subsequent bond the tendons can be set ungrouted. The *QK* action indicates variable actions based on the table of combination values. The buttons or allow you to add or delete situations. By double-clicking on a situation it can be modified subsequently.

### Partial safety factors

Table of partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$  for the actions. For actions P and CSR, the country-specific coefficients according to EN 1992-1-1, Chapter 2.4.2.2 (1), and for the other actions the nationally valid values according to EN 1990/A1, Table A.2.4 (B), are proposed.

DIN EN 1992-2:

In accordance with 2.3.1.3 (4) a partial safety factor for settlements  $\gamma_{G,Set} = 1.0$  can be assumed for concrete bridges.

SS EN 1990 (EKS 11):

The program suggests the partial safety factors as they result in accordance with Section A, Article 11, for safety class 3 from  $\gamma_d \cdot \gamma_{sup}$  with the reduction factor  $\gamma_d = 1.0$  as per Article 14. If required, lower safety classes can be taken into account entering lower values.

BS EN 1990:

If necessary, the proposed safety factors shall be adapted to the special features of Table NA.A2.4 (B).

### Combination Values

Table of the combination coefficients for the variable actions. The nationally valid values are suggested analogously to EN 1990/A1, Table A.2.1 (road bridges). The buttons or allow you to add or delete combination variants. For the calculation only the variants listed here are taken into account.

### Standard

Recommended values are assigned to the safety and combination coefficients. All relevant actions of the final state are selected for the design situation.

## Partial Safety Factors

The partial safety factors of the construction materials are preset with the nationally applicable values as specified in EN 1992-1-1, Table 2.1. In the design situations due to earthquakes, the safety factors of the accidental design situation may be assumed in accordance with EN 1998-1, Chapter 5.2.4 (3), if the strength loss is taken into account when determining the material properties. Otherwise, the factors of the permanent and temporary design situation must be applied in accordance with Chapter 5.2.4 (2). The additional safety factor against brittle failure according to EN 1998-2, Chapter 5.6.2 (2)P, is assumed to be  $\gamma_{Bd1} = 1$ .

The partial safety factors for the actions are specified in the definition of the action combinations.

OENORM B 1998-1:

In design situations resulting from earthquakes, the factors for construction materials according to OENORM B 1998-1, Chapter 5.2.4 (3), apply.

DIN EN 1998-1:

In the design situations due to earthquakes, according to the NDP to 5.2.4 (1) and (3), the safety factors of the permanent and temporary design situation generally apply.

## Section Inputs

The section inputs contain all of the specific settings made for checks in the ultimate limit and serviceability states. In addition to these specifications, the selected material properties and the properties of the reinforcing steel are also relevant for the design. An overview of the design specifications can be accessed in the *EN 1992-2 Bridge Checks* folder of the database and in the folders of the national variants.

## Checks

The following dialog is used to define which checks are available for the cross-section in the ultimate, fatigue and serviceability limit states. For composite sections, the selection is limited to the load-bearing capacity checks. The analysis settings allow to override this selection for the entire structure.

Properties for element 381 - EN 1992-2 - Checks

Section: Form, Shear stresses, Material, Bedding, EN 1992-1-1, EN 1992-2 (expanded), Checks (expanded), Base values, Shear section, Shear joint, Stresses, Crack width, Fatigue, Variation coefficients, Thermal analysis, General, Isolated column, Eccentricity

Number: 11 - M<sub>i</sub> | Section Type: Polygon | Material Type: C40/50-EN | New | Copy | Delete

Label: Main girder | Properties concern further elements.

Prestress of component: Subsequent bond | Exposure class: XC4

Ult. limit state:  Bend, long. force |  Lateral force |  Torsion |  Robustness |  Shear joint

Serviceab. limit state:  Concrete comp. stress |  Decompression |  Minimum crack reinf. |  Reinf., prestr. steel |  Lateral force |  Torsion |  Reinf. tensile stress |  Crack width |  Concr. long. stress |  Concr. comp. strut

OK | Cancel | Help

Check selection for EN 1992-2 (national variants corresponding)

### Prestressing of the component

The type of prestressing can be selected for each section separately:

- not prestressed
- subsequent bond
- without bond
- external
- mixed construction

## Exposure class

The check conditions for the decompression and crack width check are grouped by exposure class in EN 1992-2, Table 7.101N. A component can be assigned to an exposure class based on the information provided in EN 1992-1-1, Table 4.1.

DIN EN 1992-2:

For the check conditions Table 7.101DE (road bridges) and Table 7.102DE (railroad bridges) are decisive. The selection can be made in the *Type of Structure* dialog.

OENORM B 1992-2:

Table 2AT is decisive for the check conditions.

SS EN 1992-2:

In addition, the service life class as per SS EN 1992-1-1, Article 10 (EKS 11), can be selected to determine the crack width according to Table D-5 and the crack safety factor according to Table D-3 (EKS 11).

## Robustness

This check determines the minimum reinforcement against failure without warning (robustness reinforcement) based on EN 1992-2, Chapter 6.1 (109), Equation (6.101a).

## Steel tensile stresses

For components with internal prestressing, both the prestressing steel stresses and the stresses of the longitudinal reinforcement are checked.

## Minimum crack reinforcement, crack width

The crack width check is carried out according to EN 1992-1-1, Chapter 7.3.4. In this check the final longitudinal reinforcement is set as the maximum value from the bending reinforcement, robustness reinforcement and minimum crack reinforcement as per 7.3.2. The latter will be increased automatically if necessary to maintain the crack width.

## Base Values

Unless otherwise specified, the base values apply for all checks in the ultimate, fatigue and serviceability limit states.

**Properties for element 381 - EN 1992-2 - Base values**

Section: Form, Shear stresses, Material, Bedding, EN 1992-1-1, EN 1992-2 (Checks, Base values, Shear section, Shear joint, Stresses, Crack width, Fatigue, Variation coefficients, Thermal analysis), General, Isolated column, Eccentricity

Number: 11 - M<sub>i</sub> | Section Type: Polygon | Material Type: C40/50-EN | New | Copy | Delete

Label: Main girder | ⚠ Properties concern further elements.

Design mode for bend and longitudinal force: Standard | Factor for as in secondary dir.: | Reduction factor of prestr. for robustness: |  Design without considering given reinforcement ratios

Lateral force and torsion

Eff. height d [m]: | Angle cot Theta: 2.5 | Quality of stirrups: 500A | 500 | Factor for rho.w,min: 1

Asl acc. to Fig. 6.3 [cm<sup>2</sup>]: 0 | Asl extension to [cm<sup>2</sup>]: |  Design like slabs |  Design as circular cross-section

OK | Cancel | Help

**Design mode**

- *Standard*: Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary.
- *Compression member*: For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to EN 1992-1-1, Section 9.5.2 (2).

**Factor for as in secondary direction**

According to EN 1992-1-1, Section 9.3.1.1 (2), secondary longitudinal reinforcement of one-way slabs should not be less than 20% of the principal reinforcement. The examination is carried out on the program side with the results of the bending design separately for the upper and lower side of the cross-section. The direction with the largest amount of reinforcement per cross-sectional side defines each principal reinforcement direction. The assignment of the factorized reinforcement in secondary direction then takes place via corresponding reinforcement layers.

DIN EN 1992-1-1:

In the case of two-way slabs, the less stressed direction should be reinforced with at least 20% of the higher stressed direction.

**Reduction factor of prestr. for robustness**

The regulations of Chapter 6.1 (110) are decisive for the arrangement of the robustness reinforcement. Thus for the determination of the tensile zone the statically determined effect of prestressing is not taken into account. Because this cannot be determined for area elements the prestress can alternatively be reduced by a reduction factor. The specification of an appropriate value is subject to the discretion of the user.

**Design without considering given reinforcement ratios**

If selected, the reinforcement increase required in the design is performed without taking into account the reinforcement ratios specified by the basic reinforcement.

**Effective height**

Effective static height for the shear design of area elements [m].

**Angle cot Theta**

$\cot \Theta$  defines the concrete strut angle according to EN 1992-1-1, Chapter 6.2.3 (2), Equation (6.7N). The program will suggest a value of 1 (45° strut angle). You can choose to ignore the suggestion and pick any value within the permissible national limits. Entering a higher number will normally result in a lower necessary lateral force reinforcement  $A_{sw}$ , a lower absorbable lateral force  $V_{Rd,max}$  and a larger displacement  $a_1$  according to Chapter 9.2.1.3, Equation (9.2).

DIN EN 1992-2:

The strut angle is limited to  $1.0 \leq \cot \Theta \leq 1.75$  according to Eq. (6.107aDE).

Four calculation methods can be chosen for the check:

- *Standard*: The input value is limited to the range permitted in accordance with DIN EN 1992-1-1, Eq. (6.7aDE) for lateral force, torsion and combined loads (method with load-dependent strut angle).
- *Constant*: The check is carried out using the chosen value for  $\cot \Theta$  without further limitations (cf. interpretation No. 24 of NABau for DIN 1045-1).
- *Std./45°*: For lateral force  $\cot \Theta$  is limited according to DIN EN 1992-1-1, Eq. (6.7aDE). For torsion a constant strut angle of 45° is assumed for simplification according to Chapter 6.3.2 (102).
- *Std./30°*: For torsion a constant strut angle of 30° is assumed.

The actual effective angle of the concrete struts is logged for each check location.

OENORM B 1992-1-1:

The concrete strut angle is defined by  $\tan \Theta$  and should be limited according to equations (3AT) and (4AT).

SS EN 1992-1-1:

According to Article 15 and differing from Equation (6.7N), for prestressed components the condition  $1.0 \leq \cot \Theta \leq 3.0$  applies.

**Asl acc. to Fig. 6.3**

The bending tensile reinforcement to be taken into account according to EN 1992-1-1, Chapter 6.2.2, Figure 6.3 [cm²].

**Asl extension to**

You can optionally specify a maximum value for area elements and the program will automatically increase the above input

value until that maximum value is reached in order to avoid stirrup reinforcement [cm<sup>2</sup>].

### Quality of the stirrups

- 420S: Reinforcing rod with  $f_{yk} = 420$  MN/m<sup>2</sup>.
- 500A: Reinforcing rod with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- 500M: Reinforcing meshes with  $f_{yk} = 500$  MN/m<sup>2</sup>.
- *General information:* Freely definable steel quality [MN/m<sup>2</sup>].

### Design like slabs

Beams or design objects are treated like slabs, which means that a minimum lateral force reinforcement will not be determined as per EN 1992-1-1, Chapter 6.2.1 (4), if no lateral force reinforcement is required for computation.

### Factor for $\rho_{w,min}$

The minimum reinforcement level  $\rho_{w,min}$  is defined using a factor related to the standard value for beams according to EN 1992-1-1, Chapter 9.2.2 (5).

DIN EN 1992-1-1, OENORM B 1992-1-1:

For slabs with  $V_{Ed} > V_{Rd,c}$  at least the 0.6-fold value of the minimum shear reinforcement of beams is necessary.

DIN EN 1992-1-1:

For structured sections with prestressed tension chord the 1.6-fold value is to be applied according to Equation (9.5bDE).

SS EN 1992-1-1:

If the fire safety class is 1 or 2 and no shear reinforcement is required,  $\rho_{w,min}$  can be set to zero as per Article 26.

### Design as circular cross-section

For circular and annular cross-sections, the lateral force design according to Bender et al. (2010) can be selected as an alternative for the resulting shear force  $Q_T = \sqrt{(Q_y^2 + Q_z^2)}$ . The corresponding inputs are made on the *Shear Section* dialog page.

### Laying measure $c_{v,l}$

DIN EN 1992-2:

In DIN EN 1992-1-1, Chapter 6.2.3 (1), the inner lever arm  $z$  is limited to the maximum value derived from  $z = d - 2c_{v,l}$  and  $z = d - c_{v,l} - 30$  mm. Note that  $c_{v,l}$  is the laying measure of the longitudinal reinforcement in the concrete compressive zone. For  $c_{v,l}$  the program will suggest the smallest axis distance of the longitudinal reinforcement to the section edge  $d_1$ .

### Separate check for x and y direction

DIN EN 1992-2:

For two-axes stressed slabs, the lateral force check can be performed separately in the x and y stress directions as described in Chapter 6.2.1 (10) of DIN EN 1992-1-1. The user is responsible for properly aligning the reinforcement directions.

### Lever arm from bending design

DIN EN 1992-2:

The lever arm  $z$  for lateral force design of area elements is normally assumed to be  $0.9 \cdot d$ . Alternatively, the lever arm from bending design can be used. The program determines the maximum lever arm at each check location for both reinforcement directions depending on the design situation and limits the value to the range of

$$0.1 \cdot d \leq z \leq \max(d - 2c_{v,l}; d - c_{v,l} - 30 \text{ mm}).$$

When the check is performed for the resulting force  $q_T$  the minimum from  $z_x$  and  $z_y$  is used.

### Check of S2 in state I

DIN EN 1992-2:

Deviating from DIN EN 1992-2, Chapter 6.3.2 (NA.106), for *box sections* the principal compressive stress  $\sigma_2$  is calculated basically in state I from the longitudinal stress  $\sigma_{Ed}$  and the shear stress from torsion  $\tau_{Ed,T} = T_{Ed} / (2 \cdot A_k \cdot t_{eff})$  according to DIN Technical Report 102:2003, Chapter 4.3.3.2.2 (2).

## Shear Section

For polygon and composite sections, additional section dimensions are required for the lateral force and torsion design. These are explained in the following. In case of sections with internal prestressing or with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.

### Width

Section width for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Height

Section height for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Effective height

Effective static height for calculating the lateral force load-bearing capacity for  $Q_z$  [m].

### Effective width

Effective static width for calculating the lateral force load-bearing capacity for  $Q_y$  [m].

### Nominal width, nominal height

The nominal width or height of internally prestressed components as per EN 1992-1-1, Chapter 6.2.3 (6), for including the duct diameter in the calculation of the design value of the lateral load-bearing capacity  $V_{Rd,max}$ .

### Factor kb, Factor kd

Factor for calculating the inner lever arm  $z$  from the effective width  $bn$  or effective height  $d$  in the lateral loadbearing capacity check for  $Q_y$  or  $Q_z$ .

### Core section $A_k = z_1 * z_2$

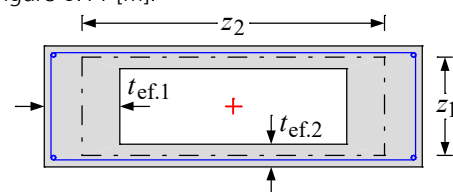
Dimensions of the core section for calculating the torsion reinforcement [m].

### tef

The effective wall thickness of the torsion section according to EN 1992-1-1, Figure 6.11 [m].

DIN EN 1992-2:

For checking combined stress from torsion and corresponding shear force, the wall thickness of box sections can be defined separately in both directions of the cross-section according to Chapter 6.8.2 (NA.102).



## Box section

Selection of the rules applicable for box sections for the check of the maximum load-bearing capacity according to Chapter 6.3.2 (4) and for the required reinforcement according to Chapter 6.3.2 (5) in case of combined stress from lateral force and torsion.

DIN EN 1992-1-1:

In accordance with Chapter 6.3.2 (NA.106) the principal compressive stress is additionally checked for box sections.

## Lever arm

DIN EN 1992-2:

The inner lever arm  $z$  can be defined in the following ways:

- $kd \cdot \text{effective height}$ ,  $kb \cdot \text{effective width}$   
The entered values are used for calculation.
- *from bending design*  
During the bending design the program determines at each check location the largest lever arm for every design situation. It is defined as the distance between the centroids of the concrete compressive and steel tensile forces.
- *RCG Eq. 12.16*  
During the bending design a lever arm weighted by the force increase in the tendons is determined according to Eq. (12.16) of the recalculation guideline for road bridges (RCG). At the check location, the maximum from the sets of internal forces with prestressing steel within the tensile zone at cracked state is decisive for each design situation. If the prestressing steel is positioned within the compressive zone for all sets of internal forces, the lever arm according to Equation (12.16) is determined by  $z = z_s$ .
- $0.9 \cdot dp$   
For the  $Q_z$  check the lever arm is determined with the effective heights  $d_p$  of the tendons with bond. Thereby  $d_p$  is assumed to be the average value weighted by the tendon areas at the check location (cf. Rossner/Graubner 2005, p. 252).

The decisive lever arm is limited to the range  $0.1 \cdot d \leq z \leq \max(d - 2c_{v,l}; d - c_{v,l} - 30 \text{ mm})$  according to DIN EN 1992-1-1, Chapter 6.2.3 (1), and documented in the detailed listing.

## Circular and annular cross-section

If the circular design according to Bender et al. (2010) was selected for the resulting lateral force  $Q_r$  on the *Base values* dialog page, the equivalent cross sections for the shear design must be defined in the following dialog.

## Width bw

Effective section width for calculation of the lateral force bearing capacity for  $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$ . According to the recommendation of the German Committee for Standardization in Civil Engineering (NABau), the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces should be selected for the effective width  $b_w$ . For circular cross-sections, the program suggests the dimension of the square inscribed in the circle ( $R \cdot \sqrt{2}$ ) for  $b_w$ , and twice the wall thickness for annular cross-sections.

### Effective height $d$

Statically effective height for calculation of the lateral force bearing capacity for  $Q_r$ . The program suggests  $d = h - d_1$ , where the height is set to  $h = R \cdot \sqrt{2}$  and  $d_1$  indicates the edge distance of the outer reinforcement layer.

### Factor $k_d$

Factor for calculating the inner lever arm  $z$  from the effective height  $d$  in the verification for  $Q_r$ .

### Efficacy factor

According to Bender et al. (2010), p. 422, the efficacy factor  $\alpha_k$  is stress-dependent ( $0.715 \leq \alpha_k \leq 0.785$ ) and can be assumed with the mean value  $\alpha_k = 0.75$ .

### Helix inclination

Angle between shear force reinforcement and component axis. When entering an inclination of  $90^\circ$ , annular single stirrups are assumed.

### $z_1, z_2, t_{ef}$

The dimensions  $z_1, z_2$  of the square core cross-section and the effective wall thickness  $t_{ef}$  of the torsion box are defined according to EN 1992-1-1, Figure 6.11. The design for torsion is carried out according to the standard for vertical stirrups.

## Shear Joint

The shear joint check is available for polygon and composite sections. The input values proposed by the program must be checked by the user and adjusted if necessary.

Properties for element 381 - EN 1992-2 - Shear joint

Section: Form, Shear stresses, Material, Bedding, EN 1992-1-1, EN 1992-2, Checks, Base values, Shear section, Shear joint, Stresses, Crack width, Fatigue, Variation coefficients, Thermal analysis, General, Isolated column, Eccentricity

Number: 11 - M<sub>i</sub> | Section Type: Polygon | Material Type: C40/50-EN | New | Copy | Delete

Label: Main girder | Properties concern further elements.

Joint location:  Between slab and web (automatic) |  Distance from top edge dz [m] | 0.4

Joint roughness: Smooth | Factor c: 0.2

Joint width b<sub>i</sub> [m]: 1 | Stress perpendicular to joint (comp. neg.) [N/mm<sup>2</sup>]: 0

Dynamic or fatigue load acc. to 6.2.5(105)

OK | Cancel | Help

### Joint location

The program can automatically determine the location of the joint at the transition between the slab and the web. Alternatively, the user can define the distance of the joint from the top edge of the cross-section  $dz$  [m].

### Joint roughness

The roughness of the joint (very smooth, smooth, rough, indented).

### Factor $c$

Factor for determining the shear resistance in the joint, which is specified depending on the joint roughness according to EN 1992-1-1, Chapter 6.2.5 (2) and can only be adjusted by the user if the joint is very smooth.

### Joint width $b_i$

Width of the joint over which shear forces are transferred between existing and new concrete [m].



### Stress perpendicular to joint (comp. neg.)

Stress  $\sigma_n$  caused by the minimum normal force perpendicular to the joint which can act simultaneously with the lateral force [N/mm<sup>2</sup>]. Compressive stresses must be entered with a negative sign and are limited in the check according to 6.2.5 (1).

### Dynamic or fatigue stress according to 6.2.5(105)

If this option is selected, a dynamic or fatigue stress on the cross-section is assumed and the factor  $c$  is adjusted according to EN 1992-2, Chapter 6.2.5 (105).

## Stresses

### perm. $\sigma_c$

The concrete compressive stress  $\sigma_c$  must be limited to  $0.60 f_{ck}$  under the characteristic action combination in the construction stages and final states according to EN 1992-1-1, Chapter 7.2 (2). This limit can be increased by 10% according to EN 1992-2, Chapter 7.2 (102), if the concrete compressive zone is helically reinforced. If stress in the concrete under quasi-continuous combination does not exceed the limit  $0.45 f_{ck}$ , linear creep can be assumed according to 7.2 (3). If this is not the case, non-linear creep must be taken into account.

OENORM B 1992-2:

An increase of the stress limit is not permitted, even if the compressive zone is helically reinforced.

### perm. $\sigma_c(t)$

Permissible concrete stress  $\sigma_c(t)$  at time  $t$  when prestressing is introduced according to EN 1992-1-1, Chapter 5.10.2.2 (5), Eq. (5.42). If the compressive stress exceeds the value  $0.45 f_{ck(t)}$ , the nonlinearity of the creep should be taken into account according to EN 1992-1-1. The program assumes that prestressing is introduced in design situation 'G1+P'.

### fck(t)

Concrete compressive strength at time  $t$  when prestressing is introduced according to EN 1992-1-1, Chapter 5.10.2.2 (5) [MN/m<sup>2</sup>].

### Reinforcing steel stresses

According to EN 1992-1-1, Chapter 7.2 (5), the tensile stresses in the reinforcement may not exceed the value  $0.8 f_{yk}$  under the characteristic action combination. For stresses resulting from indirect action, the limits can be assumed as  $1.0 f_{yk}$ .

SS EN 1992-1-1:

According to Article 19, the limit  $1.0 f_{yk}$  can be generally assumed.

## Decompression, check combination

The action combination (AC) for the decompression check normally results from the selected exposition class. Alternatively, a deviating combination can be chosen.

## Decompression, Stress

DIN EN 1992-2, OENORM B 1992-2:

Decisive stress for the decompression check for area elements ( $\sigma_1$ ,  $\sigma_x$ ,  $\sigma_y$ ).

## Principle tensile stress only in the zone of long. pressure

DIN EN 1992-2:

With this option you can apply the usage guidelines according to II-4.4.0.3 (6)P of the 2009 Edition. These guidelines allow you to limit the check to the area of longitudinal compressive stresses for prestressed railway bridges as long as no tensile-stressed chords are connected. The user is responsible for checking the usage requirements.

## Av

DIN EN 1992-2:

Area of the full section for calculating the normal stress from the longitudinal force (cf. Rossner/Graubner 2012, p. 228), if the section dimensions have been reduced to the effective width. If alternatively the section has been defined as full polygon with specification of the effective width, the input of  $A_v$  is disabled (see also chapter *Structure description - Polygon section*).

## Crack Width

These specifications apply to the minimum crack reinforcement calculation and the crack width check.

## Section edge

The following properties can be defined differently for the section edges and the reinforcement directions:

- $w_{max}$  limit for the calculated crack [mm].
- $s_{r,max}$  largest permissible crack spacing [mm].
- $k_c$  calculation method for coefficient  $k_c$ .
- $max. d_s$  largest existing bar diameter [mm].
- $max. s$  largest existing bar spacing [mm].

The following options are available for editing:

<i>Standard</i>	The standard properties are used for the unspecified edges and directions.
<i>Top, bottem, x, y</i>	Definition for the top or bottom edge in the x or y reinforcement direction.
<Add>	Starts the dialog for adding a section edge.
<Delete>	Deletes the displayed section edge.

### wmax

Limit for the calculated crack width according to EN 1992-2, Chapter 7.3.1, Table 7.101N [mm]. The program will suggest a tabular value according to the national requirements based on the selected exposure class and the prestressing of the component. This value can be modified after the input field is enabled.

SS EN 1992-2:

In addition, the service life class is taken into account to determine the suggested value according to Article 8, Table D-5.

### sr,max

When calculating the crack width, the crack spacing  $s_{r,max}$  is by default determined using Equation (7.11) of EN 1992-1-1. Alternatively, the user can specify an upper limit to take into account any special conditions of Equation (7.14) or Sections (4) and (5) of Chapter 7.3.4, for example.

### Coefficient kc

The following methods are available for calculating the coefficient  $k_c$ :

<i>auto</i>	For rectangular solid sections, kc is calculated according to EN 1992-1-1, Eq. (7.2), in all other cases according to Eq. (7.3).
<i>web</i>	kc is calculated according to Eq. (7.2).
<i>chord</i>	kc is calculated according to Eq. (7.3).

### max. ds

Largest existing bar diameter of the reinforcing steel reinforcement for evaluating Equations (7.6N), (7.7N) and (7.11) in EN 1992-1-1, Chapter 7.3 of the standard [mm].

### max. s

Largest existing bar spacing of the reinforcement for the simplified crack width check as per EN 1992-1-1, Chapter 7.3.3 (2) [mm].

### Determ. of the tensile zone

You can specify the tensile section where the minimum crack reinforcement as per EN 1992-1-1, Chapter 7.3.2, will be placed by selecting either an action combination (AC) or a restraint (bending, central tension).

### Thick component

DIN EN 1992-1-1:

Based on DIN EN 1992-1-1, Chapter 7.3.2 (5), the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined according to Equation (NA 7.5.1). Therewith a reduction compared to the calculation with Equation (7.1) can be achieved.

### Minimum reinforcement according to Eq. (16AT)

OENORM B 1992-1-1:

The minimum reinforcement for the crack width limitation under central restraint can be determined according to Equation (16AT). Therewith a reduction compared to the calculation with Equation (7.1) can be achieved.

### Coefficient k

Coefficient for taking into account nonlinear distributed concrete tensile stresses in the section in EN 1992-1-1, Chapter 7.3.2, Equation (7.1).

DIN EN 1992-1-1:

In case of restraint within the component,  $k$  can be multiplied by 0.8 whereby the minimum of the height and the width of the section or section part shall be used for  $h$ . For tensile stresses due to restraint generated outside of the component,  $k = 1.0$  applies.

SS EN 1992-1-1:

Depending of the section dimension  $h$  (flange thickness resp. web height), the factor  $k$  can be assumed between 0.50 ( $h \geq 680$  mm) and 0.90 ( $h \leq 200$  mm) according to Article 4a.

#### Factor for $f_{ctm}$

This factor is used to specify the effective concrete tensile strength  $f_{ct,eff}$  based on the average value of tensile strength  $f_{ctm}$ . This is done separately for the minimum reinforcement calculation according to Equation (7.1) and the crack width calculation according to EN 1992-1-1, Equation (7.9) of the standard. The tensile strength, which depends on the age of the concrete, is defined in Equation (3.4) of Chapter 3.1.2. If it is not certain whether crack formation will occur within the first 28 days, a tensile strength of at least 2.9 MN/m<sup>2</sup> should be assumed for Eq. (7.1). The program meets this requirement if 1.0 is entered for the reduction factor.

DIN EN 1992-1-1:

If it is not certain whether crack formation will occur within the first 28 days, a tensile strength of at least 3.0 MN/m<sup>2</sup> for normal concrete and 2.5 MN/m<sup>2</sup> for lightweight concrete is assumed for Eq. (7.1).

#### Factor for P0

DIN EN 1992-2:

According to DIN EN 1992-2, Chapter 7.3.2 (NA.111), the statically determined part of prestressing (P0) should be reduced by a factor of 0.75 on construction joints with tendon couplings.

#### $A_{c,eff}$ ring-shaped

For circular solid and hollow sections, the effective area of the reinforcement  $A_{c,eff}$  for the check of the minimum reinforcement and the crack width can be determined ring-shaped according to Wiese et al. (2004).

#### Coefficient $\xi_1$

The bond coefficient  $\xi_1$  according to EN 1992-1-1, Chapter 7.3.2, Equation (7.5), defines the extent to which prestressing steel as per 7.3.2 (3) can be taken into account for the minimum crack reinforcement. It is also used in calculating the effective reinforcement level according to Chapter 7.3.4, Equation (7.10), and thus enters into the direct calculation of the crack width. Data input is blocked for area elements since prestressing steel is normally not taken into account here.

OENORM B 1992-1-1:

The bond coefficient  $\xi_1$  is used to take into account the different bonding behavior of concrete and prestressing steel for the stress checks according to Chapter 7.2 of the standard.

#### Check combination

The action combination (AC) for the crack width check normally results from the selected exposition class. Alternatively a deviating combination can be chosen.

#### Check method

The crack width can be verified either by direct calculation according to EN 1992-1-1, Chapter 7.3.4 or simplified by limiting the bar spacing using Table 7.3N. Table 7.3N should only be used for single-layer tensile reinforcement with  $d_1 = 4$  cm under loading (cf. Zilch, Rogge (2002), p. 277; Fingerloos et al. (2012), p. 109; Book 600 of the DAfStb (2012), p. 127). For both methods, a constant average steel strain within  $A_{c,eff}$  can optionally be chosen as the basis for calculation.

OENORM B 1992-1-1:

The method is applicable to single-layer reinforcement with a bar spacing according to Table 10AT or 11AT. These are valid for concrete covers  $25 \text{ mm} \leq c_{nom} \leq 40 \text{ mm}$  with bar diameters  $8 \text{ mm} \leq d_s \leq 20 \text{ mm}$ .

#### Load duration; $k_t$

This selection defines the factor  $k_t$  in Equation (7.9) for crack width calculation.

DIN EN 1992-2:

For bridges always the factor  $k_t = 0.4$  is to be assumed.

## Fatigue

Properties for element 381 - EN 1992-2 - Fatigue

Section: Form, Shear stresses, Material, Bedding, EN 1992-1-1, EN 1992-2, Checks, Base values, Shear section, Shear joint, Stresses, Crack width, **Fatigue**, Variation coefficients, Thermal analysis, General, Isolated column, Eccentricity

Number: 11 - Mi, Section Type: Polygon, Material Type: C40/50-EN, New, Copy, Delete

Label: Main girder, Properties concern further elements.

Reinforcing steel, prestressing steel

Simplified check

Long. reinf. dSigma.Rsk,s: 70, Shear reinf. dSigma.Rsk,b: 32, Prestr. steel dSigma.Rsk,p: 70 MN/m<sup>2</sup>, Lambda.s: , Lambda.b: , Lambda.p: , Eta: 1, Factor for Qfat: ,  Check decompress.

Concrete

Simplified check

fcd,fat: 16.1 MN/m<sup>2</sup>, Lambda.c: ,  Calc Lambda.c0

Limit design variants

OK, Cancel, Help

### dSigma.Rsk,s, dSigma.Rsk,b

The permissible characteristic stress range  $\Delta\sigma_{Rsk}(N^*)$  of the longitudinal reinforcement and shear reinforcement at  $N^*$  load cycles according to the S-N curves specified in EN 1992-1-1, Chapter 6.8.4 [MN/m<sup>2</sup>]. The national decisive value found in Table 6.3N, Row 1 (beam sections) resp. Row 2 (area sections), is suggested in the dialog. For the shear reinforcement, the mandrel diameter is assumed to be four bar diameters.

OENORM B 1992-1-1:

In the dialog, the value according to Table 5, line 1 (beam sections) or line 4 (area sections) is suggested for the longitudinal reinforcement. For the shear reinforcement, the value according to line 1 is suggested.

### dSigma.Rsk,p

The permissible characteristic stress range  $\Delta\sigma_{Rsk}(N^*)$  of the prestressing steel at  $N^*$  load cycles according to the S-N curves specified in EN 1992-1-1, Chapter 6.8.4 [MN/m<sup>2</sup>]. The value found in Table 6.4N, Row 4, is suggested in the dialog.

DIN EN 1992-1-1, OENORM B 1992-1-1:

The value for prestressing steel of class 1 is suggested.

### Lambda.s, Lambda.b, Lambda.p

Correction coefficient  $\lambda_s$  for determining the damage-equivalent stress range  $\Delta\sigma_{s,eq}$  from the stress range  $\Delta\sigma_s$  of the steel according to EN 1992-2, Chapter NN.2.1 (102) and NN.3.1 (101) for longitudinal reinforcement, shear reinforcement and prestressing steel.

### Lambda.c

Correction coefficient  $\lambda_c = \lambda_{c,0} \cdot \lambda_{c,1} \cdot \lambda_{c,2} \cdot \lambda_{c,3} \cdot \lambda_{c,4}$  as per Eq. (NN.114) for determining the damage-equivalent concrete stress according to EN 1992-2, Chapter NN.3.2 (102) for railroad bridges.

### Calculate Lambda.c0

When using this option, the coefficient  $\lambda_{c,0}$  as per Eq. (NN.115) is calculated with the permanent stress  $\sigma_{c,perm}$  which is decisive at the check location. The resulting value of  $\lambda_{c,0}$  is documented in the detailed log. The aforementioned input value is then to be understood as a product of  $\lambda_{c,1} \cdot \lambda_{c,2} \cdot \lambda_{c,3} \cdot \lambda_{c,4}$ . The final multiplication with the calculated value of  $\lambda_{c,0}$  takes place during the check automatically.

### Eta

Increase factor  $\eta$  for the reinforcing steel stress of the longitudinal reinforcement. This factor is used to take into account the varying bonding behavior of concrete and prestressing steel according to EN 1992-1-1, Chapter 6.8.2 (2)P, Eq. (6.64).

**f<sub>cd,fat</sub>**

Concrete compressive strength before onset of cyclic load according to EN 1992-1-1, Chapter 6.8.7 (1), Eq. (6.76) [MN/m<sup>2</sup>]. In general, the following applies:

$$f_{cd,fat} = k_1 \cdot \beta_{cc}(t_0) \cdot f_{cd} \cdot \left(1 - \frac{f_{ck}}{250}\right) \quad (6.76)$$

with

$$\beta_{cc}(t_0) = e^{s(1-\sqrt{28/t_0})}$$

$s$  Coefficient depending on the cement type.

$t_0$  Time of the initial stressing of the concrete.

$k_1 = 0.85$

$f_{cd,fat}$  for  $s = 0.2$ ,  $t_0 = 28$  and  $f_{cd}$  according to Eq. (3.15) is suggested in the dialog.

DIN EN 1992-2, SS EN 1992-2:

$k_1 = 1.0$

BS EN 1992-2:

For the proposed value of  $f_{cd,fat}$ ,  $f_{cd}$  is determined with  $\alpha_{cc} = 1.0$  in Eq. (3.15).

**k<sub>0</sub>**

DIN EN 1992-2, OENORM B 1992-2:

The statically determined share of prestressing must be reduced in the case of beams and design objects. A base value of 0.9 (DIN) or 0.95 (OENORM) as specified in Chapter 6.8.3 (1)P is suggested in the dialog. For prestressing tendon couplers a further reduction of the base value is required.

**Simplified check**

The simplified check according to EN 1992-1-1, Chapter 6.8.6 (2) bases on the frequent action combination including the traffic loads at serviceability limit state. The method for concrete is defined in Chapter 6.8.7 (2), the permissible stress ranges for steel are suggested according to Chapter 6.8.6 (1) in the dialog. For shear reinforcement this value is reduced analogous to Table 6.3N.

**Limit design variants**

For area elements, the variants for determining the stress range can be limited to the corresponding sets of design internal forces. For more information see chapter '*Checks Against Fatigue > Special Characteristic of Shell Structures*'.

**Factor for Q<sub>fat</sub>**

Increase factor for the cyclic fatigue action defined as  $Q_{fat}$  which is taken into account during the calculation of the damage equivalent stress range  $\Delta\sigma_{s,eq}$  of the reinforcing and prestressing steel. With it, e.g. the rules for the fatigue check for road bridges according to NN.2.1 (101) can be applied.

**Check Decompression**

According to EN 1992-2, Chapter 6.8.1 (102), the fatigue check for reinforcing and prestressing steel is not necessary in areas where under the frequent action combination and  $P_k$  only compressive stress occurs at the prestressed cross-section edge.

DIN EN 1992-2:

This option is applicable at superstructures for reinforcing and prestressing steel without welding joints or couplings when decompression is checked under the frequent combination.

OENORM B 1992-2:

This option is applicable for reinforcing and prestressing steel without welding joints, as far as only compressive stress occurs under the frequent combination as per Table 2AT.

On the program side, it is checked whether the cross-sectional edge closest to the tendon is under compression under the frequent action combination with the characteristic values  $P_{k,sup}$  and  $P_{k,inf}$  of prestressing. In case of ambiguous tendon guidance, both sides are examined. The user is responsible for verifying the application requirements in accordance with the standard.

## Variation Coefficients

Properties for element 381 - EN 1992-2 - Variation coefficients

Number: 11 - M<sub>i</sub> Section Type: Polygon Material Type: C40/50-EN

Label: Main girder

Variation coefficients for the internal prestressing in the serviceability checks.

	r <sub>sup</sub>	r <sub>inf</sub>
Construction stage:	1.1	0.9
Final state:	1.1	0.9

OK Cancel Help

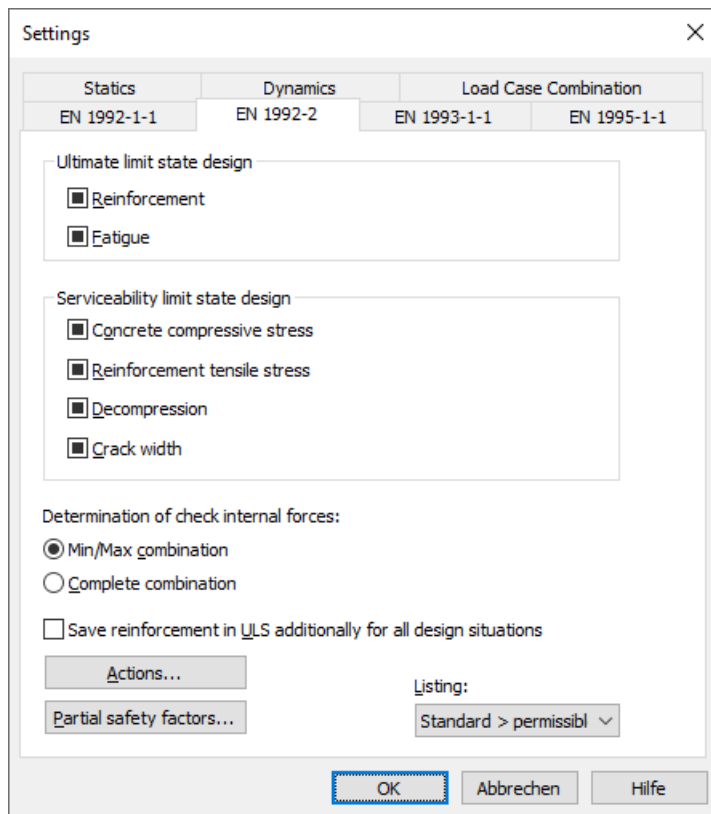
The coefficients used to take into account the variation of prestressing force are defined in EN 1992-1-1 depending on the prestressing type. In the dialog, values are suggested according to Chapter 5.10.9 (1)P for subsequent bond. In the action combinations, situations are declared as construction stage or finale state. The defined variation coefficients are taken into account for the effects from internal prestressing in the following checks:

- Decompression and concrete compressive stress check.
- Minimum reinforcement for crack width limitation.
- Crack width check.
- Check of the diagonal principal tensile stresses (DIN EN 1992-2).

Regarding the effects from external prestressing, the variation coefficients correspond to  $r_{sup} = r_{inf} = 1$ .

## Analysis Settings

The *EN 1992-2* dialog page can be opened using the *Settings* function in the *Analysis* menu.



### Check selection

When selecting checks, the following cases are to be distinguished:

- or  The check is performed according to the settings in the section dialog (see section inputs).
- or  The check is performed for all sections of the structure.
- or  The check is performed for no sections of the structure.

Corresponding section settings are bundled as follows:

- Reinforcement    Bend and longitudinal force, Lateral force, Torsion, Robustness, Shear joint
- Crack width      Minimum crack reinforcement, Calculation of the crack width

An overview of the checks can be accessed using the *Design Settings* function in the *EN 1992-2 Bridge Checks* folder of the database.

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *Check internal forces*.

### Save reinforcement in ULS additionally for all design situations

In addition to the maximum required ultimate limit state reinforcement, the reinforcement is saved separately for each design situation in the ultimate limit state.

### Type of structure...

DIN EN 1992-2:

Open the dialog for selecting the structure type.



**Actions...**

Open the dialog for describing actions.

**Partial safety factors...**

Open the dialog for modifying partial safety factors.

**Listing**

- *No*: No log is generated by the checking program.
- *Standard*: Log with tabular output of results.
- *Detailed*: Additional output of the decisive combination internal forces at the check locations.
- *Standard > permissible*: Standard log limited to check locations where the permissible limit values are exceeded.
- *Detailed > permissible*: Detailed log limited to check locations where the permissible limit values are exceeded.

## Single Design

The single design function allows you to analyze individual sections independently of the global system using predefined internal forces. Enter the information listed below in the *Single Design* table in the *EN 1992-2 Bridge Checks* folder of the database or the folders of the national variants.

**Section**

Number of the section to be designed. Both polygon and composite sections can be designed.

**Combination**

Design situation according to EN 1992-1-1, Table 2.1.

- *0*: Permanent and temporary design situation
- *1*: Accidental design situation

**Nsd, Mysd, Mzsd**

Internal forces being designed. The internal forces refer to the centroid in polygon sections or the section zero point in composite sections.

**Mode**

- *Standard*: Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members a symmetrical design is carried out taking into account the minimum reinforcement according to EN 1992-1-1, Chapter 9.5.2 (2).
- *Strains*: Determine strain state for existing reinforcing steel layers.
- *Strains SLS*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. In the compression zone, a linear strain-stress curve of the concrete with the gradient  $\tan \alpha = E_{cm}$  is used.
- *Strains SLS2*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A nonlinear strain-stress curve of the concrete is used as shown in EN 1992-1-1, Figure 3.2. Note that a horizontal progression is assumed for strains exceeding  $\epsilon_{c1}$ .
- *Load bearing capacity*: Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
- *Maximum bending moment My*: Determination of the maximum bearable bending moment  $M_y$ . The moment  $M_y$  is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.
- *Inactive*: Design disabled.

The calculation is carried out from the opened input table via the *Single Design* or *Print Preview* menu item.

OENORM B 1992-1-1:

In the modes *SLS* and *SLS2* the stress increase of the prestressing steel layers is determined according to Eq. (13AT) with the bond coefficient  $\xi_1$  specified for the section to be checked.

## Prestressed Structures

### Internal Prestressing

For internal prestressing, the tendon groups as well as the prestressing system and procedures are entered using the *Prestressing* function of the *Structure* menu. To include them in the FEM calculation, you then need to define a load case of the *Prestressing* load type. For more information, refer to the *Prestressed Concrete* section of the manual.

Prestressing with bond and prestressing without bond are differentiated in the section inputs and the specifications for the *Creep and shrinkage* load case. For prestressed components with subsequent bond the tendons can be set ungrouted for the respective situation in the action combination dialog.

### External Prestressing, Mixed Construction

External prestressing can be taken into account by entering the external forces directly in the program. For mixed construction, the additional tendons in a bond must be entered as described above.

### Variation of Prestressing

For checks in the ultimate limit state, the following applies for the design value of the prestressing force according to EN 1992-1-1, Chapter 5.10.8 (1):

$$P_{d,t}(x) = \gamma_P \cdot P_{m,t}(x)$$

with

$P_{m,t}(x)$  Mean value of prestressing force at time  $t$  and location  $x$  including prestressing losses from friction, slippage, creep, shrinkage and relaxation.

$\gamma_P$  Partial safety factor of prestressing force,  $\gamma_P = 1$  as specified in Chapter 2.4.2.2 (1).

In the serviceability limit state, two characteristic values for the prestressing force are defined in Chapter 5.10.9 (1):

$$P_{k,\text{sup}} = r_{\text{sup}} \cdot P_{m,t}(x) \quad \text{Upper characteristic value according to Equation (5.47).}$$

$$P_{k,\text{inf}} = r_{\text{inf}} \cdot P_{m,t}(x) \quad \text{Lower characteristic value according to Equation (5.48).}$$

The variation coefficients for internal prestressing are defined separately for construction stages and final states. They are used in the following checks:

- Decompression and concrete compressive stress check.
- Minimum reinforcement for crack width limitation.
- Crack width check.
- Check of the diagonal principal tensile stresses (DIN EN 1992-2).

Regarding the effects from external prestressing, the variation coefficients correspond to  $r_{\text{sup}} = r_{\text{inf}} = 1$ .

For internal prestressing, the recommended country-specific values are:

- For tendons with immediate bond or without bond:  $r_{\text{sup}} = 1.05$  and  $r_{\text{inf}} = 0.95$ .
- For tendons with subsequent bond:  $r_{\text{sup}} = 1.10$  and  $r_{\text{inf}} = 0.90$ .

DIN EN 1992-2:

In the construction stages the following values may be used according to NCI for 5.10.9 (1)P:

- For straight-lined tendons with bond and tendons without bond:  $r_{\text{sup}} = r_{\text{inf}} = 1.00$ .
- For curved tendons with bond:  $r_{\text{sup}} = 1.05$  und  $r_{\text{inf}} = 0.95$ .

OENORM B 1992-1-1:

- For tendons with immediate bond or without bond:  $r_{\text{sup}} = r_{\text{inf}} = 1.00$ .
- For tendons with subsequent bond:  $r_{\text{sup}} = 1.05$  und  $r_{\text{inf}} = 0.95$ .

## Creep and Shrinkage

Similar to prestressing, creep and shrinkage are taken into account by specifying a corresponding load case (*Creep and shrinkage* load type) in the FEM calculation. Besides the creep-generating continuous load case, you also need to specify whether the internal forces relocation between concrete and prestressing steel is to be taken into account. This option is only useful in the case of tendons with bond. The optional safety factor  $\gamma_{It}$  according to EN 1992-2, Table B.101, is not taken into account.

The decisive creep and shrinkage coefficients for calculating the *Creep and shrinkage* load case are entered in the Section dialog. Alternatively, you can also use this dialog to calculate the coefficients according to EN 1992-1-1, Chapter 3.1.4 with Annex B.

The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1+\rho \cdot \varphi} (\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,s})$$

Explanation of the individual terms:

- $\sigma_b(t)$  Concrete stress from creep and shrinkage at time  $t$ .
- $E_b$  E-modulus of the concrete.
- $\rho$  Relaxation coefficient according to Trost for time  $t$  (normally  $\rho = 0.80$ ).
- $\varphi$  Creep coefficient for time  $t$ .
- $\varepsilon_b(t)$  Concrete strain from creep and shrinkage at time  $t$ .
- $\varepsilon_{b,0}$  Concrete strain from creep-generating continuous load.
- $\varepsilon_{b,s}$  Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, yield the internal forces and deformations of the concrete. The resulting stress changes in the prestressing steel are also determined provided they are selected in the load case. Any influence from the relaxation of the prestressing steel will be ignored in this case. According to Zilch/Rogge (2002, p. 256), this influence can be calculated separately (see following section) and combined with the changes from creep and shrinkage for all time-dependent prestressing losses:

$$\Delta\sigma_{p,csr} = \Delta\sigma_{pr} + E_p \cdot \Delta\varepsilon_{cpt}$$

with

- $\Delta\sigma_{pr}$  Prestressing loss from relaxation of the prestressing steel.
- $\Delta\varepsilon_{cpt}$  Concrete strain change from creep and shrinkage.
- $E_p$  E-modulus of the prestressing steel.

## Relaxation of Prestressing Steel

According to EN 1992-1-1, Chapter 5.10.6, the stress change  $\Delta\sigma_{pr}$  in the tendons at position  $x$  and time  $t$  due to relaxation must be taken into account in addition to the stress loss from concrete creep and shrinkage. The relaxation of the steel depends on the deformation of the concrete caused by creep and shrinkage. According to 5.10.6 (1) (b), this interaction can be taken into account in a general and approximate manner by specifying a reduction coefficient of 0.8.

The stress change  $\Delta\sigma_{pr}$  can be determined for the initial stress in the tendons as a result of prestressing and quasi-continuous actions according to 5.10.6 (2). More details are provided in Chapter 3.3.2 of the standard.

The stress losses are defined in the CSR actions of the *EN 1992-2 actions* dialog.

DIN EN 1992-1-1:

The stress change  $\Delta\sigma_{pr}$  can be determined using the specifications of the prestressing steel certification for the ratio of initial stress to characteristic tensile strength ( $\sigma_{p0}/f_{pk}$ ).  $\sigma_{p0} = \sigma_{pg0}$  may be used as the initial stress, with  $\sigma_{pg0}$  referring to the initial prestressing steel stress from prestressing and the permanent action.

## Check Internal Forces

The calculation of load cases results in a set of internal forces for each load case at the check location (e.g.  $N_x$ ,  $M_y$ ). The check internal forces are then determined from the results of the load cases with the combination rules relevant for the ultimate limit state, fatigue and serviceability limit state. One of the following methods can be selected in the analysis settings:

- *Min/Max combination*  
The results of a load case are added to the set of internal forces with the minimum or maximum of an internal force, if this increases the amount of the extreme value. Result sets from traffic actions in which the control variable is less than the threshold  $10^{-3}$  are not combined. The min/max combination delivers a constant number of sets regardless of the number of load cases and thus represents a particularly economical solution for the checks.
- *Complete combination*  
To determine the evidence internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The number of records increases exponentially with the number of inclusive load cases and can therefore result in high time and memory requirements for the checks.

For beams and design objects, the resulting sets of internal forces are used directly in the checks. For area elements, *design internal forces* are derived from this, as will be described in more detail in the following section.

The internal forces relevant for the checks are documented in the detailed check listing. Regardless of the selection made, the results of the min/max combination are saved for the graphical representation. The load cases involved in the combination can be displayed using the *Combination information* context function.

The differences between the two combination methods mentioned before can be seen from the following example of a uniaxially stressed beam. The load cases 2, 3 and 4 shown can act simultaneously (inclusive). All safety and combination factors are assumed to be 1 for the example.

Action	$N_x$	$M_y$	Load case
G - permanent	-15	40	1
Q - variable	0	20	2
	5	10	3
	0	-10	4

*Internal forces of the load cases*

Extreme value	$N_x$	$M_y$	Combination
min $N_x$	-15	40	L1
max $N_x$	-10	50	L1+L3
min $M_y$	-15	30	L1+L4
max $M_y$	-10	70	L1+L2+L3

*Results of min/max combination*

Set	$N_x$	$M_y$	Combination
1	-15	40	L1
2	-15	60	L1+L2
3	-10	50	L1+L3
4	-15	30	L1+L4
5	-10	70	L1+L2+L3
6	-15	50	L1+L2+L4
7	-10	40	L1+L3+L4
8	-10	60	L1+L2+L3+L4

*Results of complete combination*

## Design internal forces for area elements

With area elements, the design internal forces correspond to the plasticity approach from Wolfensberger and Thürlimann. This approach takes into account how much the reinforcement deviates from the crack direction. Due to the current lack of precise data regarding the combined load of reinforced concrete shell structures from bending and normal force, the design internal forces for bending and normal force are calculated independently according to the static limit theorem of the plasticity theory and then used together as the basis for the design in the two reinforcement directions. This approach should always lead to results that are on the safe side.

Depending on the type of area element and reinforcement configuration, the variants of design internal forces listed below are taken into account for the checks.

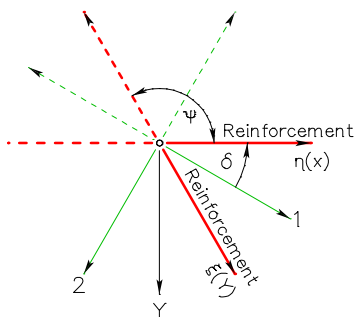
### Orthogonal area reinforcement

Slabs	$m_x \pm  m_{xy} $	
	$m_y \pm  m_{xy} $	
Plain stress elements	$n_x \pm  n_{xy} $	
	$n_y \pm  n_{xy} $	
Shells	$m_x \pm  m_{xy} $ and $n_x \pm  n_{xy} $	
	$m_y \pm  m_{xy} $ and $n_y \pm  n_{xy} $	

### Oblique area reinforcement

The bending design of slabs with oblique reinforcement assemblies is carried out according to Kuyt or Rüsç. Here the design moments are calculated with the help of the principal moments  $m_1$ ,  $m_2$  according to the equations given in Book 166 DAfStB.

For load case combinations, the calculation is based on the extreme values of  $m_1$ ,  $m_2$ . For combined loads (bending and longitudinal force), both the design moments and the normal design forces are independently derived from  $n_1$ ,  $n_2$ . The normal design forces are then used together as the basis for the design. This should also result in an upper limit for the load.



Coordinate systems

Extreme values (principal bending moments):

$$m_{1,2} = \frac{1}{2}(m_x + m_y) \pm \frac{1}{2}\sqrt{(m_x - m_y)^2 + 4m_{xy}^2}$$

with  $m_1 \geq m_2$

The angle  $\delta$  assigned to  $m_1$  is:

$$\tan \delta = \frac{2 \cdot m_{xy}}{(m_x - m_y) + \sqrt{(m_x - m_y)^2 + 4 \cdot m_{xy}^2}}$$

### Design moments:

$$m_{\eta} = \frac{1}{\sin^2 \psi} \left[ m_1 \sin^2(\delta + \psi) + m_2 \cos^2(\delta + \psi) \pm |m_1 \sin \delta \sin(\delta + \psi) + m_2 \cos \delta \cos(\delta + \psi)| \right]$$

$$m_{\xi} = \frac{1}{\sin^2 \psi} \left[ m_1 \sin^2 \delta + m_2 \cos^2 \delta \pm |m_1 \sin \delta \sin(\delta + \psi) + m_2 \cos \delta \cos(\delta + \psi)| \right]$$

The formulas apply accordingly for the normal design forces.

## Checks in the Ultimate Limit States

The following checks are performed according to EN 1992-2 in conjunction with EN 1992-1-1:

- Bending with or without normal force or normal force only (Chapter 6.1).
- Minimum reinforcement against failure without warning (Chapter 6.1 (109)).
- Lateral force (Chapter 6.2).
- Torsion and combined stressing (Chapter 6.3).
- Principal compressive stress for combined actions (DIN EN 1992-2, Chapter 6.3.2 (NA.106)).
- Shear joint (Chapter 6.2.5).
- Punching shear (Chapter 6.4).

## Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.4.3, the following combinations are taken into account in the ultimate limit states:

- For the combination of the permanent and temporary design situation either Equation (6.10) or the most unfavorable equation from (6.10a) and (6.10b) is permitted.

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10)$$

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10a)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10b)$$

For the coefficient  $\xi$  the value of  $\xi = 0.85$  results from Table A.1.2(B).

DIN EN 1990, OENORM B 1990, BS EN 1990:  
Equation (6.10) is used for the combination.

SS EN 1990 (EKS 11):

Equations (6.10a) and (6.10b) apply with following modifications:

$$\sum_{j \geq 1} \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P \quad (6.10aSS)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_d \cdot \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_d \cdot \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10bSS)$$

Assuming reliability class 3, factor  $\gamma_d$  is set to 1. (see Section A, Article 11 and 14). The coefficient  $\xi$  is set to the value of  $\xi = 0.89$ .

- Combination for accidental design situations

$$\sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.11b)$$

$\psi_{1,1} \cdot Q_{k,1}$  is used by the program for this combination.

OENORM B 1990-1:

$\psi_{2,1} \cdot Q_{k,1}$  is decisive.

- Combination for design situations caused by earthquakes

$$\sum_{j \geq 1} G_{k,j} + P + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.12b)$$

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

## Stress-Strain Curves

The following characteristics are used for section design:

- Concrete: Parabola-rectangle diagram according to EN 1992-1-1, Figure 3.3. Note that the design value for concrete compressive strength  $f_{cd}$  in EN 1992-2, Equation (3.15) is defined as  $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$  with  $\alpha_{cc} = 0.85$ .

DIN EN 1992-2:

$\alpha_{cc} = 0.85$  for normal concrete and  $\alpha_{cc} = 0.75$  for lightweight concrete.

OENORM B 1992-2:

$\alpha_{cc} = 1.0$  for normal concrete and  $\alpha_{cc} = 0.85$  for lightweight concrete.

SS EN 1992-2:

$\alpha_{cc} = 1$  for normal and lightweight concrete.

BS EN 1992-2:

According to NA to 3.1.6 (101)P conservatively,  $\alpha_{cc} = 0.85$  is always assumed for normal concrete and lightweight concrete.

- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be  $k \cdot f_{yk} / \gamma_s$  with  $k = 1.05$  as per Table C.1, class A.

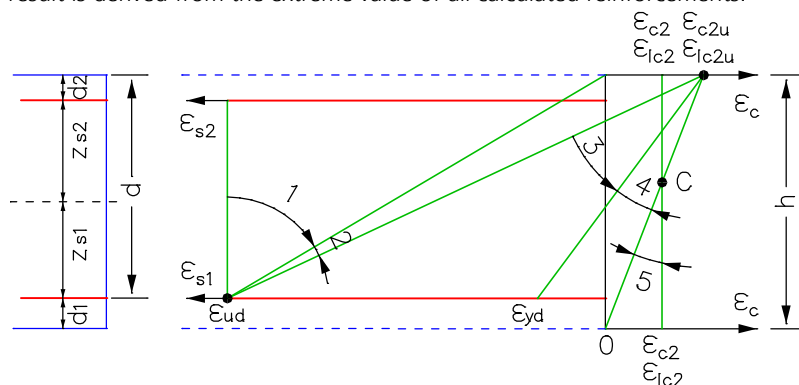
DIN EN 1992-2:

The maximum stress is assumed to be  $1.05 \cdot f_{yk} / \gamma_s$  for ductility class A according to DIN 488-1.

- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of  $f_{pd} = f_{p;0,1k} / \gamma_s$ .

## Design for Bending With or Without Normal Force or Normal Force Only

The design for longitudinal force and bending moment is performed according to EN 1992-1-1, Chapter 6.1. The reinforcement required for each internal force combination at the reinforced concrete section is determined iteratively based on the formulation of equilibrium conditions as well as the limit strain curve depicted in the illustration below. The final result is derived from the extreme value of all calculated reinforcements.



Strain areas for the design with  $\epsilon_{ud} = 0.9 \epsilon_{uk}$  and  $\epsilon_{uk} = 0.025$  as per Table C.1.

DIN EN 1992-1-1:

$\epsilon_{ud} = 0.025$

You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes. For sections subject to a compressive normal force, the minimum eccentricity defined in Chapter 6.1 (4) is taken into account. Concrete compression according to Chapter 6.1 (5) cannot be checked.

### Standard Mode

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tension section to the greatest degree possible. Given ratios between certain reinforcement layers in the tension or compression zone are maintained as far as possible, unless this is deselected in the design specifications. The procedure in strain areas 4 and 5 is the same as with symmetrical design. The required transverse reinforcement of slab as



per Section 9.3.1.1 (2) is considered during design according to user specification. However, the provision for horizontal reinforcement of walls as per Section 9.6.3 (1) is not taken into account.

DIN EN 1992-1-1:

The referenced compressive zone height  $x_d / d$  is limited according to Chapter 5.4 (NA.5) and NA.11.5.2 (1) as follows:

$$\begin{aligned} x_d / d &\leq 0.45 \text{ for concrete strength classes up to C50/60.} \\ &\leq 0.35 \text{ for concrete strength class C55/67 or higher and for lightweight concrete.} \end{aligned}$$

### Symmetrical Mode

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected unless this is deselected in the design specifications.

### Compression member Mode

The design is performed symmetrically. In addition, the minimum reinforcement required by EN 1992-1-1, Chapter 9.5.2 (2), will be calculated:

$$A_{s,\min} = 0.10 |N_{Ed}| / f_{yd} \text{ or } 0.002 A_c, \text{ depending on which value is greater} \quad (9.12N)$$

with

$N_{Ed}$  Design value of the longitudinal force to be absorbed.

$f_{yd}$  Design value for the reinforcing steel strength at the yield strength.

DIN EN 1992-2:

$$A_{s,\min} = 0.15 N_{Ed} / f_{yd} \geq 0.003 A_c \quad (9.12DE)$$

OENORM B 1992-1-1:

$$A_{s,\min} = 0.13 N_{Ed} / f_{yd} \geq 0.0026 A_c \quad (30AT)$$

SS EN 1992-1-1:

$$A_{s,\min} = 0.002 A_c \quad (\text{Article 28})$$

### Inclusion of tendons with bond

When designing beams and design objects, the internal forces of the concrete section is reduced by the statically determined portions which result from prestressing minus the losses from creep, shrinkage and prestressing steel relaxation (CSR). Situations prior to the grouting of the tendons are excluded. So only the restraint portions from 'P+CSR' and the external loads are contained in the remaining internal forces for the composite section. If necessary, the reinforcing steel positioned by the user will be increased until the composite internal forces can be absorbed. In the design mode *compression member* the prestressing steel area is taken into account when determining the minimum reinforcement according to Chapter 9.5.2 (2).

The position of the tendon groups in the section, the prestressing losses from CSR, the statically determined portions and the internal forces of the concrete section and the composite section are written to the detailed log.

As a separation into statically determined and undetermined shares of the internal forces from prestressing is not possible for shell structures, the prestressing is taken into account fully on the action side when designing the longitudinal reinforcement. As a result, on the resistance side only mild steel and concrete are considered whereas the strain reserves of the tendons with bond are not used.

## Minimum Reinforcement Against Failure Without Warning

With respect to prestressed concrete structures, a failure without warning can be prevented by adding a minimum reinforcement as described in EN 1992-2, Chapter 6.1 (109). This reinforcement is determined based on Equation (6.101a):

$$A_{s,\min} = M_{\text{rep}} / (z_s \cdot f_{yk}) \quad (6.101a)$$

with

$M_{\text{rep}}$  Crack moment without allowance for prestressing force and under the assumption that the edge tensile stress corresponds to  $f_{\text{ctm}}$ . According to EN 1992-1-1, Chapter 9.2.1.1 (4), the 1.15-fold crack moment is used for components with unbonded tendons or with external prestressing.

$z_s$  Lever arm of the internal forces in the ultimate limit state.

According to EN 1992-2, Chapter 6.1 (110), the minimum reinforcement should be added in areas where tensile stresses in the concrete occur under the characteristic action combination. This process should take into account the statically undetermined prestressing effect and ignore the statically determined effect.

The program determines all stresses at the gross section. The statically determined prestressing effect can only be subtracted for beams and design objects. For area elements the prestress is alternatively reduced by a user-defined reduction factor. The lever arm  $z_s$  of the internal forces is assumed as  $0.9 \cdot d$  for the sake of simplicity. The calculated reinforcement is evenly distributed to the reinforcement layers in the tensile zone. In the design mode *symmetrical* reinforcement is also applied to the remaining layers. This will not affect the predefined relationships between the individual reinforcement layers. For sections with mode *compression member* the robustness reinforcement is not checked because minimum reinforcement is already determined during the design for bending with longitudinal force.

DIN EN 1992-2:

The crack moment is determined considering an edge tensile stress of  $f_{\text{ctk},0.05}$  without raising as per NCI for Chapter 9.2.1.1 (4).

SS EN 1992-1-1:

According to Article 13, method D (proofs concerning the reliability of the tendons), in combination with at least one of the other methods, should be used. The second condition can be covered by adding the minimum reinforcement as described in Chapter 9.2.1 (method A) or by use of the above-mentioned robustness reinforcement.

## Surface Reinforcement

To prevent concrete spalling, a surface reinforcement may be necessary according to EN 1992-1-1, Chapter 9.2.4. For more information, refer to Annex J. The reinforcement determined in this manner can be incorporated into the program by specifying a base reinforcement in the reinforcing steel description.

DIN EN 1992-2:

For bridges constructive minimum reinforcement according to Chapter 9.1 (NA.104) is required. In case of prestressed members always a surface reinforcement according to Annex J, Table NA.J.4.1, is to be installed.

OENORM B 1992-1-1:

The guidelines set forth in Annex J are not normative.

## Design for Lateral Force

Lateral force design involves determining the lateral force reinforcement and includes a concrete strut check according to EN 1992-2, Chapter 6.2. The following special conditions apply:

- The angle of the lateral force reinforcement is assumed to be 90°.
- The value for  $\cot\Theta$  can be selected by the user within the permissible national limits of Eq. (6.7N) of EN 1992-1-1.
 

DIN EN 1992-2:  
In the calculation, the specified value for  $\cot\Theta$  is limited to the range permitted in accordance with Equation (6.107aDE) (method with load-dependent strut angle), unless the check with a constant value is selected in the section dialog. The actual effective angle of the concrete struts is logged for each check location.
- The minimum reinforcement according to EN 1992-1-1, Chapter 9.2.2 (5) is included in the calculated stirrup reinforcement. For areas, the minimum reinforcement as per Chapter 6.2.1 (4) will only be determined if the lateral force reinforcement is necessary for computation. For beams no minimum reinforcement is calculated for the direction with  $M = Q = 0$ .
- Slab and shell elements are designed for lateral force  $q_r = \sqrt{(q_{x^2} + q_{y^2})}$ . Depending on which has a negative effect, either the principal compressive force or principal tensile force is used for the associated longitudinal force.
 

DIN EN 1992-2:  
If selected, the check will be carried out separately for the reinforcement directions x and y in accordance with Chapter 6.2.1 (10). In this case, the normal force in reinforcement direction is used for the associated longitudinal force. If lateral force reinforcement is necessary, it must be added from both directions.
- There is **no** reduction of the action from loads near supports as specified in EN 1992-1-1, Chapter 6.2.1 (8).
- For beams and design objects, the decisive values of the equivalent rectangle are determined by the user independently of the normal section geometry. The coefficients for calculating the inner lever arm  $z$  based on the effective width and effective height must also be specified. For area elements, the calculation is generally performed with the lever arm  $z = 0.9 d$ .
 

DIN EN 1992-2:  
Alternatively, the lever arm from bending design can be used. According to DIN EN 1992-1-1, Chapter 6.2.3 (1), the inner lever arm is limited to the maximum value derived from  $z = d - c_{v,1} - 30$  mm and  $z = d - 2c_{v,1}$ . Note that  $c_{v,1}$  refers to the extent to which longitudinal reinforcement is laid in the concrete compressive zone.
- For beam sections with internal prestressing, the design value of lateral load-bearing capacity  $V_{Rd,max}$  according to EN 1992-1-1, Chapter 6.2.3 (6) is determined using the nominal value  $b_{w,nom}$  of the section width.
- The necessity of a lateral force reinforcement is analyzed according to EN 1992-1-1, Chapter 6.2.2 (1). The special conditions listed in Sections (2) through (7) are not used in this case.
- The interaction between lateral force and lateral bending in the webs of hollow box sections according to EN 1992-2, Chapter 6.2.106 is not considered. The same applies to the special case of straight tendons as per Fig. 6.101.
- BS EN 1992-2:  
The shear strength of concrete of strength classes higher than C50/60 is limited to the value of class C50/60, according to NA to 3.1.2 (102)P. The concrete compressive strength  $f_{cd}$  according to Eq. (3.15) is determined conservatively with  $\alpha_{cc} = 0.85$ .

The used formulas of the standard are listed below.

### Components without computationally necessary lateral force reinforcement

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}] b_w \cdot d \quad (6.2a)$$

with at least

$$V_{Rd,c} = (v_{min} + k_1 \cdot \sigma_{cp}) b_w \cdot d \quad (6.2b)$$

For lightweight concrete applies:

$$V_{IRd,c} = [C_{IRd,c} \cdot \eta_1 \cdot k \cdot (100 \rho_1 \cdot f_{lck})^{1/3} + k_1 \cdot \sigma_{cp}] b_w \cdot d \quad (11.6.2)$$

$$\geq (\eta_1 \cdot v_{1,min} + k_1 \cdot \sigma_{cp}) b_w \cdot d$$

where

$f_{ck}, f_{lck}$  is the characteristic concrete strength [N/mm<sup>2</sup>].

$k = 1 + \sqrt{(200 / d)} \leq 2.0$  with  $d$  specified in mm.

$\rho_1 = A_{s1} / (b_w \cdot d) \leq 0.02$ .

- $A_{sl}$  is the area of the tensile reinforcement that extends at least  $(l_{bd} + d)$  beyond the analyzed section (see Figure 6.3).
- $b_w$  is the smallest section width in the tensile zone of the section [mm].
- $\sigma_{cp} = N_{Ed} / A_c < 0.2 f_{cd}$  [N/mm<sup>2</sup>].
- $N_{Ed}$  is the normal force in the section due to loading or prestressing [N] ( $N_{Ed} > 0$  for compression). The influence of the forced deformations on  $N_{Ed}$  can be ignored.
- $A_c$  is the entire area of the concrete section [mm<sup>2</sup>].
- $V_{Rd,c}, V_{IRd,c}$  is the design value of the lateral force resistance [N].
- $\eta_1$  is the reduction coefficient for lightweight concrete according to Eq. (11.1).

The recommended values are:

- $C_{Rd,c} = 0.18 / \gamma_c$  for normal concrete.
- $C_{IRd,c} = 0.15 / \gamma_c$  for lightweight concrete.
- $k_1 = 0.15$
- $v_{min} = 0.035 k^{3/2} \cdot f_{ck}^{1/2}$  for normal concrete (6.3N)
- $v_{l,min} = 0.028 k^{3/2} \cdot f_{lck}^{1/2}$  for lightweight concrete according to 11.6.1 (1).

DIN EN 1992-1-1:

- $C_{Rd,c} = C_{IRd,c} = 0.15 / \gamma_c$
- $k_1 = 0.12$
- $v_{min} = (\kappa_1 / \gamma_c) k^{3/2} \cdot f_{ck}^{1/2}$
- $v_{l,min} = (\kappa_1 / \gamma_c) k^{3/2} \cdot f_{lck}^{1/2}$
- with
- $\kappa_1 = 0.0525$  for  $d < 600$  mm
- $\kappa_1 = 0.0375$  for  $d > 800$  mm

For  $600 \text{ mm} < d \leq 800 \text{ mm}$  can be interpolated.

### Components with computationally necessary lateral force reinforcement

The angle  $\theta$  between the concrete struts and the component axis perpendicular to the lateral force must be limited:

$$1 \leq \cot \theta \leq 2.5 \quad (6.7N)$$

DIN EN 1992-2:

$$1.0 \leq \cot \theta \leq (1.2 + 1.4 \sigma_{cp} / f_{cd}) / (1 - V_{Rd,cc} / V_{Ed}) \leq 1.75 \quad (6.107aDE)$$

with

$$V_{Rd,cc} = c \cdot 0.48 \cdot f_{ck}^{1/3} (1 - 1.2 \sigma_{cp} / f_{cd}) \cdot b_w \cdot z \quad (6.7bDE)$$

where

- $c = 0.5$
- $\sigma_{cp}$  The design value of the concrete longitudinal stress at the level of the centroid of the section with  $\sigma_{cp} = N_{Ed} / A_c$  in N/mm<sup>2</sup>.
- $N_{Ed}$  The design value of the longitudinal force in the section caused by external actions ( $N_{Ed} > 0$  as longitudinal compressive force).

For lightweight concrete the input value  $V_{Rd,cc}$  from Eq. (6.7bDE) is to be multiplied by  $\eta_1$  according to Eq. (11.1).

OENORM B 1992-1-1:

$$0.6 \leq \tan \theta \leq 1.0 \quad (3AT)$$

If the section is completely under pressure, then  $\theta$  in the area

$$0.4 \leq \tan \theta \leq 1.0 \quad (4AT)$$

may be selected.

SS EN 1992-1-1:

According to Article 15 and differing from Equation (6.7N), for prestressed components the condition  $1.0 \leq \cot \theta \leq 3.0$  applies.

For components with lateral force reinforcement perpendicular to the component axis, the lateral force resistance  $V_{Rd}$  is the smaller value from

$$V_{Rd,s} = (A_{sw} / s) \cdot z \cdot f_{ywd} \cdot \cot \theta \quad (6.8)$$

and

$$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta) \quad (6.9)$$

where

$A_{sw}$  is the section area of the lateral force reinforcement.

$s$  is the distance of the stirrups to each other.

$f_{ywd}$  is the design value for the yield strength of the lateral force reinforcement.

$v_1$  is a reduction coefficient for concrete strength when shear cracks occur.

$\alpha_{cw}$  is a coefficient for taking into account the stress state in the compression chord.

The recommended values are:

$$v_1 = v$$

with

$$v = 0.6 (1 - f_{ck} / 250) \quad (f_{ck} \text{ in N/mm}^2) \quad (6.6N)$$

$$\alpha_{cw} = 1 \quad \text{for non-prestressed components}$$

$$= (1 + \sigma_{cp} / f_{cd}) \quad \text{for } 0 < \sigma_{cp} \leq 0.25 f_{cd} \quad (6.11aN)$$

$$= 1.25 \quad \text{for } 0.25 f_{cd} < \sigma_{cp} \leq 0.5 f_{cd} \quad (6.11bN)$$

$$= 2.5 (1 - \sigma_{cp} / f_{cd}) \quad \text{for } 0.5 f_{cd} < \sigma_{cp} \leq 1.0 f_{cd} \quad (6.11cN)$$

where

$\sigma_{cp}$  is the average compressive stress in the concrete (indicated as a positive value) as a result of the design value for the normal force.

The maximum effective section area of the lateral force reinforcement  $A_{sw,max}$  for  $\cot \theta = 1$  is derived from:

$$(A_{sw,max} \cdot f_{ywd}) / (b_w \cdot s) \leq \frac{1}{2} \alpha_{cw} \cdot v_1 \cdot f_{cd} \quad (6.12)$$

The additional tensile force in the longitudinal reinforcement due to lateral force according to Eq. (6.18) is

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) \quad (6.18)$$

DIN EN 1992-2:

$$v_1 = \eta_1 \cdot 0.75 \text{ for concrete up to C50/60-EN-D}$$

$$= \eta_1 \cdot 0.75 \cdot \min(1.0; 1.1 - f_{ck} / 500) \text{ from C55/67-EN-D as per DIN EN 1992-1-1, NDP 6.2.3 (3).}$$

$$\eta_1 = 1.0 \text{ for normal concrete and as per Eq. (11.1) for lightweight concrete.}$$

$$\alpha_{cw} = 1.0$$

Equation (6.12) is not applied.

BS EN 1992-2:

$$v_1 = v \cdot (1 - 0.5 \cos \alpha)$$

The further regulations of NDP to 6.2.3 (3) are not taken into account.

**Lateral force reinforcement (Standard design)**

The lateral force reinforcement level is derived from Equation (9.4):

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) \tag{9.4}$$

where

$\rho_w$  is the reinforcement level of the lateral force reinforcement. In general, this level may not be smaller than

$$\rho_{w,min}$$

$A_{sw}$  is the section area of the lateral force reinforcement per length  $s$ .

$s$  is the distance of the lateral force reinforcement as measured along the component axis.

$b_w$  is the web width of the component.

$\alpha$  is the angle between the lateral force reinforcement and the component axis.

The recommended value for the minimum reinforcement is:

$$\rho_{w,min} = 0.08 \sqrt{f_{ck}} / f_{yk} \tag{9.5N}$$

DIN EN 1992-1-1:

$$\rho_{w,min} = 0.16 f_{ctm} / f_{yk} \tag{9.5aDE}$$

With respect to slabs, the value can vary between zero and the above value as described in Chapter 9.3.2 (2).

For structured sections with prestressed tension chord, the following applies:

$$\rho_{w,min} = 0.256 f_{ctm} / f_{yk} \tag{9.5bDE}$$

OENORM B 1992-1-1:

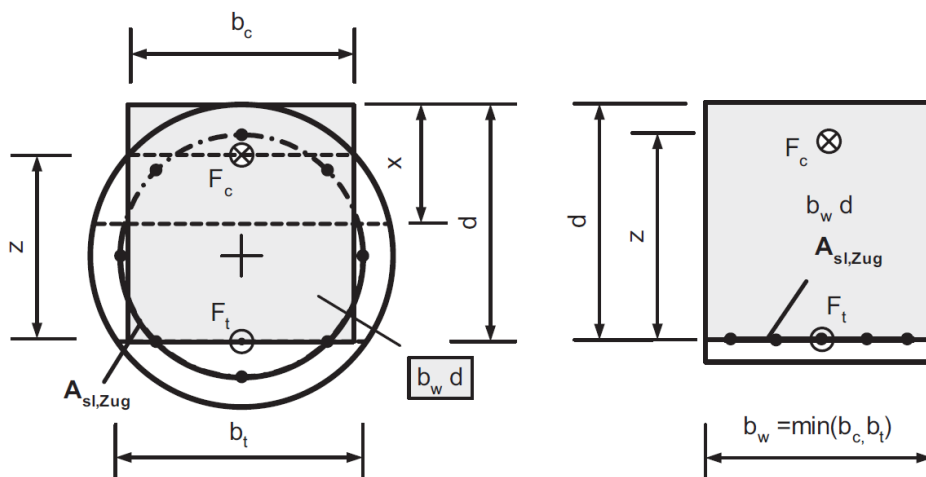
$$\rho_{w,min} = 0.15 f_{ctm} / f_{yd} \tag{24AT}$$

For slabs with a calculated shear reinforcement at least the 0.6-fold value of the minimum shear reinforcement of beams is to be applied.

**Lateral force design for circular and annular cross-sections according to Bender et al.**

For circular and annular cross sections with annular single stirrups or helixes, the lateral force design is optionally carried out according to Bender et al. (2010) for the resulting shear force  $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$ .

In its interpretation of 1 June 2012 of Chapter 10.3 of DIN 1045-1:2008, the German Committee for Structural Engineering (NABau) recommends using the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces for the effective width  $b_w$  (see following figure). The values for the width  $b_w$ , the effective height  $d$  and the inner lever arm  $z$  are defined in the cross-section dialog.



Definition of the effective width bw as per NABau (2012) [Fig. from: Bender et al. (2006), p. 87]

For structural members without shear reinforcement, the resistance  $V_{Rd,ct}$  is given according to Bender et al. (2006), Equ. (1), in accordance with DIN 1045-1:2008, Equ. (70). Therefore, the program uses the above equations (6.2a), (6.2b) and (11.6.2) of EN 1992-1-1 with the selected value for  $b_w$ .

For structural members with shear reinforcement, the design is carried out according to Bender et al. (2010):

$$V_{Rd,sy} = \alpha_k \cdot A_{sw} / s_w \cdot f_{yd} \cdot z \cdot \cot \Theta \cdot \sin \alpha \quad (10)$$

$$V_{Rd,max} = \alpha_k \cdot b_w \cdot z \cdot \alpha_c \cdot f_{cd} \cdot \cot \Theta / [(\cot \Theta \cdot \cot \alpha)^2 + 1] \quad (11)$$

where

$\alpha_k$  is an efficacy factor, which is stress-dependent ( $0.715 \leq \alpha_k \leq 0.785$ ) according to Bender et al. (2010), p. 422, and can be assumed with the mean value  $\alpha_k = 0.75$ .

$A_{sw}$  is the section area of the lateral force reinforcement per length  $s_w$ .

$s_w$  is the distance of the lateral force reinforcement as measured along the component axis.

$b_w$  is the effective cross-section width.

$z$  is the inner lever arm.

$\Theta$  is the inclination of the concrete compressive struts.

$\alpha$  is the angle between the lateral force reinforcement and the component axis (helix inclination).

$f_{yd}$  is the design value for the yield strength of the lateral force reinforcement.

$f_{cd}$  is the design value of the concrete compressive strength.

$\alpha_c$  is a coefficient to account for the stress state in the compression chord.

The additional tensile force in the longitudinal reinforcement due to lateral force  $Q_r$  is determined according to equation (6.18) of the standard. In case of simultaneous loading by lateral force and torsion, the torsion design is carried out according to the standard for vertical stirrups assuming a square torsion box.

The design results are stored separately from the standard design results.

## Design for Torsion and Combined Stressing

The design for torsion is carried out according to EN 1992-1-1, Chapter 6.3. It includes the calculation of the diagonal tensile reinforcement and the longitudinal reinforcement based on Equation (6.28) and the concrete strut check under lateral force based on Formula (6.29) of the standard.

The equivalent section on which this design is based is defined by the user independently of the normal section geometry. The check of box sections according to EN 1992-2, Figure 6.104 is not implemented.

### Strut angle

According to 6.3.2 (2), the rules set forth in Chapter 6.2.3 (2) for lateral force shall also apply for the strut angle.

DIN EN 1992-2:

For combined stress from torsion and proportional lateral force,  $V_{Ed}$  in Equation (6.7aDE) must include the shear force of the wall  $V_{Ed,T+V}$  based on Equation (NA.6.27.1) and  $b_w$  in Equation (6.7bDE) must include the effective thickness of wall  $t_{ef}$ .

The check for both lateral force and torsion must be carried out using the selected angle  $\Theta$ . The reinforcements determined in this manner are to be added together.

$$V_{Ed,T+V} = V_{Ed,T} + V_{Ed} \cdot t_{ef} / b_w \quad (NA.6.27.1)$$

Alternatively a strut angle of 45° or 30° for torsion (cf. Chapter 6.3.2 (102)) or a constant value  $\cot \Theta$  for lateral force and torsion (cf. interpretation No. 24 of NABau for DIN 1045-1) can be chosen in the section dialog. For box sections the wall thickness  $t_{ef}$  can be defined separately for both directions of the cross-section.

### Torsion reinforcement

The necessary reinforcement is to be determined according to Chapter 6.3.2 (3):

$$\Sigma A_{sl} \cdot f_{yd} / u_k = T_{Ed} / 2A_k \cdot \cot \Theta \quad (6.28)$$

or

$$A_{sw} \cdot f_{yd} / s = T_{Ed} / 2A_k \cdot \tan \Theta$$

where

$A_{sl}$  is the section area of the longitudinal torsional reinforcement. The possibility of reducing the longitudinal torsional reinforcement in compression chords is not used.

$A_{sw}$  is the section area of the torsion reinforcement perpendicular to the component axis.

$u_k$  is the perimeter of area  $A_k$ .

$s$  is the distance of the torsion reinforcement as measured along the component axis.

$A_k$  is the area enclosed by the center lines of the walls.

For approximately rectangular full sections, only the minimum reinforcement defined in Section (5) is necessary if the condition expressed by Equation (6.31) is met:

$$T_{Ed} / T_{Rd,c} + V_{Ed} / V_{Rd,c} \leq 1.0 \quad (6.31)$$

where

$T_{Rd,c}$  is the torsion crack moment which, according to Zilch (2006, p. 290), is defined as  $T_{Rd,c} = f_{ctd} \cdot W_T$ .

$V_{Rd,c}$  is the lateral force resistance according to Equation (6.2).

DIN EN 1992-1-1:

The condition (6.31) is supplemented with the following equations:

$$T_{Ed} \leq \frac{V_{Ed} \cdot b_w}{4.5} \quad (NA.6.31.1)$$

$$V_{Ed} \left[ 1 + \frac{4.5 T_{Ed}}{V_{Ed} \cdot b_w} \right] \leq V_{Rd,c} \quad (NA.6.31.2)$$

### Strut load-bearing capacity

To avoid exceeding the strut load-bearing capacity of a component subject to torsion and lateral force, the following condition must be met:

$$T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max} \leq 1.0 \quad (6.29)$$

where

$T_{Ed}$  is the design value of the torsion moment.

$V_{Ed}$  is the design value of the lateral force.

$T_{Rd,max}$  is the design value of the absorbable torsion moment based on

$$T_{Rd,max} = 2 v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t_{ef,i} \cdot \sin \Theta \cdot \cos \Theta \quad (6.30)$$

with  $\alpha_{cw}$  according to Equation (6.9) and  $v$  according to Eq. (6.6N) for normal concrete and according to Eq. (11.6.6N) for lightweight concrete.

DIN EN 1992-1-1:

For compact sections, the interaction Equation (NA.6.29.1) is used:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \quad (NA.6.29.1)$$

In Equation (6.30)  $v = \eta_1 \cdot 0.75$  is used for box sections and  $v = \eta_1 \cdot 0.525 \cdot \min(1.0; 1.1 - f_{ck} / 500)$  in all other cases according to the NPD for 6.2.2 (6) with  $\eta_1 = 1.0$  for normal concrete and as per Eq. (11.1) for lightweight concrete.

OENORM B 1992-1-1:

For full sections the following interaction equation can be used:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \quad (9AT)$$



## Check of Principal Compressive Stress for Combined Actions

If box sections are stressed by torsion in combination with lateral forces, bending moments and normal forces, this may result in critical principal stresses within the compressive zone according to DIN EN 1992-2, Chapter 6.3.2 (NA.106). In such cases, the principal compressive stresses in uncracked areas shall not exceed the value  $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$ .

The principal compressive stresses  $\sigma_2$  are to be determined in state I from the average longitudinal stress  $\sigma_{Ed}$  and the shear stress  $\tau_{Ed,T+V}$  from lateral force  $\tau_{Ed,V}$  and torsion  $\tau_{Ed,T} = T_{Ed} / (2 \cdot A_k \cdot t_{eff})$ . If the principal tensile stress  $\sigma_1$  exceeds the characteristic concrete tensile strength  $f_{ctk;0.05}$ , the principal compressive stress is to be calculated in state II based on the truss analogy. In this case, the design value of the compressive strength  $f_{cd}$  should be reduced adequately.

In the program, the check is carried out during torsional design, if the option *box section* is chosen on the shear section tab of the cross-section dialog. When performing the check, three cases are distinguished, with the main tensile stresses  $\sigma_1$  being determined in cross-section areas under longitudinal pressure.

- $\sigma_1 \leq f_{ctk;0.05}$   
To determine the principal compressive stress  $\sigma_2$  according to state I, the longitudinal compressive stress  $\sigma_{Ed}$  is determined at a distance  $t_{eff} / 2$  from the cross-sectional edge. The shear stress from lateral force  $\tau_{Ed,V}$  is calculated according to the boundary element method and averaged over the cross-sectional width. The constant shear stress from torsion is assumed to be  $\tau_{Ed,T} = T_{Ed} / (2 \cdot A_k \cdot t_{eff})$ .
- $\sigma_1 > f_{ctk;0.05}$   
In state II, the principal compressive stress is determined on the truss model. The strut angles  $\theta_V$  and  $\theta_T$  are assumed as in the shear design. The compressive strength is reduced to  $\alpha_{cw} \cdot v_1 \cdot f_{cd}$  with  $\alpha_{cw}$  as per NDP to EN 1992-2, Chapter 6.2.3 (103), and  $v_1$  as per NCI to EN 1992-2, Chapter 6.3.2 (104).
- Selection of "*Check  $\sigma_2$  in state I*" in the cross-section dialog  
According to DIN Technical Report 102:2003, the principal compressive stress  $\sigma_2$  is calculated basically in state I from the longitudinal stress  $\sigma_{Ed}$  at a distance  $t_{eff} / 2$  from the cross-sectional edge and the constant shear stress from torsion  $\tau_{Ed,T} = T_{Ed} / (2 \cdot A_k \cdot t_{eff})$ .

## Shear Joint Check

The shear joint check is performed for beam elements and design objects in accordance with EN 1992-1-1, Chapter 6.2.5. It is carried out for the shear force in the z-direction of the cross-section and is only useful for components that are mainly stressed in this direction. The transmission of shear force through the joint is ensured if the following conditions are met:

$$v_{Edi} \leq v_{Rdi} \quad (6.23)$$

**Design value of the shear stress in the joint  $v_{Edi}$ :**

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) \quad (6.24)$$

where

$\beta$  is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered. The ratio factor  $\beta$  is calculated depending on whether the shear joint is in the compression or tension zone and whether reinforcement in the existing concrete or in the concrete supplement was determined from the other checks with the internal forces associated with  $V_{Ed}$  in state II as follows (see also Booklet 600 to 6.2.5 (1)):

Outer edge of concrete supplement	Shear joint	Tensile reinforcement		$\beta$
		Concrete supplement	Existing Concrete	
Tension	Ten. / Comp.	-	-	1
Compression	Compression	-	-	$0 < F_{cdi} / F_{cd} < 1$
	Tension	Yes	Yes	$0 < F_{sd} / (F_{sd} + F_{sdi}) < 1$
		Yes	No	0 (No check)
		No	Yes	1
		No	No	0 (No check)

$V_{Ed}$  is the design value of the lateral force in the z direction.

$z$  is the lever arm of the composite section. The lever arm is assumed as in the lateral force check according to the specifications in the shear section.

$b_i$  is the width of the joint.

$F_{cdi}$  is the concrete compression force in the concrete supplement in the compression zone.

$F_{cd}$  is the total concrete compression force in the compression zone.

$F_{sdi}$  is the tension force of the reinforcing steel layers in the concrete supplement in the tensile zone.

$F_{sd}$  is the total tension force of the reinforcing steel layers in the tensile zone.

**Design value of the shear resistance in the joint  $v_{Rdi}$ :**

$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} (\mu \cdot \sin \alpha + \cos \alpha) \leq 0.5 \cdot v \cdot f_{cd} \quad (6.25)$$

DIN EN 1992-1-1:

The load bearing component of the transverse reinforcement from the shear friction in Eq. (6.25) may be increased to  $\rho \cdot f_{yd} (1.2 \cdot \mu \cdot \sin \alpha + \cos \alpha)$ .

For very smooth joints without external compression normal force perpendicular to the joint, the friction component in Eq. (6.25) may be taken into account up to the limit  $\mu \cdot \sigma_n \leq 0.1 \cdot f_{cd}$  according to NPD 6.2.2 (6).

where

$c, \mu$  are factors which depend on the roughness of the joint (see 6.2.5 (2)). Under dynamic or fatigue loads, the value  $c = 0$  is assumed according to EN 1992-2, Chapter 6.2.5 (105).

$f_{ctd}$  is the design value of the concrete tensile strength according to 3.1.6 (2)P.

$\sigma_n$  is the stress caused by the minimum normal force perpendicular to the joint, which can act simultaneously with the lateral force (positive for compression with  $\sigma_n < 0.6 f_{cd}$  and negative for tension). If  $\sigma_n$  is a tensile stress  $c \cdot f_{ctd}$  should usually be set to 0.

$$\rho = A_s / A_i$$

$A_s$  is the area of reinforcement crossing the joint, including ordinary shear reinforcement, with adequate anchorage at both sides of the interface.

$A_i$  is the area of the joint.

$\alpha$  the angle of inclination of the transverse reinforcement. This is set at 90° by the program.

$v$  is a strength reduction factor according to 6.2.2 (6).

OENORM B 1992-1-1:

If reinforcement is required, the following minimum reinforcement must be inserted perpendicular to the joint for beam-like components:

$$\rho_{\min} = 0.2 \cdot f_{\text{ctm}} / f_{\text{yk}} \geq 0.001$$

## Punching Shear

The punching shear check is based on EN 1992-1-1, Chapter 6.4. For more information on this, refer to section *EN 1992-1-1* of the manual. This check is accessed from the *Structure* menu.

## Checks Against Fatigue

The following checks according to EN 1992-2 in conjunction with EN 1992-1-1 are available:

- Fatigue of longitudinal reinforcement, shear reinforcement and prestressing steel (Chapter 6.8.5, 6.8.6)
- Fatigue of concrete under compressive stress (Chapter 6.8.7)
- Fatigue of the concrete compressive struts under lateral force and torsion (Chapter 6.8.7 (3))

The user can select two alternative methods for design:

- Simplified check for the frequent action combination according to EN 1992-1-1, Chapter 6.8.6 (2), and EN 1990, Eq. (6.15b), taking the relevant traffic loads at serviceability limit state into account.
- Check with damage equivalent stress ranges for the fatigue combination according to EN 1992-1-1, Chapter 6.8.3, Eq. (6.69), considering the specific fatigue load  $Q_{fat}$ .

The concrete compressive stresses are determined for both cases in state II. Differing from Chapter 5.10.9 the variation of prestressing is not taken into account.

In the case of road bridges  $Q_{fat}$  corresponds to fatigue model 3 according to EN 1991-2, Chapter 4.6.4. The increase factors according to EN 1991-2, Chapter 4.6.1 (6), must be taken into account when entering the load coordinates while the factors according to NN.2.1 (101) are defined in the section dialog.

According to EN 1991-2, Chapter 6.9 in conjunction EN 1992-2, Annex NN.3, the traffic model 71 including the dynamic factor as per EN 1991-2 plays the determinant role in calculating the stress range for railroad bridges.

DIN EN 1992-2:

According to Chapter 6.8.3 (1)P, the 0.9-fold statically determined portion of the prestressing force is to be used in the check. For construction joints with tendon couplers, this value is to be reduced by the factor 0.75. The decisive reduction factor is defined in the Section dialog.

## Design Combinations

For the check against fatigue two alternative action combinations can be used:

- Frequent combination for simplified checks according to EN 1992-1-1, Chapter 6.8.6 (2) in conjunction with EN 1990, Chapter 6.5.3.

$$\sum_{j \geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} \cdot Q_{k,1} \text{ "+" } \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.15b)$$

- Fatigue combination for checks with damage equivalent stress ranges according to EN 1992-1-1, Chapter 6.8.3.

$$\left( \sum_{j \geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} \cdot Q_{k,1} \text{ "+" } \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right) \text{ "+" } Q_{fat} \quad (6.69)$$

In this equation  $Q_{k,1}$  and  $Q_{k,i}$  are non-cyclic, non-permanent actions whereas  $Q_{fat}$  defines the relevant fatigue load.

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

## Stress-Strain Curves

For checks against fatigue the following characteristics apply:

- Concrete: Stress-strain curve according to EN 1992-1-1, Figure 3.2, where a horizontal curve is assumed for strains of  $\epsilon_{c1}$  or higher (cf. Rossner, Graubner 2012, p. 511 and Nguyen et al. 2015, p. 158).
- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be  $k \cdot f_{yk}$  with  $k = 1.05$  as per Table C.1, class A.

DIN EN 1992-2:

The maximum stress is assumed to be  $1.05 \cdot f_{yk} / \gamma_s$  for ductility class A according to DIN 488-1.

- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of  $f_{p,0,1k}$ .

## Fatigue of Longitudinal Reinforcement, Shear Reinforcement and Prestressing Steel

The fatigue check is carried out according to EN 1992-1-1, Chapter 6.8. The steel stresses are calculated for longitudinal reinforcement from bending and longitudinal force as well as for prestressing steel in beams and design objects under the assumption of a cracked concrete section. For shear and longitudinal reinforcement from lateral force and torsion, the stresses are calculated according to 6.8.2 (3) based on a truss model with the strut angle  $\tan \Theta_{\text{fat}} = \sqrt{\tan \Theta} \leq 1$  acc. to Eq. (6.65). Where  $\Theta$  is the angle between the concrete compression struts and the beam axis used in the corresponding ultimate limit state design. The prestressing steel stresses in area elements are determined at the uncracked concrete section. Tendons without bond and external tendons are not checked.

DIN EN 1992-1-1:

The strut angle is to be determined according to Eq. (H.6-26) of Book 600 of the DAfStb.

### Simplified check

According to Chapter 6.8.6, adequate fatigue resistance may be assumed if the stress range under the frequent action combination does not exceed 70 MN/m<sup>2</sup> for unwelded reinforcing bars and 35 MN/m<sup>2</sup> for welded bars. The condition described in Chapter 6.8.6 (3) for couplings in prestressed components is not examined by the program.

DIN EN 1992-1-1:

The simplified check is not permitted for welded reinforcing bars.

### Check with damage equivalent stress ranges

According to Chapter 6.8.5 (3), for reinforcing and prestressing steel adequate fatigue resistance should be assumed if Eq. (6.71) is satisfied:

$$\gamma_{F,\text{fat}} \cdot \Delta\sigma_{s,\text{equ}}(N^*) \leq \Delta\sigma_{\text{Rsk}}(N^*) / \gamma_{s,\text{fat}} \quad (6.71)$$

with

$$\gamma_{F,\text{fat}} = 1 \text{ according to Chapter 2.4.2.3 and 6.8.4 (1).}$$

$$\gamma_{s,\text{fat}} = 1.15 \text{ for reinforcing and prestressing steel according to Chapter 2.4.2.4.}$$

$\Delta\sigma_{\text{Rsk}}(N^*)$  Permitted characteristic stress range at  $N^*$  load cycles based on the S-N curves specified in Tab. 6.4N for prestressing steel or Tab. 6.3N for reinforcing steel.

$\Delta\sigma_{s,\text{equ}}(N^*)$  Damage equivalent stress range with  $\Delta\sigma_{s,\text{equ}} = \lambda_s \cdot \Delta\sigma_s$  according to EN 1992-2, Eq. (NN.101) and Eq. (NN.106).

$\lambda_s$  Correction coefficient according to EN 1992-2, Annex NN.2 and NN.3.

$\Delta\sigma_s$  Maximum stress range from the cyclic fatigue action  $Q_{\text{fat}}$

### Calculation method

The maximum from the robustness, crack and bending reinforcement is taken as the existing bending reinforcement. If as a result the load from the fatigue combination in state II cannot be absorbed, the design will be repeated using the existing reinforcement and the check internal forces.

The maximum stress range per steel layer that results from the strain state in state II or the truss model is determined separately for each check situation. For longitudinal reinforcement the varying bond behavior of reinforcing and prestressing steel is taken into account by increasing the steel stress by the coefficient  $\eta$  from Eq. (6.64). If for longitudinal and shear reinforcement the resulting stress range exceeds the permitted stress range, the necessary reinforcement will be iteratively increased until the check succeeds for all situations. In the *Symmetrical* and *Compression member* design modes the longitudinal reinforcement is applied at all predefined locations. This will not affect the predefined relationships between the individual reinforcement layers.

The permitted stress ranges and the coefficients  $\eta$  and  $\lambda$  are specified by the user in the Section dialog. The decisive reinforcement used for the check, which may have been increased, is recorded in the check log and saved for graphical representation.

### Special Case

According to EN 1992-2, Chapter 6.8.1 (102), the fatigue check for reinforcing and prestressing steel is not necessary in areas where under the frequent action combination and  $P_k$  only compressive stress occurs at the prestressed cross-section edge. Selection of this option is provided in the cross-section dialog.

DIN EN 1992-2:

This option is applicable at superstructures for reinforcing and prestressing steel without welding joints or couplings when decompression is checked under the frequent combination.

OENORM B 1992-2:

This option is applicable for reinforcing and prestressing steel without welding joints, as far as only compressive stress occurs under the frequent combination as per Table 2AT.

On the program side, it is checked whether the cross-sectional edge closest to the tendon is under compression under the frequent action combination with the characteristic values  $P_{k,sup}$  and  $P_{k,inf}$  of prestressing. In case of ambiguous tendon guidance, both sides are examined. The user is responsible for verifying the application requirements in accordance with the standard.

## Fatigue of Concrete Under Longitudinal Compressive Stress

The fatigue check of concrete that is subject to compressive stress is performed for bending and longitudinal force at the cracked section. This check takes into account the final longitudinal reinforcement including an potential increase applied during the fatigue check of reinforcing steel.

DIN EN 1992-2, OENORM B 1992-2:

In the case of road bridges, the check according to NDP to Chapter (102) is not necessary if the concrete compressive stresses under the characteristic action combination are limited to  $0,6 \cdot f_{ck}$ . This is the case if the check of the concrete compressive stresses in the serviceability state is carried out with the limit of  $0,6 \cdot f_{ck}$  according to Chapter 7.2.

### Simplified check

Adequate fatigue resistance may be assumed if the following condition is satisfied:

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.5 + 0.45 \frac{\sigma_{c,min}}{f_{cd,fat}} \leq 0.9 \text{ for } f_{ck} \leq 50 \text{ MN/m}^2 \quad (6.77)$$

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.8 \text{ for } f_{ck} > 50 \text{ MN/m}^2$$

with

$\sigma_{c,max}$  the maximum compressive stress at a fibre under the frequent load combination (compression measured positive).

$\sigma_{c,min}$  the minimum compressive stress at the same fibre where  $\sigma_{c,max}$  occurs ( $\sigma_{c,min} = 0$  if  $\sigma_{c,min}$  is a tensile stress).

$f_{cd,fat}$  the design fatigue strength of concrete according to Eq. (6.76). This value is entered by the user in the Section dialog.

$$f_{cd,fat} = k_1 \cdot \beta_{cc}(t_0) \cdot f_{cd} \cdot (1 - f_{ck}/250) \text{ with } \beta_{cc}(t_0) \text{ as per Eq. (3.2) and } f_{cd} \text{ as per Eq. (3.15)} \quad (6.76)$$

$$k_1 = 0.85$$

DIN EN 1992-1-1, OENORM B 1992-1-1:

$$k_1 = 1.0$$

### Check with damage equivalent concrete compressive stresses

The check for railroad bridges has to be executed according to Annex NN3.2 of EN 1992-2. A satisfactory fatigue resistance may be assumed for concrete under compression, if following equation is fulfilled:

$$14 \cdot (1 - E_{cd,max,equ}) / \sqrt{1 - R_{equ}} \geq 6 \quad (NN.112)$$

where

$R_{equ} = E_{cd,min,equ} / E_{cd,max,equ}$  is the stress ratio.

$E_{cd,min,equ} = \gamma_{sd} \cdot \sigma_{cd,min,equ} / f_{cd,fat}$  is the minimum compressive stress level.

$E_{cd,max,equ} = \gamma_{sd} \cdot \sigma_{cd,max,equ} / f_{cd,fat}$  is the maximum compressive stress level.

$\sigma_{cd,min,equ}$  is the lower stress of the damage equivalent stress range for  $N = 10^6$  cycles.

$\sigma_{cd,max,equ}$  is the upper stress of the damage equivalent stress range for  $N = 10^6$  cycles.

$f_{cd,fat}$  is the design fatigue strength of concrete according to Eq. (6.76).

$\gamma_{sd}$  a coefficient which is assumed to be  $\gamma_{sd} = 1$ .

DIN EN 1992-2:

Coefficient  $\gamma_{sd}$  corresponds to coefficient  $\gamma_{Ed,fat}$ . Eq. (NN.112) is equivalent to Eq. (NA.NN.12).

$$E_{cd,max,equ} + 0.43 \cdot \sqrt{1 - R_{equ}} \leq 1,0 \quad (NA.NN.12)$$

The upper and lower stress of the damage equivalent stress range shall be calculated with the equation (NN.113).

$$\begin{aligned}\sigma_{cd,max,eq} &= \sigma_{c,perm} + \lambda_c (\sigma_{c,max,71} - \sigma_{c,perm}) \\ \sigma_{cd,min,eq} &= \sigma_{c,perm} - \lambda_c (\sigma_{c,perm} - \sigma_{c,min,71})\end{aligned}\quad (NN.113)$$

with

$\sigma_{c,perm}$	Compressive stress under the fatigue combination without load model 71.
$\sigma_{c,max,71}$	Minimum and maximum compressive stress under the fatigue combination with load model 71 and the dynamic coefficient $\Phi$ .
$\sigma_{c,min,71}$	
$\lambda_c$	Correction coefficient for the calculation of the stresses caused by load model 71.

## Fatigue of the Concrete Compressive Struts Under Lateral Force and Torsion

Fatigue of the concrete compressive struts is examined for beams and design objects. The check differentiates between components with and without calculatory required lateral force. In the case of combined loads from lateral force and torsion, the supplementary condition according to EN 1992-1-1, Chapter 6.3.2 (5) is checked in addition to the condition in Chapter 6.2.1 (5).

DIN EN 1992-1-1: In addition, the equations according to NCI for 6.3.2 (5) are evaluated.

### Components with required lateral force reinforcement

The fatigue check for concrete under compressive stress as per Chapter 6.8.7, is also applicable for verifying the concrete compressive struts of components with required lateral force reinforcement as per Chapter 6.8.7 (3).

In the case of vertical stirrups ( $\alpha = 90^\circ$ ), the design values  $\sigma_{cd,max}$  and  $\sigma_{cd,min}$  of the maximal and minimal compressive stress may be determined according to the following equations while assuming an identical compressive strut angle  $\theta$  for lateral force and torsion as well:

$$\sigma_{cd,T} = \frac{T_{Ed}}{2 \cdot A_k \cdot t_{ef}} \cdot (\cot \Theta + \tan \Theta)$$

$$\sigma_{cd,V} = \frac{V_{Ed}}{b_w \cdot z} \cdot (\cot \Theta + \tan \Theta)$$

$$\sigma_{cd,max} = \begin{cases} \max \sigma_{cd,T} + \text{cor} \cdot \sigma_{cd,V} \\ \max \sigma_{cd,V} + \text{cor} \cdot \sigma_{cd,T} \end{cases}$$

$$\sigma_{cd,min} = \begin{cases} \min \sigma_{cd,T} + \text{cor} \cdot \sigma_{cd,V} \\ \min \sigma_{cd,V} + \text{cor} \cdot \sigma_{cd,T} \end{cases}$$

The program performs the check depending on the user's selection either according to the simplified method as per Eq. (6.77), for the frequent combination or by using the damage equivalent stress range as per EN 1992-2, Annex NN.3.2, for the fatigue combination given in EN 1992-1-1, Chapter 6.8.3, Eq. (6.69).

When performing the simplified check under pure lateral force load, the concrete strength  $f_{cd,fat}$  should be reduced by the factor  $v \cdot \eta_1$  as per Chapter 6.2.2 (6). In case of combined stressing from lateral force and torsion, the reduction factor  $v \cdot \alpha_{cw} \cdot \eta_1$  with  $\alpha_{cw}$  as per Eq. (6.9) applies. The coefficient  $\eta_1$  should be set to 1 for normal concrete and according to Eq. (11.1), for light weight concrete. In case of internal prestressing, the nominal dimension according to user specification is used for  $b_w$ .

DIN EN 1992-2:

The following factors apply:

$$v_1 = 0.75 \cdot \eta_1 \quad \text{in case of pure lateral force as per NDP 6.2.3 (103).}$$

$$v_2 = (1.1 - f_{ck} / 500) \leq 1.0 \quad \text{acc. to DIN EN 1992-1-1, NCI 6.8.7 (3).}$$

$$v \cdot \alpha_{cw} \cdot \eta_1 = 0.525 \cdot \eta_1 \quad \text{in case of combined stressing as per NDP 6.2.2 (6).}$$

### Components without required lateral force reinforcement

For components without required lateral force reinforcement at ultimate limit state, adequate fatigue resistance against lateral force load may be assumed according to EN 1992-1-1, Chapter 6.8.7 (4), if the following conditions are satisfied:

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} \geq 0: \frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5 + 0.45 \cdot \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \leq 0.9 \text{ for concrete up to } C50/60 \quad (6.78)$$

$$\frac{|V_{Ed,min}|}{|V_{Rd,c}|} \leq 0.8 \text{ for concrete } C55/67 \text{ or higher}$$

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} < 0: \frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5 - \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \quad (6.79)$$

where

$V_{Ed,max}$  is the design value of the maximum lateral force under the frequent action combination.

$V_{Ed,min}$  is the design value of the minimum lateral force under the frequent action combination at the cross-section where  $V_{Ed,max}$  occurs.

$V_{Rd,c}$  is the design value of the absorbable lateral force without shear reinforcement as per Eq. (6.2a).

For performing the check, the program selects automatically the simplified method with the frequent action combination.

## Special Characteristic of Shell Structures

In shell structures the strain state at the cracked concrete section under general stress cannot be determined unambiguously. The design is therefore carried out separately for the reinforcement directions x and y with the design internal forces from Wolfensberger/Thürlimann or Rüsçh as described above. The reinforcement calculated in this manner yields a reliable load-bearing capacity.

When calculating the stress range for reinforcing steel and concrete, this method can lead to unrealistic results in the case of torsional or shear stresses as shown in the following example:

Assume two identical sets of slab internal forces:

Set	$m_x$ [kNm/m]	$m_y$ [kNm/m]	$m_{xy}$ [kNm/m]
1	300	200	100
2	300	200	100

According to Wolfensberger/Thürlimann, this results in design variants for the x direction:

Set	Variant	$m$ [kNm/m]
1	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$
2	1	$m_x +  m_{xy}  = 400$
	2	$m_x -  m_{xy}  = 200$

The torsional moments generate a variation of the design moments and thus a calculatory stress range. This may lead to a necessary reinforcement increase in the fatigue check due to apparent overstressing. For normal design forces, this applies correspondingly to the shear forces.

Selecting **Limit design variants** in the Section dialog allows you to avoid the described effect. In this case only the corresponding variants are compared when determining the stress range, i.e. only the first and second variants of both sets in this example. Assuming constant stress, the stress range is thus correctly determined to be zero.

**This alternative, however, does not ensure that all conceivable stress fluctuations are analyzed. You should therefore be particularly careful when assessing the results. For this purpose the detailed log indicates the main variants and design internal forces used for the check.**

When determining the design internal forces according to Rüsçh for inclined reinforcement, the described relationships apply accordingly.

If the check is performed with the action combination according to equation (6.69) of the standard, the design internal forces are determined from the sum of non-cyclic and cyclic internal forces of the area elements.



## Checks in the Serviceability Limit States

The following checks are performed according to EN 1992-2 in conjunction with EN 1992-1-1:

- Limiting the concrete compressive stresses (Chapter 7.2).
- Limiting the reinforcing steel stresses (Chapter 7.2).
- Limiting the prestressing steel stresses (Chapter 7.2).
- Decompression check (Chapter 7.3.1).
- Minimum reinforcement for crack width limitation (Chapter 7.3.2).
- Crack width calculation (Chapter 7.3.4).
- Limiting diagonal principle tensile stresses (DIN EN 1992-2, Chapter 7.3.1).
- Limiting deformations (Chapter 7.4).

## Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.5.3, the following combinations are taken into account in the serviceability limit states:

- Combination for characteristic situations

$$\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i > 1} \psi_{0,i} \cdot Q_{k,i} \quad (6.14b)$$

- Combination for frequent situations

$$\sum_{j \geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.15b)$$

- Combination for quasi-continuous situations

$$\sum_{j \geq 1} G_{k,j} + P + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.16b)$$

DIN EN 1992-2:

Additionally, the design combinations according to Tab. 7.101DE for road bridges and Tab. 7.102DE for railroad bridges are taken into account.

OENORM EN 1992-2:

Additionally, the design combinations according to Tab. 2AT are taken into account.

For each combination you can define different design situations for the construction stages and final states. If necessary, the combination required by the check will automatically be determined from the section specifications. Each check is carried out for all the situations of a combination.

## Stress-Strain Curves

For checks in the serviceability limit states the following characteristics apply:

- Concrete: Stress-strain curve according to EN 1992-1-1, Figure 3.2, where a horizontal curve is assumed for strains of  $\epsilon_{c1}$  or higher (cf. Interpretation No. 098 of the NABau for DIN TR 102).
- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be  $k \cdot f_{yk}$  with  $k = 1.05$  as per Table C.1, class A.

DIN EN 1992-2:

The maximum stress is assumed to be  $1.05 \cdot f_{yk}$  for ductility class A according to DIN 488-1.

- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of  $f_{p;0,1k}$ .

## Stress Analysis

For uncracked concrete sections, the program assumes that concrete and steel under tensile and compressive stress behave elastically. As for cracked concrete sections, the concrete compressive stresses are determined using the aforementioned stress-strain curve.

### Area elements

For area elements the concrete stresses are calculated at the gross section. The steel stress check is carried out for reinforcing steel by determining the strain state at the cracked concrete section and for the prestressing steel at the uncracked concrete section.

### Beams and design objects

The action combination stresses that can be determined without checks are always calculated at the gross section.

Conversely, in the checks the stresses are determined as follows and are graphically displayed or logged:

- When checking the crack reinforcement and crack width, the concrete stress is calculated at the gross section
- When checking the decompression, concrete compressive stresses and the diagonal principal tensile stresses the bending stress is calculated
  - without internal tendons at the gross section
  - with internal tendons without bond at the net section
  - with internal tendons with bond for situations before being grouted at the net section or otherwise at the ideal section
- The reinforcing and prestressing steel stresses are checked by determining the strain state at the cracked concrete section

When checking the crack reinforcement and crack width, the decompression, the concrete compressive and diagonal tensile stresses, the longitudinal force is based on the area of the full section (cf. Rossner/Graubner 2012, p. 228).

OENORM B 1992-1-1:

If the stresses according to Chapter 7.2 are calculated at the cracked section the different bonding behavior of concrete and prestressing steel is to be taken into account. The increase of tension force  $\Delta F_{tp}$  in the prestressing steel is to be calculated as follows:

$$\Delta F_{tp} = \xi_1^2 \cdot A_p \cdot \varepsilon(y_p) \cdot E_p \quad (13AT)$$

where

$\xi_1$  is the bond coefficient according to Eq. (7.5); the value can be entered in the crack width check dialog;

$A_p$  is the section area of the tendon with bond;

$\varepsilon(y_p)$  is the strain of the concrete section at the location  $y_p$  of the tendon;

$E_p$  is the elasticity modulus of the tendon.

For beams and design objects this rule is taken into account by the program for situations after grouting. For area elements it is not used because prestressing steel is only checked at the uncracked section.

## Limiting the Concrete Compressive Stresses

The concrete compressive stress check is carried out according to EN 1992-1-1, Chapter 7.2. As described in Chapter 7.1 (2), a cracked section is assumed if the tensile stress calculated in the uncracked state exceeds  $f_{ctm}$ .

The calculation in the cracked state is performed by determining the strain state with the final longitudinal reinforcement (maximum from robustness, crack and bending reinforcement including a possible increase from the fatigue check). For beams and design objects, the tendons with bond are taken into account on the resistance side provided that they are grouted in the check situation. For area elements, the compressive stress for both reinforcement directions is determined separately and the extreme value is checked since the general strain state cannot be determined unambiguously.

In the construction stages and final states for exposure classes XD, XF and XS the concrete compressive stress  $\sigma_c$  as defined in EN 1992-2, Chapter 7.2 (102) is to be limited to  $0.60 f_{ck}$  under the characteristic combination. The limit may be increased by 10% if the concrete compressive zone is helically reinforced (e.g., by lateral reinforcement). If stress in the concrete under quasi-continuous combination does not exceed the limit  $0.45 f_{ck}$ , linear creep can be assumed according to EN 1992-1-1, Chapter 7.2 (3). If this is not the case, non-linear creep must be taken into account. Both conditions are considered based on the user's specifications.

In prestressed concrete components as per EN 1992-1-1, Chapter 5.10.2.2, the maximum concrete compressive stress must be limited to  $0.60 f_{c(t)}$  when entering the average prestressing value. If the concrete compressive stress exceeds the value  $0.45 f_{c(t)}$ , the nonlinearity of the creep must be taken into account.  $f_{c(t)}$  indicates the average value of the concrete compressive strength at time  $t$  when the prestressing is entered.

The program assumes the time of introducing the prestressing to coincide with situation  $G1+P$ . If a situation  $G1+P$  is defined in the combination selected above, the concrete stress is checked against the limit value  $0.45 f_{c(t)}$  or  $0.60 f_{c(t)}$  for this situation depending on the user's specification. The value for  $f_{c(t)}$  is also defined in the dialog.

OENORM B 1992-2:

An increase of the stress limit is not permitted, even if the compressive zone is helically reinforced.

## Limiting the Reinforcing and Prestressing Steel Stresses

### Reinforcing steel

For reinforcing steel, the limitation of steel stress under the characteristic combination is checked for  $0.8 f_{yk}$  or  $1.0 f_{yk}$  depending on the user's selection according to EN 1992-1-1, Chapter 7.2 (5). The increased limit is permissible for stresses from indirect actions. In this check the reinforcement corresponds to the maximum value from the robustness, crack and bending reinforcement, including a possible increase as a result of the fatigue check. The determination of the strain state is performed at the cracked concrete section. If tendons with bond are grouted in the check situation, they will be taken into account on the resistance side for beam and design objects.

SS EN 1992-1-1:

According to Article 19, the limit  $1.0 f_{yk}$  is generally assumed.

### Prestressing steel

For tendons with bond, the limitation of steel stress is checked at the cracked concrete section for beams and design objects and at the uncracked concrete section for area elements. This check is based on the limit  $0.75 f_{pk}$  under the characteristic action combination.

DIN EN 1992-1-1:

The check is carried out for the quasi-continuous combination with the limit  $0.65 f_{pk}$ . In addition, the stresses are checked against the minimum of  $0.9 f_{p0,1k}$  and  $0.8 f_{pk}$  under the characteristic combination.

For situations before prestressing and for tendons without bond, the stress  $\sigma_{pm0}(x)$  is checked according to EN 1992-1-1, Equation (5.43). External tendons are not checked.

## Decompression Check

This check is to be carried out for prestressed components with bond of exposure classes XC2-XC4, XD1-XD3 and XS1-XS3 as per Table 7.1N in Chapter 7.3.1. According to this, it is to prove that the concrete section within a distance of 100 mm from the tendon or duct remains under compression. The decisive action combination results from the selected exposure class or according to the user specification.

For beams and design objects, the analysis is carried out for stresses resulting from bending and normal force. A cracked section according to Chapter 7.1 (2) is assumed in this analysis in case the tensile stress under the decisive action combination exceeds  $f_{ctm}$ . In addition, the rules for stress analysis indicated above apply.

For area sections, an uncracked section is assumed. The 2D concrete stress applied in the direction of the tendon is decisive for the check.

The result is indicated as the 'compression depth' which refers to the shortest distance between the tendon or duct and the tensile zone or section edge. This value is negative if the tendon is in the tensile zone.

OENORM B 1992-2:

Table 2AT is decisive with the action combination according to the exposure class and the supplements of note c. In accordance with Chapter 9.2.1 it is to prove that the concrete section within a distance of 200 mm from the tendon or duct remains under compression. For area elements the check is performed for the stress according to the user's selection. The simplification permitted by the National Annex, to limit the stress of the edge fiber in the precompressed tensile zone alternatively, is not used in the program.

DIN EN 1992-2:

The check conditions result either from the table 7.101DE (road bridges) or 7.102DE (railroad bridges) depending on the selected type of structure and are independent from the exposure class. In accordance with 7.3.1 (105) in the final state tensile stresses are not permitted to occur at the edge that is directly adjacent to the tendon. For construction stages edge stresses of  $0.85 \cdot f_{ctk;0.05}$  or the limitation stresses specified in Table 7.103DE are permitted depending on the type of member. For the check, a non-cracked concrete section is assumed.

The program determines the above edge as follows:

- Beams and design objects: If the edge point next to the tendon is above the centroid, the stress on the upper side of the section will be checked. If not, the lower side of the section will be checked.
- Area elements: The check will be carried out for the upper or lower section edge if the tendon next to the check point is located above or below the centroid level of the element in question. Tendons outside of the element are taken into account at a distance of up to five times the section height.

If the tendon guide is ambiguous, the check will be carried out for both sides. For area elements, the principal tensile stress  $\sigma_1$  or one of the longitudinal tensile stresses  $\sigma_x$  or  $\sigma_y$  is decisive depending on the user's selection. The latter can be used to limit the check to the direction of the prestressing if the internal force systems are appropriately aligned.

## Concrete Tensile Stresses in Bridge Transverse Direction

DIN EN 1992-2:

If prestressing occurs in the bridge longitudinal direction and the bridge lateral direction is designed as a non-prestressed or as a non-bonded prestressed construction, a check must be carried out as per Table 7.101DE (road bridges) or Table 7.102DE (railroad bridges) to ensure that the concrete tensile stresses in the bridge lateral direction determined in state I under the rare action combination do not exceed the values found in Table 7.103DE.

If necessary, the check can be carried out in the graphical user interface by controlling the edge stresses from the rare combination.

## Minimum Reinforcement for Crack Width Limitation

The minimum reinforcement for crack width limitation is defined in EN 1992-1-1, Chapter 7.3.2. According to 7.3.2 (1), the minimum reinforcement is to be applied in areas where tension is expected. Tension areas can be defined in the section dialog by choosing either an action combination or a restraint (bending, central tension). Reinforcing steel layers that are not under tension are also provided with reinforcement in the *symmetrical* and *compression member* design modes. This will not affect the predefined relationships between the individual reinforcement layers.

For profiled sections, each subsection (web or flange) should be checked individually in accordance with EN 1992-2, Chapter 7.3.2, Figure 7.101. This cannot be done if any polygonal section geometries are taken into consideration. For this reason, the program always determines the minimum reinforcement based on the entire section. The coefficient  $k_c$  is calculated according to user specification either as per EN 1992-1-1, Eq. (7.2) or as per Eq. (7.3), optionally different for the top and bottom of the cross-section.

SS EN 1992-2:

The permissible crack widths are defined in Article 8, Table D-5, depending on the exposure class, the service life class and the type of prestressing. If the tensile stress does not exceed  $f_{ctk} / \zeta$  with  $\zeta$  as per SS EN 1992-1-1, Article 21, Table D-3 (EKS 11), the concrete may be regarded as uncracked. In this case no minimum reinforcement is determined. The program assumes  $f_{ctk} = f_{ctk;0,05}(t) = 0,7 \cdot f_{ctm}(t) = 0,7 \cdot f_{ct,eff}$  with  $f_{ct,eff}$  according to Equation (7.1).

### Determining the minimum reinforcement

Minimum reinforcement  $A_{s,min}$  is determined using Equation (7.1) of the standard:

$$A_{s,min} \cdot \sigma_s = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} \quad (7.1)$$

where

$A_{ct}$  is the area of the concrete tensile zone during initial crack formation in state I. To determine the value, the program scales the bending moment of the action combination until the maximum edge stress in state I corresponds to the value  $f_{ct,eff}$ .

$\sigma_s$  is the maximum permitted stress in the reinforcing steel reinforcement in relation to the limit diameter of the reinforcing steel.

$k$  is the coefficient for factoring in nonlinearly distributed tensile stresses based on the user's input, which can vary between 0.65 and 1.0 depending on the section height.

DIN EN 1992-2:

In case of restraint within the component, this value can be assumed between 0.5 and 0.8. For tensile stresses due to restraint generated outside of the component,  $k = 1.0$  shall be assumed.

SS EN 1992-1-1:

According to Article 4a, the value can be assumed to be between 0.50 and 0.90.

$f_{ct,eff}$  is the effective concrete tensile strength at the time of crack formation based on the user's input. The tensile strength is assumed to be  $f_{ctm}$  or lower in case the crack formation is expected to occur within the first 28 days. The tensile strength, which depends on the age of the concrete, is defined in Equation (3.4) of Chapter 3.1.2. A minimum tensile strength of 2.9 MN/m<sup>2</sup> should be assumed to consider shrinkage.

DIN EN 1992-1-1:

If it is not certain whether crack formation will occur within the first 28 days, a tensile strength of at least 3 MN/m<sup>2</sup> for normal concrete and 2.5 MN/m<sup>2</sup> for lightweight concrete should be assumed.

$k_c$  is the coefficient for consideration of stress distribution prior to crack formation.

$k_c = 1.0$  for tension only

For rectangular sections and webs of box girders or T sections:

$$k_c = 0.4 (1 - \sigma_c / (k_1 \cdot h / h^*)) / f_{ct,eff} \leq 1 \quad (7.2)$$

For flanges of box girders and T sections:

$$k_c = 0.9 \cdot F_{cr} / A_{ct} / f_{ct,eff} \geq 0.5 \quad (7.3)$$

$\sigma_c$  is the average concrete stress in the analyzed part of the section with

$$\sigma_c = N_{Ed} / (b \cdot h) \quad (7.4)$$

$N_{Ed}$  is the normal force in the analyzed part of the section (compressive force positive) under the decisive action combination.

$h^* = \min(h; 1.0 \text{ m})$ .

- $k_1$  is the coefficient for taking into account the effects of normal force  $N_{Ed}$  on the stress distribution:
- $$k_1 = 1.5 \quad \text{for compressive normal force}$$
- $$k_1 = 2 h^* / (3h) \quad \text{for tensile normal force}$$
- $F_{cr}$  is the absolute value of the tensile force in the chord directly before crack formation. The tensile force is generated through the integration of tensile stresses within area  $A_{ct}$ .

The largest existing bar diameter  $\phi_s$  is specified in the Section dialog (where it is labeled with  $d_s$ ). It is used in the following equations to determine the limit diameter  $\phi_s^*$  as an input value for Table 7.2N:

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot h_{cr} / (2(h-d)) \quad \text{for bending} \quad (7.6N)$$

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot h_{cr} / (8(h-d)) \quad \text{for central tension} \quad (7.7N)$$

where

$h$  is the overall section height.

$d$  is the static effective height up to the centroid of the outermost reinforcement layer.

$h_{cr}$  is the height of the tensile zone directly before crack formation under the decisive action combination.

The limit diameter  $\phi_s^*$  and the permissible crack width  $w_{max}$  are used to determine the permissible reinforcing steel stress  $\sigma_s$  for Equation (7.1) according to Table 7.2N. The values within the table are interpolated linear, beyond the table they are extrapolated linear for  $w_k$  and quadratic for  $\sigma_s$ .

When determining the minimum reinforcement under central tension, the steel stress  $\sigma_s$  is calculated with the minimum of the permissible crack width and the maximum of the other quantities, provided they have been defined differently for the cross-section edges.

If the crack width check is to be carried out at the same time, the program will determine whether the specified crack width according to Chapter 7.3.4 is maintained by inserting the calculated minimum reinforcement. If necessary, the minimum reinforcement is increased iteratively until the check limit is reached. The increased reinforcement is indicated by an exclamation mark "!" in the log.

DIN EN 1992-1-1:

The limit diameter  $\phi_s^*$  for Table 7.2DE is determined using the following equations:

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot k \cdot h_{cr} / (4(h-d)) \geq \phi_s^* \cdot f_{ct,eff} / 2.9 \quad \text{for bending} \quad (7.6DE)$$

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot k \cdot h_{cr} / (8(h-d)) \geq \phi_s^* \cdot f_{ct,eff} / 2.9 \quad \text{for central tension} \quad (7.7DE)$$

The steel stress  $\sigma_s$  is calculated with the equation from Table 7.2DE and limited to the mean yield strength  $f_{yk}$  of the steel layers to be dimensioned.

Based on Chapter 7.3.2 (NA.5), the minimum reinforcement for the crack width limitation in the case of thicker components under central restraint can be determined according to Equation (NA.7.5.1). It is not necessary to insert more reinforcing steel as results from Equation (7.1). The rules specified before will be used, if the option is selected by the user, whereas the possibility of lower reinforcement for slowly hardening concrete according to Section (NA.6) will not be used. The evaluation of Figure NA.7.1d to determine the effective tensile zone  $A_{c,eff}$  is performed with the smallest edge distance  $d_1$  of the reinforcement.

OENORM B 1992-1-1:

Table 7.2N is replaced by Table 8AT. The steel stress  $\sigma_s$  is determined according to Equation (19AT) and limited to the mean yield strength  $f_{yk}$  of the steel layers to be dimensioned. The limit diameter is to be modified as follows:

$$\phi_s = \phi_s^* \cdot f_{ct,eff} / 2.9 \cdot k_c \cdot k \cdot h_{cr} / (4(h-d)) \geq \phi_s^* \cdot f_{ct,eff} / 2.9 \quad (21AT)$$

For central tension  $h_{cr} = h / 2$  for each member side is applied.

For members under central restraint the minimum reinforcement for the crack width limitation can be determined according to Equation (16AT). This rule will be used, if the option is selected by the user, assuming the smallest edge distance of the reinforcement  $d_1$  for  $h-d$ . The program does not take into account the possibility of reducing the reinforcement for slowly hardening concrete.

### Special characteristic of prestressed concrete structures

According to the guidelines set forth in Chapter 7.3.2 (3), tendons with bond in the tensile zone may be added to the minimum reinforcement as long as their axis distance to the reinforcing steel layer does not exceed 150 mm. To include the

tendons, add the term

$$\xi_1 \cdot A_p' \cdot \Delta\sigma_p$$

on the left side of Equation (7.1). In this formula

$A_p'$  is the section area of the tendons with bond located in  $A_{c,eff}$ .

$A_{c,eff}$  is the effective area of the reinforcement according to Figure 7.1. The section after the next describes how  $A_{c,eff}$  is determined.

$\xi_1$  is the adjusted ratio of bond strengths between reinforcing steel and prestressing steel according to Equation (7.5).

$\Delta\sigma_p$  is the stress change in the tendons.

For beams and design objects, the tendon layers with bond can be added using the  $\xi_1$  value specified in the Section dialog as long as they are grouted in the check situation. For area elements, prestressing steel can never be taken into account.

According to Section (4) of Chapter 7.3.2, prestressed concrete components do not require a minimum reinforcement in sections where the absolute value of concrete tensile stress  $\sigma_{ct,p}$  under the characteristic action combination and characteristic prestressing is less than  $f_{ct,eff}$ . This condition is automatically checked by the program.

DIN EN 1992-2:

According to Section (NA.104) no minimum reinforcement is required in sections where the absolute value of concrete compressive stress under the characteristic combination and where applicable under the characteristic prestressing on the section edge is greater than 1 N/mm<sup>2</sup>. This condition is taken into consideration by the program for components with and without prestressing. The special rules for construction joints according to (NA.110) and (NA.111) are not applied.

OENORM B 1992-2:

The value is specified as  $\sigma_{ct,p} = -1$  N/mm<sup>2</sup> according to Chapter 9.2.2.1.

SS EN 1992-1-1:

The value is specified as  $\sigma_{ct,p} = f_{ctk} / \zeta$  with  $\zeta$  the crack safety factor according to Article 21, Table D-3.

The program assumes  $f_{ctk} = f_{ctk;0,05}(t) = 0,7 \cdot f_{ctm}(t) = 0,7 \cdot f_{ct,eff}$  with  $f_{ct,eff}$  according to Equation (7.1).

To delimit the areas where no minimum reinforcement is required, the concrete compressive stresses in state I are calculated at the gross cross-section with the mean characteristic prestress. The affected structural areas can be evaluated in the graphical stress representation for the characteristic combination. In the remaining areas, minimum reinforcement is determined if concrete tensile stresses occur in the selected check combination.

## Crack Width Calculation

The crack width check is performed through direct calculation in accordance with EN 1992-1-1, Chapter 7.3.4, for all sections where tensile stresses in state I occur under the action combination specified by the user. The bar diameter  $\phi$  ( $d_s$  in the dialog) of the reinforcing steel reinforcement and the decisive  $f_{ct,eff}$  concrete tensile strength are defined in the section dialog.

SS EN 1992-1-1:

The check is performed according to Article 20 for the quasi-continuous action combination.

The program performs the check according to the following steps:

- Determine strain state II under the check combination with the stress-strain curve shown in Figure 3.2. For beams and design objects, all tendons with bond are considered on the resistance side.
- Define the effective area of reinforcement  $A_{c,eff}$  shown in Figure 7.1 (see next section), determine the reinforcing steel layers and prestressing steel layers within  $A_{c,eff}$ .
- Calculate reinforcement level:

$$\rho_{p,eff} = (A_s + \xi_1^2 \cdot A_p') / A_{c,eff} \quad (7.10)$$

$\xi_1$  Bond coefficient according to user specification.

$A_s, A_p'$  Reinforcing steel and prestressing steel within  $A_{c,eff}$ .

- Determine individually for each reinforcing steel layer:

#### Difference of the average strain for concrete and reinforcing steel

$$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - k_t \cdot f_{ct,eff} / \rho_{p,eff} (1 + \alpha_e \cdot \rho_{p,eff})] / E_s \geq 0.6 \sigma_s / E_s \quad (7.9)$$

Where

$$\alpha_e = E_s / E_{cm}$$

$\sigma_s$  is the reinforcing steel stress from strain state II.

For tendons, this value is replaced by the stress increase  $\Delta\sigma_p$ .

DIN EN 1992-1-1:

$$\sigma_s = \sigma_{s2} + 0.4 f_{ct,eff} (1/\rho_{p,eff} - 1/\rho_{tot}) \quad (NA. 7.5.3)$$

$\sigma_s$  is limited to  $f_{yk}$  in the program

$\sigma_{s2}$  = Reinforcing steel stress from strain state II

$f_{ct,eff}$  is the effective concrete tensile strength as per specifications.

$k_t$  is the factor for the duration of the load action:

0.6 for short-term and 0.4 for long-term load action.

DIN EN 1992-2:

For bridges always the factor  $k_t = 0.4$  is to be assumed.

#### Maximum crack spacing

$$s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot \phi / \rho_{p,eff} \quad (7.11)$$

Where

$\phi$  is the bar diameter specified by the user.

$c$  is the concrete cover with respect to the longitudinal reinforcement. The concrete cover is set to  $d_1 - \phi / 2$  in the program, where  $d_1$  is the smallest axis distance of the reinforcing steel reinforcement of the section edge within  $A_{c,eff}$ .

$k_1$  is the coefficient for consideration of the bond properties of the reinforcement. The coefficient is set to 0.8 in the program, which is the recommended value for good bond properties.

$k_2$  is the coefficient for taking strain distribution into account:  
0.5 for bending and 1.0 for pure tension.

$k_3, k_4$  The recommended national values are  $k_3 = 3.4$  and  $k_4 = 0.425$ .

DIN EN 1992-1-1:

$$k_1 \cdot k_2 = 1, k_3 = 0 \text{ and } k_4 = 1 / 3.6$$

$$s_{r,max} \leq \sigma_s \cdot \phi / (3.6 \cdot f_{ct,eff})$$

OENORM B 1992-1-1:

$$k_3 = 0 \text{ and } k_4 = 1 / (3.6 \cdot k_1 \cdot k_2) \leq \rho_{p,eff} \cdot \sigma_s / (3.6 \cdot k_1 \cdot k_2 \cdot f_{ct,eff}) \quad (22AT)$$

$$s_{r,max} = \phi / (3.6 \cdot \rho_{p,eff}) \leq \sigma_s \cdot \phi / (3.6 \cdot f_{ct,eff}) \quad (23AT)$$

SS EN 1992-1-1:

$$k_3 = 7 \cdot \phi / c \quad (\text{Article 22})$$

If an upper limit for the crack spacing in Equation (7.11) was specified in the section dialog, this allows the special features of Equations (7.13) and (7.14) and sections (4) and (5) of Chapter 7.3.4 to be taken into consideration.

#### Calculated value of the crack width

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (7.8)$$

The layer with the largest calculated crack width is shown in the log. If selected in the cross-section dialog, a constant mean steel strain within  $A_{c,eff}$  is assumed during calculation.

- For sections completely under tension, the check is performed separately for each of the two effective tensile zones. The maximum value is shown in the log.



If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the crack width. For that purpose a design is carried out using the decisive check combination for calculating the crack width. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The crack width is checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check) and saved for graphical representation together with the decisive reinforcing steel stress.

## Crack Width Check by Limitation of the Bar Distances

As an alternative to the direct crack width calculation described in EN 1992-1-1, Section 7.3.4, you can choose the simplified check according to Section 7.3.3 (2) through limitation of the bar spacing as shown in Table 7.3N in the cross-section dialog.

The program performs the check according to the following steps:

- Determine strain state II under the check combination defined by the requirement class with the stress-strain curve according to Figure 3.2. For beams and design objects, all tendons in a bond are considered on the resistance side.
- Determine the reinforcing steel stress  $\sigma_s$  for each reinforcement layer. If selected in the cross-section dialog, a constant mean steel stress within  $A_{c,eff}$  is assumed for calculating.

DIN EN 1992-1-1:

$$\sigma_s = \sigma_{s2} + 0,4 f_{ct,eff} (1/\rho_{p,eff} - 1/\rho_{tot}) \quad (NA. 7.5.3)$$

$\sigma_{s2}$  = Reinforcing steel stress from strain state II

- Compare the value entered in the dialog (**max. s**) with the table value (**perm. s**), which is derived from the calculated steel stress  $\sigma_s$  and the permissible crack width  $w_k$ . In the log, the location with the largest quotient (**max. s / perm. s**) is checked.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the permissible bar spacing. For this purpose, a design is carried out with the decisive action combination for the check. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The bar spacings are then checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check).

### Note

According to EN 1992-1-1, Section 7.3.3 (2), the simplified check can only be used in the event of crack formation resulting from mostly direct actions (restraint). In addition, Table 7.3N should only be applied for single-layer tensile reinforcement with  $d_1 = 4$  cm (cf. Zilch, Rogge (2002), p. 277; Fingerloos et al. (2012), p. 109; Book 600 of the DAfStb (2012), p. 127).

OENORM B 1992-1-1:

The method is applicable for single-layer reinforcement with bar spacings according to Table 10AT resp. 11AT. These are valid for concrete covers  $25 \text{ mm} \leq c_{nom} \leq 40 \text{ mm}$  with bar diameters  $8 \text{ mm} \leq d_s \leq 20 \text{ mm}$ .

The user is responsible for the evaluation of these requirements.

## Determining the Effective Area $A_{c,eff}$

According to EN 1992-1-1, Figure 7.1, the effective area of reinforcement  $A_{c,eff}$  defines the area of a rectangular, uniaxially stressed concrete section in which the model assumptions in Book 466 of the German Committee for Reinforced Concrete (DAfStb) are applicable. Although the program can apply this model to any section and stress situation, the user has the responsibility and discretion to do so.

When determining  $A_{c,eff}$ , the following steps are performed by the program:

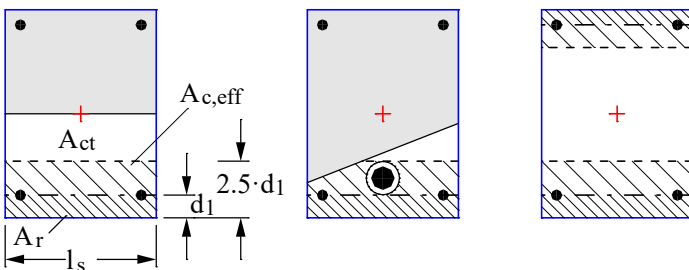
- Determine tensile zone  $A_{ct}$  in state I: When calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the exposure class. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Define the centroid line of the reinforcement as a regression line through the reinforcing steel layers in the tensile zone. In 2D frameworks and for area elements, a horizontal line through the centroid of the reinforcement layers under tension is assumed.
- Determine the truncated residual area  $A_r$  to the edge and the sum of section lengths  $l_s$ . The average edge distance is then assumed as  $d_1 = A_r / l_s$ , but not less than the smallest edge distance of the reinforcing steel layers in the tensile zone.
- Shift the centroid line in parallel by  $1.5 \cdot d_1$ . Assuming  $h - d = d_1$ , the height of  $A_{c,eff}$  is determined as per 7.3.2 (3) by  $h_{c,ef} = 2.5 \cdot (h - d) \leq h / 2$ . According to DIN EN 1992-1-1 and OENORM B 1992-1-1, Section 7.3.2 (3), this value is limited to  $(h - x) / 2$  ( $x$  = compressive zone height in state I).

DIN EN 1992-1-1 and OENORM B 1992-1-1:

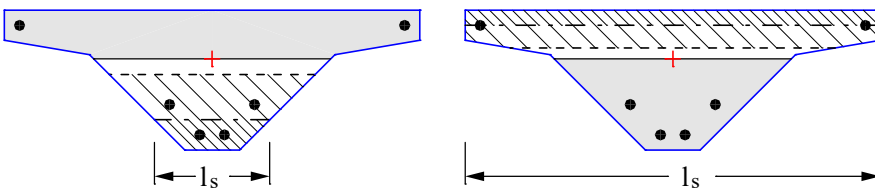
If the minimum reinforcement for thicker components under central restraint is selected in the section dialog, the height of  $A_{c,eff}$  is  $h_{c,ef} \geq 2.5 d_1$  according to Figure NA.7.1 d) or Eq. (16AT). In the crack width check, this increase of  $h_{c,ef}$  does not apply (see comments in Book 600 for Chapter 7.3.2 (NA.5) and 7.3.4 (2)).

- The resulting polygon is intersected with the tensile zone and then defines the effective area  $A_{c,eff}$
- If all the reinforcing steel layers of the section are under tension, then two zones will be determined; one for the layers above the centroid and the other for layers below the centroid. The area of each zone is limited to  $A_c / 2$ .

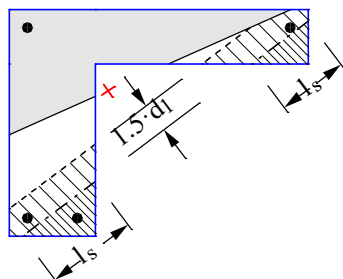
The following illustrations show the effective areas determined by the program for a few representative situations. The last case (edge beam) deviates from the model assumptions in Book 466 to such a degree that it is questionable as to whether it should be used.



Effective area of the reinforcement at a rectangular section under uniaxial bending, normal force with double bending and central tension



Effective area of the reinforcement at a bridge section under uniaxial bending



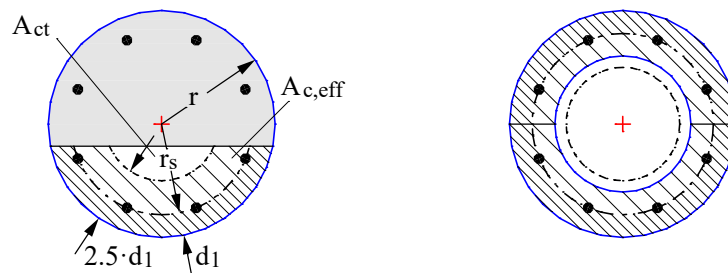
Effective area of the reinforcement at an edge beam under uniaxial bending

### Ring-shaped determination of $A_{c,eff}$

For circular solid and hollow cross-sections, the cross-section dialog allows that the effective area of the reinforcement  $A_{c,eff}$  for checking of the minimum reinforcement and the crack width is determined ring-shaped according to Wiese et al. (2004). This can be used e.g. for considering the specifics of bored piles and spun concrete columns. In order to determine  $A_{c,eff}$  the following steps are performed by the program:

- Determine tensile zone  $A_{ct}$  in state I: When calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the exposure class. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Calculate the mean radius  $r_s$  of the reinforcing layers within the tensile zone. Assuming the circular radius  $r$  of the outer edge, the mean edge distance is determined by  $d_1 = r - r_s$ .
- The effective area  $A_{c,eff}$  is then assumed to be ring-shaped with a width of  $2.5 \cdot d_1$  and finally intersected with the tensile zone  $A_{ct}$ .
- If all the reinforcing steel layers of the section are under tension, then two ring-shaped zones will be determined; one for the layers above the centroid and the other for layers below the centroid.

The following figures show ring-shaped effective areas exemplarily.



*Effective area of the reinforcement at a solid section under bending with normal force as well as a hollow section under central tension.*

## Limiting Diagonal Principal Tensile Stresses

DIN EN 1992-2:

For prestressed road bridges with thin flanges, the formation of shear cracks must be limited in accordance with Chapter 7.3.1 (NA.111). It is to be ensured that the diagonal principal tensile stresses affected by lateral force and torsion do not exceed the values  $f_{ctk;0,05}$ . The check is to be carried out in state I for the frequent combination.

For prestressed railway bridges, the principal tensile stresses as per Chapter 7.3.1 (NA.112), must be limited as well. Note, however, that you can limit the check to the area of longitudinal compressive stresses as long as no tensile-stressed chords are connected. You can choose this option in the section dialog. The user is responsible for verifying the usage requirements.

### Beams and design objects

The program analyzes the section to determine all shear parameters from lateral force and torsion for the section edge. Calculation points are all points of the cross-section polygon, the edge centers and a variable count of additional edge points depending on the edge length. The shear stresses from lateral force are determined on the basis of either the theory of thick-walled profiles or an average across the section width, depending on which option is selected in the Section dialog. The diagonal principal tensile stress can thus be calculated for each section point:

$$\sigma_1 = \frac{\sigma_x}{2} + \frac{1}{2} \sqrt{\sigma_x^2 + 4\tau_{xy}^2 + 4\tau_{xz}^2}$$

with

$\sigma_x$  Longitudinal stress from bending and longitudinal force in accordance with the rules for stress analysis indicated above.

$\tau_{xy}$  Edge shear stress in the y direction from lateral force and torsion.

$\tau_{xz}$  Edge shear stress in the z direction from lateral force and torsion.

The angle between  $\sigma_1$  and  $\sigma_x$  is derived with:

$$\tan 2\vartheta_1 = \frac{-2\tau}{\sigma_x}$$

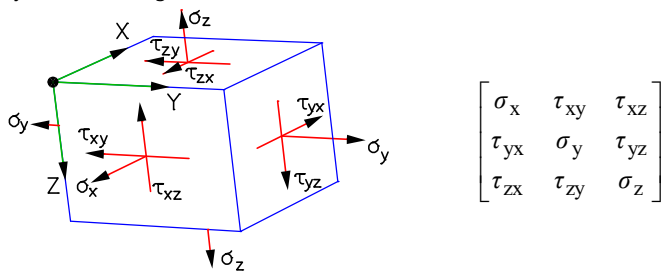
with

$$\tau = \sqrt{\tau_{xy}^2 + \tau_{xz}^2} \text{ as the resulting edge shear stress.}$$

The maximum value for  $\sigma_1$  is listed in the log together with the relevant section point.

### Area elements

For area elements the check is carried out for principal tensile stresses at the gross section. The 3D stress state is described by the following stress tensor:



with

$\sigma_x, \sigma_y$  Longitudinal stress from bending and normal force.

$\sigma_z = 0$ .

$\tau_{xy} = \tau_{yx}$  Shear stress from torsional moment and shear force with a linear curve along the section height.

$\tau_{xz} = \tau_{zx}$  Shear stress from lateral force  $q_x$  with a parabolic curve along the section height and the maximum value  $1.5 \cdot q_x / h$  in the centroid level.

$\tau_{yz} = \tau_{zy}$  Shear stress from lateral force  $q_y$  with a parabolic curve along the section height and the maximum value  $1.5 \cdot q_y / h$  in the centroid level.

The maximum principal stress  $\sigma_1$  with its height level  $z$  in relation to the upper section edge is calculated by determining the stress tensor for each layer and solving the eigenvalue problem for the three principal stresses. These are shown together with the associated stress components in the results log.

## Limiting Deformations

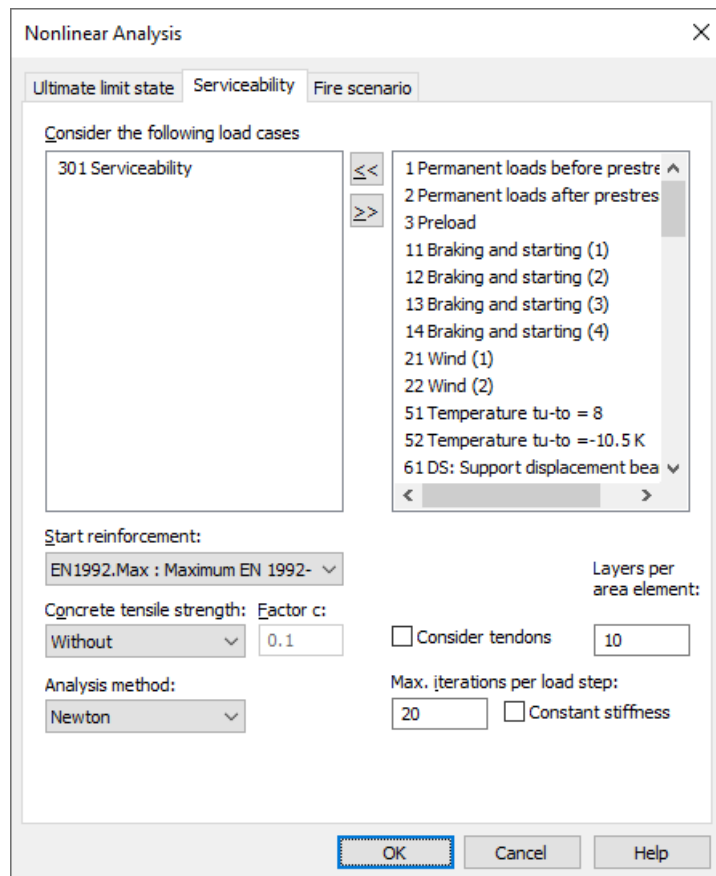
According to EN 1992-1-1, Chapter 7.4.1, the deformations of a component or structure may not impair its proper functioning or appearance.

The standard does not include a method for direct calculation of deformations in accordance with Chapter 7.4.3.

The InfoCAD program system allows you to perform a realistic check as part of a nonlinear system analysis for beam and shell structures that takes geometric and physical nonlinearities into account. The resistance of the tendons within the bond is currently not included in the calculation.

Editing is performed in the following steps:

- Define the check situation using the *Load Group* function in the Load dialog by grouping the decisive individual load cases. The variable loads must first be weighted with the combination coefficients  $\psi_2$  for the quasi-continuous combination.
- Select the check load cases in the *Nonlinear Analysis / Serviceability* dialog in the analysis settings for the FEM or framework analysis.
- Set the reinforcement determined in the ultimate limit state in the *Start reinforcement* selection field (maximum from bending, robustness, crack check and fatigue).
- Perform the FEM or framework analysis to determine the deformations in state II.
- Check the system deformations displayed graphically or in tabular form.



For a detailed description of nonlinear system analysis, refer to the relevant chapter of the manual.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The resulting bending, robustness and crack reinforcement, the decisive maximum value and the stirrup and torsion reinforcement are provided for the graphical representation as well.

The log shows the design internal forces and necessary reinforcements, checked stresses or crack widths at each result location. If the permissible limit values are exceeded, they are reported as warnings and indicated at the check location. The detailed log also lists the decisive combination internal forces of all design situations.

### Tendon reactions

$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$d_p, d_{p,\min}$	Depth of the tendons or ducts in the concrete compressive zone in the decompression check [mm].

### Stresses for beams and design objects

$\sigma_x$	Longitudinal stresses in the decompression and concrete compressive stress checks [MN/m <sup>2</sup> ].
$\sigma_1, \sigma_x, \tau_{\max}$	DIN EN 1992-2: Diagonal principal tensile stresses and related longitudinal and shear stresses [MN/m <sup>2</sup> ].
$\Delta\sigma_1$	DIN EN 1992-2: Stress ranges of principal stresses in the fatigue check under lateral forces [MN/m <sup>2</sup> ].
$\sigma_2$	DIN EN 1992-2: Principal compressive stresses of box girders [MN/m <sup>2</sup> ].
$\sigma_s, \Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd}, \Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete [MN/m <sup>2</sup> ].
$\sigma_{c,\text{perm}}$	Permanent stress during fatigue check for concrete under longitudinal pressure [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,y}, \Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from $Q_y$ and $Q_z$ [MN/m <sup>2</sup> ].
$\Delta\sigma_{sb,T}, \Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and longitudinal torsion reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{\text{perm}}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{\text{perm}}$	Stress range utilization.

### Stresses for area elements

$\sigma_t$	Concrete stress in the tendon direction in the decompression check [MN/m <sup>2</sup> ].
$\sigma_x, \sigma_y, \sigma_1$	DIN EN 1992-2, ÖNORM B 1992-2: Longitudinal stress in x or y direction or principal tensile stresses in the decompression check (depending on user specification) [MN/m <sup>2</sup> ].
$\sigma_2$	Principal compressive stresses in the concrete compressive stress check [MN/m <sup>2</sup> ].
$\sigma_1, \sigma_x, \sigma_y$	DIN EN 1992-2: Diagonal principal tensile stresses and related longitudinal and
$\tau_{xy}, \tau_{xz}, \tau_{yz}$	shear stresses [MN/m <sup>2</sup> ].
$\sigma_{sx}, \Delta\sigma_{sx}$	Stresses and stress ranges for reinforcing steel in the x direction [MN/m <sup>2</sup> ].
$\sigma_{sy}, \Delta\sigma_{sy}$	Stresses and stress ranges for reinforcing steel in the y direction [MN/m <sup>2</sup> ].
$\sigma_p, \Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m <sup>2</sup> ].
$\sigma_{cd,x}, \Delta\sigma_{cd,x}$	Stresses and stress ranges in the concrete fatigue check under longitudinal compression in the
$\sigma_{cd,y}, \Delta\sigma_{cd,y}$	x- and y-direction [MN/m <sup>2</sup> ].
$\sigma_{cx,\text{perm}}$	Permanent stress during fatigue check for concrete under longitudinal pressure in x-
$\sigma_{cy,\text{perm}}$	and y-direction [MN/m <sup>2</sup> ].
$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m <sup>2</sup> ].
$\sigma / \sigma_{\text{perm}}$	Stress utilization.
$\Delta\sigma / \Delta\sigma_{\text{perm}}$	Stress range utilization.

**Bending reinforcement**

$A_s$	Bending reinforcement [cm <sup>2</sup> ] for beams and design objects.
$a_{sx}, a_{sy}$	Bending reinforcement [cm <sup>2</sup> /m] for area elements in the x and y direction.

**Reinforcement from lateral force**

$a_{sbx}, a_{sby}, a_{sb}$	Stirrup reinforcement [cm <sup>2</sup> /m <sup>2</sup> ] of area elements from $q_x, q_y$ and $q_r$ .
$A_{sb,y}, A_{sb,z}$	Stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $Q_y$ and $Q_z$ .
$A_{sl}$ for $a_{sb}=0$	Longitudinal reinforcement [cm <sup>2</sup> ] of area elements.
$z_y, z_z$	Inner lever arm [m] for lateral force $Q_y$ and $Q_z$ .
$\Delta F_{tdy}, \Delta F_{tdz}$	Additional tensile force [kN] in the longitudinal reinforcement as a result of lateral force $Q_y$ and $Q_z$ .

**Torsional reinforcement**

$A_{sb,T}$	Torsional stirrup reinforcement [cm <sup>2</sup> /m] of beams and design objects from $M_x$ .
$A_{sl,T}$	Torsional longitudinal reinforcement [cm <sup>2</sup> ] of beams and design objects from $M_x$ .

**Design values**

$V_{Rd,ct}, v_{Rd,ct}$	Absorbable design lateral force without shear reinforcement [kN, kN/m].
$v_{Rd,max}$	Absorbable design lateral force of concrete struts for area elements [kN/m].
$V_{Rd,max}$	Absorbable design lateral force of concrete struts for beams and design objects [kN].
$T_{Rd,max}$	Design value of the maximum absorbable torsion moment [kNm].
$Q/V_{Rd} + M_x/T_{Rd}$	For compact and box sections: $Q/V_{Rd,max} + M_x/T_{Rd,max}$
	DIN EN 1992-1-1: For compact sections: $(Q/V_{Rd,max})^2 + (M_x/T_{Rd,max})^2$
	OENORM B 1992-1-1: For full sections: $(Q/V_{Rd,max})^2 + (M_x/T_{Rd,max})^2$

**Crack width**

$w_{k,top}, w_{k,bottom}$	Computed crack width at the top and bottom of the cross-section separately for the x and y reinforcement direction for area elements.
$w_k / w_{per}$	Crack width utilization.

# Examples

## Road Bridge in Solid Construction

In this example the features and capabilities of the *EN 1992-2 Bridge Checks* program module are demonstrated based on a simple bridge slab.

It was chosen inspired by part 1 of the book

Bauer, Thomas / Müller, Michael, Straßenbrücken in Massivbauweise nach DIN-Fachbericht (Road Bridges in Solid Construction According to DIN Technical Report)

**Beispiele prüffähiger Standsicherheitsnachweise (Examples of Verifiable Stability Safety Checks).**

Stahlbeton- und Spannbetonüberbau nach DIN-Fachbericht 101 und 102

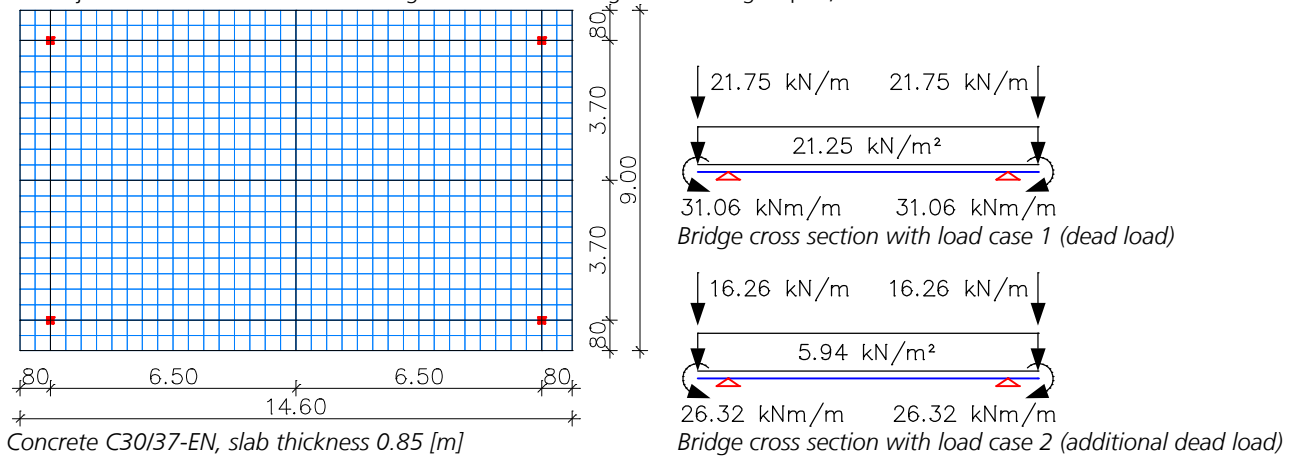
(Reinforced Concrete and Prestressed Concrete Superstructures According to DIN Technical Reports 101 and 102). 2nd Revised Edition. Bauwerk Verlag GmbH, Berlin 2003.

The bridge checks are divided into the following sections in the program:

- Section-dependent check specifications
- Selection of the structure type
- Load model 1
- Combination of actions
- Performing checks

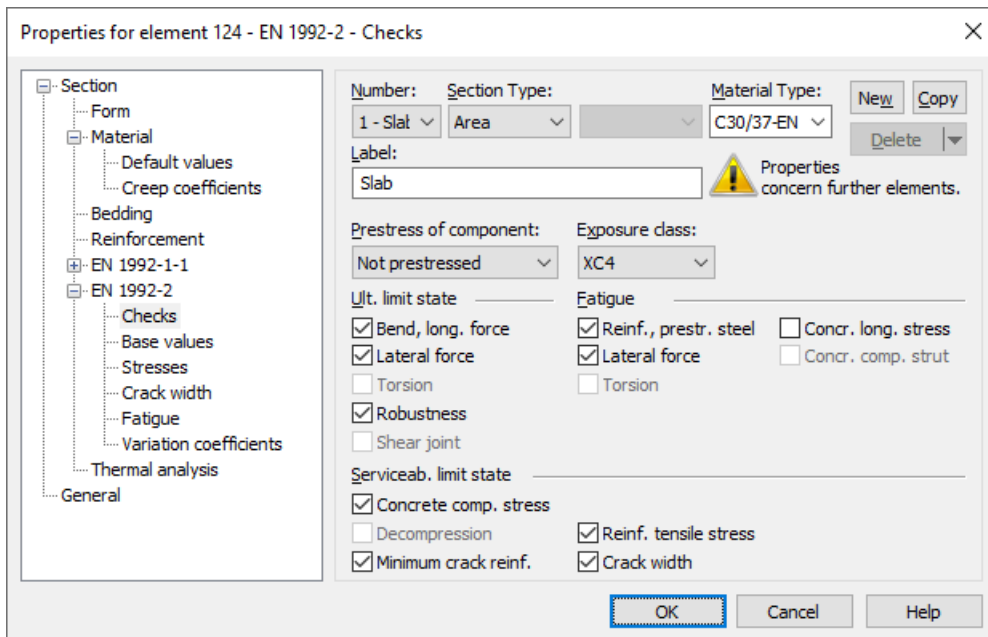
### The Task

The objective is to calculate a road bridge that is to be designed as a single-span, reinforced concrete slab.



The depicted FEM system is generated using shell elements. Make sure that the support for the system is free of restraint. The load cases for dead loads, additional loads and all loads that do not belong to load model 1 according to EN 1991-2 are specified as usual.

After that complete the section-dependent check specifications:



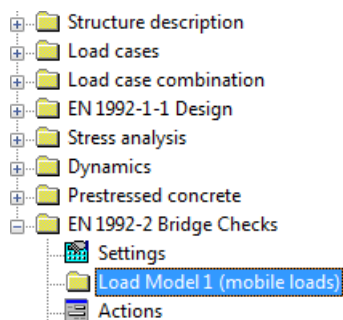


## Load model 1

Load model 1 consists of two parts:

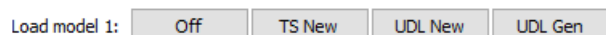
- (a) Twin axle (TS tandem system)
- (b) Uniformly distributed area loads UDL

These loads should be applied in both the longitudinal and lateral directions of the bridge in the least favorable position. In the lateral direction, the load positions are determined by dividing the roadway into computational lanes. Since the decisive lane division is not always known in advance, you can define different load position variants.

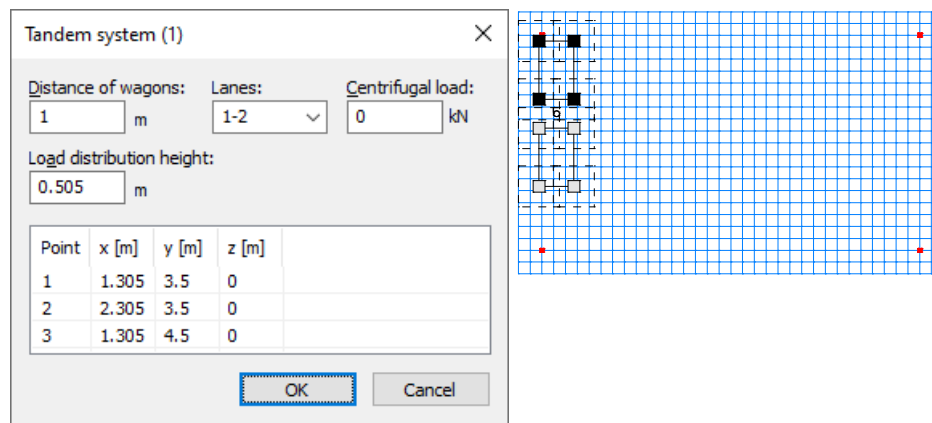


In the *EN 1992-2 Bridge Checks* folder right-click *Load model 1* and then select *New* from the context menu.

This creates the 1st variant of the load model and opens an input dialog.

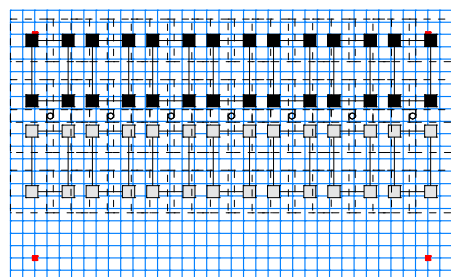


Click *TS New* to specify the centroid and the direction of traffic for the 1st tandem system.



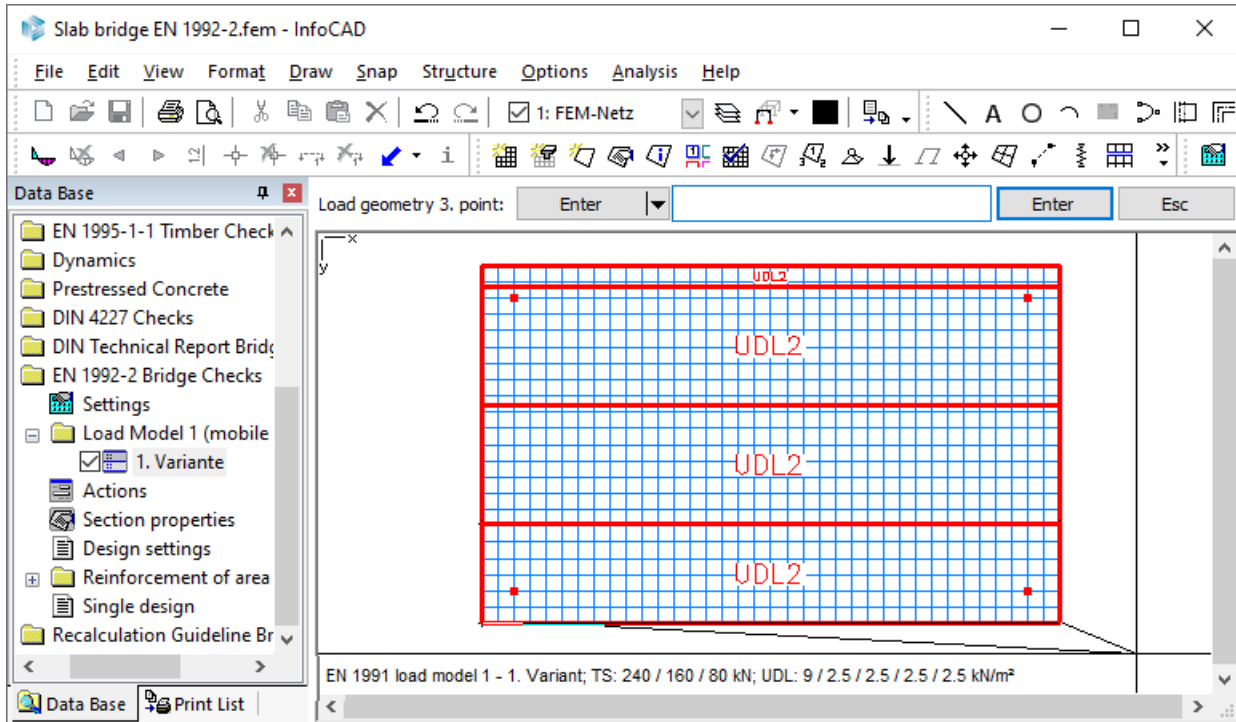
The properties of the tandem system position are set in the dialog.

The Lane 1 is defined left of the traffic direction and indicated by a dark hatching pattern. The area resulting from the load distribution height is shown in dashes.

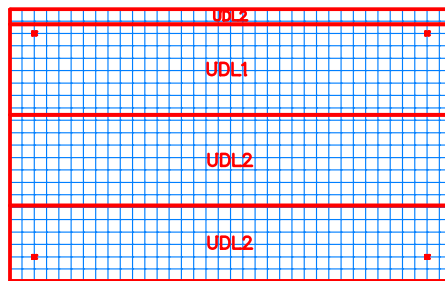
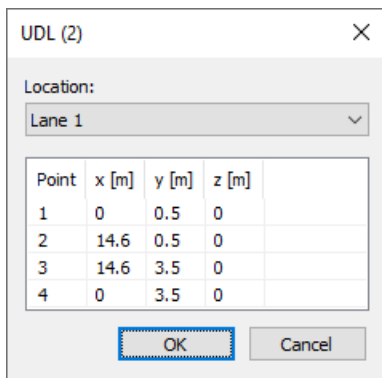


Once the tandem system is selected, you can either copy or generate the desired load positions. The usual snap functions such as *Mid-point* or *Endpoint* can be used for this purpose.

Now click *UDL New* to define the load areas of the UDL system in consecutive order.



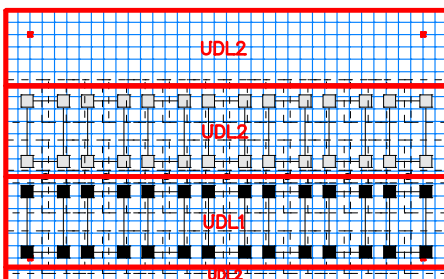
Initially, all the load areas are accepted as UDL2 (lanes 2). Double-click the second UDL area and specify *Lane 1* for this.



Completed UDL areas (TS hidden)

This done, the 1st variant of the load model has been defined. To enter the 2nd variant, right-click *Load model 1* in the database again and select *New* from the context menu.

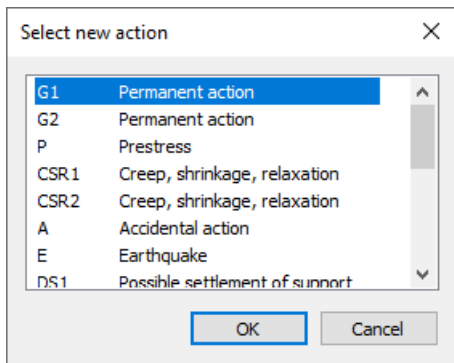
All of the TS and UDL positions can be copied to the 2nd variant via the clipboard and then modified (in this case: rotated by 180°).



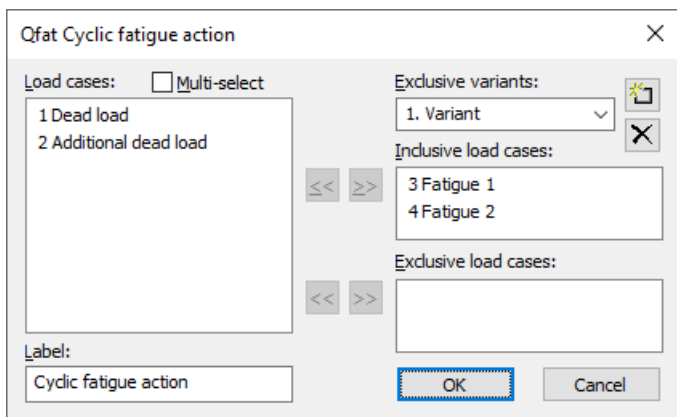
Load model 1 is now fully described and can be calculated. This is done in conjunction with the load cases as part of the FEM calculation. The results of all load positions are individually saved and grouped in the *Load model 1 (Mobile loads)* folder.

## Combination of actions

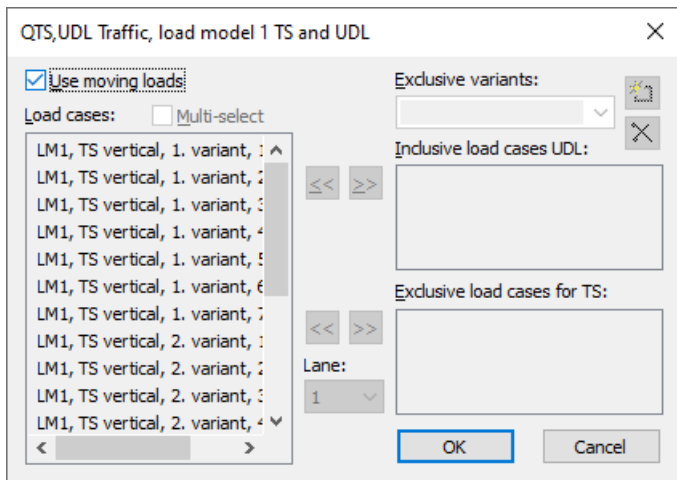
To describe the actions and their combinations, click the *Actions* option in the database and then select *Insert*.



The actions to be considered are selected in the dialog. You can now assign load cases to them.



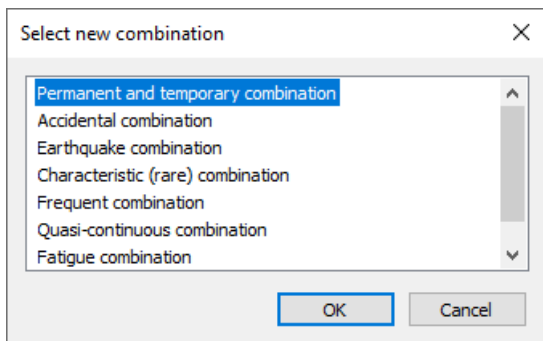
For variable actions, you can add multiple load cases to form inclusive and exclusive groups. If required, you can also define multiple mutually exclusive variants.



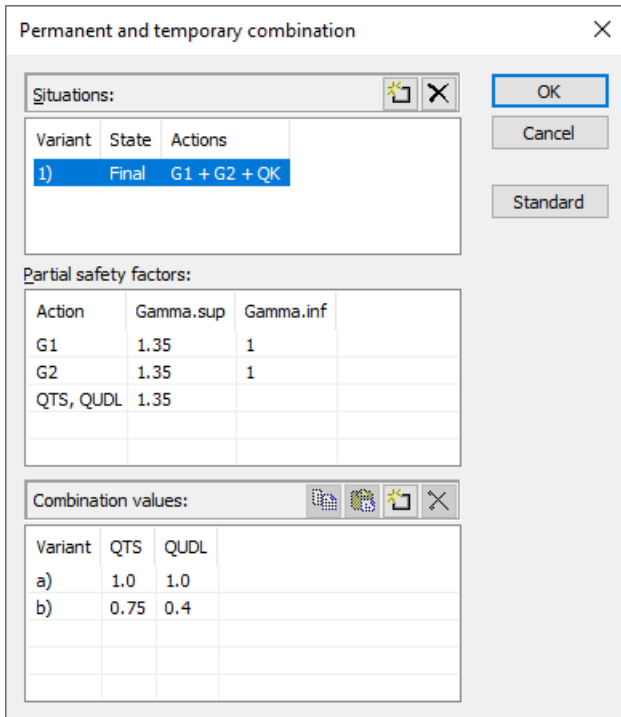
The variable actions QTS and QUDL are taken directly from load model 1.

The *Use moving loads* option must be enabled for this to work.

Alternatively, you can also use freely-definable load cases or combinations.

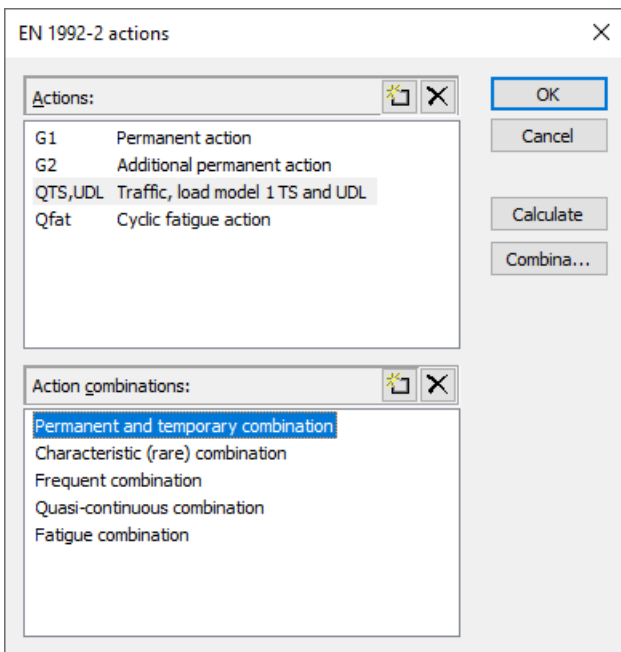


Once the respective loads have been assigned to the actions, you can add the combinations required for the checks.



The following variants are derived using the safety factors and combination coefficients according to EN 1990.

You can accept the suggestion by clicking *OK* in order to continue selecting more combinations.



To determine the extremal internal forces from the defined combinations, click *Calculate*.

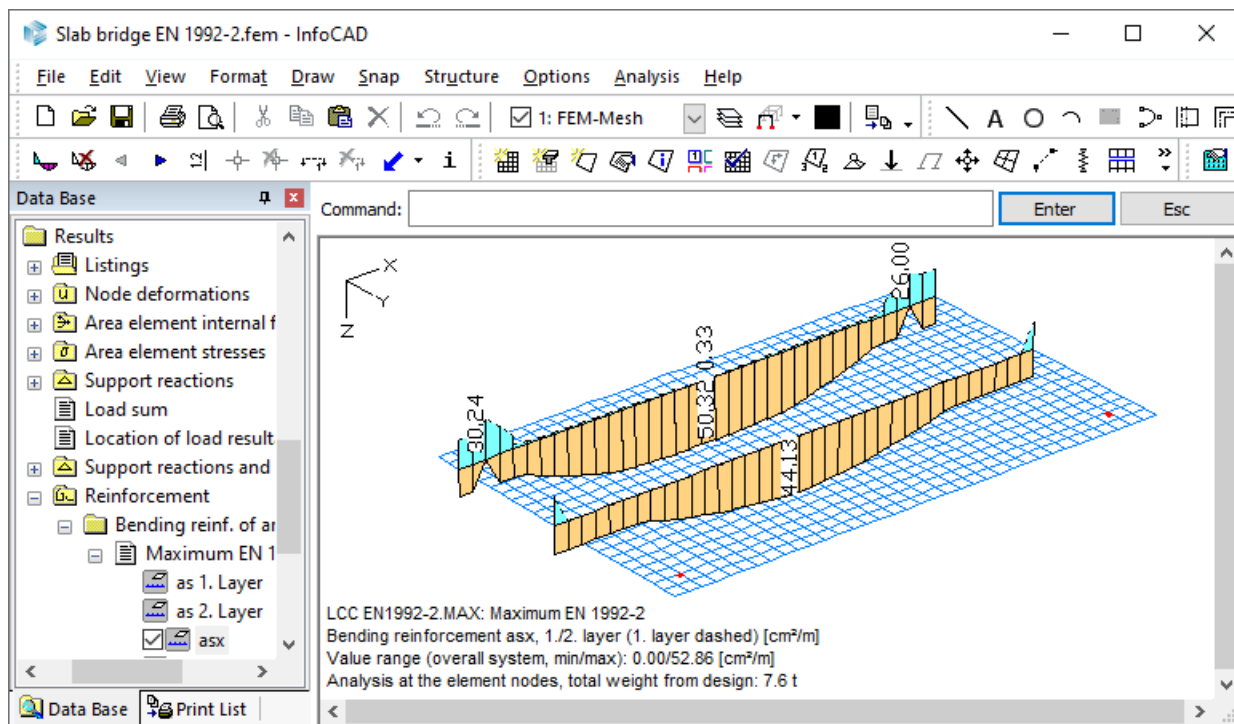
The program combines the results of the load cases, combinations and load models that are used and then makes them available in the usual manner:

- Area element internal forces
  - 1. Permanent and temporary situation, EN 1992-2
  - 1. Rare (characteristic) situation, EN 1992-2
  - 1. Frequent situation, EN 1992-2
  - 1. Quasi-continuous situation, EN 1992-2
  - 1. Fatigue situation, basic part, EN 1992-2
  - 1. Fatigue situation, cyclic part, EN 1992-2

The calculation of combinations is optional here since it will be performed by the checking program in any case.

## Performing checks

The EN 1992-2 Bridge Checks checking program is started from the Analysis menu. Afterwards, the results can be obtained from the data base.



Maximum value of the upper and lower longitudinal reinforcement from bending with normal force, crack check and fatigue.

The summary log for a single element in the midspan is provided below:

### Design according to EN 1992-2:2008

The design is applicable for reinforced and prestressed concrete bridges with and without bond. The actions are combined according to EN 1990, Eq. (6.10), using the specific partial safety and combination factors.

All checks are made for the extreme values of actions.

### Design overview

Se.	Expos. class	Prestress of component	Reinforcem. M R B Q T S	Fatigue B Q T P C V	Cr. wi.	De- co.	Stress C B P
1	XC4	Not prestressed	x x x x . .	x x . . . .	x	.	x x .

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state, fatigue and stress check.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.
- (T) Torsional reinforcement at ultimate limit and fatigue state.
- (S) Shear joint check.
- (P) Prestressing steel at fatigue and stress check.
- (C) Concrete comp. stress, concrete at fatigue check under long. compression.
- (V) Concrete at fatigue check under lateral force.

### Settings for flexural and shear reinforcement

M,N Design mode for bend and longitudinal force:  
 (ST) Standard, (SY) Symmetrical, (CM) Compression member.  
 (\*) Design without considering specified ratio between reinf. layers.

fyk Quality of stirrups.

Theta Angle of concrete truss.

S. Beams are designed like slabs.

C. Design for resulting lateral force at circular resp. ring section.

Asl Given reinforcement according to picture 6.3, increase to maximum.

rhov Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).

as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).

Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den- sity [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. S.C.	Asl [cm²] Pic. 6.3 given max	Red. pre- str.
1	C30/37-EN	.	ST	500	1.75	.	0.00 0.00 1.00 0.20	.

**Shear sections**

bw.nom Nominal width of the prestressed section according to 6.2.3(6).  
 h.nom Nominal height of the prestressed section according to 6.2.3(6).  
 kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.  
 z1, z2 Height and width of the core section for torsion.  
 tef Thickness of the torsion box.  
 B. Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width [m]	Eff. width	Height[m]	Eff.height	Torsion section [m]
	bw bw.nom	bn [m] kb	h h.nom	d [m] kd	z1 z2 tef B.
1	1.000 .	. .	0.850 .	0.780 0.90	. . . .

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel [mm].  
 max.s Maximal given bar spacing of the reinforcing steel [mm].  
 sr,max Upper limit for the crack spacing from Eq. (7.11) [mm].  
 Xil Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 kt Coefficient for the duration of the load to calculate the crack width.  
 Fact. Reduction factor for fctm as per Chapt. 7.3.2 (As) resp. 7.3.4 (wk).  
 Comb. Combination for verifying the minimum reinf. (As) and crack width (wk):  
 CC, TC, QC = Characteristic, frequent, quasi-continuous combination,  
 CT, TT, TB = Central tension, tension on top side, tension on bottom,  
 CL = Action combination according to exposure class.  
 Method Check method for minimum reinf. (kc) and crack width (wk):  
 kc Determination of coefficient kc for webs/chords per Eq. (7.2/7.3).  
 auto = Eq. (7.2) for rectangular sections, Eq. (7.3) for others.  
 wk Calc. = Direct calculation of crack width as per Chapter 7.3.4,  
 Bar sp. = Limiting the bar spacing as per Table 7.3N,  
 Cal.(m) = Direct calculation for mean steel strain within Ac,eff,  
 Spc.(m) = Lim. the bar spacing for mean steel strain within Ac,eff.  
 RS Ring-shaped determination of Ac,eff according to Wiese et al.,  
 Beton- und Stahlbetonbau 2004, Issue 4, p 253 ff.

Se.	wmax [mm]	ds	max s	sr	Coefficient	Fact.fctm	Comb.	Method	RS
					Xil k kt As wk	As wk kc wk	As wk kc wk	auto calc.	
1	0.30	28	. .	. .	1.00 0.4	1.00 1.00	CL CL	auto calc.	. .

**Settings for the check of concrete and steel stresses**

Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.s Reinforcing steel stress in the serviceability limit state.  
 (CC),(QC) Characteristic, quasi-continuous combination.  
 (TC),(CL) Frequent combination, combination according to exposure class.

Se.	fck(t) [MN/m²]	per.sigma.c(t) (CC, QC)	per.sigma.c (CC) (QC)	per.sigma.s (CC)	Decompression Comb. Stress
1	. .	. .	0.60 fck 0.45 fck	0.80 fyk	. .

**Settings for fatigue check**

The concrete compressive stresses are calculated according to fig. 3.2.

fcd,fat Compr. strength of concrete before beginning of cyclic actions.  
 dSigma.Rsk,s Permissible charact. stress fatigue range of longitudinal reinf.  
 dSigma.Rsk,b Permissible charact. stress fatigue range of shear reinf.  
 dSigma.Rsk,p Permissible charact. stress fatigue range of prestr. steel.  
 Lambda.s,b,p Corresp. correction coeff. for damage equivalent fatigue range.  
 Lambda.c Correction coeff. for damage equivalent concrete stresses.  
 Eta Increase factor for reinforcing steel stress acc. to 6.8.2(2)P.  
 Check SPCVQT Check for (S) flexural reinf., (P) prestr. steel, (C) concrete, (V) compr. strut, (Q) Lateral force reinf., (T) Torsional reinf.  
 (x) Check with damage equivalent fatigue ranges - fatigue comb.  
 (+) Simplified check acc. to 6.8.6(2) - frequent combination.  
 Factor Qfat Factor for Qfat to calculate the damage equ. fatigue range for reinf. and prestr. steel as per Annex NN.2.1(101).

Se.	Check SPCVQT	fcd,fat [MN/m²]	dSigma.Rsk [MN/m²]	Lambda	Eta	Factor Qfat
		s b p	s b p c	b p c		Qfat
1	x...x.	. . .	162.50 73.00 .	1.26 1.26 .	. . .	1.40 *

\*) For this section, the calculation of the fatigue range is reduced to corresponding variants of design forces.

**Stress calculation for area elements**

Stresses of concrete are calculated at gross section.  
 The design of reinf. steel stresses is examined in the condition of cracked concrete section.

**Partial safety factors for material at ultimate limit state**

	concrete (gamma.c)	Reinf.steel (gamma.s)	Prestr.steel (gamma.s)
Permanent and temporary combination	1.50	1.15	1.15
Accidental combination	1.20	1.00	1.00
Earthquake combination	1.50	1.15	1.15
Fatigue check	1.50	1.15	1.15

**Reinforcing steel of area elements**

Se.	Lay.	Qua.	E-Modul [MN/m <sup>2</sup> ]	dt x [m]	db x [m]	asx [cm <sup>2</sup> /m]	dt y [m]	db y [m]	asy [cm <sup>2</sup> /m]	as fix
1	1	500M	200000	0.060	.	0.00	0.080	.	0.00	.
	2	500M	200000	.	0.060	0.00	.	0.080	0.00	.

**DESIGN FOR AREA ELEMENTS**

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

- (M) Nominal reinf. for robustness as per EN 1992-2, 6.1 (109) (Charact. C.)
- (R) Nominal/requ. reinforcement as per 7.3.2/4 for crack width limitation  
Increase of reinforcement due to crack width check is marked by "!".
- (B) Design of reinforcement at ultimate limit state  
In case of dominant bending, compression reinforcement is marked with "\*".  
For section areas acc. to 6.1 (5) the correct strain is not limited.  
The minimum reinforcement acc. to 9.2.1.1 and 9.3.1.1 is not determined.  
For compressive members the minimum reinf. acc. to 9.5.2 is considered.  
For walls the minimum reinforcement as per 9.6.3 (1) is not determined.  
For the less stressed direction of slabs, the minimum reinforcement as per 9.3.1.1 (2) is considered.

Element No.	Se.	Lo.	Lay.	Type	Reinforcem. Reinf. for x-direction			Reinf. for y-direction		
					nsd [kN/m]	msd [kNm/m]	req.asx [cm <sup>2</sup> /m]	nsd [kN/m]	msd [kNm/m]	req.asy [cm <sup>2</sup> /m]
124	1	1	1	M	0.00	760.28	0.00	0.00	-47.71	10.08
				R	0.00	760.28	0.00	0.00	-41.88	22.08
	2	1	M	0.00	1618.41	0.37	0.00	-64.40	1.86	
			R	0.00	1198.82	9.82	0.00	21.95	10.08	
	2	1	M	0.00	778.28	45.58!	0.00	-23.88	0.00	
			R	0.00	1618.41	50.29	0.00	37.99	10.06	
124	1	2	1	M	0.00	769.11	0.00	0.00	-41.00	10.08
				R	0.00	769.11	0.00	0.00	-35.95	22.08
	2	1	M	0.00	1616.96	0.32	0.00	-55.35	1.60	
			R	0.00	1197.75	9.82	0.00	13.24	10.08	
	2	1	M	0.00	775.27	45.44!	0.00	-29.79	0.00	
			R	0.00	1616.96	50.24	0.00	28.30	10.05	
124	1	3	1	M	0.00	757.32	0.00	0.00	-24.58	10.08
				R	0.00	757.32	0.00	0.00	-24.02	22.08
	2	1	M	0.00	1591.86	0.20	0.00	-34.74	1.00	
			R	0.00	1179.15	9.82	0.00	39.02	10.08	
	2	1	M	0.00	762.99	44.86!	0.00	-18.35	0.00	
			R	0.00	1591.86	49.35	0.00	62.11	9.87	
124	1	4	1	M	0.00	748.50	0.00	0.00	-30.47	10.08
				R	0.00	748.50	0.00	0.00	-29.92	22.08
	2	1	M	0.00	1593.34	0.24	0.00	-41.19	1.19	
			R	0.00	1180.25	9.82	0.00	47.67	10.08	
	2	1	M	0.00	766.00	45.01!	0.00	-12.42	0.00	
			R	0.00	1593.34	49.40	0.00	73.23	9.88	

**Design of shear reinforcement**

The percentage of nominal reinforcement as per Eq. (9.5N) is considered

- vRd Absorbable lateral force of comp. struts per 6.2.3 (3) [kN/m]
- Angle Angle cot Theta between the compressive strut and the element plane
- As1 Req. longitudinal reinf. as per Fig. 6.3 for req. asb [cm<sup>2</sup>]
- qr Lateral force for design = (qx<sup>2</sup>+qy<sup>2</sup>)<sup>1/2</sup> [kN/m]
- req.asb Req. stirrup reinforcement [cm<sup>2</sup>/m<sup>2</sup>]  
Exceedings as per Eq. (6.12) are marked by "!"
- req.As1 Req. longitudinal reinf. as per Fig. 6.3 [cm<sup>2</sup>] for req. asb

Element No.	Loc.	qx [kN/m]	qy [kN/m]	n [kN/m]	qr [kN/m]	qr/vRd	Angle [cm <sup>2</sup> /m <sup>2</sup> ]	req. asb [cm <sup>2</sup> ]	As1 f. asb=0 [cm <sup>2</sup> ]	req. As1 [cm <sup>2</sup> ]
124	1	63.31	47.08	0.00	78.90	0.03	1.75	0.00	.	.
	2	63.31	48.58	0.00	79.81	0.03	1.75	0.00	.	.
	3	63.44	48.58	0.00	79.90	0.03	1.75	0.00	.	.
	4	63.44	47.08	0.00	79.00	0.03	1.75	0.00	.	.

**Fatigue check for longitudinal reinforcement**

For the check, a cracked concrete section is assumed.

- dSigma.s, equ Damage equivalent stress range [MN/m<sup>2</sup>]
- dSigma.s, zul = dSigma.Rsk,s / gamma.s, fat [MN/m<sup>2</sup>]
- (simplified) = dSigma.Rsk,s as per 6.8.6 (1) [MN/m<sup>2</sup>]

Element No.	Lo.	Lay.	Check in x-direction			Check in y-direction				
			as giv. [cm <sup>2</sup> /m]	dSigma.s req. [MN/m <sup>2</sup> ]	dSigma.s equ. [MN/m <sup>2</sup> ]	as giv. [cm <sup>2</sup> /m]	dSigma.s req. [MN/m <sup>2</sup> ]	dSigma.s equ. [MN/m <sup>2</sup> ]		
124	1	1	0.37	.	20.15	141.30	22.08	.	19.21	141.30
		2	50.29	.	86.75	141.30	10.08	.	39.86	141.30
124	2	1	0.32	.	21.48	141.30	22.08	.	23.78	141.30
		2	50.24	.	92.46	141.30	10.08	.	33.92	141.30
124	3	1	0.20	.	21.30	141.30	22.08	.	18.96	141.30
		2	49.35	.	92.97	141.30	10.08	.	72.60	141.30
124	4	1	0.24	.	19.91	141.30	22.08	.	17.00	141.30
		2	49.40	.	86.91	141.30	10.08	.	78.05	141.30

**Fatigue check for shear reinforcement**

The stress calculation is based on a framework model.

dSigma.s, equ Damage equivalent fatigue stress range [MN/m<sup>2</sup>]  
 dSigma.s, per = dSigma.Rsk,b / gamma.s, fat [MN/m<sup>2</sup>]  
 (simplified) = dSigma.Rsk,b as per 6.8.6 (1) [MN/m<sup>2</sup>]

Element			asb [cm <sup>2</sup> /m <sup>2</sup> ]		dSigma.s [MN/m <sup>2</sup> ]	
No.	Se.	Loc.	giv.	req.	equ.	per.
124	1	1	0.00	.	.	63.48
		2	0.00	.	.	63.48
		3	0.00	.	.	63.48
		4	0.00	.	.	63.48

**Check of crack widths**

The check is led by direct calculation of the crack width.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

wk Calculated value of crack width as per 7.3.4 [mm]  
 wmax Permissible crack width as per specification [mm]  
 nsd, msd Longitudinal force and moment for design [kN/m, kNm/m]  
 (CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

Element				Check in x-direction			Check in y-direction				
No.	Se.	C.	Loc.	nsd	msd	wk	wmax	nsd	msd	wk	wmax
				[kN/m]	[kNm/m]	[mm]	[mm]	[kN/m]	[kNm/m]	[mm]	[mm]
124	1	QC	1	0.00	778.28	0.26	0.30	0.00	-41.88	0.05	0.30
			2	0.00	775.27	0.26	0.30	0.00	-35.95	0.04	0.30
			3	0.00	762.99	0.26	0.30	0.00	-24.02	0.03	0.30
			4	0.00	766.00	0.26	0.30	0.00	-29.92	0.04	0.30

**Check of concrete compressive stress**

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked. On cracked section, the stress min(Sigma.x, Sigma.y) as the extremal value of the two reinforcing directions is checked.

Sigma,min Total maximal longitudinal compressive stress [MN/m<sup>2</sup>]  
 Sigma,zul = 0.60\*fck resp. 0.66\*fck for Charact. C. (CC) as per 7.2 (102)  
 = 0.45\*fck for Q.-cont. C. (QC) as per 7.2 (3)  
 (t,b) Upper, lower edge of section

Element			Stress	min Sigma	Sigma,per	Side	Period	Situation
No.	Se.	Loc.		[MN/m <sup>2</sup> ]	[MN/m <sup>2</sup> ]	t	b	
124	1	1	Sigma.x(II)	-16.29	-18.00	x	.	Final CC.1
			Sigma.x(II)	-10.89	-13.50	x	.	Final QC.1
		2	Sigma.x(II)	-16.28	-18.00	x	.	Final CC.1
			Sigma.x(II)	-10.86	-13.50	x	.	Final QC.1
		3	Sigma.x(II)	-16.16	-18.00	x	.	Final CC.1
			Sigma.x(II)	-10.77	-13.50	x	.	Final QC.1
		4	Sigma.x(II)	-16.17	-18.00	x	.	Final CC.1
			Sigma.x(II)	-10.80	-13.50	x	.	Final QC.1

**Check of reinforcing steel stress for the Characteristic (rare) combination**

For the check, a cracked concrete section is assumed.

Sigma.s, per = 0.80 \* fyk resp. 1.0 \* fyk (CK) as per 7.2 (5)

Element				Check in x-direction			Check in y-direction		
No.	Se.	Lo.	Lay.	as	Sigma.s	per.	as	Sigma.s	per.
				[cm <sup>2</sup> /m]	[MN/m <sup>2</sup> ]	[MN/m <sup>2</sup> ]	[cm <sup>2</sup> /m]	[MN/m <sup>2</sup> ]	[MN/m <sup>2</sup> ]
124	1	1	1	0.37	-45.68	400.00	22.08	29.73	400.00
			2	50.29	329.30	400.00	10.08	29.45	400.00
124	1	2	1	0.32	-46.25	400.00	22.08	25.55	400.00
			2	50.24	329.33	400.00	10.08	17.77	400.00
124	1	3	1	0.20	-45.70	400.00	22.08	15.31	400.00
			2	49.35	329.82	400.00	10.08	52.36	400.00
124	1	4	1	0.24	-45.13	400.00	22.08	18.99	400.00
			2	49.40	329.79	400.00	10.08	63.97	400.00



## Railroad Overpass With Prestressed Concrete Superstructure

This example was chosen on the basis of part 2 of the book

Müller, Michael / Bauer, Thomas

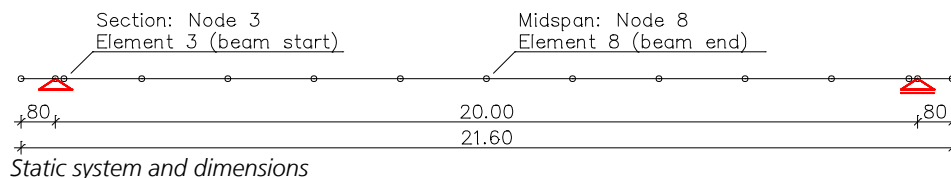
Eisenbahnbrückenbau nach DIN-Fachbericht (Railroad Bridge Construction According to DIN Technical Report)

**Beispiele prüffähiger Standsicherheitsnachweise. (Examples of Verifiable Stability Safety Checks)**

Band 1: Stahlbeton- und Spannbetonüberbau nach DIN-Fachbericht 101 und 102. (Volume 1: Reinforced Concrete and Prestressed Concrete Superstructures According to DIN Technical Reports 101 and 102)

2nd Revised Edition. Bauwerk Verlag GmbH, Berlin 2003.

The construction is a prestressed beam in the longitudinal direction with subsequent bond in exposure class XC4. The lateral direction is not analyzed in this example. The support is free floating in the longitudinal direction and fixed in the lateral direction.



### Material

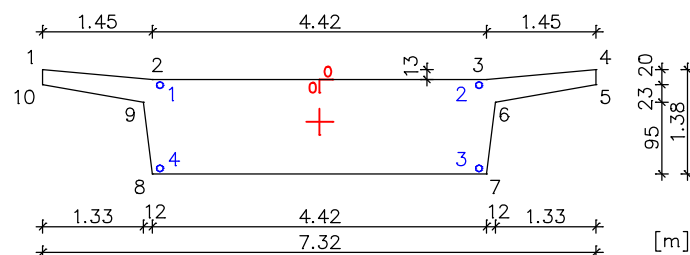
Concrete

C40/50-EN

Reinforcing steel

BSt 500/550, axis distance from edge 7.5 cm

### Section



According to the draft specification the superstructure must be designed for load model 71 with a classification coefficient of  $\alpha = 1.0$  and for load model SW/2.

The main checks are carried out for the following times:

$t_0 = 10$  days: Time of prestressing

$t_1 = 100$  days: Time of traffic transfer

$t_\infty = 100$  years: Time after conclusion of creep and shrinkage

The following checks are carried out as a part of this example:

Ultimate limit state

- Bending with longitudinal force
- Lateral force
- Failure without warning (robustness)
- Fatigue of longitudinal reinforcement and prestressing steel
- Fatigue of shear reinforcement
- Fatigue of concrete under compressive stress

Serviceability limit state

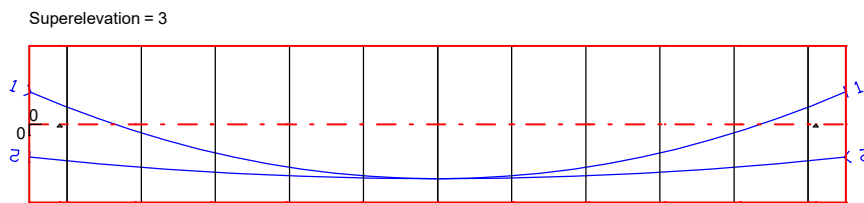
- Decompression
- Minimum reinforcement for the crack width limitation
- Crack width calculation
- Limiting the concrete compressive stresses
- Limiting the reinforcing and prestressing steel stresses

Because of their low influence on the selected checks, actions arising from centrifugal loads, lateral impacts, wind loads, the 'unloaded train' load model, starting and braking, derailment, actions on rails, traffic loads on service sidewalks and soil pressure are ignored.

**Prestressing steel and prestressing system**

Prestressing steel quality	St 1500/1770
Certification of the prestressing system	SUSPA EC 140
Number of tendons in the bundle	6
Section surface $A_p$	2660 mm <sup>2</sup>
E-modulus of the prestressing steel	190000 MN/m <sup>2</sup>
0.1% strain limit (yield strength) of the prestressing steel $f_{p0.1k}$	1500 MN/m <sup>2</sup>
Tensile strength of the prestressing steel $f_{pk}$	1770 MN/m <sup>2</sup>
Permissible prestressing force of a tendon $P_{m0}$	3391.5 kN
Friction coefficients when prestressing and releasing $\mu$	0.21
Unintentional deviation angle of a tendon $\beta'$	0.3 °/m
Slippage at prestressing anchor	6 mm
Duct diameter $d_h$	97 mm
Variation factors of internal prestressing	
Construction stage ( $r_{sup} / r_{inf}$ )	1.1 / 0.9
Final state ( $r_{sup} / r_{inf}$ )	1.1 / 0.9

The tendon guide is shown in the next figure. 2 tendon groups with 6 bundled tendons each are arranged such that they stretch across the entire girder length and are alternately prestressed at both girder ends. The prestressing systems, prestressing procedures and prestressing curves for both tendon groups are also shown.



Tendon group ordinates  $z_v$  [cm] at the base points

xv	0.00	1.00	2.96	4.92	6.88	8.84	10.80	12.76	14.72	16.68	18.64	20.60	21.60
1	-28.6	-15.1	7.6	25.3	37.9	45.5	48.0	45.5	37.9	25.3	7.6	-15.1	-28.6
2	28.8	32.2	37.9	42.3	45.5	47.4	48.0	47.4	45.5	42.3	37.9	32.2	28.8

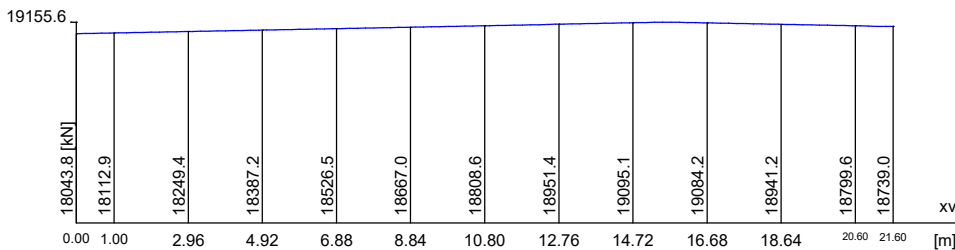
*Tendon guide in the longitudinal section*

Force function of tendon group 1 (6 tendon(s),  $l = 21.67$  m)

Prestressing system 1 - SUSPA EC 140. Certification according to EC2.  
 $P_{m0} = 3391.5$  kN,  $A_p = 2660.0$  mm<sup>2</sup>,  $\mu_a = 0.21$ , Angle  $\beta' = 0.30$  °/m  
 E-Modulus = 190000 MN/m<sup>2</sup>,  $A_h = 7389.8$  mm<sup>2</sup>,  $\mu_n = 0.21$ , Slippage = 6.00 mm

Prestressing procedure 1 -  $P_{m0}$

Pre. anchor : Start  
 Normal. force : 1.000  
 Pre. force [kN]: 3391.5  
 Extension [mm]: 139.6



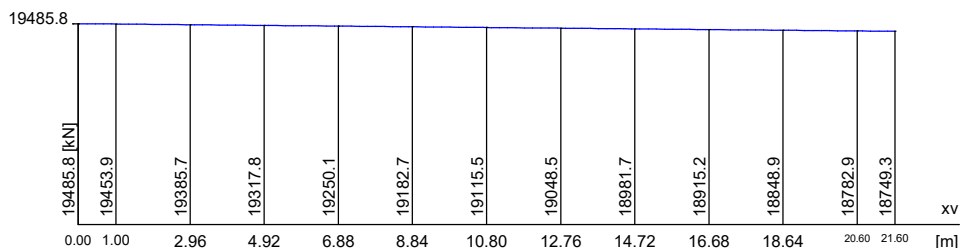
*Prestressing curve of tendon group 1 in the longitudinal section*

Force function of tendon group 2 (6 tendon(s), l = 21.60 m)

Prestressing system 1 - SUSPA EC 140. Certification according to EC2.  
 Pm0 = 3391.5 kN, Ap = 2660.0 mm<sup>2</sup>, μa = 0.21, Angle β = 0.30 °/m  
 E-Modulus= 190000 MN/m<sup>2</sup>, Ah = 7389.8 mm<sup>2</sup>, μn = 0.21, Slippage = 6.00 mm

Prestressing procedure 1 - Pm0

Pre. anchor : Start  
 Normal. force : 1.000  
 Pre. force [kN]: 3391.5  
 Extension [mm]: 142.2

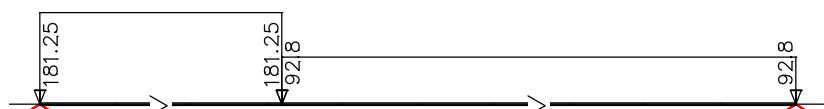


Prestressing curve of tendon group 2 in the longitudinal section

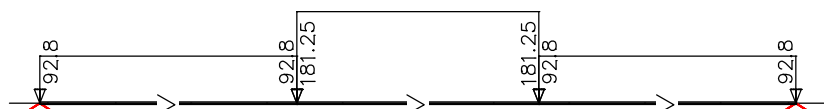
**Loads**

- Load case 1      Dead load (G1).
- Load case 2      Additional loads 103,7 kN/m (G2).
- Load case 3      Prestressing (P).
- Load case 4      Creep-generating permanent load (G1+G2+P)
- Load case 5      Creep and shrinkage (CS, t<sub>∞</sub>). The specifications  
 RH=80%, t<sub>0</sub>=10 d, t<sub>s</sub>=1 d, cement hardening = normal  
 result in the following coefficients: φ<sub>t∞</sub> = 1.57; ε<sub>t∞</sub> = -23.7·10<sup>-5</sup>; ρ = 0.8  
 Creep-generating permanent load case: 4,  
 The redistribution of internal forces between concrete and prestressing steel are taken into account.
- Load case 11-13    Positions of load model 71 (Q1)
- Load case 21      Load model SW/2 (Q2)
- Load case 31      Temperature Δx<sub>neg</sub> (T)
- Load case 32      Temperature Δx<sub>pos</sub> (T)

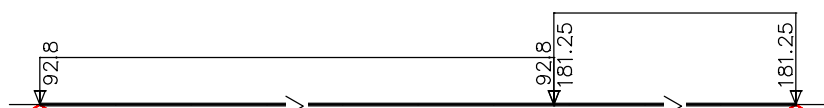
**Load model 71**



Load case 11: Load model 71, left  
 The dynamic coefficient Φ of 1.16 is already considered in the load ordinates.

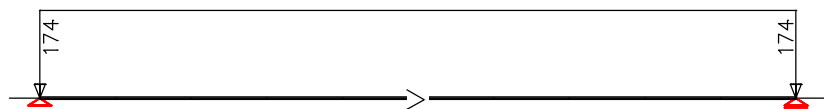


Load case 12: Load model 71, middle



Load case 13: Load model 71, right

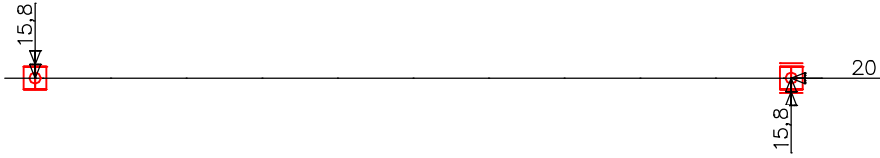
**Load model SW/2**



Load case 21: The dynamic coefficient Φ of 1.16 is already considered in the load ordinates.

**Temperature**

Due to the free-floating support, the temperature loads  $\Delta T_{Mz}$  and  $\Delta T_N$  that are to be applied result in support movements and therefore restoring forces according to Müller, pp. 71-72.



Load case 31: Restoring forces as external support reactions

The actions and action combinations defined for the checks are listed in the following.

**EN 1992-2 actions**

**G1 - Permanent action, dead load**

- 1 G1

**G2 - Permanent action, additional dead load**

- 2 G2

**P - Prestress, intern**

- 3 P

**CSR1 - Creep, shrinkage, relaxation, t1**

CS as constant reduction of the prestressing with 4 %.  
 Prestressing loss from relaxation of prestressed steel: 2.2 %.

**CSR2 - Creep, shrinkage, relaxation, too**

- 5 CS,too

Prestressing loss from relaxation of prestressed steel: 6.2 %.

**T - Temperature, 1. Variante, exclusive**

- 31 Temperature dTN (dx,neg)
- 32 Temperature dTN (dx,pos)

**Q1 - Load group gr11-gr14 (1 track), 1. Variante, exclusive**

- 11 Qvk LM 71,1
- 12 Qvk LM 71,2
- 13 Qvk LM 71,3

**Q2 - Load group gr16-gr17 (1 track), 1. Variante, exclusive**

- 21 Qvk LM SW/2

**Qfat - Cyclic fatigue action, 1. Variante, exclusive**

- 11 Qvk LM 71,1
- 12 Qvk LM 71,2
- 13 Qvk LM 71,3

**Permanent and temporary combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) t1	Final	G1 + G2 + P + CSR1 + QK
3) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Permanent and temporary combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1.35	1
G2	1.35	1
P, CSR1, CSR2	1	1
T	1.5	0
Q1	1.45	0
Q2	1.2	0

**Permanent and temporary combination, combination coefficients**

Variant	T	Q1	Q2
a) T	1	0.8	0
b) T	1	0	0.8
c) Q1	0.6	1	0
d) Q2	0.6	0	1

**Characteristic (rare) combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) t1	Final	G1 + G2 + P + CSR1 + QK
3) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Characteristic (rare) combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0

**Characteristic (rare) combination, combination coefficients**

Variant	T	Q1	Q2
a) T	1	0.8	0
b) T	1	0	0.8
c) Q1	0.6	1	0
d) Q2	0.6	0	1

**Frequent combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) t1	Final	G1 + G2 + P + CSR1 + QK
3) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Frequent combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0

**Frequent combination, combination coefficients**

Variant	T	Q1	Q2
a) T	0.6	0	0
b) Q1	0.5	0.8	0
c) Q2	0.5	0	0.8

**Quasi-continuous combination, situations**

Variant	State	Actions
1) t0	Constr. - Ungr.	G1 + P
2) too	Final	G1 + G2 + P + CSR2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Quasi-continuous combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0

**Quasi-continuous combination, combination coefficients**

Variant	T	Q1	Q2
a)	0.5	0	0

**Fatigue combination, situations**

Variant	State	Actions
1) too	Final	G1 + G2 + P + CSR2 + QK + Qfat

QK means changeable actions in accordance with the table of the combination coefficients.

**Fatigue combination, safety coefficients**

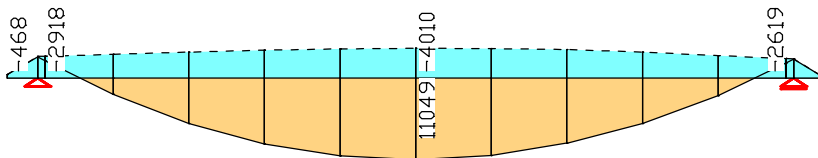
Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0
Qfat	1	0

**Fatigue combination, combination coefficients**

Variant	T	Q1	Q2
a)	0.6	0	0

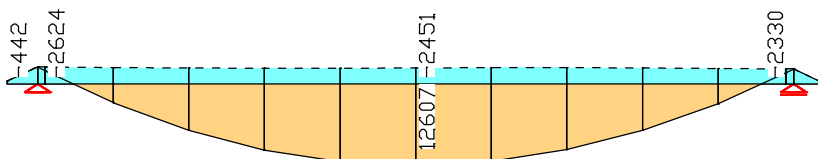
Below you will find an example of the curve of bending moment  $M_y$  for design situations in the ultimate limit states.

**2. Permanent and temporary combination – t1**



Bending moment  $M_y$  [kNm]

**3. Permanent and temporary combination - too**



Bending moment  $M_y$  [kNm]

**Design according to EN 1992-2:2008**

The design is applicable for reinforced and prestressed concrete bridges with and without bond. The actions are combined according to EN 1990, Eq. (6.10), using the specific partial safety and combination factors.

**All checks are made for the extreme values of actions.**

For the flexural design of prestressed beam sections with bond, the internal forces of the bond section and the tendon groups with their prestressing are considered.

With this, the situations before pressure grouting are excluded.

**Design overview**

Se.	Expos. class	Prestress of component	Reinforcem. M R B Q T S	Fatigue B Q T P C V	Cr. wi.	De-co.	Stress C B P
1	XC4	Subsequent bond	x x x x . .	x x . x x .	x	x	x x x

(M) Nominal reinforcement to guarantee robustness.

(R) Nominal reinforcement for crack width limitation.

(B) Flexural reinforcement at ultimate limit state, fatigue and stress check.

(Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.

(T) Torsional reinforcement at ultimate limit and fatigue state.

(S) Shear joint check.

(P) Prestressing steel at fatigue and stress check.

(C) Concrete comp. stress, concrete at fatigue check under long. compression.

(V) Concrete at fatigue check under lateral force.

**Variation of prestressing**

The variation of prestressing is considered at the following checks:

- Check of decompression and concrete compressive stress

- Nominal reinforcement for crack width limitation

- Check of crack width

All other checks are made using the mean value  $P_{m,t}$  of prestressing.

Se.	Prestressing of component	Const.period r.sup	r.inf	Final state r.sup	r.inf
1	Subsequent bond	1.10	0.90	1.10	0.90

**Settings for flexural and shear reinforcement**

M,N Design mode for bend and longitudinal force:  
(ST) Standard, (SY) Symmetrical, (CM) Compression member.  
(\*) Design without considering specified ratio between reinf. layers.

fyk Quality of stirrups.

Theta Angle of concrete truss.

S. Beams are designed like slabs.

C. Design for resulting lateral force at circular resp. ring section.

Asl Given reinforcement according to picture 6.3, increase to maximum.

rhov Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).

as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).

Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den-sity [kg/m <sup>3</sup> ]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. S.C.	Asl [cm <sup>2</sup> ] given max	Factor rhov	as	Red. pre-str.
1	C40/50-EN	.	ST	500	1.75	.	0.00	1.00	.	.

**Shear sections**

bw.nom Nominal width of the prestressed section according to 6.2.3(6).

h.nom Nominal height of the prestressed section according to 6.2.3(6).

kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.

z1, z2 Height and width of the core section for torsion.

tef Thickness of the torsion box.

B. Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width [m]	Eff. width bw	Height [m] h	Eff.height d	Torsion section [m] z1	z2	tef	B.				
1	4.420	4.129	4.345	0.90	1.250	1.250	1.175	0.90	1.100	4.270	0.150	.

**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel [mm].

max.s Maximal given bar spacing of the reinforcing steel [mm].

sr,max Upper limit for the crack spacing from Eq. (7.11) [mm].

Xil Bond coefficient of prestressing steel for beam sections.

k Coefficient for consideration of non-linear distributed tensile stress.

kt Coefficient for the duration of the load to calculate the crack width.

Fact. Reduction factor for fctm as per Chapt. 7.3.2 (As) resp. 7.3.4 (wk).

Comb. Combination for verifying the minimum reinf. (As) and crack width (wk):  
CC, TC, QC = Characteristic, frequent, quasi-continuous combination,  
CT, TT, TB = Central tension, tension on top side, tension on bottom,  
CL = Action combination according to exposure class.

Method Check method for minimum reinf. (kc) and crack width (wk):  
Determination of coefficient kc for webs/chords per Eq. (7.2/7.3).  
auto = Eq. (7.2) for rectangular sections, Eq. (7.3) for others.

wk Calc. = Direct calculation of crack width as per Chapter 7.3.4,  
Bar sp. = Limiting the bar spacing as per Table 7.3N,  
Calc.(m) = Direct calculation for mean steel strain within Ac,eff,  
Sp.(m) = Lim. the bar spacing for mean steel strain within Ac,eff.

RS Ring-shaped determination of Ac,eff according to Wiese et al.,  
Beton- und Stahlbetonbau 2004, Issue 4, p 253 ff.

Se.	wmax [mm]	ds	max s	sr	Coefficient Xil	Fact.fctm As	Comb. As	Method wk	RS					
1	0.20	16	.	.	0.31	1.00	0.4	1.00	1.00	CL	CL	auto	calc.	.

**Settings for the check of concrete and steel stresses**

fck(t) Compressive strength of concrete at the time t of prestressing.  
 Sigma.c(t) Concrete compressive stress at the time t of prestressing.  
 Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.x Concrete stress from bending and normal force in beam direction.  
 Sigma.s Reinforcing steel stress in the serviceability limit state.  
 (CC), (QC) Characteristic, quasi-continuous combination.  
 (TC), (CL) Frequent combination, combination according to exposure class.

Se.	fck(t) [MN/m <sup>2</sup> ]	per.sigma.c(t) (CC, QC)	per.sigma.c (CC) (QC)	per.sigma.s (CC)	Decompression Comb.	Stress
1	36.00	0.45 fck(t)	0.60 fck	0.45 fck	0.80 fyk	CL Sigma.x

**Settings for the fatigue check**

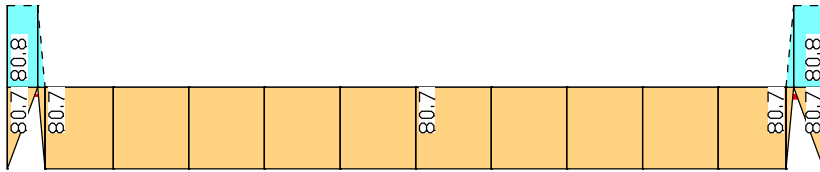
The concrete compressive stresses are calculated according to fig. 3.2.

fcd,fat Compr. strength of concrete before beginning of cyclic actions.  
 dSigma.Rsk,s Permissible charact. stress fatigue range of longitudinal reinf.  
 dSigma.Rsk,b Permissible charact. stress fatigue range of shear reinf.  
 dSigma.Rsk,p Permissible charact. stress fatigue range of prestr. steel.  
 Lambda.s,b,p Corresp. correction coeff. for damage equivalent fatigue range.  
 Lambda.c Correction coeff. for damage equivalent concrete stresses.  
 Eta Increase factor for reinforcing steel stress acc. to 6.8.2(2)P.  
 Check SPCVQT Check for (S) flexural reinf., (P) prestr. steel, (C) concrete,  
 (V) compr. strut, (Q) Lateral force reinf., (T) Torsional reinf.  
 (x) Check with damage equivalent fatigue ranges - fatigue comb.  
 (+) Simplified check acc. to 6.8.6(2) - frequent combination.  
 Factor Qfat Factor for Qfat to calculate the damage equ. fatigue range  
 for reinf. and prestr. steel as per Annex NN.2.1(101).

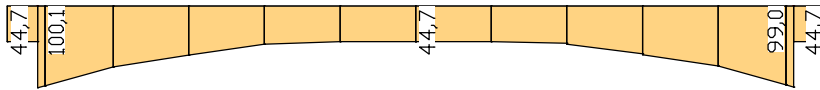
Se.	Check	fcd,fat [MN/m <sup>2</sup> ]	dSigma.Rsk s b p	Lambda s b p c	Eta	Factor Qfat
1	xxx.x.	13.68	162.00 73.00 120.00	0.65 0.65 0.70 0.90	2.00	1.00

**Results**

The following illustrations show the curve of the required longitudinal and shear reinforcement. With respect to design in the ultimate limit state, the strain reserves of the tendons are enough to perform the check without additional bending reinforcement. Hence only the robustness reinforcement are necessary.



Minimum reinforcement  $A_s$  for ensuring robustness (ductility) [cm<sup>2</sup>] (upper reinforcement with dashed lines).



Necessary lateral force reinforcement  $A_{sbz}$  to satisfy the fatigue check [cm<sup>2</sup>/m]

Excerpts from the standard check log for beam 3 at location 1 (first cut, x = 0.2 m) and beam 8 at location 1 (midspan) are provided below.

**Design of longitudinal reinforcement**

The calculated requ. reinforcement includes the specified basic reinforcement.

(M) Nominal reinf. for robustness as per EN 1992-2, 6.1 (109) (Charact. C.)  
 (R) Nominal/requ. reinforcement as per 7.3.2/4 for crack width limitation  
 Increase of reinforcement due to crack width check is marked by "!"  
 Ap' Part of prestr. steel area  $X_{il} \cdot A_p$  which was used to reduce req.As  
 Xil Bond coefficient for prestressing steel as per Eq. (7.5)  
 (B) Design of reinforcement at ultimate limit state  
 In case of dominant bending, compression reinforcement is marked with "\*\*\*".  
 For section areas acc. to 6.1 (5) the concrete strain is not limited.  
 The minimum reinforcement acc. to 9.2.1.1 and 9.3.1.1 is not determined.  
 For compressive members the minimum reinf. acc. to 9.5.2 is considered,  
 whereby the prestressing steel area of bonded tendons is taken into account.

Beam	No.	Se.	Lo.	Reinforcement Lay. Type	Nx [kN]	My [kNm]	Mz [kNm]	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]
3	1	1	1	M	-1.37	265.35	0.00	.	0.00
				R	0.00	0.00	0.00	.	0.00
				B	-32185.62	-2624.16	0.00	.	0.00
				M	-1.37	265.35	0.00	.	0.00
				R	0.00	0.00	0.00	.	0.00
				B	-32185.62	-2624.16	0.00	.	0.00
			2	M	64.14	788.22	0.00	.	40.35
				R	0.00	0.00	0.00	.	0.00
				B	-32137.62	-2019.75	0.00	.	0.00
				M	64.14	788.22	0.00	.	40.35
				R	0.00	0.00	0.00	.	0.00
				B	-32137.62	-2019.75	0.00	.	0.00



Beam No.	Se.	Lo.	Reinforcement Lay.	Type	Nx [kN]	My [kNm]	Mz [kNm]	Ap' [cm <sup>2</sup> ]	req.As [cm <sup>2</sup> ]	
8	1	1	1	M	0.00	7936.54	0.00	.	0.00	
				R	-41716.52	-12087.39	0.00	.	0.00	
				B	-37924.11	-10267.04	0.00	.	0.00	
				2	M	0.00	7936.54	0.00	.	0.00
					R	-41716.52	-12087.39	0.00	.	0.00
					B	-37924.11	-10267.04	0.00	.	0.00
			3	M	12.00	21797.83	0.00	.	40.35	
				R	0.00	0.00	0.00	.	0.00	
				B	-35554.82	11048.54	0.00	.	0.00	
				4	M	12.00	21797.83	0.00	.	40.35
					R	0.00	0.00	0.00	.	0.00
					B	-35554.82	11048.54	0.00	.	0.00

**Design of shear reinforcement**

The percentage of nominal reinforcement acc. to Eq. (9.5N) is considered.

VRd, TRd Design value of maximum absorbable lateral force, torsional moment  
 Angle Angle cot Theta between the compressive strut and the beam axis  
 Asb,Asl.T Req. stirrup reinf. from lateral force and torsion, torsional reinf.  
 Exceedings as per Eq. (6.12) are marked by "!"  
 Asl Req. longitudinal reinf. acc. to Fig. 6.3 for req. Asb.

Beam No.	Loc.	Qy/VRd	Angle [cm <sup>2</sup> /m]	Asb.y VRd	Qz/VRd	Angle [cm <sup>2</sup> /m]	Asb.z [cm <sup>2</sup> ]	Asl Mx/TRd	Q/VRd+	Asb.T [cm <sup>2</sup> /m]	Asl.T [cm <sup>2</sup> ]
3	1	0.00	1.75	0.00	0.11	1.75	44.73	.	.	.	.
8	1	0.00	1.75	0.00	0.01	1.75	44.73	.	.	.	.

**Fatigue check for longitudinal reinforcement and prestressing steel**

For the check, a cracked concrete section is assumed.

Type S Longitudinal reinforcement from N and M, layer number  
 Type P Prestressing steel, tendon number  
 dSigma.s, equ Damage equivalent stress range [MN/m<sup>2</sup>]  
 dSigma.s, zul = dSigma.Rsk,s / gamma.s, fat [MN/m<sup>2</sup>]  
 (simplified) = dSigma.Rsk,s as per 6.8.6 (1) [MN/m<sup>2</sup>]

Beam No.	Se.	Loc.	Steel Type	No.	As [cm <sup>2</sup> ] giv.	req.	dSigma.s [MN/m <sup>2</sup> ] equ.	per.			
3	1	1	S	1	0.00	.	140.87				
				2	0.00	.	140.87				
				3	40.35	1.31	140.87				
				4	40.35	1.31	140.87				
			P	1	159.60	0.17	104.35				
				2	159.60	0.35	104.35				
				8	1	S	1	0.00	.	140.87	
							2	0.00	.	140.87	
							3	40.35	31.64	140.87	
							4	40.35	31.65	140.87	
P	1	159.60	12.54			104.35					
	2	159.60	12.54			104.35					

**Fatigue check for shear reinforcement**

The stress calculation is based on a framework model.

Type BY, BZ Shear reinforcement from Qy or Qz [cm<sup>2</sup>/m]  
 Asb giv. Given reinforcement from preceded dimensioning [cm<sup>2</sup>/m]  
 Asb req. Requ. reinf. for compl. with the permiss. stress range [cm<sup>2</sup>/m]  
 dSigma.s, equ Damage equivalent fatigue stress range [MN/m<sup>2</sup>]  
 dSigma.s, per = dSigma.Rsk,b / gamma.s, fat [MN/m<sup>2</sup>]  
 (simplified) = dSigma.Rsk,b as per 6.8.6 (1) [MN/m<sup>2</sup>]

Beam No.	Se.	Loc.	Rei. Type	Asb [cm <sup>2</sup> /m] giv.	req.	dSigma.s [MN/m <sup>2</sup> ] [MN/m <sup>2</sup> ]
3	1	1	BY	0.00	.	63.48
			BZ	44.73	100.08	63.48
8	1	1	BY	0.00	.	63.48
			BZ	44.73	44.73	11.00

**Fatigue check for concrete under longitudinal compressive stress**

For the check, a cracked concrete section is assumed.

Sigma.cd,min Absolute value of damage equiv. min. compressive stress [MN/m<sup>2</sup>]  
 Sigma.cd,max Absolute value of damage equiv. max. compressive stress [MN/m<sup>2</sup>]  
 Sigma.cd,per = fcd, fat \* (1 - 0.43 \* (1 - Sigma.cd,min/Sigma.cd,max)<sup>1/2</sup>) [MN/m<sup>2</sup>]  
 t, b Position of the edge point: above, below of centre

Beam No.	Se.	Loc.	Sigma.cd min [MN/m <sup>2</sup> ]	Sigma.cd max [MN/m <sup>2</sup> ]	Sigma.cd,per [MN/m <sup>2</sup> ]	Se.- pnt.	Side t b
3	1	1	6.46	6.64	12.72	8	. x
8	1	1	3.49	8.00	9.27	4	x .

**Check of crack widths**

The check is led by direct calculation of the crack width.  
 The final long. reinforcement as the maximum from robustness, crack and bending reinf. incl. a possible increase resulting from the fatigue check is decisive.

wk Calculated value of crack width as per 7.3.4 [mm]  
 wmax Permissible crack width as per specification [mm]  
 Sigma.c Maximal concrete edge stress in state I [MN/m<sup>2</sup>]  
 (CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

Beam	No.	Se.	C.	Lo.	Reinf. Layer	Nx [kN]	My [kNm]	Mz [kNm]	Sigma.x [MN/m <sup>2</sup> ]	wk [mm]	wmax [mm]
	3	1	TC	1	.	-41150.8	-3641.36	0.00	-3.75	0.00	0.20
	8	1	TC	1	.	-34131.7	-8446.68	0.00	0.89	-.	0.20

**Check of decompression**

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked. If the minimal edge distance limits the compressive depth, "!" is marked.

(TC), (QC) Frequent combination, Quasi-continuous combination  
 dp Depth of the tendon or duct within the concrete compressive zone [mm]  
 dp,min Minimal value for dp as per 7.3.1 (105) [mm]

Beam	No.	Se.	Loc.	Sigma.x [MN/m <sup>2</sup> ]	Compr.depth [mm]	Tendon group	Period	Situation	
				top	bottom	dp	dp,min		
	3	1	1 (I)	-2.75	-6.10	319.9	100	2 Final	QC.2
	8	1	1 (I)	-3.82	-5.07	161.8	100	1 Final	QC.2

**Check of concrete compressive stress**

For the check, a cracked concrete section (II) is assumed if the tensile stress from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section (I) is used. If the strain is not absorbable on cracked section, (I\*) is marked.

Sigma.x,min Total maximal longitudinal compressive stress [MN/m<sup>2</sup>]  
 Sigma.x,zul = 0.60\*fck resp. 0.66\*fck for Charact. C. (CC) as per 7.2 (102)  
 = 0.45\*fck for Q.-cont. C. (QC) as per 7.2 (3)  
 (t,b) Position of the edge point: above, below of centre

Beam	No.	Se.	Loc.	Sigma.x,min [MN/m <sup>2</sup> ]	Sigma.x,per [MN/m <sup>2</sup> ]	Se.-Pnt.	Side	Period	Situation
						t	b		
	3	1	1 (I)	-9.24	-16.20	7	.	x Constr.	CC.1
			(I)	-9.24	-16.20	7	.	x Constr.	QC.1
	8	1	1 (I)	-15.84	-16.20	7	.	x Constr.	CC.1
			(I)	-15.84	-16.20	7	.	x Constr.	QC.1

**Check of steel stress**

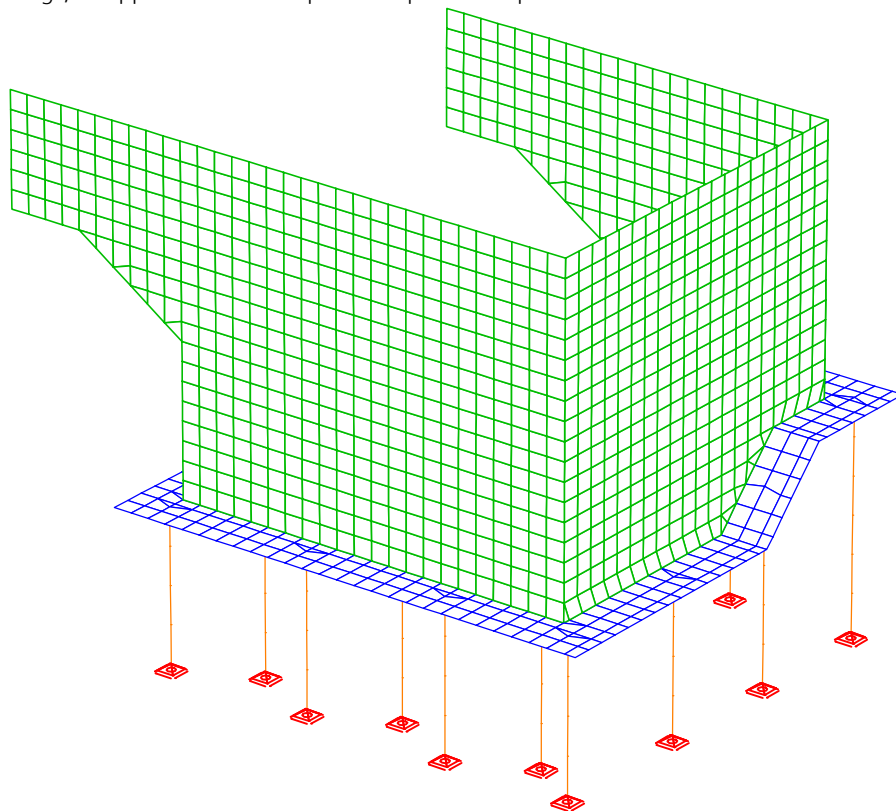
For the check, a cracked concrete section is assumed.  
 For tendon groups without bond and/or for situations before grouting, the prestressing steel stress is checked acc. to Eq. (5.43).

Type S Long. reinf. from N and M, layer number, Charact. C. (CC)  
 Type P Prestressing steel, Tendon number, Charact. C. (CC)  
 Sigma.s,per = 0.80 \* fyk resp. 1.0 \* fyk (CK) as per 7.2 (5)  
 Sigma.p,per = 0.75 \* fpk as per 7.2 (5)

Beam	No.	Se.	Lo.	Type	Steel No.	As [cm <sup>2</sup> ]	Sigma.s [MN/m <sup>2</sup> ]	per. [MN/m <sup>2</sup> ]	Situation
	3	1	1	S	1	0.00	.	400.00	CC.3
				S	2	0.00	.	400.00	CC.3
				S	3	40.35	-34.71	400.00	CC.3
				S	4	40.35	-34.71	400.00	CC.3
				P	1	159.60	1134.89	1275.00	CC.1
				P	2	159.60	1218.92	1275.00	CC.1
	8	1	1	S	1	0.00	.	400.00	CC.1
				S	2	0.00	.	400.00	CC.1
				S	3	40.35	-4.13	400.00	CC.3
				S	4	40.35	-4.13	400.00	CC.3
				P	1	159.60	1178.49	1275.00	CC.1
				P	2	159.60	1197.71	1275.00	CC.1

## Bridge Abutment

This example demonstrates how the EN 1992-2 applies to road bridge abutments. The construction consists of abutment wings, a support block and a pile head plate with piles. The dimensions are shown in the following figures.



*Static System*

### Material

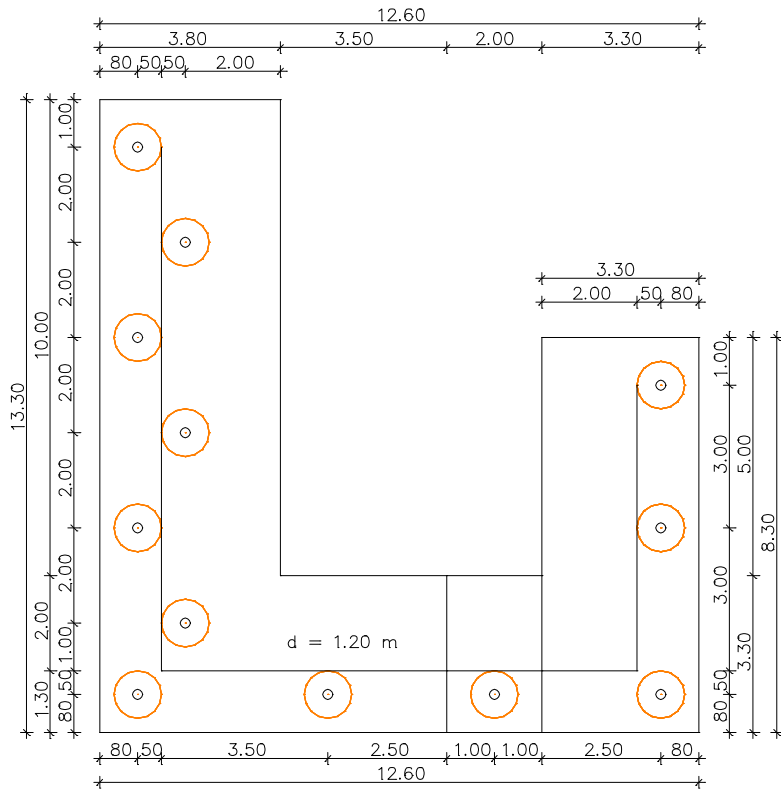
Concrete	C 30/37-EN (areas), C 40/50-EN (piles)
Reinforcing steel	BSt 500/550, axis distance from edge 8 cm
Bedding	50 MN/m <sup>3</sup> horizontal bedding module

### Section

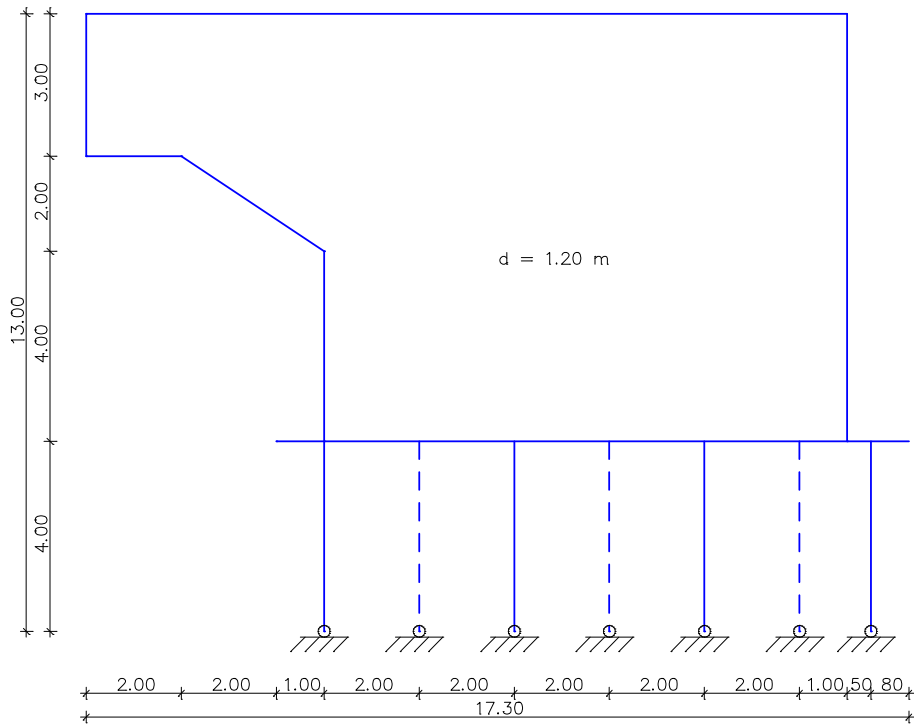
Wing walls	d = 1.2 m (section 1)
Support block	d = 1.5 m (section 2)
Pile head plate	d = 1.2 m (section 3)
Piles	∅ = 1.0 m (section 4)
Exposure class	XC4

The depicted FEM system is generated with shell elements in conjunction with beam elements. A 3D drawing consisting of edges and model surfaces formed the basis for creating the structure. It was important to position the model objects on the center lines or centroid levels of the structural components since they are used to describe the elements.

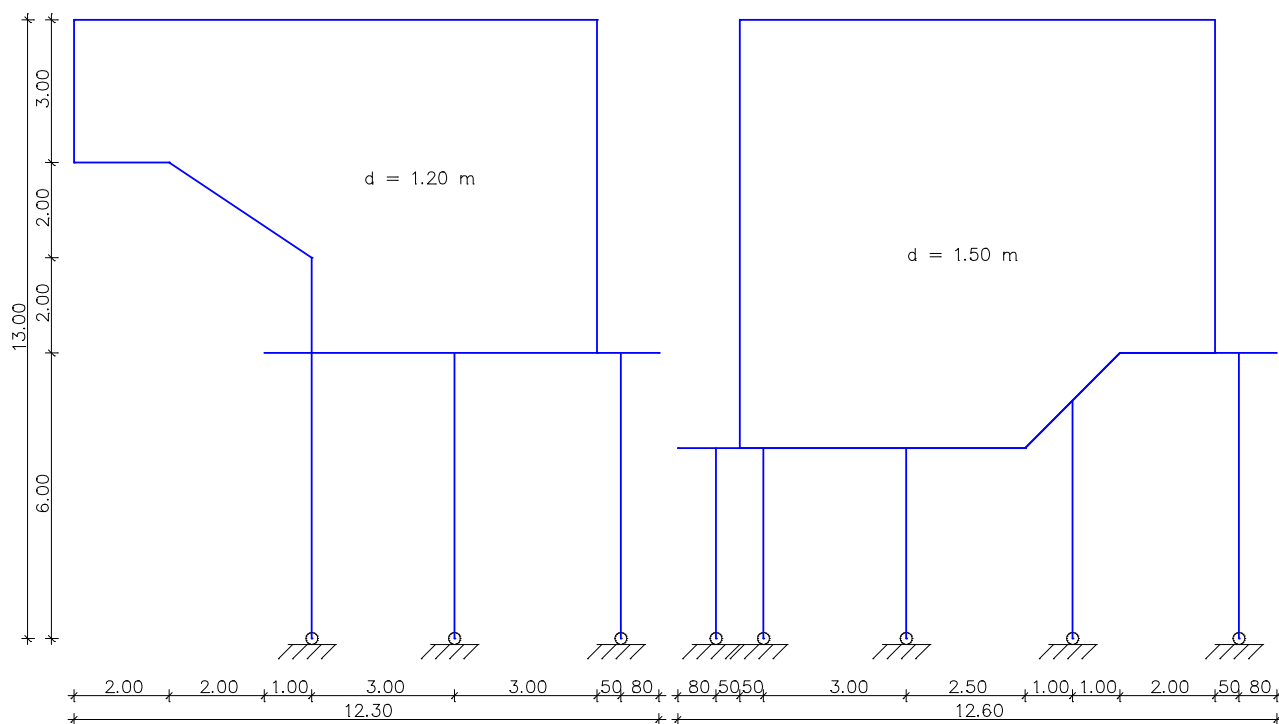
**Dimensions**



Top view of the pile head plate



View of the long wing wall



View of the short wing wall and the support block

## Checks

The following checks are carried out as a part of this example:

Ultimate limit state

- Bending with longitudinal force
- Lateral force
- Failure without warning (robustness)

Serviceability limit state

- Minimum reinforcement for the crack width limitation
- Crack width calculation
- Limiting the concrete compressive stresses
- Limiting the reinforcing steel stresses

Due to the intersection the checks are not carried out for the pile head plate. The required specifications are listed further below.

## Loads

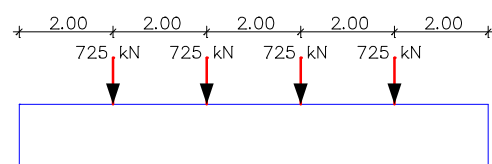
Due to the scope of the project, the actions were limited to the decisive loads.

- Load case 1 G: Dead load
- Load case 2 G: Soil pressure
- Load case 3 G: Superstructure
- Load case 4 Q: UDL (uniformly distributed load)
- Load case 5 Q: TS position 1
- Load case 6 Q: TS position 2

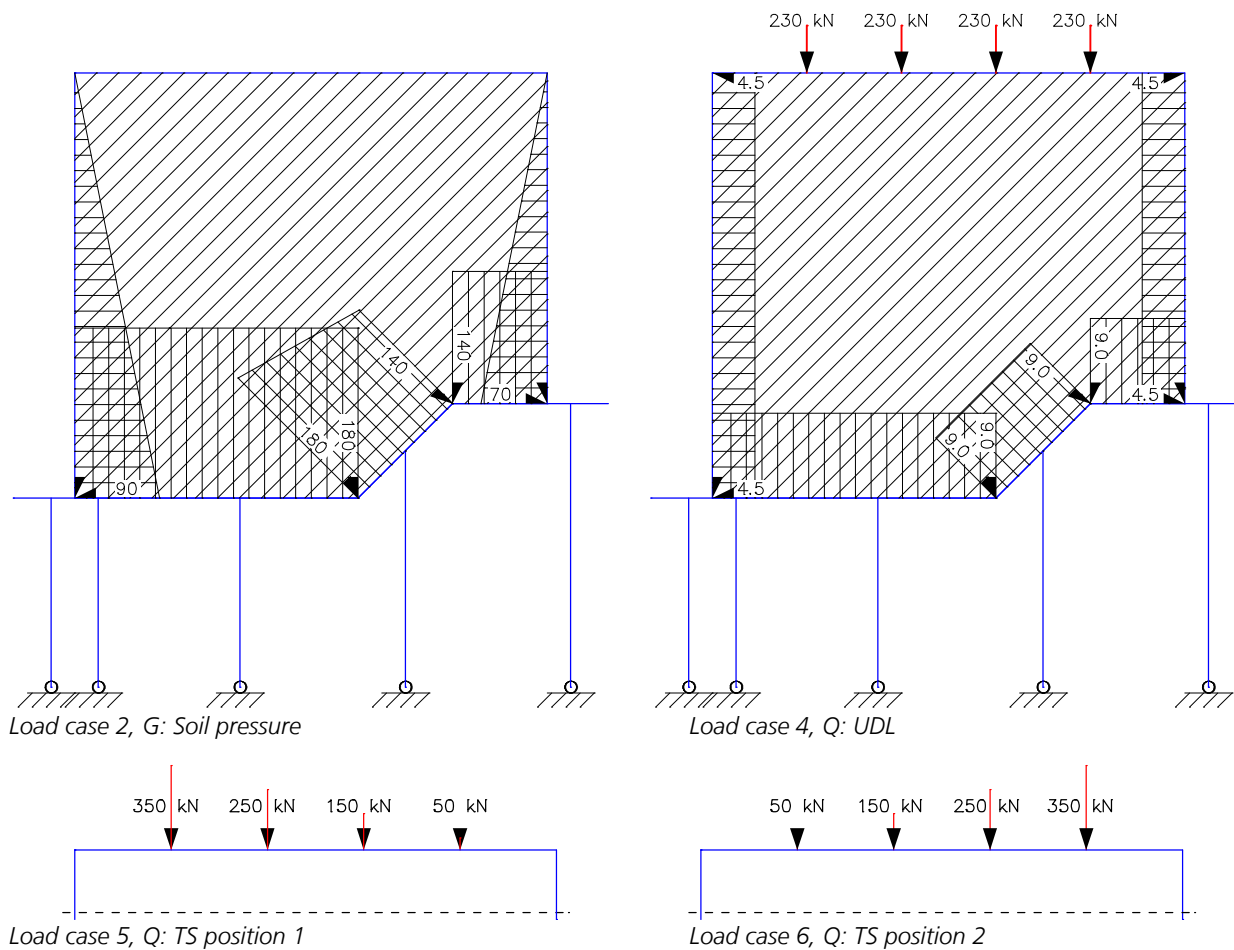
### Load data load case 1: G: Dead load

No.	Dead load (EG) referring to material and cross-section properties weighting factor in direction		
	X [-]	Y [-]	Z [-]
1	0.0000	0.0000	1.0000

Load case 1, G: Dead load



Load case 3, G: Superstructure



The actions and action combinations defined for the checks are listed in the following.

**EN 1992-2 actions**

**G1 - Permanent action**

- 1 G: Dead load
- 2 G: Earth pressure

**G2 - Superstructure**

- 3 G: Superstructure

**QUDL - Traffic, load model 1 distributed load, 1. Variant, inklusive**

- 4 Q: Uniformly distributed load

**QTS - Traffic, load model 1 tandem system, 1. Variant, exklusive**

- 5 Q: TS position 1
- 6 Q: TS position 2

**Permanent and temporary combination, situations**

Variant	State	Actions
1)	Constr.	G1
2)	Final	G1 + G2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Permanent and temporary combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1.35	1
G2	1.35	1
QTS, QUDL	1.35	0

**Permanent and temporary combination, combination coefficients**

Variant	QTS	QU DL
a)	1	1

**Characteristic (rare) combination, situations**

Variant	State	Actions
1)	Final	G1 + G2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Characteristic (rare) combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
QTS, QU DL	1	0

**Characteristic (rare) combination, combination coefficients**

Variant	QTS	QU DL
a)	1	1

**Frequent combination, situations**

Variant	State	Actions
1)	Final	G1 + G2 + QK

QK means changeable actions in accordance with the table of the combination coefficients.

**Frequent combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
QTS, QU DL	1	0

**Frequent combination, combination coefficients**

Variant	QTS	QU DL
a)	0.75	0.4

**Quasi-continuous combination, situations**

Variant	State	Actions
1)	Final	G1 + G2

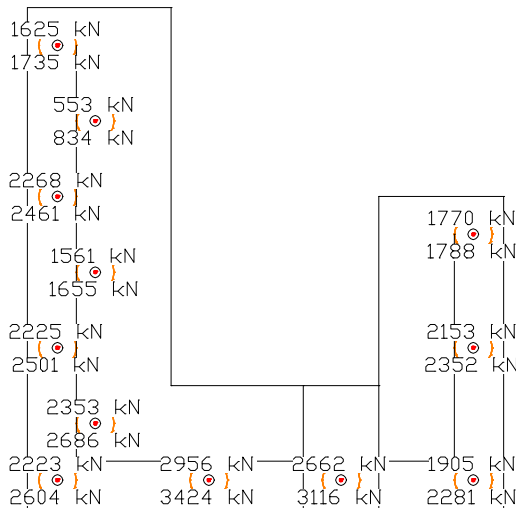
**Quasi-continuous combination, safety coefficients**

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
QTS, QU DL	1	0

Due to the low compressive forces and the high bending moments, the construction stage (G1) is decisive for load-bearing safety in several areas.

The following shows an example of the support reactions  $R_z$  for the characteristic (rare) combination.

1. Characteristic (rare) combination



Support reactions  $R_z$

Design overview EN 1992-2

Se.	Expos. class	Prestress of component	Reinforcem. M R B Q T S	Fatigue B Q T P C V	Cr. wi.	De-co.	Stress C B P
1	XC4	Not prestressed	x x x x . . . . .	. . . . .	x	.	x x .
2	XC4	Not prestressed	x x x x . . . . .	. . . . .	x	.	x x .
4	XC4	Not prestressed	. x x x . . . . .	. . . . .	x	.	x x .

- (M) Nominal reinforcement to guarantee robustness.
- (R) Nominal reinforcement for crack width limitation.
- (B) Flexural reinforcement at ultimate limit state, fatigue and stress check.
- (Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.
- (T) Torsional reinforcement at ultimate limit and fatigue state.
- (S) Shear joint check.
- (P) Prestressing steel at fatigue and stress check.
- (C) Concrete comp. stress, concrete at fatigue check under long. compression.
- (V) Concrete at fatigue check under lateral force.

Settings for flexural and shear reinforcement

- M,N Design mode for bend and longitudinal force:  
(ST) Standard, (SY) Symmetrical, (CM) Compression member.  
(\* Design without considering specified ratio between reinf. layers.
- f<sub>yk</sub> Quality of stirrups.
- Theta Angle of concrete truss.
- S. Beams are designed like slabs.
- C. Design for resulting lateral force at circular resp. ring section.
- As1 Given reinforcement according to picture 6.3, increase to maximum.
- rho<sub>w</sub> Factor for minimum reinf. rho<sub>w,min</sub> acc. to Chapter 9.3.2(2).
- as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).
- Red. Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Den-sity [kg/m <sup>3</sup> ]	Dsn. M,N	f <sub>yk</sub> [MPa]	cot Theta	Dsn. S.C.	As1 [cm <sup>2</sup> ] Pic. 6.3 given max	Factor rho <sub>w</sub> as	Red. pre-str.
1	C30/37-EN	.	SY	500	1.75	.	0.00 0.00	1.00 0.20	.
2	C30/37-EN	.	SY	500	1.75	.	0.00 0.00	1.00 0.20	.
4	C40/50-EN	.	CM	500	1.75	. x	0.00 .	1.00 .	.

Shear sections (standard design)

- bw<sub>nom</sub> Nominal width of the prestressed section according to 6.2.3(6).
- h<sub>nom</sub> Nominal height of the prestressed section according to 6.2.3(6).
- kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.
- z1, z2 Height and width of the core section for torsion.
- tef Thickness of the torsion box.
- B. Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width bw [m]	Eff. width bn [m]	Height h [m]	Eff.height d [m]	Torsion. section [m]
1	1.000	.	1.200	1.120 0.90	. . . . .
2	1.000	.	1.500	1.420 0.90	. . . . .

Shear sections (circular, circular ring design as per Bender et al. <sup>1</sup>)

- D, Di Circle diameter, inner diameter of circular ring.
- Incl. Inclination of the helical reinforcement: 90° = annular single stirrups.
- Factor Efficacy factor for calc. resistance of tension, compression strut.
- bw Effective cross-section width, for ring max. double wall thickness.
- kd Factor to calculate the inner lever arm z from the eff. height d.
- z1, z2 Height and width of the core section for torsion.
- tef Thickness of the torsion box.
- R. Circular ring section; calc. of bearing capacity acc. to Eq. (6.29).
- <sup>1</sup> Bender et al., Beton- und Stahlbetonbau 105 (2010), (7), pp. 421-432.

Se.	D [m]	Di [m]	Helix Incl.[°]	Effic. Factor	Width bw [m]	Eff.height d [m]	Torsion section [m]
4	1.000	.	90.00	0.75	0.707	0.627 0.90	0.530 0.530 0.177 .



**Settings for the check of crack widths**

ds Maximal given bar diameter of the reinforcing steel [mm].  
 max.s Maximal given bar spacing of the reinforcing steel [mm].  
 sr,max Upper limit for the crack spacing from Eq. (7.11) [mm].  
 Xil Bond coefficient of prestressing steel for beam sections.  
 k Coefficient for consideration of non-linear distributed tensile stress.  
 kt Coefficient for the duration of the load to calculate the crack width.  
 Fact. Reduction factor for fctm as per Chapt. 7.3.2 (As) resp. 7.3.4 (wk).  
 Comb. Combination for verifying the minimum reinf. (As) and crack width (wk):  
 CC, TC, QC = Characteristic, frequent, quasi-continuous combination,  
 CT, TT, TB = Central tension, tension on top side, tension on bottom,  
 CL = Action combination according to exposure class.  
 Method Check method for minimum reinf. (kc) and crack width (wk):  
 kc Determination of coefficient kc for webs/chords per Eq. (7.2/7.3).  
 auto = Eq. (7.2) for rectangular sections, Eq. (7.3) for others.  
 wk Calc. = Direct calculation of crack width as per Chapter 7.3.4,  
 Bar sp. = Limiting the bar spacing as per Table 7.3N,  
 Cal.(m) = Direct calculation for mean steel strain within Ac,eff,  
 Spc.(m) = Lim. the bar spacing for mean steel strain within Ac,eff.  
 RS Ring-shaped determination of Ac,eff according to Wiese et al.,  
 Beton- und Stahlbetonbau 2004, Issue 4, p 253 ff.

Se.	wmax	ds	max	sr	Coefficient			Fact.fctm	Comb.	Method	RS	
	[mm]		s	max	Xil	k	kt	As	wk	As wk	kc wk	
1	0.30	16	.	.	.	0.65	0.4	0.71	1.00	CL CL	auto calc.	.
2	0.30	16	.	.	.	0.65	0.4	0.71	1.00	CL CL	auto calc.	.
4	0.30	16	.	.	.	0.65	0.4	0.71	1.00	CL CL	auto calc.	x

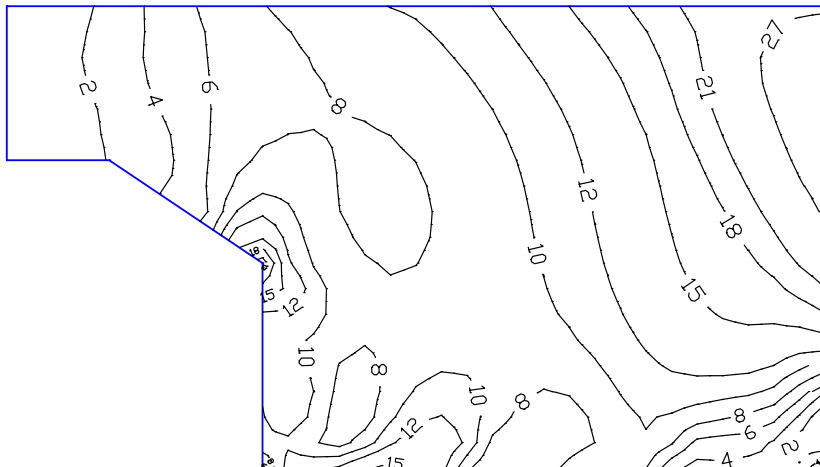
**Settings for the check of concrete and steel stresses**

Sigma.c Concrete compressive stress in the serviceability limit state.  
 Sigma.s Reinforcing steel stress in the serviceability limit state.  
 (CC),(QC) Characteristic, quasi-continuous combination.  
 (TC),(CL) Frequent combination, combination according to exposure class.

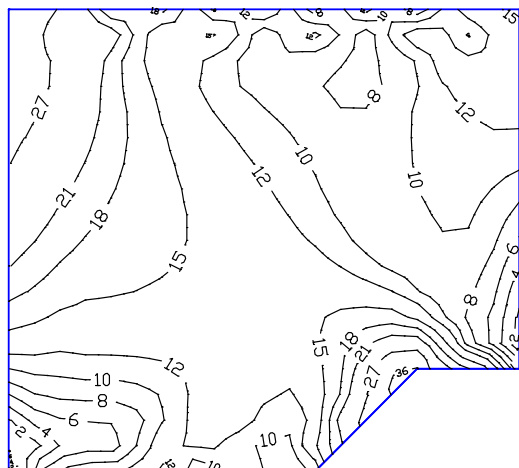
Se.	fck(t)	per.sigma.c(t)	per.sigma.c	per.sigma.s	Decompression
	[MN/m <sup>2</sup> ]	(CC, QC)	(CC) (QC)	(CC)	Comb. Stress
1	.	.	0.60 fck	0.80 fyk	.
2	.	.	0.60 fck	0.80 fyk	.
4	.	.	0.60 fck	0.80 fyk	.

**Results**

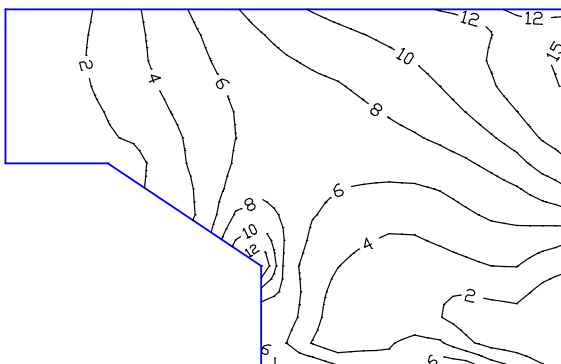
The following figures show the curve of the required longitudinal reinforcement.



Long wing wall

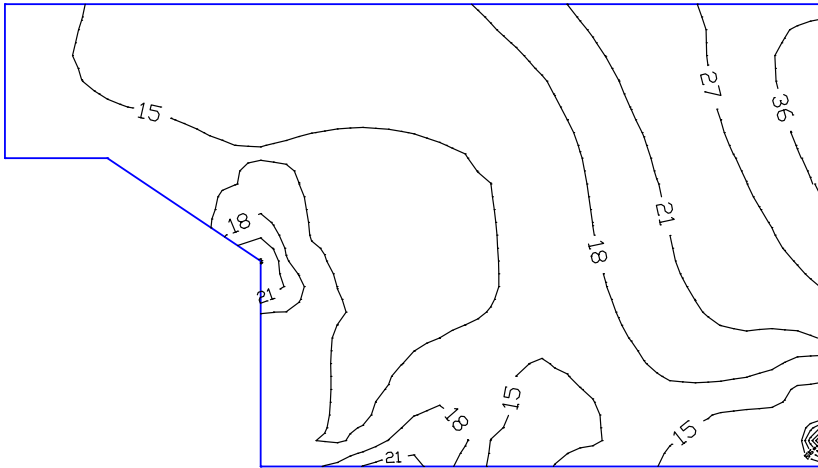


Support block

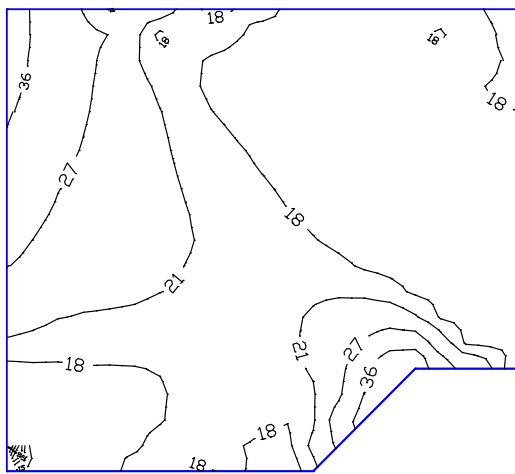


Short wing wall

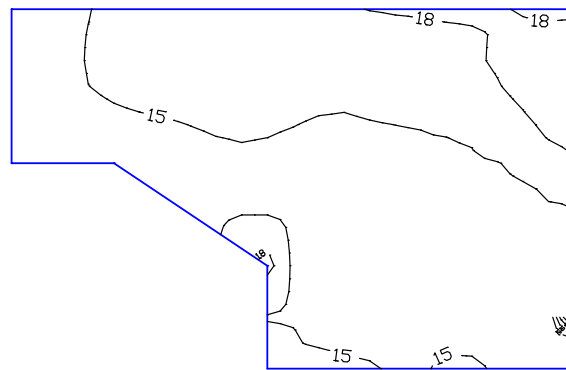
Required horizontal internal longitudinal reinforcement for the load-bearing capacity [cm<sup>2</sup>/m]



Long wing wall

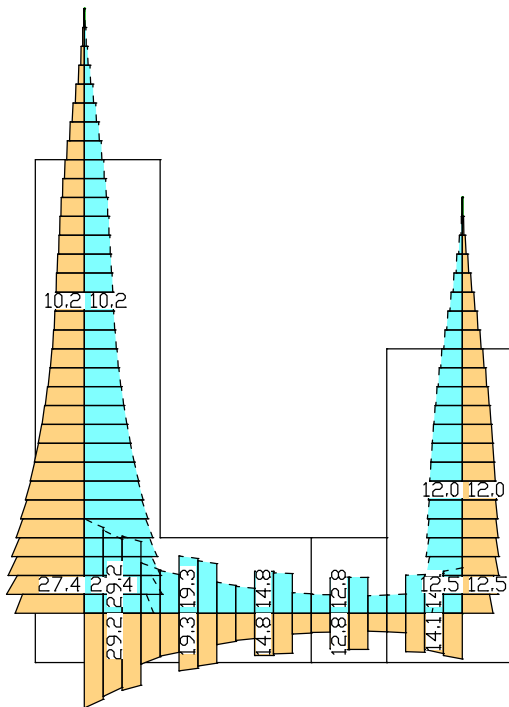


Support block

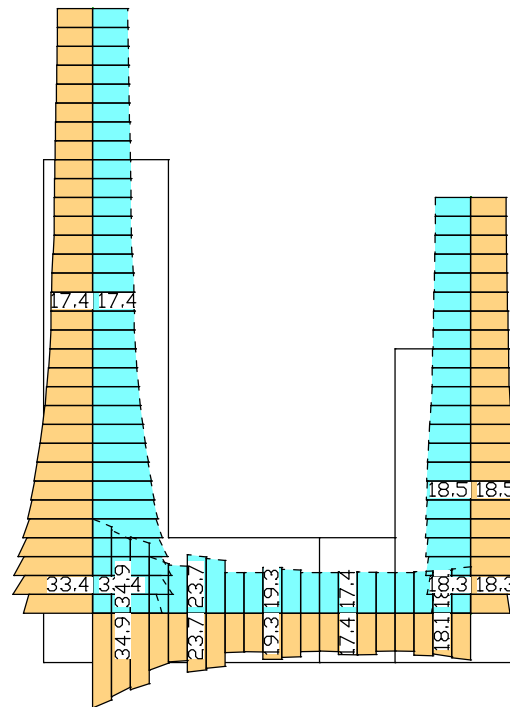


Short wing wall

Maximum required horizontal internal longitudinal reinforcement [cm<sup>2</sup>/m]

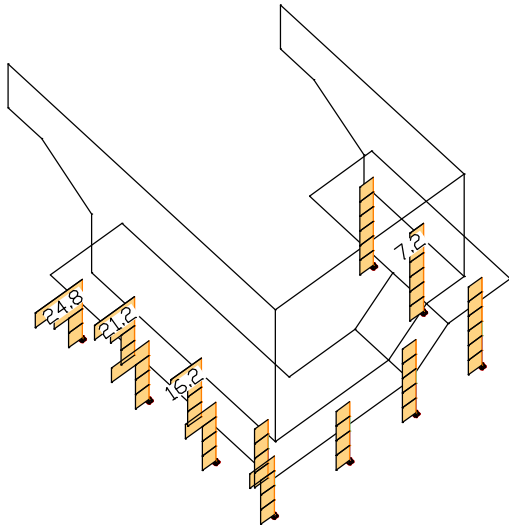


Reinforcement for the load-bearing capacity

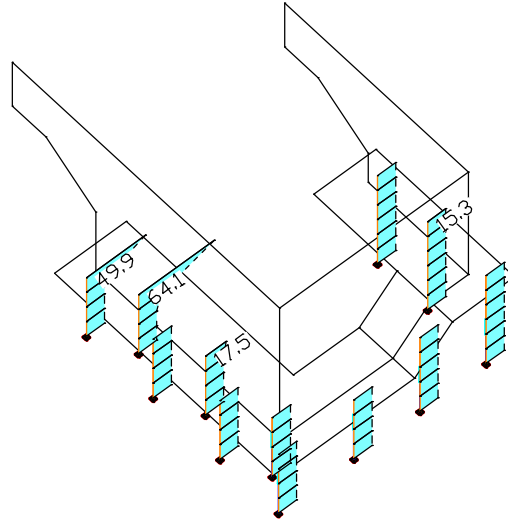


Maximum required reinforcement

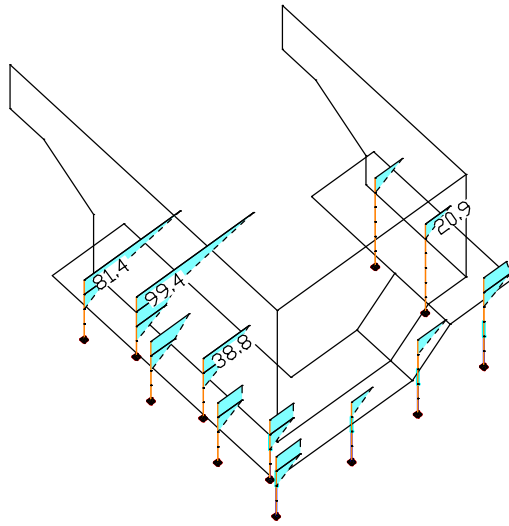
Horizontal longitudinal reinforcement along the upper edge [cm<sup>2</sup>/m]



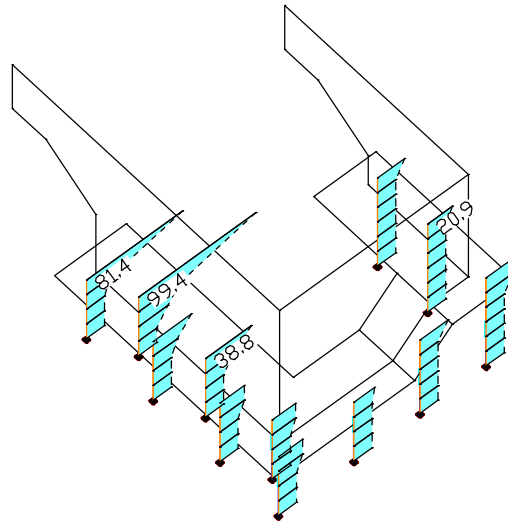
Lateral force reinforcement [cm<sup>2</sup>/m]



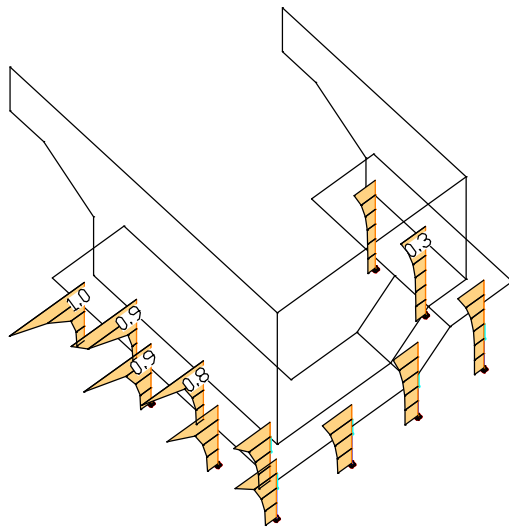
Load-bearing capacity reinforcement [cm<sup>2</sup>]



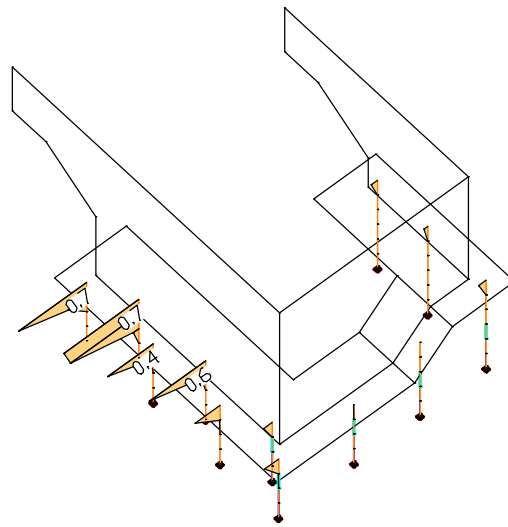
Crack check reinforcement [cm<sup>2</sup>]



Maximum required reinforcement [cm<sup>2</sup>]



Concrete compressive stresses  $\sigma_c / \sigma_{c,perm}$  [MN/m<sup>2</sup>]



Reinforcing steel stresses  $\sigma_s / \sigma_{c,perm}$  [MN/m<sup>2</sup>]

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# DIN 18800 Steel Checks

## Basics

The checks according to DIN 18800 can be applied to polygonal beam sections and are carried out according to the elastic-elastic method as per Section 7.5.2 with the Clauses 746 to 749 and the elastic-plastic method as per Section 7.5.3 with Clauses 753 and 755 (DIN 18800, Part 1, published November 2008).

Subsequent to the static analysis of the framework, the results of the load cases are combined into logical action groups and partial safety factors and combination coefficients are applied. The actions and other input data are defined in the database. Use the *Settings* in the *Analysis* menu to specify the type of check and the scope of the log.

The predefined steel sections, project-specific parameter and polygon sections or sections from the user database can be used as sections. Beams with the *Beam* section type are not checked as the section geometry is not known for them.

For area sections, the extremal internal forces for the defined action combinations are determined without performing a check and saved in the database for graphical representation.

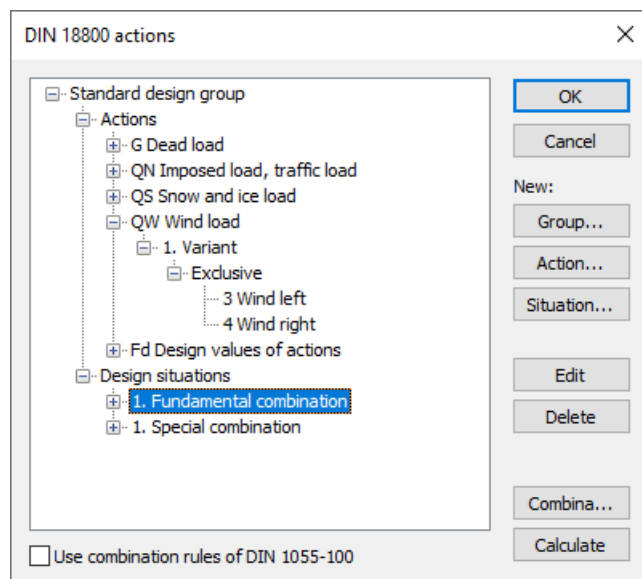
The lateral torsional buckling check is carried out on the fork supported equivalent beam with an independent program, which can be opened from the *Infograph* folder in the start menu.

## Input for Checks on the Entire Structure

### Actions and Design Situations

The load design values are calculated based on the internal forces of individual load cases and load case combinations. To do so, the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



#### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Design values of actions (Fd)

The assigned load cases should contain a design-relevant set of loads with combination coefficients and partial safety factors for actions and material such as for example a load group for the stability check stability check on the entire structure according to DIN 18800, Part 2, Clause 117. The selected load cases are combined exclusively.

**Group...**

Open the dialog for entering a new design group. According to e.g. standard DIN 1055-3, Chapter 6.3 (4), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

**Situation...**

Open the dialog for entering new design situations.

**Edit**

Open the Edit dialog for the selected action or situation.

**Delete**

Delete the selected action or situation.

**Combinations...**

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

**Calculate**

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to open the checking module. Each time you open the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

**Use combination rules of DIN 1055-100**

The fundamental combination (permanent and temporary combination) and the accidental combination are determined in accordance with DIN 1055-100, Section 9.4, Equations (14) and (15). For the coefficients Tables A.2 and A.3 of the standard are decisive. Alternatively, the simplified combination rules of DIN 18800, Chapter 7.2.2 can be used.

## Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.

**Label**


User-defined label for the action.

**Gamma.sup, Gamma.inf**


Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$



**Combination coefficients psi for:**

Input fields for selecting the combination coefficients for variable actions. The  button allows you to view and change the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .



**Load cases**

List of possible load cases or load case combinations. You can choose an item from the list by selecting it and clicking the  button or by using drag & drop.

**Multi-select**

Load cases and combinations can be added to the actions more than once.

**Exclusive variants**

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the  or  buttons.

**Inclusive load cases**

Selected load cases and combinations that can act simultaneously.

**Exclusive load cases**

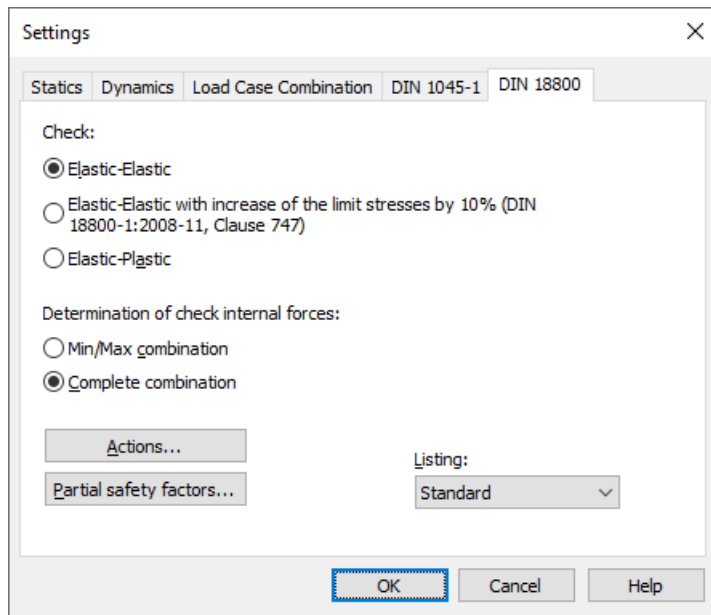
Selected load cases and combinations that exclude each other.

## Partial Safety Factors

The partial safety factor of steel is preset with the value specified by DIN 18800-1, Clause 720, and can be modified if necessary. In version 10.3 or higher, the partial safety factors for actions are determined by the definition of actions. These factors appear in the Partial Safety Factors dialog for compatibility reasons only and therefore cannot be modified.

## Analysis Settings

The *DIN 18800* dialog field can be accessed using the *Settings* option of the *Analysis* menu.



### Elastic-Elastic

A stress check according to the elastic-elastic method with the Clauses 746 to 749 DIN 18800 Part 1 is performed, including determination of the elastic utilization.

### Elastic-Elastic with increase of the limit stresses by 10% (DIN 18800-1:2008-11)

For the fundamental combination and the accidental combination, the limit stresses of Equations (33) to (35) in Clause 747 are increased by 10% in the elastic-elastic check. The Clauses 749 and 750 are not used. An increase of the limit stresses is not permissible in the special combination and is therefore ignored.

### Elastic-Plastic

The check is carried out according to the elastic-plastic method with the Clauses 753 and 755 of DIN 18800 Part 1. The full-plastic limit internal forces and a factor for the plastic utilization of the section are determined iteratively. The full-plastic limit internal forces are provided in the detailed log.

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *EN 1993-1-1 Steel Checks > Check internal forces*.

### Actions...

Open the dialog for describing actions.

### Partial safety factors...

Open the dialog for modifying partial safety factors.

### Listing

- *No*: No log is generated by the design program.
- *Standard*: During analysis a log with a tabular output of the calculated utilizations is created.
- *Detailed*: This log differs from the standard log by additionally offering information on the analyzed internal force combinations.
- *Standard>permissible*: Standard log of the check locations at which the permissible limits are exceeded.
- *Detailed>permissible*: Detailed log of the check locations at which the permissible limits are exceeded.

# Input for Checks on the Equivalent Beam Settings

**Listing**

Number of beam result points:

Detailed listing

Listing for all result points

---

**Page Frame**

Page no.:  
  Date, Time

Project:

Title:

Take user name from InfoCAD page setup

## Number of beam result points

The internal forces are calculated at equidistant points in the beam. For each result location the check is performed with the corresponding internal forces and the corresponding  $M_{K_{i,y,d}}$  (see DIN 18800, Part 2, Section 1.3, Note 5 and Explanations to DIN 18800, Page 180).

## Detailed listing

A detailed log with all necessary values is generated during calculation.

## Listing for all result points

In addition to the determinant result location, the results for all result locations are documented.

## Page no.

Page number used for the 1st side to be printed. The number is increased incrementally for each printing operation. You can enter '0' to suppress the numbering.

## Date, time

This information appears automatically in the standard page frame.

## Project, title

This text appears automatically in the standard page frame.

## Take user name from InfoCAD page setup

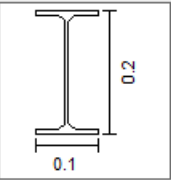
If the mark is set, the user name is automatically retrieved from the *lgraph.dat* file and included in the standard page frame. By removing the mark a different user name can be defined.

## Section & Material

**Cross-section**

ipe 200

- [-] Narrow I-sections
- [-] Average broad I-sections
  - [-] IPE
    - 80
    - 100
    - 120
    - 140
    - 160
    - 180
    - 200
    - 220
    - 240
    - 270



A = 28.5 cm<sup>2</sup>

I<sub>x</sub> = 7.02 cm<sup>4</sup>

I<sub>y</sub> = 1940 cm<sup>4</sup>

I<sub>z</sub> = 142 cm<sup>4</sup>

I<sub>w</sub> = 12990.0 cm<sup>6</sup>

**Material**

S235

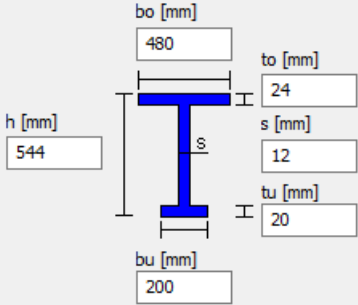
Gamma.M: 1.1

f<sub>yk</sub> [N/mm<sup>2</sup>]: 240

E-Modul [N/mm<sup>2</sup>]: 210000

G-Modul [N/mm<sup>2</sup>]: 81000

**User-defined section** ✕



bo [mm] 480

h [mm] 544

to [mm] 24

s [mm] 12

tu [mm] 20

bu [mm] 200

Cancel

OK

### Section

Selection of a steel section from the section library.

### Material

- S235, S275, S355, S450: Construction steel according to DIN 18800, Part 1, Tab. 1.
- Steel: User-defined steel.

### Gamma.M

The material-specific coefficient  $\gamma_M$  is used to obtain the design value for the strength  $f_{y,d}$ .

### f<sub>yk</sub>

Characteristic yield strength [N/mm<sup>2</sup>].

### E-Modulus

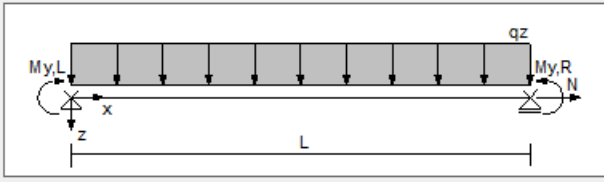
Modulus of elasticity [N/mm<sup>2</sup>].

### G-Modulus

Shear modulus [N/mm<sup>2</sup>].

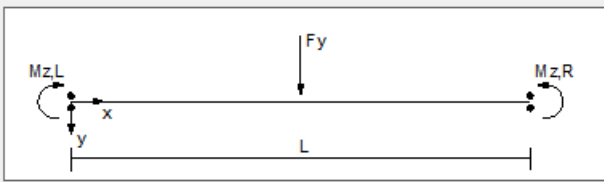
## Load & System

Load & System



Load in z-direction:   My,d left [kNm]:  qz,d [kN/m]:  My,d right [kNm]:

Length L [m]:  Nd [kN]:



Load in y-direction:   Mz,d left [kNm]:  Fy,d [kN]:  Mz,d right [kNm]:

### Load in z-direction

- *Distributed load:* A uniformly distributed load  $q_{z,d}$  [kN/m] is applied to the equivalent beam.
- *Concentrated load:* A point load  $F_{z,d}$  [kN/m] is applied to the equivalent beam. The load lever arm  $z_p$  is always set to zero for this selection.

### My,d left, My,d right

Fixed end moments  $M_{y,d}$  [kNm] including the partial safety factors  $\gamma_F$ . The fixed end moments can be determined, if required, using the second-order theory (DIN 18800, Part 2, Clause (303)).

### qz,d or Fz,d

Distributed load  $q_{z,d}$  [kN/m] or point load  $F_{z,d}$  [kN] including the partial safety factors  $\gamma_F$ .

### Nd

Normal force  $N_d$  [kN] including the partial safety factors  $\gamma_F$  (negative compression). Only compressive forces can be considered. The normal force can be determined, if required, using the second-order theory (Explanations to DIN 18800, Page 140).

### Length L

Equivalent beam length [m].

### Load in y-direction

- *Distributed load:* A uniformly distributed load  $q_{y,d}$  [kN/m] is applied to the equivalent beam.
- *Concentrated load:* A point load  $F_{y,d}$  [kN/m] is applied to the equivalent beam.

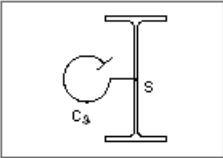
### Mz,d left, Mz,d right

Fixed end moments  $M_{z,d}$  [kNm] including the partial safety factors  $\gamma_F$ . The fixed end moments can be determined, if required, using the second-order theory (DIN 18800, Part 2, Clause (303)).

### qy,d or Fy,d

Distributed load  $q_{y,d}$  [kN/m] or point load  $F_{y,d}$  [kN] including the partial safety factors  $\gamma_F$ .

## Lateral Torsional Buckling Parameters

<b>Moment coefficients</b> Automatic <input type="button" value="v"/> Beta My: <input type="text" value="1.3"/> Beta Mz: <input type="text" value="1.4"/>	<b>Torsion spring</b> Null <input type="button" value="v"/>  C.g.theta [kNm/m]: <input type="text" value="0"/> <input type="button" value="Edit..."/>
<b>Load application point zp</b> At the top chord <input type="button" value="v"/> zp [m]: <input type="text" value="-0.1"/>	<b>Girder coefficient n</b> Hot rolled girder <input type="button" value="v"/> n: <input type="text" value="2.5"/>

### Moment coefficients

- *Automatic*: The moment coefficients  $\beta_y$  and  $\beta_z$  are automatically calculated based on the load data.
- *User defined*: The moment coefficients  $\beta_y$  and  $\beta_z$  are applied automatically from the input fields.

### Load application point zp

- *Top chord*: The load acts at the top chord.
- *Shear center*: The load acts at the shear center.
- *Center of gravity*: The load acts at the center of gravity.
- *Bottom chord*: The load acts at the bottom chord.
- *User-defined*: User-defined load application point.

### zp

The load lever arm  $z_p$  is calculated for the corresponding load application point.

### Torsion spring

- *User-defined*: The torsion spring  $C_g$  entered by the user is taken into account.
- *Compute*: The torsion spring  $C_g$  is calculated from the input data (see the elastic rotational bedding dialog). For U profiles the elastic rotational bedding from the profile deformation of the supported girder is not taken into account.
- *Null*: No torsion spring is taken into account.

### Edit...

Open the dialog to enter the torsion spring  $C_g$  [kNm/m].

### C.theta

The torsion spring  $C_g$  is taken into account using an ideal second-degree torsional area moment  $I_{T,ideal}$ :

$$I_{T,ideal} = I_T + C_g \cdot L^2 / (\pi^2 \cdot G)$$

Using  $I_{T,ideal}$ ,  $M_{Ki,y}$  is calculated for girders without elastic rotational bedding. This approximation can only be applied for small values of elastic rotational bedding  $C_g$  for instance for values of elastic rotational bedding for trapezoidal profiles, calculated according to DIN 18800-2 Clause (309); for larger values of  $C_g$  the lateral torsional buckling moments calculated with  $I_{T,ideal}$  can be unreliable (see Meister).

### Girder coefficient n

- *Hot rolled girder*: Girder coefficient  $n$  for rolled girders  $n = 2.5$ .
- *Welded girder*: Girder coefficient  $n$  for welded girders  $n = 2.0$ .
- *Honeycomb girder*: Girder coefficient  $n$  for honeycomb girders  $n = 1.5$ .
- *Notched girder*: Girder coefficient  $n$  for notched girders  $n = 2.0$ .
- *User defined*: Girder coefficient  $n$  is entered by the user.

## Torsion Spring

Torsion spring as per DIN 18800-2, Element 309

Supporting component  
 Trapezoidal sheet steel

Effect of continuity: Exterior span  
 Material: S235

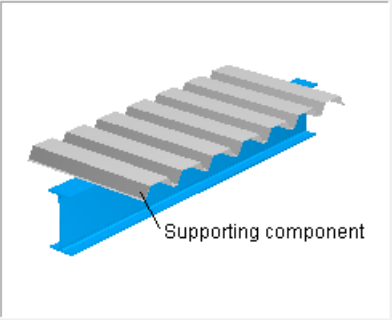
Span a [m]: 2.5  
 fyk [N/mm<sup>2</sup>]: 240

Ia [cm<sup>4</sup>/m]: 250  
 E-Modul [N/mm<sup>2</sup>]: 210000

Purlin distance e [m]:

Connection stiffness  
 C.theta.A,k [kNm/m]: 5.2  
 Table 7...

OK Cancel



### Supporting component

- *Trapezoidal sheet steel*
- *Purlins*: The connection strength is not taken into account.

### Effect of continuity

- *Exterior span*:  $k = 2$  for one-span and two-span girders (see DIN 18800, Part 2, Clause (309)).
- *Inside span*:  $k = 4$  for continuous girders with three or more spans (see DIN 18800, Part 2, Clause (309)).

### Span a

Span [m] of the supporting component.

### Ia resp. Iy

Second-degree area moment [cm<sup>4</sup>/m] of the supporting component.

### Purlin distance e

Distance between the purlins perpendicular to the direction of load [m].

### Material

Material of the supporting component.

- *S235, S275, S355, S450*: Construction steel according to DIN 18800, Part 1, Tab. 1.
- *Steel*: User-defined steel.

### fyk

Characteristic yield strength of the supporting component [N/mm<sup>2</sup>].

### E-Modulus

E-modulus [N/mm<sup>2</sup>] of the supporting component.

### C theta,A,k

Connection strength [kNm/m] according to DIN 18800, Part 2, Table 7.

## Checks in the Ultimate Limit States

Depending on the user's selection the checks in the ultimate limit states are carried out with the combinations of DIN 1055-100, Chapter 9.4 or with the simplified combination rules of DIN 18800, Chapter 7.2.2.

### Design Combinations According to DIN 1055-100

In DIN 1055-100, Chapter 9.4 the following combinations are defined:

- Combination for permanent and temporary design situations (fundamental combination)

$$E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_P \cdot P_k \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \right\} \quad (14)$$

- Combination for accidental design situations (accidental combination)

$$E \left\{ \sum_{j \geq 1} \gamma_{GA,j} \cdot G_{k,j} \oplus \gamma_{PA} \cdot P_k \oplus A_d \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (15)$$

Additionally, for the results of a stability check according to the second-order theory the

- Special combination

$$E \left\{ F_{d,1} \right\}; \dots; E \left\{ F_{d,n} \right\}$$

is available. In this combination the action ( $F_d$ ) are superposed exclusively.

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations applies. The partial safety factors and combination coefficients are defined in Tables A.2 and A.3 of the standard.

### Design Combinations According to DIN 18800-1

According to DIN 18800-1, Clause 710, the following combination rules result for the fundamental combination. The maximum value of each rule is decisive for the design:

$$E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \sum_{i \geq 1} \gamma_{Q,i} \cdot \psi_i \cdot Q_{k,i} \right\}$$

$$E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_{Q,1} \cdot Q_{k,1} \right\}; \dots; E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_{Q,n} \cdot Q_{k,n} \right\}$$

For the accidental combination according to Clause 714, an additional accidental action  $F_{A,k}$  must be taken into account. The combination rule is then defined as follows:

$$E \left\{ \sum_{j \geq 1} \gamma_{GA,j} \cdot G_{k,j} \oplus \gamma_{FA} \cdot F_{A,k} \oplus \sum_{i \geq 1} \gamma_{QA,i} \cdot \psi_i \cdot Q_{k,i} \right\}$$

The partial safety factors and combination coefficients result from the Equations (12) to (17) of DIN 18800. The *Special combination* defined above is available for results of a stability check according to the second-order theory.



## Design Values According to the Second-Order Theory

The calculation according to the second-order theory according to Clause 728 is carried out as part of the internal forces calculation. For this you need to create appropriate load cases from decisive loads (see also function *Combinations* in the action dialog).

As the partial safety factors are already to be taken into account for the internal forces calculation, the respective actions must be multiplied with their corresponding partial safety factor  $\gamma_G$  or  $\gamma_Q$  as well as with  $\gamma_M = 1.10$ . This procedure is permissible according to DIN 18800, Part 2, Clause 117. As a simplification  $\gamma_G = \gamma_Q$  is often set to 1.35 for the calculation.

In this case, the following factor is to be collectively applied to the load ordinates:  $1.35 \cdot 1.1 \cong 1.50$ .

In the check the load cases calculated according to the second-order theory are assigned to an action  $F_d$  and superposed in the *Special combination* as an exclusive selection. The results are checked against the characteristic values  $\sigma_{RK}$  and  $\tau_{RK}$  without taking the partial safety factors into account.

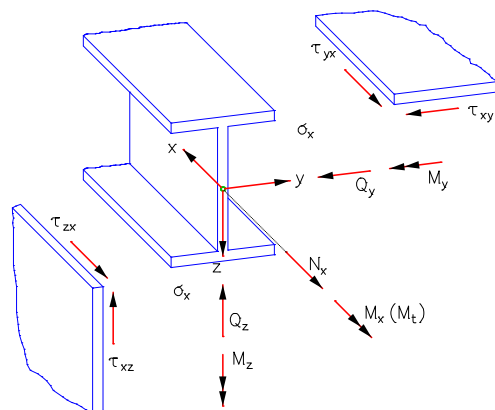
## Characteristic Values

For sections made out of construction steel S235, S275, S355 or S450, the yield strength is taken from DIN 18800-1 or EN 10025-2 for product thicknesses  $t \leq 40$  mm. If other sizes are used, the general material type *Stahl* is to be chosen as it allows for freely defining all properties.

## Stress Determination

### Coordinate Systems

The following illustration shows the definition of the coordinate systems for internal forces, loads and stresses using the example of a beam with an I profile. The section edge is defined as a closed polygon.



The longitudinal force  $N_x$  runs through the centroid, the lateral forces  $Q_y$  and  $Q_z$  run through the shear center of the section which, in this case, coincides with the centroid. The vectors of the moments  $M_x$ ,  $M_y$ ,  $M_z$  and stresses  $\sigma_x$ ,  $\tau_{xy}$ ,  $\tau_{xz}$  run parallel to the coordinate axes.

## Longitudinal Stresses

The necessary section properties for determination of the longitudinal stresses from longitudinal force and bending are determined elementarily with the segmentation method from the coordinates of the polygon.

More specifically, these are:

$y_s, z_s$	Centroid coordinates.
$A$	Area of the section.
$I_y, I_z$	Moments of inertia in relation to the coordinate axes.
$I_{yz}$	Deviation moment of inertia.
$I_1, I_2$	Moments of inertia in relation to the main axes.
$\varphi$	Twisting angle of the main axes.
$W_y, W_z$	Moments of resistance for the calculation of the extremal longitudinal stresses from $M_y, M_z$ .

For the analysis of a section, the stresses are determined at the automatically defined calculation points. The longitudinal stress  $\sigma$  for a point  $(y,z)$  of the section results from

$$\sigma(y,z) = N_x \frac{1}{A} + M_y \frac{\Delta z I_z - \Delta y I_{yz}}{I_y I_z - I_{yz}^2} + M_z \frac{\Delta y I_y - \Delta z I_{yz}}{I_y I_z - I_{yz}^2}$$

with  $\Delta y = y - y_s$  and  $\Delta z = z - z_s$ .

## Shear Stresses

### Lateral force

For load as a result of lateral force, the differential equation of the so-called shear warpage  $\omega_\tau$

$$\frac{\partial^2 \omega_\tau}{\partial y^2} + \frac{\partial^2 \omega_\tau}{\partial z^2} = -\frac{Q_z z}{G I_y} \quad \text{or} \quad -\frac{Q_y y}{G I_z}$$

is solved numerically with the help of the boundary element method. From this the following shear characteristics are determined:

$\tau_{Qy,1}$	Shear stress for the lateral force $Q_y = 1$ .
$\tau_{Qz,1}$	Shear stress for the lateral force $Q_z = 1$ .
$W_{qy}, W_{qz}$	Area values for calculation of the extremal shear stresses from $Q_y$ and $Q_z$ with $\tau_{Qy} = Q_y / W_{qy}$ and $\tau_{Qz} = Q_z / W_{qz}$ .

If selected in the Section dialog, the shear stresses across the section width will be averaged.

### Torsion

For load as a result of St. Venant torsion, the differential equation of the unit warpage  $\omega$

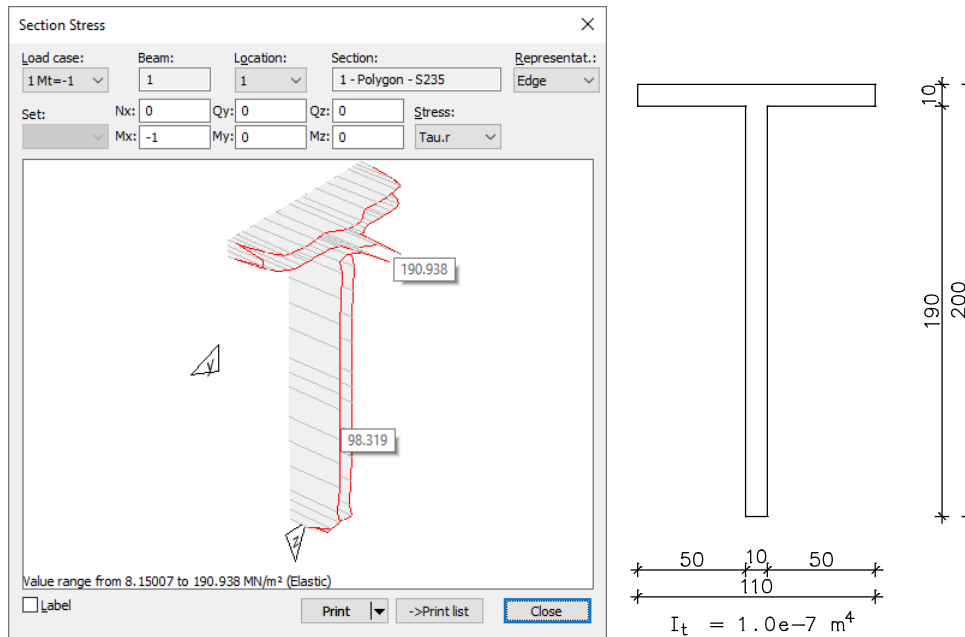
$$\frac{\partial^2 \omega}{\partial y^2} + \frac{\partial^2 \omega}{\partial z^2} = 0$$

is decisive. Its solution leads to the following characteristics:

$I_t$	Torsion moment of inertia.
$\tau_{Mx,1}$	Shear stress for the torsional moment $M_x = 1$ .
$W_t$	Moment of resistance for the calculation of the extremal shear stress from $M_x$ .

As a result, in the fillet area of rolled sections or in blunt corners of welded profiles larger torsional shear stresses occur than those according to the theory of thin-walled profiles. As an option in the section dialog, specifying the maximum sheet thickness (open profiles) or the torsion resistance moment (box profiles) allows you to limit the stresses to the maximum value according to the theory of thin-walled profiles.

The following figure shows the stress curve for a load of  $M_x = 1$  (Section Stress function):



The numerically determined value for the web in the example corresponds to the theoretical shear stress for open thin-walled sections  $\tau = M_x \cdot t / I_t = 100 \text{ MN/m}^2$ . The peak value of  $190 \text{ MN/m}^2$  results from a singularity in the blunt corners.

All unit shear stresses are calculated at discrete points on the section edges. The discretization is automatically carried out by the program. The unit shear stresses are edge stresses which run along the section edge. Their sign depends on the direction of the edge. The resultant shear stress  $\tau$  from lateral force and torsion is then determined by the following equation:

$$\tau = Q_y \cdot \tau_{Qy,1} + Q_z \cdot \tau_{Qz,1} + M_x \cdot \tau_{Mx,1}$$

## Check 'Elastic-Elastic'

### Limit stresses

The limit stresses for the fundamental combination and the accidental combination are determined according to Clause 746:

Limit normal stress

$$\sigma_{R,d} = f_{y,d} = f_{y,k} / \gamma_M$$

Limit shear stress

$$\tau_{R,d} = f_{y,d} / \sqrt{3}$$

In the *Special combination* the characteristic values replace the design values:

$$\sigma_{R,k} = f_{y,k}$$

$$\tau_{R,k} = f_{y,k} / \sqrt{3}$$

### Design

In the checks the conditions according to Clause 747 are checked; in this case  $\sigma_y$ ,  $\sigma_z$  and  $\tau_{yz}$  are assumed to be zero for the beam models analyzed:

for the normal stress  $\sigma_x$

$$|\sigma / \sigma_{R,d}| \leq 1$$

for the shear stresses  $\tau_{xy}$  and  $\tau_{xz}$

$$|\tau / \tau_{R,d}| \leq 1$$

for the simultaneous action of several stresses

$$\sigma_v / \sigma_{R,d} \leq 1$$

with the comparison stress  $\sigma_v$  according to Clause 748

$$\sigma_v = \sqrt{\sigma_x^2 + 3 \cdot \tau_{xy}^2 + 3 \cdot \tau_{xz}^2}$$

The last condition is fulfilled if  $\sigma_x$  and  $\tau$  are the only actions and either:

$$|\sigma_x / \sigma_{R,d}| \leq 0.5 \text{ or } |\tau / \tau_{R,d}| \leq 0.5$$

According to Clause 749, the comparison stress  $\sigma_v$  may exceed the limit stress  $\sigma_{R,d}$  by 10% if for beams with normal force and bending the following holds true **simultaneously**:

$$|\sigma_{Nx} + \sigma_{My}| \leq 0.8 \sigma_{R,d} \text{ and } |\sigma_{Nx} + \sigma_{Mz}| \leq 0.8 \sigma_{R,d}$$

This rule is only used if the global increase of the limit stresses by 10 % according to Clause 747 is not selected in the analysis settings. The options for beams with I-Section in Clause 750 are not used generally.

In the case of the *Special combination*, the limit stresses  $\sigma_{R,d}$  and  $\tau_{R,d}$  are replaced with the characteristic values  $\sigma_{R,k}$  and  $\tau_{R,k}$ .

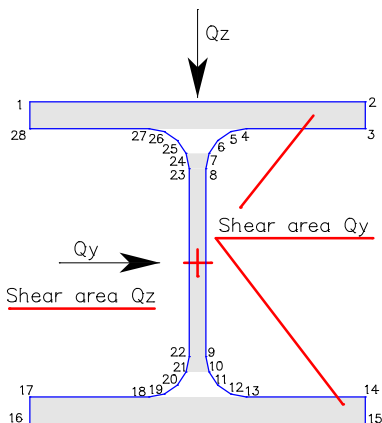
## Check 'Elastic-Plastic'

### Determination of the Plastic Limit Internal Forces

The *Elastic-Plastic* check is carried out by comparison of the absorbable internal forces with the load internal forces of a section. Here it is to be checked that the limit internal forces in the plastic state are not exceeded (DIN 18800, Section 7.5.3). The calculation of the absorbable internal forces is performed by integration of the stresses on the section polygon and equilibrium iteration taking into account the conditions according to Clause 755.

- Linear-elastic ideal plastic stress-strain relationship.
- Section remains planar.
- Huber-von Mises yield criterion.
- The full-plastic limit moments are limited to 1.25 times the elastic limit moment.

The interaction of all 6 internal forces  $N_x, Q_y, Q_z, M_x, M_y, M_z$  is to be taken into account. To ensure this, simplified assumptions with respect to the shear stress distribution from lateral force and torsion are to be made as a closed solution of the interaction problem is not available. Because in steel construction thin-walled sections are common, subareas can be defined as a good approximation and then used to dissipate the shear stresses:



The steel sections in the section library have predefined subareas which are listed in the section table of the log. The following assumptions apply:

- The lateral forces create constant shear stresses  $\tau_{xy} = Q_y/A_{qy}$  or  $\tau_{xz} = Q_z/A_{qz}$  in their corresponding subareas.
- The torsion moment  $M_x$  results in local, constant shear stress states in the subareas and the remaining area with the maximum edge shear stress  $\tau_t$  acting on the area.

For polygon sections and sections from the user database, subareas cannot be taken into account. In this case the following assumptions apply for the entire section:

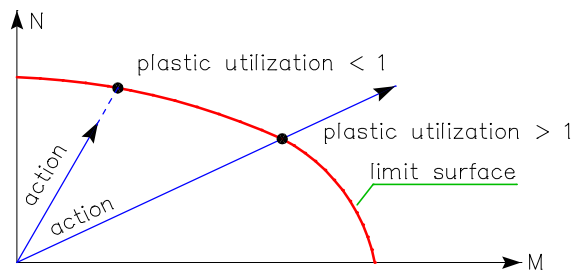
- The lateral forces create constant shear stresses  $\tau_{xy} = Q_y/W_{qy}$  or  $\tau_{xz} = Q_z/W_{qz}$ .
- The torsion moment  $M_x$  creates the constant shear stress state  $\tau_t = M_x/W_t$ .

The design values of the strengths are each reduced locally by the shear stress described above. To allow calculation of the absorbable internal forces, the remaining available strength is used for each area. The calculation of the shear characteristics is carried out for determining the section properties as described above.

The calculation approach described delivers limit internal forces that are always on the safe side; however, in individual cases it could be that not all reserves are used.

To help estimate the degree of plastic utilization of a section, the load vector is intersected with the limit area of the 6-dimensional internal force space. This can be used to calculate a factor for plastic utilization, which is  $\leq 1$  if the limit area is not exceeded and  $> 1$  if it is exceeded.

The following illustration shows the iteration of the limit area using the example of an N-M interaction.



## Check

According to Clause 753, the check evaluates if the load internal forces exceed the limit internal forces taking into account the interaction of all internal forces in the full plastic state.

The plastic internal forces are calculated according to Clause 755 with the design value of the yield strength  $f_{y,d}$ . In the *Special Combination*  $f_{y,k}$  replaces  $f_{y,d}$ . The limit bending moments in the plastic state are limited to 1.25 times the elastic limit bending moment.

## Check Against Lateral Torsional Buckling

According to DIN 18800 Part 2, the check for potentially unstable frameworks can be carried out in two steps. First the lateral buckling check is performed on the entire system according to the second-order theory. Subsequently, the lateral torsional buckling check is carried out on the equivalent beam based on the previously determined fixed end forces. The field moments from any beam loads that might be present can be calculated according to the first-order theory.

The calculation program carries out the check for lateral torsional buckling on the equivalent beam according to the 'elastic-plastic' method. The following requirements must be met:

- The calculation of the internal forces is carried out according to the theory of elasticity.
- The equivalent beam is assumed to be straight.
- Section and material are constant along the length of the beam.
- The dimensions of the section are small compared with the other dimensions.
- The mathematical curvature is linearized.
- The influence of shear deformations on the internal forces are not considered.
- The load is slowly increased to its final value and does not undergo any deviation in direction as a result of the system deformation.
- Actions in the z direction act at a distance  $z_p$  from the shear center.
- The actions in the y direction act at the shear center.
- The equivalent beam rests on a fork support at both ends.
- Sheet buckling is not included.
- The effects from torsion are not accounted for.

### Calculation of the plastic internal forces

The determination of the plastic internal forces is made via integration of the stresses at the section polygon, as described above.

According to DIN 18800, Part 2 limiting the full plastic limit moments is not necessary.

If the utilization of the lateral force is greater than 0.33 in the z direction or 0.25 in the y direction, then, according to DIN 18800, Part 2, Section 3.4.2.2, Clause (315), the influence of the lateral forces on the load-bearing capacity of the section must be accounted for. This occurs through a reduction of the yield stress in the web or chords.

### Calculation of $N_{Ki,z}$

According to DIN 18800 Part 2 Section 3.4.3,  $N_{Ki}$  is the normal force under the smallest critical load for branching perpendicular to the z axis or the torsional buckling load.

$$N_{Ki,z,d} = \frac{E \cdot I_z \cdot \pi^2}{L^2}$$

It should be noted that only buckling perpendicular to the z axis is analyzed, even if buckling perpendicular to the y axis has a smaller buckling load (see Explanations to DIN 18800, Page 178).

### Calculation of the ideal lateral torsional buckling moment $M_{Ki,y}$

DIN 18800 provides the following formula for calculating  $M_{Ki,y}$ :

$$M_{Ki,y} = \zeta \cdot N_{Ki,z} \cdot \left( \sqrt{c^2 + 0.25 \cdot z_p^2} + 0.5 \cdot z_p \right)$$

This formula is only applicable for double-symmetric I profiles. Additionally, for a general moment curve, the moment coefficient  $\zeta$  is quite difficult to determine. For single-symmetric profiles, it is no longer possible to determine  $M_{Ki,y}$  using the abovementioned approach.

In order to be able to calculate any single-symmetric profile under general load, this program module contains a method for the direct determination of  $M_{Ki,y}$ .

The solution is derived by varying the elastic potential  $\Pi$  while neglecting the terms from the calculated bending  $w$ . For the unknown functions  $v$  and  $\varphi$  a multiple-term power series is used that fulfills the boundary conditions.

$$\Pi = \frac{1}{2} \int_0^L \left\{ \left[ G I_x + (r_y - 2z_m) M_y(x) \eta_{cr} \right] \left( \frac{\partial \varphi}{\partial x} \right)^2 + E I_w \left( \frac{\partial^2 \varphi}{\partial x^2} \right)^2 + E I_z \left( \frac{\partial^2 v}{\partial x^2} \right)^2 + 2 M_y(x) \eta_{cr} \left( \frac{\partial^2 v}{\partial x^2} \right) \varphi + q \eta_{cr} z_p \varphi^2 \right\} dx$$

The geometrically nonlinear share comprising actions in the z direction is included via a uniformly distributed load  $q_z$ . This is chosen such that the same maximum moment of span is created as through the given action.

The resulting eigenvalue problem provides the smallest positive critical load factor  $\eta_{K_{i,y}}$  and thus the required lateral torsional buckling moment  $M_{K_{i,y}}$ . In compliance with DIN 18800, Part 2, Section 1.3, Note 5, this is calculated for each check location.

$$M_{K_{i,y,d}} = \frac{\eta_{K_{i,y}} \cdot M_{y,d}}{\gamma_M}$$

The method described offers the advantage that the user does not need to specify the moment coefficient  $\zeta$ .

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The detailed log also lists the decisive combination internal forces of all design situations for each result location.

### Stresses

$\sigma_x, \sigma_y$	Extremal normal stresses from bending and normal force.
$\sigma_{xy}$	Extremal shear stresses from torsional moment and shear force.
$\tau_{xy}, \tau_{xz}$	Extremal shear stresses from lateral force and torsion.
$\sigma_v$	Maximum comparison stress.
$\sigma_1, \sigma_2$	Extremal principal stresses.

All stresses are given in [MN/m<sup>2</sup>].

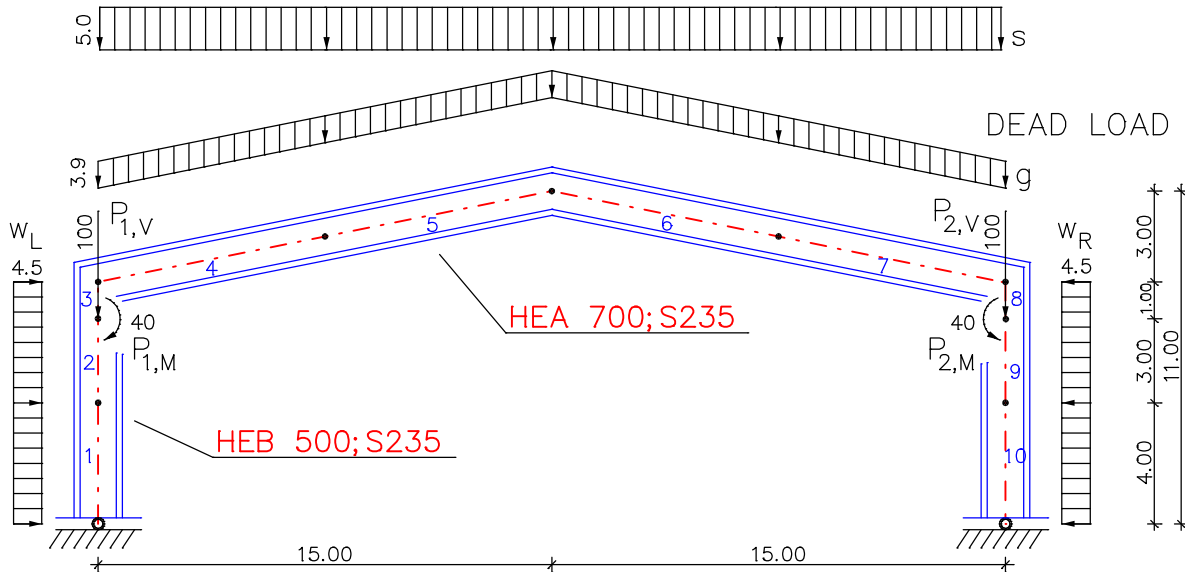
### Utilizations

$\sigma/\sigma_R, \tau/\tau_R, \sigma_v/\sigma_R$	Maximum elastic utilizations based on the limit stresses according to Clause 746. Stresses of the fundamental and accidental combination are divided by the design values $\sigma_{Rd}$ and $\tau_{Rd}$ , stresses of the special combination by the characteristic values $\sigma_{Rk}$ and $\tau_{Rk}$ .
plastic utiliz.	Maximum plastic utilization, defined as the ratio between the load internal forces and the full-plastic limit internal forces.

# Examples

## Steel Checks on the Entire System

In the hall frame illustrated below a framework analysis and a stability check are carried out according to the second-order theory. The checks are performed according to the *Elastic-Elastic* and *Elastic-Plastic* methods with combination rules according to DIN 18800.



Static system and dimensions

### Material properties

No.	Material type	EModule [MN/m <sup>2</sup> ]	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]	k <sub>bx</sub> -a	k <sub>bx</sub> -e [MN/m <sup>3</sup> ]	k <sub>bz</sub> -a	k <sub>bz</sub> -e	b <sub>x</sub>	b <sub>z</sub> [m]
1	S235	210000	1.200e-05	78.500	0	0	0	0	1.00	1.00
2	S235	210000	1.200e-05	78.500	0	0	0	0	1.00	1.00

### Section properties

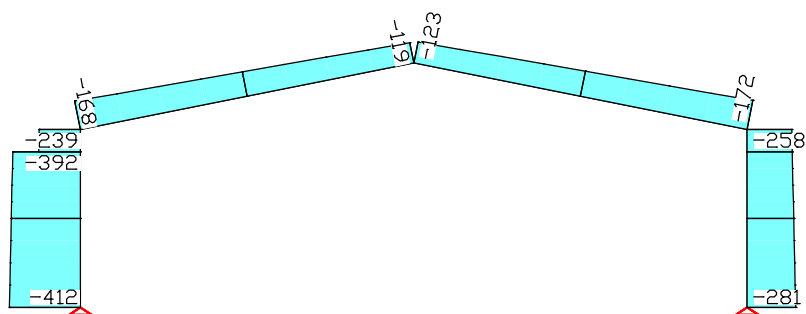
No. 1	HEB 500	A = 2.390e-02 [m <sup>2</sup> ], I <sub>y</sub> = 1.072e-03 [m <sup>4</sup> ]
No. 2	HEA 700	A = 2.600e-02 [m <sup>2</sup> ], I <sub>y</sub> = 2.153e-03 [m <sup>4</sup> ]

### Sum of installed loads and support reactions

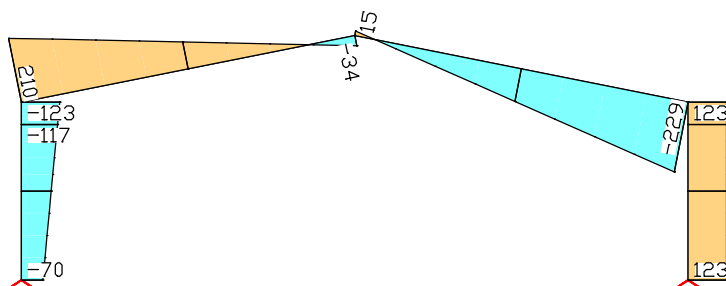
LC	Label	F <sub>x</sub> [kN]	F <sub>z</sub> [kN]
1	Dead load	0.000	211.778
	Support reactions	-0.000	211.778
2	Snow	0.000	150.000
	Support reactions	-0.000	150.000
3	Wind left	36.000	0.000
	Support reactions	36.000	-0.000
4	Wind right	-36.000	0.000
	Support reactions	-36.000	0.000
5	Crane track left	0.000	100.000
	Support reactions	-0.000	100.000
6	Crane track right	0.000	100.000
	Support reactions	-0.000	100.000
7	Crane track center:(LC5+LC6)/2	0.000	100.000
	Support reactions	-0.000	100.000
10	(LC1+2+3+5)*1.5; 2.order	54.000	692.667
	Support reactions	54.000	692.667

For the stability check according to the second-order theory, a decisive load case no. 10 made up of *dead load*, *snow*, *wind from left* and *crane way position left* with a uniform safety factor of 1.5 was constructed. The following internal force curves result:

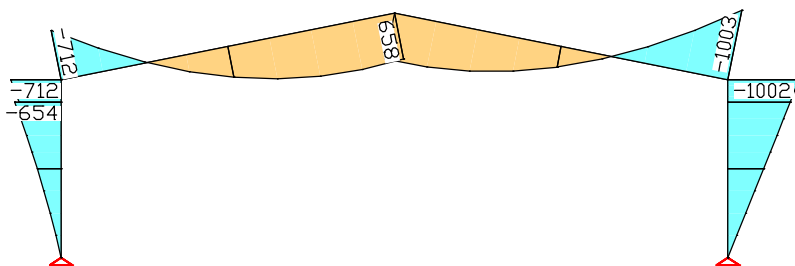




Second-order theory, internal forces  $N_x$



Second-order theory, internal forces  $Q_z$



Second-order theory, internal forces  $M_y$

To perform the checks the actions must be assigned as shown in the following log. The checking program makes the necessary internal force combinations and determines the permissible stresses. Additionally, the full-plastic internal forces are determined and the level of plastic utilization is specified.

**Design per DIN 18800-1 (2008)**

The design according to the method Elastic-Elastic is carried out considering chapter 7.5.2 with the Clauses 746 to 749.

The design according to the method Elastic-Plastic is carried out considering chapter 7.5.3 with the Clauses 753 and 755.

The action combinations are calculated according to chapter 7.2.2 with the Equations (12)-(15) and (17). For the design, the appropriate extremum of Equations (13) and (14) is decisive.

**Designing occurs for all possible combinations of actions.**

**Terms in Printout**

- A : Sectional area.
- $I_y, I_z, I_{yz}$  : Moments of inertia.
- $A_{qz}$  : Shear subarea of the polygon section for  $Q_z$ .
- $W_{qz}$  : Resistance momentum of the ext. shear stress of  $Q_z$ .
- Pl. Util. : Plastic utilization, defined as the ratio of the load internal forces and the plastic limit state internal forces.
- y, z : Location in the sectional polygon [m].

**Stress Calculation**

The normal stresses and shear stresses are calculated for homogeneous polygon sections from normal force, bending moment and lateral force. The shear characteristics are calculated acc. to the Boundary-Element-Method. The calculation points for all stresses are edge points of the cross-section.

For the following cross-sections the shear stresses from lateral force are averaged over the section width.

Cross-section	Cross-section	Cross-section
1 HEB 500	2 HEA 700	-

**Calculation of the Plastic Limit State Internal Forces**

The plastic limit state internal forces are calculated by integration on the polygon section. With it the interaction of all internal forces  $N_x$ ,  $Q_z$  und  $M_y$  is considered.

For simplification the following assumptions are made:  
 - The lateral force produces with its corresponding subarea the constant shear stress  $\tau = Q_z/A_{qz}$ .  
 - If no subareas are defined, a constant shear stress  $\tau = Q_z/W_{qz}$  is assumed for the whole section area.

The shear characteristics are calculated with use of the Boundary-Element-Method at the section edge.

The Huber-v. Mises-Condition as per Eq. (36) is used as yield criterion. The plastic limit state moments are limited to the 1.25-fold value of the elastic limit state moments (DIN 18800, Clause 755).

**Partial Safety Factors for Steel**

	gamma.M
Fundamental combination	1.10
Accidental combination	1.10
Special combination	1.00

**Characteristic Material Properties**

Yield stresses of steel as per Table 1 for  $t \leq 40$  mm [MN/m<sup>2</sup>].  
 Limiting stresses according to Clause 746.

		Fundam. C.		Accid. C.		Special C.	
Material	fyk	Sigma.Rd	Tau.Rd	Sigma.Rd	Tau.Rd	Sigma.Rk	Tau.Rk
S235	240	218.18	125.97	218.18	125.97	240.00	138.56

**Cross-Section Properties**

Cross-section	Material	A[mm <sup>2</sup> ]	Aqz[mm <sup>2</sup> ]	Iy[m <sup>4</sup> ]	Iz[m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
1 HEB 500	S235	23900	5997	1.0720e-03	1.2620e-04	0.0000e+00
2 HEA 700	S235	26000	8781	2.1530e-03	1.2180e-04	0.0000e+00

**DIN 18800 actions**

**Standard design group**

G - Dead load

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

-----  
 1 Dead load

**QN - Imposed load, traffic load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Standard

Standard  
 Psi.0 / Psi.1 / Psi.2 = 0.9 / 0 / 0

Load cases 1. Variant, exclusive

-----  
 5 Crane track left  
 6 Crane track right  
 7 Crane track center:(LC5+LC6)/2

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Standard

Standard  
 Psi.0 / Psi.1 / Psi.2 = 0.9 / 0 / 0

Load cases 1. Variant, inclusive

-----  
 2 Snow

**QW - Wind load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Standard

Standard  
 Psi.0 / Psi.1 / Psi.2 = 0.9 / 0 / 0

Load cases 1. Variant, exclusive

-----  
 3 Wind left  
 4 Wind right

**Fd - Design values of actions**

Load cases  
 -----  
 10 (LC1+2+3+5)\*1.5;Theory 2.order

**1. Fundamental combination**

Final state  
 G - Dead load  
 QN - Imposed load, traffic load  
 QS - Snow and ice load  
 QW - Wind load

**1. Special combination**

Final state  
 Fd - Design values of actions

**Check for Beams**

The results represent the extrema of all combinations.  
 Permissible exceedings according to Clause 749 are marked with '\*'.  
 S.x, S.v Extremal normal stress, max. comparison stress acc. to Eq. (36).  
 T.xy, T.xz Max. absolute values of shear stresses from lateral force.  
 S.R, T.R Limit normal stress as per Eq. (31), limit shear stress as per Eq. (32).

**Elastic Stresses [MN/m²] and Plastic Utilization**

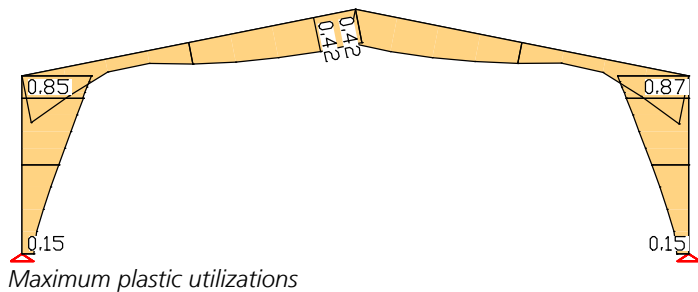
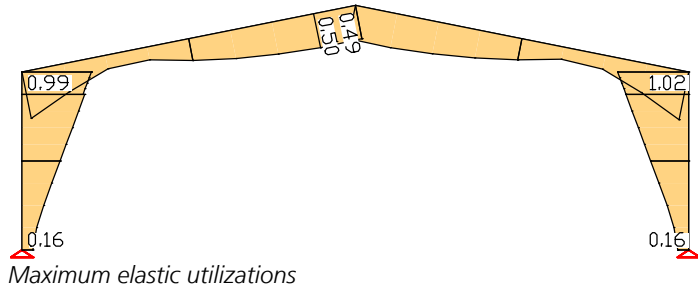
Beam	Se.	Lo.	S.x ext	T.xy  max	T.xz  max	S.v max	S.x /S.R	T.xy /T.R	T.xz /T.R	S.v /S.R	Pl. Util.
1	1	1	-16.06	2.83	17.19	33.84	0.07	0.02	0.14	0.16	0.15
		2	-41.85	2.83	17.19	44.66	0.19	0.02	0.14	0.20	0.18
		3	-67.64	2.83	17.19	67.79	0.31	0.02	0.14	0.31	0.26
		4	-93.43	2.83	17.19	93.54	0.43	0.02	0.14	0.43	0.35
		5	-119.22	2.83	17.19	119.31	0.55	0.02	0.14	0.55	0.45
2	1	1	-119.22	2.83	17.19	119.31	0.55	0.02	0.14	0.55	0.45
		2	-138.57	2.83	17.19	138.64	0.64	0.02	0.14	0.64	0.52
		3	-157.91	2.83	17.19	157.97	0.72	0.02	0.14	0.72	0.60
		4	-177.25	2.83	17.19	177.31	0.81	0.02	0.14	0.81	0.67
		5	-196.59	2.83	17.19	196.65	0.90	0.02	0.14	0.90	0.75
3	1	1	-191.10	2.83	17.19	191.15	0.88	0.02	0.14	0.88	0.74
		2	-197.54	2.83	17.19	197.60	0.91	0.02	0.14	0.91	0.77
		3	-203.99	2.83	17.19	204.04	0.93	0.02	0.14	0.94	0.79
		4	-210.44	2.83	17.19	210.49	0.96	0.02	0.14	0.96	0.82
		5	-216.89	3.14	19.12	216.93	0.99	0.02	0.14	0.99	0.85
4	2	1	-148.29	3.75	23.70	148.41	0.68	0.03	0.19	0.68	0.58
		2	-89.19	3.23	20.46	89.33	0.41	0.03	0.16	0.41	0.35
		3	-38.49	2.72	17.21	41.14	0.18	0.02	0.14	0.19	0.16
		4	-44.56	2.21	13.97	44.68	0.19	0.02	0.11	0.19	0.16
		5	-76.58	1.71	10.82	76.62	0.32	0.01	0.09	0.32	0.27
5	2	1	-76.71	1.71	10.82	76.75	0.32	0.01	0.09	0.32	0.27
		2	-99.04	1.22	7.73	99.06	0.41	0.01	0.06	0.41	0.35
		3	-112.04	0.73	4.64	112.04	0.47	0.01	0.04	0.47	0.39
		4	-108.13	0.25	1.55	108.13	0.50	0.00	0.01	0.50	0.42
		5	-106.18	0.60	3.79	106.18	0.49	0.00	0.03	0.49	0.42
6	2	1	-106.18	0.53	3.37	106.18	0.49	0.00	0.03	0.49	0.42
		2	-108.13	0.28	1.78	108.13	0.50	0.00	0.01	0.50	0.42
		3	-101.25	0.82	5.21	101.26	0.46	0.01	0.04	0.46	0.39
		4	-87.70	1.37	8.64	87.71	0.40	0.01	0.06	0.40	0.34
		5	-67.63	1.91	12.07	67.67	0.31	0.01	0.09	0.31	0.26
7	2	1	-67.63	1.91	12.09	67.67	0.31	0.01	0.09	0.31	0.26
		2	-39.16	2.45	15.52	39.28	0.18	0.02	0.11	0.18	0.15
		3	-44.02	2.99	18.94	46.62	0.18	0.02	0.14	0.19	0.17
		4	-100.97	3.23	20.46	101.13	0.42	0.03	0.16	0.42	0.36
		5	-167.27	3.75	23.70	167.40	0.70	0.03	0.19	0.70	0.60
8	1	1	-244.50	3.14	19.12	244.55	1.02>1	0.02	0.14	1.02>1	0.87
		2	-237.38	3.14	19.12	237.43	0.99	0.02	0.14	0.99	0.84
		3	-230.26	3.14	19.12	230.32	0.96	0.02	0.14	0.96	0.82
		4	-223.14	3.14	19.12	223.20	0.93	0.02	0.14	0.93	0.79
		5	-216.02	3.14	19.12	216.08	0.90	0.02	0.14	0.90	0.76
9	1	1	-196.59	3.14	19.13	196.65	0.90	0.02	0.14	0.90	0.77
		2	-177.25	3.14	19.13	177.31	0.81	0.02	0.14	0.81	0.68
		3	-157.91	3.14	19.13	157.97	0.72	0.02	0.14	0.72	0.61
		4	-138.57	3.14	19.12	138.64	0.64	0.02	0.14	0.64	0.53
		5	-119.22	3.14	19.12	119.31	0.55	0.02	0.14	0.55	0.45
10	1	1	-119.22	3.15	19.14	119.31	0.55	0.02	0.14	0.55	0.45
		2	-93.43	3.15	19.13	93.54	0.43	0.02	0.14	0.43	0.35
		3	-67.64	3.14	19.13	67.79	0.31	0.02	0.14	0.31	0.26
		4	-41.85	3.14	19.12	44.66	0.19	0.02	0.14	0.20	0.18
		5	-16.06	3.14	19.12	33.84	0.07	0.02	0.14	0.16	0.15

**Max. Cross-Sections Utilization**

Sigma.x,v Extremal normal stresses, max. comparison stresses acc. to Eq. (36).  
 Tau.xy,xz Max. absolute values of shear stresses from lateral force.  
 Sigma.R Limit normal stress according to Eq. (31).  
 Tau.R Limit shear stress according to Eq. (32).  
 x Distance from the beam startpoint [m].  
 FC,SC Fundamental combination, Special combination

Cross-section	Material	Result	Beam	Loc.	x [m]	Comb.	Util.
1 HEB 500	S235	Sigma.x/Sigma.R	8	1	0.00	SC.1	1.02 > 1
		Tau.xy/Tau.R	10	1	0.00	SC.1	0.02
		Tau.xz/Tau.R	10	1	0.00	SC.1	0.14
		Sigma.v/Sigma.R	8	1	0.00	SC.1	1.02 > 1
		Pl. Utilization	8	1	0.00	SC.1	0.87
2 HEA 700	S235	Sigma.x/Sigma.R	7	5	7.65	SC.1	0.70
		Tau.xy/Tau.R	4	1	0.00	FC.1	0.03
		Tau.xz/Tau.R	4	1	0.00	FC.1	0.19
		Sigma.v/Sigma.R	7	5	7.65	SC.1	0.70
		Pl. Utilization	7	5	7.65	SC.1	0.60

The stress check reveals that the elastic limit stresses are exceeded in beam 8. Only after reconciliation with the plastic limit internal forces does a permissible utilization of the section result.



Additionally, an excerpt of the detailed listing for beam 8 is shown below:

**Check for Beams**

The results represent the extrema of all combinations.

Sigma.x,v Extremal normal stresses, max. comparison stresses acc. to Eq. (36).  
 Tau.xy,xz Max. absolute values of shear stresses from lateral force.  
 Sigma.R Limit normal stress according to Eq. (31).  
 Tau.R Limit shear stress according to Eq. (32).  
 x Distance from the beam startpoint [m].

Beam 8: x = 0.00 m (Beam Length 1.00 m)  
 Cross-section 1: HEB 500, S235; fyk=240 MN/m<sup>2</sup>; A=23900 mm<sup>2</sup>

**1. Fundamental combination (FC.1): G+QN+QS+QW**

No set of internal forces in this combination was relevant.

**1. Special combination (SC.1): Fd**

Relevant values from 1 set of internal forces

Set	Nx [kN]	Qz [kN]	My [kNm]
1	-258.49	123.50	-1002.03

Load case combination for the relevant set of internal forces

Set	Combination
1	L10

**Check of the Normal Stresses [MN/m<sup>2</sup>]**

Min. Sigma.x	: -244.50	Sigma.x/Sigma.R	: 1.02 > 1
Sigma.x(Nx)	: -10.82	Se.point y [m]	: 0.150
Sigma.x(My)	: -233.68	z [m]	: 0.250
		Situation	: SC.1,1
Max. Sigma.x	: 222.87	Sigma/Sigma.R	: 0.93
Sigma.x(Nx)	: -10.82	Se.point y [m]	: -0.150
Sigma.x(My)	: 233.68	z [m]	: -0.250
		Situation	: SC.1,1

Check of the Shear Stresses [MN/m<sup>2</sup>]

```
Ext. Tau.xy      :      3.14      Tau.xy/Tau.R    :      0.02
                                     Se.point y [m]   :      3.144
                                     z [m]           :     -0.222
                                     Situation        :      SC.1,1

Ext. Tau.xz      :     19.12      Tau.xz/Tau.R    :      0.14
                                     Se.point y [m]   :     19.122
                                     z [m]           :     -0.000
                                     Situation        :      SC.1,1
```

Check of the Comparison Stress [MN/m<sup>2</sup>]

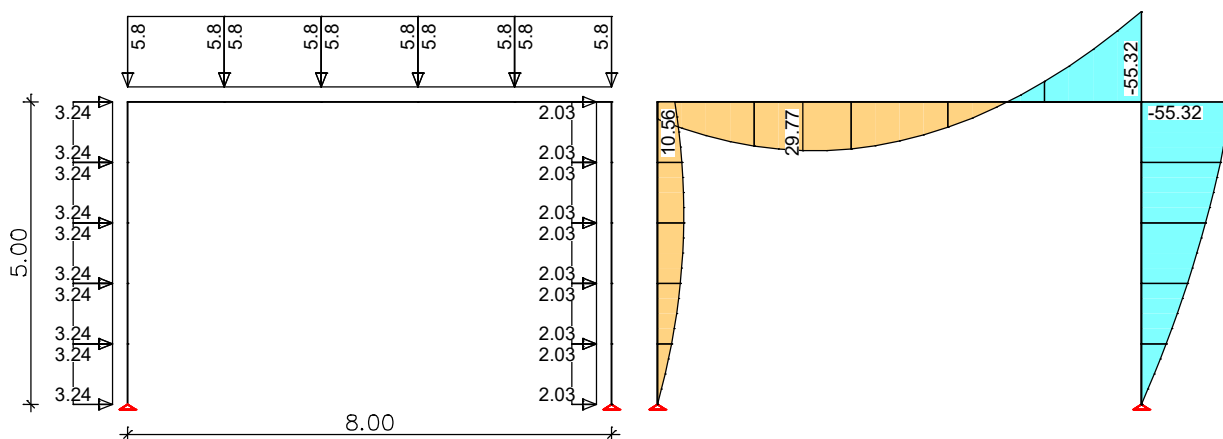
```
Max. Sigma.v    :     244.55      Sigma.v/Sigma.R :     1.02 > 1
Sigma.x (Nx)    :     -10.82      Se.point y [m]   :     0.045
Sigma.x (My)    :    -233.68      z [m]           :     0.250
Tau.xy (Qz)    :       2.91      Situation        :      SC.1,1
Tau.xz (Qz)    :       0.00
```

Check of the Plastic Limit State Forces and of the Plastic Utilization

```
Pl. Nx [kN]     :    -296.62      Pl. Utilization  :     0.87
Pl. Qz [kN]     :     141.72      Situation        :      SC.1,1
Pl. My [kNm]    :   -1149.85
```

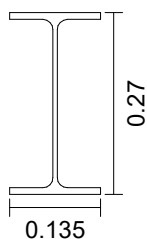
## Lateral Torsional Buckling Check of a Two-Hinged Frame

The input values correspond to example 2b on page 79 from the essay of W. Martin.



### Log Printout:

Cross section: IPE 270



h = 270.0 mm  
 b = 135.0 mm  
 s = 6.6 mm  
 t = 10.2 mm  
 r = 15.0 mm

A = 45.9 cm<sup>2</sup>      I<sub>y</sub> = 5790.0 cm<sup>4</sup>      I<sub>z</sub> = 420.0 cm<sup>4</sup>  
 I<sub>T</sub> = 16.0 cm<sup>4</sup>      I<sub>ω</sub> = 70580 cm<sup>6</sup>  
 i<sub>z</sub> = 3.02 cm      i<sub>p</sub> = 11.63 cm      i<sub>M</sub> = 11.63 cm

Material: S235

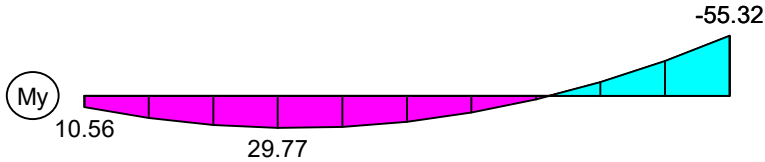
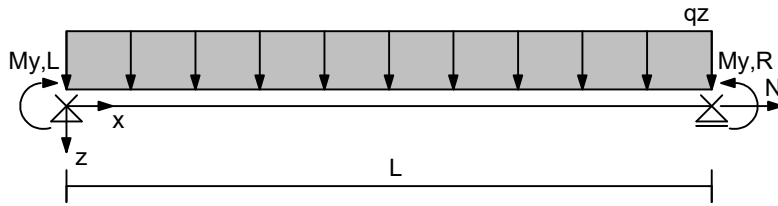
f<sub>y,k</sub> = 240 N/mm<sup>2</sup>  
 E-Modulus = 210000 N/mm<sup>2</sup>  
 G-Modulus = 81000 N/mm<sup>2</sup>  
 γ<sub>M</sub> = 1.10

Load in z-direction

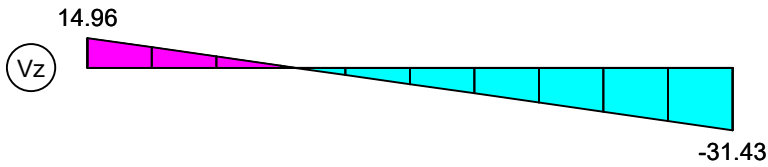
L = 8.00 m      N<sub>d</sub> = 0.00 kN      q<sub>z,d</sub> = 5.80 kN/m  
 M<sub>y,d,left</sub> = 10.56 kNm      M<sub>y,d,right</sub> = -55.32 kNm

The load application point is in the shear center  $\Rightarrow z_p = 0.00$  cm

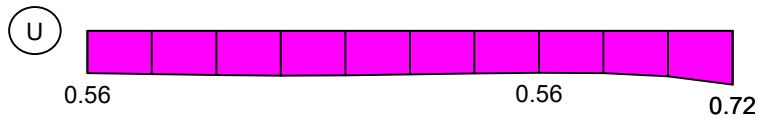
The ends of the beam are forked supported.



Max  $M_{y,d} = 29.77$  kNm, Min  $M_{y,d} = -55.32$  kNm



Max  $V_{z,d} = 14.96$  kN, Min  $V_{z,d} = -31.43$  kN



Maximum utilization = 0.72

**Buckling load  $N_{K_{i,z,d}}$**

$$N_{K_{i,z,d}} = \pi^2 \cdot (E \cdot I_z / \gamma_M) / L^2$$

with  $I_z = 420.0$  cm<sup>4</sup>       $E = 210000$  N/mm<sup>2</sup>  
 $L = 8.00$  m                       $\gamma_M = 1.1$

$N_{K_{i,z,d}} = 123.65$  kN

**Critical load factor  $\eta_{K_{i,y}}$  for  $M_{K_{i,y,d}}$**

$\eta_{K_{i,y}} = 1.97$

**Decisive check at location  $x = 8.00$  m**

**Internal forces**

$N_d = 0.00$  kN  
 $V_{z,d} = -31.43$  kN  
 $M_{y,d} = -55.32$  kNm

**Plastic internal forces**

$N_{pl,k} = \pm 1101.60$  kN               $N_{pl,d} = \pm 1001.45$  kN  
 $V_{pl,z,k} = \pm 228.26$  kN               $V_{pl,z,d} = \pm 207.51$  kN  
 $M_{pl,y,k} = \pm 116.16$  kNm               $M_{pl,y,d} = \pm 105.60$  kNm

**Reduction factor  $\kappa_z$** 

$$\bar{\lambda}_{k,z} = \sqrt{N_{pl,d} / N_{Ki,z,d}}$$

with  $N_{pl,d} = 1001.45 \text{ kN}$      $N_{Ki,z,d} = 123.65 \text{ kN}$

$$\bar{\lambda}_{k,z} = 2.85 > 0.2$$

$$\kappa_z = 1 / (k + \sqrt{k^2 - \bar{\lambda}_{k,z}^2}) \quad k = 0.5 \cdot (1 + \alpha \cdot (\bar{\lambda}_{k,z} - 0.2) + \bar{\lambda}_{k,z}^2)$$

Buckling stress line b  $\Rightarrow \alpha = 0.34$

$$k = 5.00$$

$$\kappa_z = 0.11$$

**Lateral torsional buckling moment  $M_{Ki,y,d}$** 

$$M_{Ki,y,d} = \frac{\eta_{Ki,y} \cdot M_{y,d}}{\gamma_M}$$

with  $\eta_{Ki,y} = 1.97$      $M_{y,d} = -55.32 \text{ kNm}$      $\gamma_M = 1.1$

$$M_{Ki,y,d} = -98.85 \text{ kNm}$$

**Reduction factor  $\kappa_M$** 

$$\bar{\lambda}_M = \sqrt{M_{pl,y,d} / M_{Ki,y,d}}$$

with  $M_{pl,y,d} = -105.60 \text{ kNm}$      $M_{Ki,y,d} = -98.85 \text{ kNm}$

$$\bar{\lambda}_M = 1.03 > 0.4$$

$$\kappa_M = (1 / (1 + \bar{\lambda}_M^{2n}))^{1/n}$$

Moment ratio  $\psi = -0.19 \leq 0.5$

Factor  $K_n = 1.00$

Coefficient for hot rolled girder  $n = 2.50 \Rightarrow n_{red} = n \cdot K_n = 2.50$

$$\kappa_M = 0.73$$

**Coefficient  $k_y$** 

$$k_y = 1 - N_d / (\kappa_z \cdot N_{pl,d}) \cdot a_y \leq 1 \quad a_y = 0.15 \cdot \bar{\lambda}_{k,z} \cdot \beta_{M,y} - 0.15 \leq 0.9$$

with  $N_d = 0.00 \text{ kN}$      $N_{pl,d} = 1001.45 \text{ kN}$   
 $\kappa_z = 0.11$      $\bar{\lambda}_{k,z} = 2.85$

Moment ratio  $\psi_y = -0.19 \Rightarrow \beta_{M,y} = 1.59$

$$a_y = 0.53$$

$$k_y = 1.00$$

**Check**

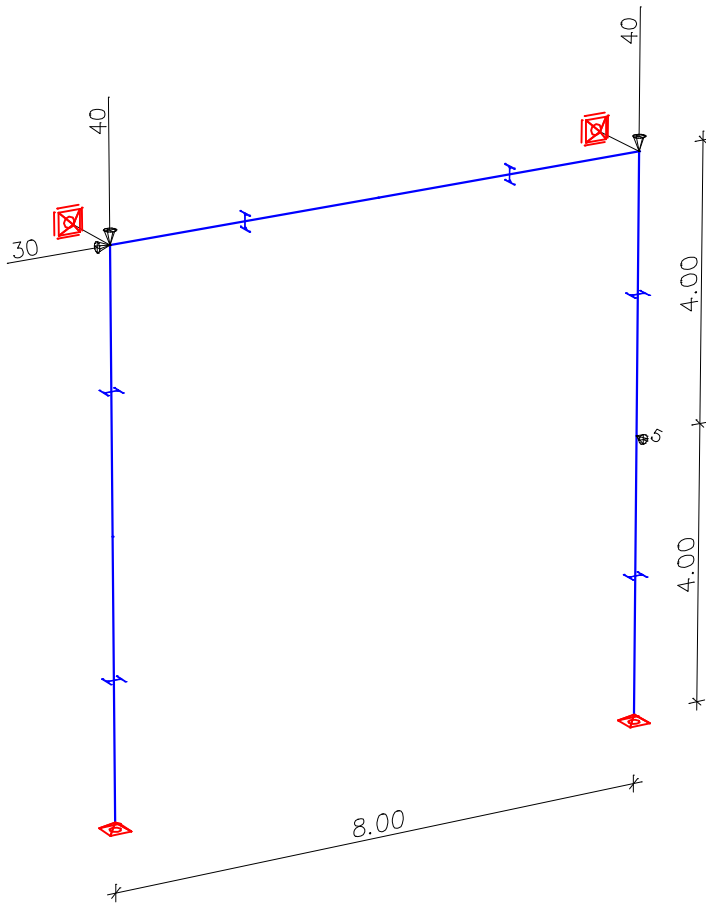
$$\frac{N_d}{\kappa_z \cdot N_{pl,d}} + \frac{M_{y,d}}{\kappa_M \cdot M_{pl,y,d}} \cdot k_y \leq 1$$

$N_d = 0.00 \text{ kN}$      $M_{y,d} = -55.32 \text{ kNm}$   
 $N_{pl,d} = 1001.45 \text{ kN}$      $M_{pl,y,d} = -105.60 \text{ kNm}$   
 $\kappa_z = 0.11$      $\kappa_M = 0.73$      $k_y = 1.00$

$$0.00 + 0.72 = 0.72 \leq 1.0 \quad \text{The check is OK !}$$

## Lateral Torsional Buckling Check of a Frame Column With Two-Axis Bending and Normal Force

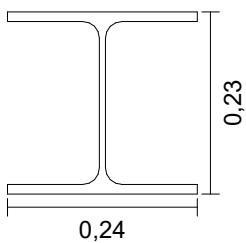
The input values correspond to example 7c (check with corresponding internal forces) on page 262 from the book of J. Meister.



The right frame column is checked.  
 $\gamma_M = 1.00$  (simplified assumption for the example)

### Log Printout:

**Cross section: HEA 240**



$h = 230,0 \text{ mm}$   
 $b = 240,0 \text{ mm}$   
 $s = 7,5 \text{ mm}$   
 $t = 12,0 \text{ mm}$   
 $r = 21,0 \text{ mm}$

$A = 76.8 \text{ cm}^2$   
 $I_T = 41.7 \text{ cm}^4$   
 $i_z = 6.01 \text{ cm}$

$I_y = 7760.0 \text{ cm}^4$   
 $I_{\omega} = 328500 \text{ cm}^6$   
 $i_p = 11.71 \text{ cm}$

$I_z = 2770.0 \text{ cm}^4$   
 $i_M = 11.71 \text{ cm}$

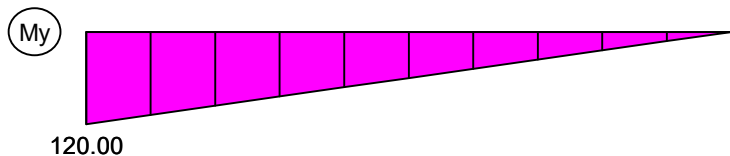
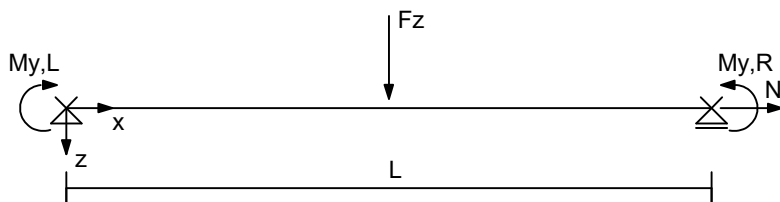
**Material: S235**

$f_{y,k} = 240 \text{ N/mm}^2$   
 $E\text{-Modul} = 210000 \text{ N/mm}^2$   
 $G\text{-Modul} = 81000 \text{ N/mm}^2$   
 $\gamma_M = 1.00$

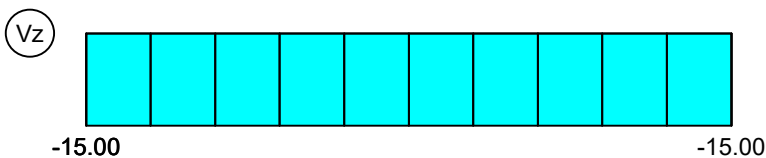


**Load in z-direction**

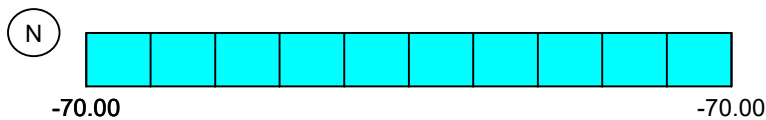
$L = 8.00 \text{ m}$                        $N_d = -70.00 \text{ kN}$                        $F_{z,d} = 0.00 \text{ kN}$   
 $M_{y,d,\text{left}} = 120.00 \text{ kNm}$                        $M_{y,d,\text{right}} = 0.00 \text{ kNm}$   
 The load application point is in the shear center  $\Rightarrow z_p = 0.00 \text{ cm}$   
 The ends of the beam are forked supported.



Max  $M_{y,d} = 120.00 \text{ kNm}$



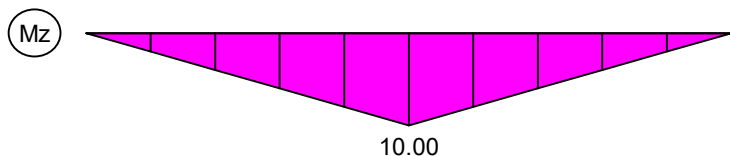
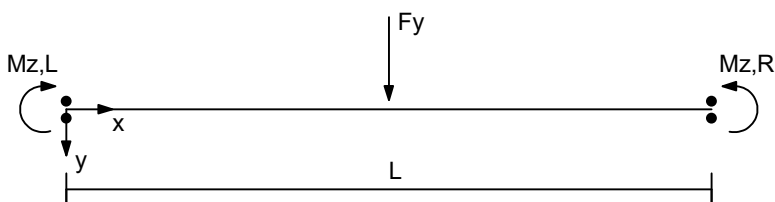
Max  $V_{z,d} = -15.00 \text{ kN}$



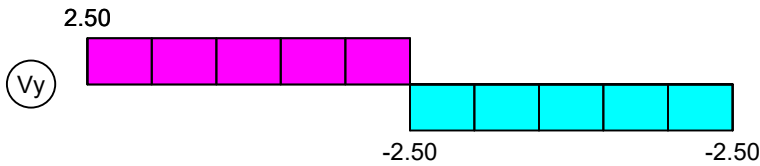
Min  $N_{,d} = -70.00 \text{ kN}$

**Load in y-direction**

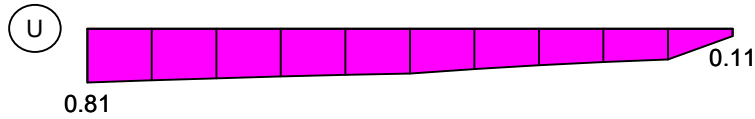
$L = 8.00 \text{ m}$                        $F_{y,d} = 5.00 \text{ kN}$   
 $M_{z,d,\text{left}} = 0.00 \text{ kNm}$ ;                       $M_{z,d,\text{right}} = 0.00 \text{ kNm}$



Max  $M_{z,d} = 10.00 \text{ kNm}$



Max  $V_{y,d} = 2.50$  kN, Min  $V_{y,d} = -2.50$  kN



Maximum utilization = 0.81

**Buckling load  $N_{K_{i,z,d}}$**

$$N_{K_{i,z,d}} = \pi^2 \cdot (E \cdot I_z / \gamma_M) / L^2$$

with  $I_z = 2770.0$  cm<sup>4</sup>       $E = 210000$  N/mm<sup>2</sup>  
 $L = 8.00$  m       $\gamma_M = 1.0$

$N_{K_{i,z,d}} = 897.05$  kN

**Critical load factor  $\eta_{K_{i,y}}$  for  $M_{K_{i,y,d}}$**

$\eta_{K_{i,y}} = 3.03$

-----  
**Decisive check at location  $x = 0.00$  m**  
 -----

**Internal forces**

$N_d = -70.00$  kN  
 $V_{z,d} = -15.00$  kN  
 $M_{y,d} = 120.00$  kNm

$V_{y,d} = 2.50$  kN  
 $M_{z,d} = 0.00$  kNm

**Plastic internal forces**

$N_{pl,k} = \pm 1843.20$  kN       $N_{pl,d} = \pm 1843.20$  kN  
 $V_{pl,z,k} = \pm 214.08$  kN       $V_{pl,z,d} = \pm 214.08$  kN  
 $M_{pl,y,k} = \pm 178.71$  kNm       $M_{pl,y,d} = \pm 178.71$  kNm

$V_{pl,y,k} = \pm 798.13$  kN       $V_{pl,y,d} = \pm 798.13$  kN  
 $M_{pl,z,k} = \pm 84.41$  kNm       $M_{pl,z,d} = \pm 84.41$  kNm

**Reduction factor  $\kappa_z$**

$$\bar{\lambda}_{k,z} = \sqrt{N_{pl,d} / N_{K_{i,z,d}}}$$

with  $N_{pl,d} = 1843.20$  kN       $N_{K_{i,z,d}} = 897.05$  kN

$$\bar{\lambda}_{k,z} = 1.43 > 0.2$$

$$\kappa_z = 1 / (k + \sqrt{k^2 - \bar{\lambda}_{k,z}^2}) \quad k = 0.5 \cdot (1 + \alpha \cdot (\bar{\lambda}_{k,z} - 0.2) + \bar{\lambda}_{k,z}^2)$$

Buckling stress line c  $\Rightarrow \alpha = 0.49$

$k = 1.83$

$\kappa_z = 0.34$

**Lateral torsional buckling moment  $M_{k_i,y,d}$** 

$$M_{k_i,y,d} = \frac{\eta_{k_i,y} \cdot M_{y,d}}{\gamma_M}$$

with  $\eta_{k_i,y} = 3.03$                        $M_{y,d} = 120.00 \text{ kNm}$                        $\gamma_M = 1.0$

$$M_{k_i,y,d} = 363.60 \text{ kNm}$$

**Reduction factor  $\kappa_M$** 

$$\bar{\lambda}_M = \sqrt{M_{pl,y,d} / M_{k_i,y,d}}$$

with  $M_{pl,y,d} = 178.71 \text{ kNm}$      $M_{k_i,y,d} = 363.60 \text{ kNm}$

$$\bar{\lambda}_M = 0.70 > 0.4$$

$$\kappa_M = (1 / (1 + \bar{\lambda}_M^{-2n}))^{1/n}$$

Coefficient for hot rolled girder  $n = 2.50$

$$\kappa_M = 0.94$$

**Coefficient  $k_y$** 

$$k_y = 1 - N_d / (\kappa_z \cdot N_{pl,d}) \cdot a_y \leq 1 \quad a_y = 0.15 \cdot \bar{\lambda}_{k,z} \cdot \beta_{M,y} - 0.15 \leq 0.9$$

with  $N_d = 70.00 \text{ kN}$                        $N_{pl,d} = 1843.20 \text{ kN}$   
 $\kappa_z = 0.34$                        $\bar{\lambda}_{k,z} = 1.43$

Moment ratio  $\psi_{y} = 0.00 \Rightarrow \beta_{M,y} = 1.80$

$$a_y = 0.24$$

$$k_y = 0.97$$

**Coefficient  $k_z$** 

$$k_z = 1 - N_d / (\kappa_z \cdot N_{pl,d}) \cdot a_z \leq 1.5 \quad a_z = \bar{\lambda}_{k,z} \cdot (2 \cdot \beta_{M,z} - 4) + (\alpha_{pl,z} - 1) \leq 0.8$$

with  $N_d = 70.00 \text{ kN}$                        $N_{pl,d} = 1843.20 \text{ kN}$                        $\alpha_{pl,z} = 1.52$   
 $\kappa_z = 0.34$                        $\bar{\lambda}_{k,z} = 1.43$

Moment ratio  $\psi_{z} = 1.00 \Rightarrow \beta_{M,z} = 1.40$

$$a_z = -1.20$$

$$k_z = 1.13$$

**Check**

$$\frac{N_d}{\kappa_z \cdot N_{pl,d}} + \frac{M_{y,d}}{\kappa_M \cdot M_{ply,d}} \cdot k_y + \frac{M_{z,d}}{M_{plz,d}} \cdot k_z \leq 1$$

$$N_d = 70.00 \text{ kN}$$

$$N_{pl,d} = 1843.20 \text{ kN}$$

$$\kappa_z = 0.34$$

$$M_{y,d} = 120.00 \text{ kNm}$$

$$M_{ply,d} = 178.71 \text{ kNm}$$

$$\kappa_M = 0.94 \quad k_y = 0.97$$

$$M_{z,d} = 0.00 \text{ kNm}$$

$$M_{plz,d} = 84.41 \text{ kNm}$$

$$k_z = 1.13$$

$$0.11 + 0.70 + 0.00 = 0.81 \leq 1.0 \quad \text{The check is OK!}$$

**Check at location  $x = 4.00 \text{ m}$** **Internal forces**

$$N_d = -70.00 \text{ kN}$$

$$V_{z,d} = -15.00 \text{ kN}$$

$$M_{y,d} = 60.00 \text{ kNm}$$

$$V_{y,d} = -2.50 \text{ kN}$$

$$M_{z,d} = 10.00 \text{ kNm}$$

**Plastic internal forces**

$$N_{pl,k} = \pm 1843.20 \text{ kN}$$

$$N_{pl,d} = \pm 1843.20 \text{ kN}$$

$$V_{pl,z,k} = \pm 214.08 \text{ kN}$$

$$V_{pl,z,d} = \pm 214.08 \text{ kN}$$

$$M_{pl,y,k} = \pm 178.71 \text{ kNm}$$

$$M_{pl,y,d} = \pm 178.71 \text{ kNm}$$

$$V_{pl,y,k} = \pm 798.13 \text{ kN}$$

$$V_{pl,y,d} = \pm 798.13 \text{ kN}$$

$$M_{pl,z,k} = \pm 84.41 \text{ kNm} \quad M_{pl,z,d} = \pm 84.41 \text{ kNm}$$

**Reduction factor  $\kappa_z$** 

$$\bar{\lambda}_{k,z} = \sqrt{N_{pl,d} / N_{ki,z,d}}$$

$$\text{with } N_{pl,d} = 1843.20 \text{ kN} \quad N_{ki,z,d} = 897.05 \text{ kN}$$

$$\bar{\lambda}_{k,z} = 1.43 > 0.2$$

$$\kappa_z = 1 / (k + \sqrt{k^2 - \bar{\lambda}_{k,z}^{-2}}) \quad k = 0.5 \cdot (1 + \alpha \cdot (\bar{\lambda}_{k,z} - 0.2) + \bar{\lambda}_{k,z}^2)$$

Buckling stress line c  $\Rightarrow \alpha = 0.49$

$$k = 1.83$$

$$\kappa_z = 0.34$$

**Lateral torsional buckling moment  $M_{ki,y,d}$** 

$$M_{ki,y,d} = \frac{\eta_{ki,y} \cdot M_{y,d}}{\gamma_M}$$

$$\text{with } \eta_{ki,y} = 3.03 \quad M_{y,d} = 60.00 \text{ kNm} \quad \gamma_M = 1.0$$

$$M_{ki,y,d} = 181.80 \text{ kNm}$$

**Reduction factor  $\kappa_M$** 

$$\bar{\lambda}_M = \sqrt{M_{pl,y,d} / M_{ki,y,d}}$$

$$\text{with } M_{pl,y,d} = 178.71 \text{ kNm} \quad M_{ki,y,d} = 181.80 \text{ kNm}$$

$$\bar{\lambda}_M = 0.99 > 0.4$$

$$\kappa_M = (1 / (1 + \bar{\lambda}_M^{-2n}))^{1/n}$$

Coefficient for hot rolled girder  $n = 2.50$

$$\kappa_M = 0.76$$

**Coefficient  $k_y$** 

$$k_y = 1 - N_d / (\kappa_z \cdot N_{pl,d}) \cdot a_y \leq 1 \quad a_y = 0.15 \cdot \bar{\lambda}_{k,z} \cdot \beta_{M,y} - 0.15 \leq 0.9$$

$$\text{with } N_d = 70.00 \text{ kN} \quad N_{pl,d} = 1843.20 \text{ kN}$$

$$\kappa_z = 0.34 \quad \bar{\lambda}_{k,z} = 1.43$$

$$\text{Moment ratio } \psi_{y} = 0.00 \Rightarrow \beta_{M,y} = 1.80$$

$$a_y = 0.24$$

$$k_y = 0.97$$

**Coefficient  $k_z$** 

$$k_z = 1 - N_d / (\kappa_z \cdot N_{pl,d}) \cdot a_z \leq 1.5 \quad a_z = \bar{\lambda}_{k,z} \cdot (2 \cdot \beta_{M,z} - 4) + (\alpha_{pl,z} - 1) \leq 0.8$$

$$\text{with } N_d = 70.00 \text{ kN} \quad N_{pl,d} = 1843.20 \text{ kN} \quad \alpha_{pl,z} = 1.52$$

$$\kappa_z = 0.34 \quad \bar{\lambda}_{k,z} = 1.43$$

$$\text{Moment ratio } \psi_{z} = 1.00 \Rightarrow \beta_{M,z} = 1.40$$

$$a_z = -1.20$$

$$k_z = 1.13$$

**Check**

$$\frac{N_d}{\kappa_z \cdot N_{pl,d}} + \frac{M_{y,d}}{\kappa_M \cdot M_{ply,d}} \cdot k_y + \frac{M_{z,d}}{M_{plz,d}} \cdot k_z \leq 1$$

$$N_d = 70.00 \text{ kN}$$

$$N_{pl,d} = 1843.20 \text{ kN}$$

$$\kappa_z = 0.34$$

$$M_{y,d} = 60.00 \text{ kNm}$$

$$M_{ply,d} = 178.71 \text{ kNm}$$

$$\kappa_M = 0.76 \quad k_y = 0.97$$

$$M_{z,d} = 10.00 \text{ kNm}$$

$$M_{plz,d} = 84.41 \text{ kNm}$$

$$k_z = 1.13$$

$$0.11 + 0.43 + 0.13 = 0.67 \leq 1.0 \quad \text{The check is OK !}$$

## References

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Verlag Ernst & Sohn, Berlin 1978.

# EN 1993-1-1 Steel Checks

## Basics

The steel checks according to EN 1993-1-1 (Eurocode 3) can be used for buildings and engineering constructions under observance of the following standards:

- EN 1993-1-1:2005/A1:2014 as the base document
- DIN EN 1993-1-1:2014 with the National Annex Germany 2022-10
- OENORM EN 1993-1-1:2014 with the National Annex Austria B 1993-1-1:2017-11
- SS EN 1993-1-1:2014 with the National Annex Sweden 2019-01 (EKS 11)
- BS EN 1993-1-1:2005 with the National Annex Great Britain 2008-12

The desired rule is selected in the *Settings* dialog in the *Analysis* menu. When selecting the material the following alternatives are available:

- *S235-EN* to *S500-EN* for construction steel as per EN 1993-1-1, Table 3.1 or EN 10025-2
- *Stahl* for a free definition of the material characteristics

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with EN 1991-1-1:2002/AC:2009 (Eurocode 1). The program will take into account the preset safety factors and combination coefficients defined in EN 1990:2021 (Eurocode 0) for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements. The stress checks are performed for all combinations in the ultimate limit state including the *special combination*. For every set of internal forces of these design combinations the cross-section class according to EN 1993-1-1, Chapter 5.5, is automatically determined and optionally the elastic or plastic cross-section resistance in accordance with Chapter 6.2 is checked. The shear buckling check according to EN 1993-1-5 is not carried out.

The predefined steel sections, project-specific parameter and polygon sections or sections from the user database can be used as cross-sections. Beams with the *Beam* section type are not checked as the section geometry is not known for them. For area sections, the extremal internal forces for the defined action combinations are determined without performing a check and saved in the database for graphical representation.

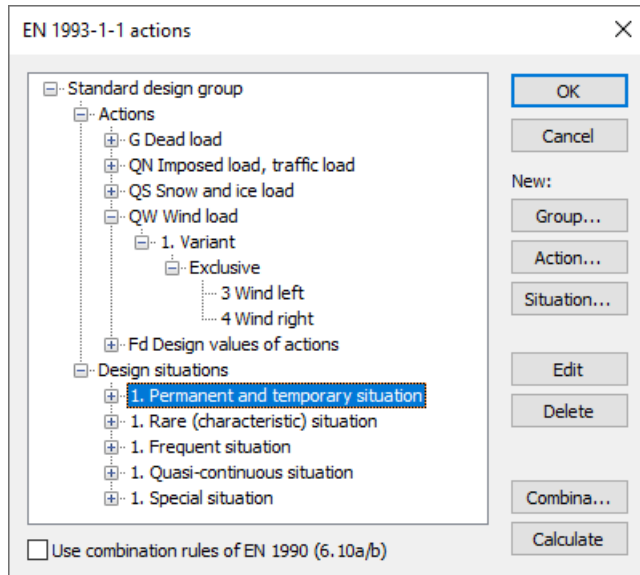
The buckling and lateral torsional buckling check is carried out on the fork supported equivalent beam with an independent program, which can be opened from the *Infograph* folder in the start menu.

The EN 1993-1-1 guidelines are primarily cited for the following descriptions. Reference to the relevant national settings is only made if they contain different or complementary rules. The passages in question are marked by a vertical line left of the text.

# Input for Checks on the Entire Structure

## Actions and Design Situations

The load design values are calculated based on the internal forces of individual load cases and load case combinations. To do so, the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations. The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)

The assigned load cases should contain a design-relevant set of loads with combination coefficients and partial safety factors for actions and material such as for example a load group for the stability check on the entire structure according to EN 1993-1-1, Clause 5.2.2 (3). The selected load cases are combined exclusively.

### Group...

Open the dialog for entering a new design group. According to e.g. standard EN 1991-1-1, Chapter 6.2.2 (2), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

### Situation...

Open the dialog for entering new design situations.

### Edit

Open the Edit dialog for the selected action or situation.

### Delete

Delete the selected action or situation.

### Combinations...

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

The following example shows the total variants of the *permanent and temporary situation* according to Eq. (6.10) to be examined with the load cases (L1...L6) involved and their weighting factors.

Actions	Load cases	$\gamma_{sup}$	$\gamma_{inf}$	$\Psi_0$
Dead load	1	1.35	1.0	-
Imposed load, traffic load	2, 3 (inclusive)	1.5	0	0.7
Wind load	4	1.5	0	0.6
$F_d$ Design values of actions	5, 6	1.0	1.0	-

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
1	1.35					
2	1.35	1.50	1.50	0.90		
3	1.35	1.50	1.50			
4	1.35	1.50		0.90		
5	1.35	1.50				
6	1.35		1.50	0.90		
7	1.35		1.50			
8	1.35	1.05	1.05	1.50		
9	1.35	1.05		1.50		
10	1.35		1.05	1.50		
11	1.35			1.50		
12	1.00					

OK Cancel Load group...

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
13	1.00	1.50	1.50	0.90		
14	1.00	1.50	1.50			
15	1.00	1.50		0.90		
16	1.00	1.50				
17	1.00		1.50	0.90		
18	1.00		1.50			
19	1.00	1.05	1.05	1.50		
20	1.00	1.05		1.50		
21	1.00		1.05	1.50		
22	1.00			1.50		
23					1.00	
24						1.00

OK Cancel Load group...

**Calculate**

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to open the checking module. Each time you open the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

**Use combination rules of EN 1990 (6.10a/b)**

Optionally the Eq. (6.10a/b) are used for the combination of the permanent and temporary situation, otherwise Eq. (6.10).



## Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.

### Label

User-defined label for the action.

### Gamma.sup, Gamma.inf

Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$

SS EN 1990:

The program suggests the partial safety factors as they result in accordance with Section A, Article 11, for safety class 3 from  $\gamma_d \cdot \gamma_{sup}$  with the reduction factor  $\gamma_d = 1.0$  as per Article 14. If required, lower safety classes can be taken into account entering lower values.

### Combination coefficients psi for:

Input fields for selecting the combination coefficients for variable actions. The button allows you to view and change the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .

### Load cases

List of possible load cases or load case combinations. You can choose an item from the list by selecting it and clicking the button or by using drag & drop.

### Multi-select

Load cases and combinations can be added to the actions more than once.

### Exclusive variants

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the or buttons.

### Inclusive load cases

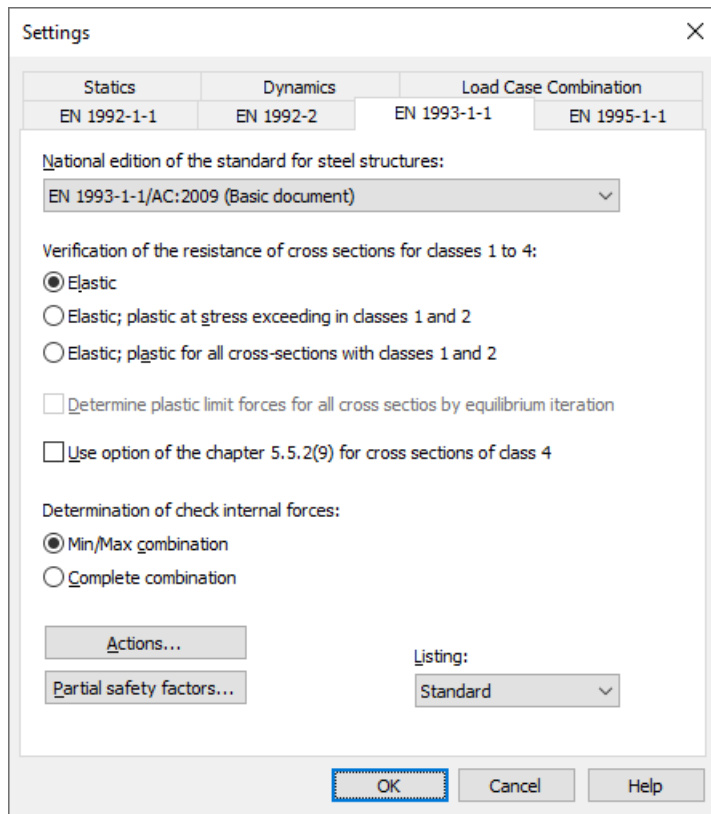
Selected load cases and combinations that can act simultaneously.

### Exclusive load cases

Selected load cases and combinations that exclude each other.

## Analysis Settings

The *EN 1993-1-1* dialog field can be accessed using the *Settings* option of the *Analysis* menu.



### National edition of the standard

The edition you select will be used for all subsequent entries and calculations.

The following checks can be chosen:

#### Elastic

In accordance with Chapter 6.2.1 of the standard, the elastic cross-section resistance is verified for classes 1 to 4. For class 4 the check is carried out with effective cross-section properties as per 1993-1-5, Clause 4.3.

#### Elastic; plastic at stress exceeding in classes 1 and 2

In accordance with Chapter 6.2.1 of the standard, the elastic cross-section resistance is verified for classes 1 to 4. If the comparison stress in classes 1 and 2 exceeds the permissible limit, the plastic cross-section resistance will be verified.

#### Elastic; plastic for all cross-sections with classes 1 and 2

Generally the plastic cross-section resistance will be verified for class 1 and 2, also if the comparison stress does not exceed the permissible limit.

#### Determine plastic limit forces for all cross-sections by equilibrium iteration

When selected, for verifying the cross-section resistance, the plastic limit forces are determined by stress integration and equilibrium iteration. Otherwise, for double symmetrical I and H profiles with constant flange thickness as well as for rectangular box sections, the check is performed using the interaction equations according to EN 1993-1-1, Chapter 6.2.3 to 6.2.10. Equilibrium iteration is always used for all other cross sections.

#### Use option of the chapter 5.5.2(9) for cross-sections of class 4

Cross-sections of class 4 will be treated like cross-sections of class 3, if the  $c/t$  ratio does not exceed the limits of class 3 increased by a factor according to Chapter 5.5.2 (9).

### Determination of the check internal forces

- *Min/Max combination*  
The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*  
To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *Check internal forces*.

### Actions...

Open the dialog for describing actions.

### Partial safety factors...

Open the dialog for modifying the partial safety factor  $\gamma_{M0}$ . He is preset to 1.0 according to Chapter 6.1 (1).

### Listing

- *No*: No log is generated by the design program.
- *Standard*: During analysis a log with a tabular output of the calculated utilizations is created.
- *Detailed*: This log differs from the standard log by additionally offering information on the analyzed internal force combinations.
- *Standard>permissible*: Standard log of the check locations at which the permissible limits are exceeded.
- *Detailed>permissible*: Detailed log of the check locations at which the permissible limits are exceeded.

## Section Inputs

Properties for element 16 - Section - Form

Section

- Form
- Shear stresses
- Material
- Bedding
- Thermal analysis
- General
- Eccentricity

Number: 1 - 2 x Section Type: Polygon Material Type: S235-EN

Label: 2 x IPE 200

Properties concern further elements.

A: 0.0057199 m<sup>2</sup>

I<sub>x</sub>: 1.74216e-05 m<sup>4</sup>

I<sub>y</sub>: 3.90387e-05 m<sup>4</sup>

I<sub>z</sub>: 1.71486e-05 m<sup>4</sup>

I<sub>yz</sub>: 0 m<sup>4</sup>

Factor for I<sub>x</sub>: 1

Ignore in member stiffness

Reduce dead load

Reinforcem...

Cross-section class as per EN 1993-1-1:

OK Cancel Help

### Cross-section class

For polygonal sections and sections from the user database the classification into the cross-section class according to EN 1993-1-1, Table 5.2, is carried out by the user. The check of the plastic cross-section resistance is always performed by equilibrium iteration, regardless of the calculation settings.

# Input for Checks on the Equivalent Beam Settings

**Listing**

Number of beam result points:

Detailed listing

Listing for all result points

**Page Frame**

Page no.:  
  Date, Time

Project:

Title:

Take user name from InfoCAD page setup

## Number of beam result points

The internal forces are calculated at equidistant points in the beam. For each result location the checks are performed with the corresponding internal forces and the corresponding  $M_{cr}$ .

## Detailed listing

A detailed log with all necessary values is generated during calculation.

## Listing for all result points

In addition to the determinant result location, the results for all result locations are documented.

## Page no.

Page number used for the 1st side to be printed. The number is increased incrementally for each printing operation. You can enter '0' to suppress the numbering.

## Date, time

This information appears automatically in the standard page frame.

## Project, title

This text appears automatically in the standard page frame.

## Take user name from InfoCAD page setup

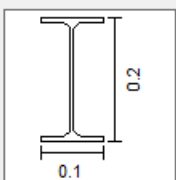
If the mark is set, the user name is automatically retrieved from the *lgraph.dat* file and included in the standard page frame. By removing the mark a different user name can be defined.

## Section & Material

**Cross-section**

IPE 200

- IPE 80
- IPE 100
- IPE 120
- IPE 140
- IPE 160
- IPE 180
- IPE 200
- IPE 220
- IPE 240
- IPE 270
- IPE 300



A = 28.5 cm<sup>2</sup>

I<sub>x</sub> = 7.02 cm<sup>4</sup>

I<sub>y</sub> = 1940 cm<sup>4</sup>

I<sub>z</sub> = 142 cm<sup>4</sup>

I<sub>w</sub> = 12990.0 cm<sup>6</sup>

**Material**

S235-EN

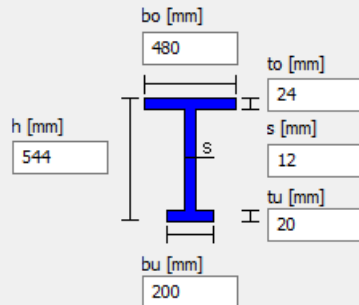
Gamma.M1: 1

f<sub>yk</sub> [N/mm<sup>2</sup>]: 235

E-Modul [N/mm<sup>2</sup>]: 210000

G-Modul [N/mm<sup>2</sup>]: 81000

**User-defined section** ✕



bo [mm]: 480

h [mm]: 544

to [mm]: 24

s [mm]: 12

tu [mm]: 20

bu [mm]: 200

**Properties:**

welded

Cancel

OK

### Section

Selection of a steel section from the section library or input of a user-defined profile by specification of given dimensions.

### User-defined profile

For the defined I profile the property *welded* or *rolled* for the assignment of buckling curves according to EN 1993-1-1, Tables 6.2 and 6.5 can be specified.

### Material

- S235-EN to S500-EN: Construction steel according to EN 1993-1-1, Tab. 3.1 or EN 10025-2.
- Steel: User-defined steel.

### Gamma.M1

The material-specific coefficient  $\gamma_{M1}$  is used to obtain the design value for the strength  $f_{y,d}$ .

### f<sub>yk</sub>

Characteristic yield strength [N/mm<sup>2</sup>] of construction steel S235-EN to S450-EN according to EN 1993-1-1, Table 3.1 for product thicknesses  $t \leq 40$  mm and of S460-EN and S500-EN according to EN 10025-2 for product thicknesses  $t \leq 16$  mm. Other values can be taken into account selecting the material type *Steel*.

### E-Modulus

Modulus of elasticity [N/mm<sup>2</sup>].

### G-Modulus

Shear modulus [N/mm<sup>2</sup>].

## Load & System

Load & System

Load in z-direction: My,d left [kNm]: qz,d [kN/m]: My,d right [kNm]:

Distributed load:

Length L [m]:  Nd [kN]:

Load in y-direction: Mz,d left [kNm]: Fy,d [kN]: Mz,d right [kNm]:

Concentrated:

### Load in z-direction

- *Distributed load:* A uniformly distributed load  $q_{z,d}$  [kN/m] is applied to the equivalent beam.
- *Concentrated load:* A point load  $F_{z,d}$  [kN/m] is applied to the equivalent beam. The load lever arm  $z_p$  is always set to zero for this selection.

### My,d left, My,d right

Fixed end moments  $M_{y,d}$  [kNm] including the partial safety factors  $\gamma_F$ . The fixed end moments can be determined, if required, using the second-order theory.

### qz,d or Fz,d

Distributed load  $q_{z,d}$  [kN/m] or point load  $F_{z,d}$  [kN] including the partial safety factors  $\gamma_F$ .

### Nd

Normal force  $N_d$  [kN] including the partial safety factors  $\gamma_F$  (negative compression). Only compressive forces can be considered. The normal force can be determined, if required, using the second-order theory.

### Length L

Equivalent beam length [m].

### Load in y-direction

- *Distributed load:* A uniformly distributed load  $q_{y,d}$  [kN/m] is applied to the equivalent beam.
- *Concentrated load:* A point load  $F_{y,d}$  [kN/m] is applied to the equivalent beam.

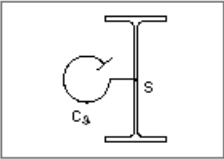
### Mz,d left, Mz,d right

Fixed end moments  $M_{z,d}$  [kNm] including the partial safety factors  $\gamma_F$ . The fixed end moments can be determined, if required, using the second-order theory.

### qy,d or Fy,d

Distributed load  $q_{y,d}$  [kN/m] or point load  $F_{y,d}$  [kN] including the partial safety factors  $\gamma_F$ .

## Lateral Torsional Buckling Parameters

<b>Calculation Method</b> 2 - Annex B	<b>Torsion spring</b> Null
<b>Interaction factors:</b> B.2 - tors. flexible	
<b>Load application point zp</b> At the top chord zp [m]: -0.1	
	C.theta [kNm/m]: 0 <input type="button" value="Edit..."/>

### Calculation method

- 1 - Annex A: Determination of interaction factors  $k_{ij}$  with method 1 according to EN 1993-1-1, Annex A.
- 2 - Annex B: Determination of interaction factors  $k_{ij}$  with method 2 according to EN 1993-1-1, Annex B.

Only method 1 can be applied for tube sections.

OENORM B 1993-1-1:

Method 2 is to be used. Tube sections can therefore only be tested for buckling.

SS EN 1993-1-1:

Method 1 is to be used.

### Interaction factors

When selecting method 2, it is also necessary to specify whether the component is susceptible to torsional deformations or not according to EN 1993-1-1, Section 6.3.3 (1).

- B.1 - tors. rigid: The interaction factors  $k_{ij}$  for components that are not susceptible to torsional deformations (torsional rigid) are calculated according to Table B.1.
- B.2 - tors. flexible: The interaction factors  $k_{ij}$  for components that are susceptible to torsional deformations (torsional flexible) are calculated according to Table B.2.

Rectangular hollow sections are automatically assumed to be torsional rigid.

### Load application point zp

- Top chord: The load acts at the top chord.
- Shear center: The load acts at the shear center.
- Center of gravity: The load acts at the center of gravity.
- Bottom chord: The load acts at the bottom chord.
- User-defined: User-defined load application point.

### zp

The load lever arm  $z_p$  is calculated for the corresponding load application point.

### Correction factor kc

DIN EN 1993-1-1, BS EN 1993-1-1:

- Automatic: Calculation corresponding to the moment distribution according to Tab. 6.6.
- User-defined: Factor  $0 \leq k_c \leq 1$ , e.g. as per DIN EN 1993-1-1, Eq. (NA.4) or BS EN 1993-1-1, Chapter NA.2.18.

### Torsion spring

- User-defined: The torsion spring  $C_{9,k}$  entered by the user is taken into account.
- Compute: The torsion spring  $C_{9,k}$  is calculated from the input data (see the elastic rotational bedding dialog). For U profiles the elastic rotational bedding from the profile deformation of the supported girder is not taken into account.
- Null: No torsion spring is taken into account.

### Edit...

Open the dialog to enter the torsion spring  $C_{9,k}$  [kNm/m].

### C.theta

The torsion spring  $C_{9,k}$  is taken into account using an ideal second-degree torsional area moment  $I_{T,ideal}$ .

$$I_{T,ideal} = I_T + C_{9,k} \cdot L^2 / (\pi^2 \cdot G)$$

Using  $I_{T,ideal}$ ,  $M_{cr}$  is calculated for girders without elastic rotational bedding. This approximation can only be applied for

small values of elastic rotational bedding  $C_{9,k}$  for instance for values of elastic rotational bedding for trapezoidal profiles, calculated according to EN 1993-1-1, Chapter 10.1.5.2; for larger values of  $C_{9,k}$  the lateral torsional buckling moments calculated with  $I_{T,ideal}$  can be unreliable (see Meister).

## Torsion Spring

Torsion spring as per EN 1993-1-3, Chapter 10.1.5.2

Supporting component  
 Trapezoidal sheet steel

Factor k, Fig. 10.7: 2 Endspan t. Material: S235-EN

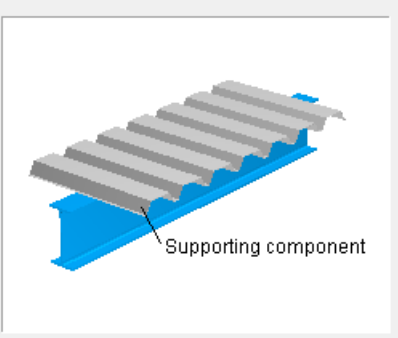
Span s [m]: 2.5  $f_{yk}$  [N/mm<sup>2</sup>]: 240

$I_{eff}$  [cm<sup>4</sup>/m]: 250 E-Modul [N/mm<sup>2</sup>]: 210000

Purlin distance e [m]:

Connection stiffness  
 $C_{\theta}, C_{\theta,k} = C_{D,A}$  from Eq. (10.17): 5.2 kNm/m

OK Cancel



### Supporting component

- *Trapezoidal sheet steel*
- *Purlins*: The connection strength is not taken into account.

### Factor k, Fig. 10.7

Factor for the determination of  $C_{D,C}$  according to EN 1993-1-3, Eq. (10.16):

- *2 Endspan t.*:  $k = 2$ ; Endspan and rotation according to EN 1993-1-3, Fig. 10.7 top.
- *3 Endspan t.*:  $k = 3$ ; Endspan and rotation according to EN 1993-1-3, Fig. 10.7 bottom.
- *4 Inside span t.*:  $k = 4$ ; Inside span and rotation according to EN 1993-1-3, Fig. 10.7 top.
- *6 Inside span t.*:  $k = 6$ ; Inside span and rotation according to EN 1993-1-3, Fig. 10.7 bottom.

### Span s

Span [m] of the supporting component.

### $I_{eff}$

Effective second-degree area moment [cm<sup>4</sup>/m] of the supporting component.

### Purlin distance e

Distance between the purlins perpendicular to the direction of load [m].

### Material

Material of the supporting component.

- *S235-EN, S275-EN, S355-EN, S450-EN*: Construction steel according to EN 1993-1-1, Tab. 3.1.
- *Steel*: User-defined steel.

### $f_{yk}$

Characteristic yield strength of the supporting component [N/mm<sup>2</sup>].

### E-Modulus

E-modulus [N/mm<sup>2</sup>] of the supporting component.

### $C_{\theta}, C_{\theta,k} = C_{D,A}$ from Eq. (10.17)

Connection strength [kNm/m] for trapezoidal sheet steel according to EN 1993-1-3, Eq. (10.17).



## Check Internal Forces

The calculation of load cases results in a set of internal forces for each load case at the check location (e.g.  $N_x$ ,  $M_y$ ). The check internal forces are then determined from the results of the load cases with the combination rules relevant for the check situations. One of the following methods can be selected in the analysis settings:

- *Min/Max combination*  
The results of a load case are added to the set of internal forces with the minimum or maximum of an internal force, if this increases the amount of the extreme value. Result sets from traffic actions in which the control variable is less than the threshold  $10^{-3}$  are not combined. The min/max combination delivers a constant number of sets regardless of the number of load cases and thus represents a particularly economical solution for the checks.
- *Complete combination*  
To determine the evidence internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The number of records increases exponentially with the number of inclusive load cases and can therefore result in high time and memory requirements for the checks.

The internal forces relevant for the checks are documented in the detailed check listing. Regardless of the selection made, the results of the min/max combination are saved for the graphical representation. The load cases involved in the combination can be displayed using the *Combination information* context function.

The differences between the two combination methods mentioned before can be seen from the following example of a uniaxially stressed beam. The load cases 2, 3 and 4 shown can act simultaneously (inclusive). All safety and combination factors are assumed to be 1 for the example.

Action	$N_x$	$M_y$	Load case
G - permanent	-15	40	1
Q - variable	0	20	2
	5	10	3
	0	-10	4

*Internal forces of the load cases*

Extreme value	$N_x$	$M_y$	Combination
min $N_x$	-15	40	L1
max $N_x$	-10	50	L1+L3
min $M_y$	-15	30	L1+L4
max $M_y$	-10	70	L1+L2+L3

*Results of min/max combination*

Set	$N_x$	$M_y$	Combination
1	-15	40	L1
2	-15	60	L1+L2
3	-10	50	L1+L3
4	-15	30	L1+L4
5	-10	70	L1+L2+L3
6	-15	50	L1+L2+L4
7	-10	40	L1+L3+L4
8	-10	60	L1+L2+L3+L4

*Results of complete combination*

# Ultimate Limit States

## Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.4.3, the following combinations are taken into account in the ultimate limit states:

- For the combination of the permanent and temporary design situation either Equation (6.10) or the most unfavorable equation from (6.10a) and (6.10b) is permitted.

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10)$$

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10a)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10b)$$

For the coefficient  $\xi$  the value of  $\xi = 0.85$  results from Table A.1.2(B).

DIN EN 1990, OENORM B 1990:

Equation (6.10) is used for the combination.

SS EN 1990:

Equations (6.10a) and (6.10b) apply with following modifications:

$$\sum_{j \geq 1} \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P \quad (6.10aSS)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_d \cdot \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_d \cdot \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10bSS)$$

Assuming reliability class 3, factor  $\gamma_d$  is set to 1. (see Section A, Article 11 and 14). The coefficient  $\xi$  is set to the value of  $\xi = 0.89$ .

BS EN 1990:

The coefficient  $\xi$  in Equation (6.10b) is set to the value of  $\xi = 0.925$ .

- Combination for accidental design situations

$$\sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.11b)$$

$\psi_{1,1} \cdot Q_{k,1}$  is used by the program for this combination.

OENORM B 1990-1:

$\psi_{2,1} \cdot Q_{k,1}$  is decisive.

- Combination for design situations caused by earthquakes

$$\sum_{j \geq 1} G_{k,j} + P + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.12b)$$

Additionally, for the results of a stability check according to the second-order theory the

- Special combination

$$F_{d,1}; \dots; F_{d,n}$$

is available. In this combination the action ( $F_d$ ) are superposed exclusively.

For each combination you can define different design situations for the construction stages and final states. Each check is performed for all situations of a combination.

## Design Values According to the Second-Order Theory

The calculation according to the second-order theory as per Clause 5.2.2 is carried out as part of the internal forces calculation. For this you need to create – with the help of load groups – appropriate load cases from decisive loads (see also function *Combinations* in the action dialog).

As the partial safety factors are already to be taken into account for the internal forces calculation, within the load groups the respective actions must be multiplied with their corresponding partial safety factor  $\gamma_G$  or  $\gamma_Q$  as well as with  $\gamma_{M0}$  according to Clause 6.1 (1).

DIN EN 1993-1-1:

Instead of  $\gamma_{M0}$  the value  $\gamma_{M1} = 1.1$  is to be used.

In the check the load cases calculated according to the second-order theory are assigned to an action  $F_d$  and superposed in the *Special Combination* as an exclusive selection. The results are checked against the characteristic values of the cross-section resistances.

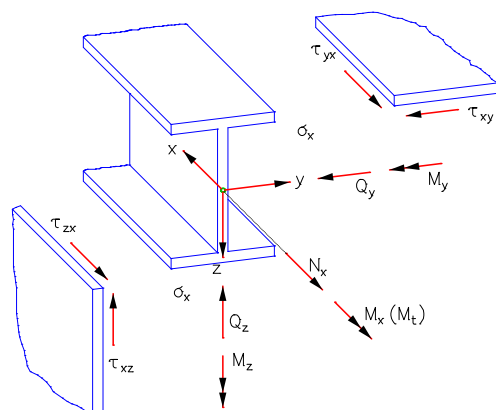
## Characteristic Values

For sections made out of construction steel *S235-EN* to *S450-EN*, the yield strength is taken from EN 1993-1-1, Table 3.1 for product thicknesses  $t \leq 40$  mm. For *S460-EN* and *S500-EN* the yield strength according to EN 10025-2 for product thicknesses  $t \leq 16$  mm is assumed. If other sizes are used, the general material type *Steel* is to be chosen as it allows for freely defining all properties.

## Stress Determination

### Coordinate Systems

The following illustration shows the definition of the coordinate systems for internal forces, loads and stresses using the example of a beam with an I profile. The section edge is defined as a closed polygon.



The longitudinal force  $N_x$  runs through the centroid, the lateral forces  $Q_y$  and  $Q_z$  run through the shear center of the section which, in this case, coincides with the centroid. The vectors of the moments  $M_x$ ,  $M_y$ ,  $M_z$  and stresses  $\sigma_x$ ,  $\tau_{xy}$ ,  $\tau_{xz}$  run parallel to the coordinate axes.

## Longitudinal Stresses

The necessary section properties for determination of the longitudinal stresses from longitudinal force and bending are determined elementarily with the segmentation method from the coordinates of the polygon. They are to be regarded as gross section properties in the sense of Clause 6.2.2.1 of the standard.

More specifically, these are:

$y_s, z_s$	Centroid coordinates.
$A$	Area of the section.
$I_y, I_z$	Moments of inertia in relation to the coordinate axes.
$I_{yz}$	Deviation moment of inertia.
$I_1, I_2$	Moments of inertia in relation to the main axes.
$\varphi$	Twisting angle of the main axes.
$W_y, W_z$	Moments of resistance for the calculation of the extremal longitudinal stresses from $M_y, M_z$ .

For the analysis of a section, the stresses are determined at the automatically defined calculation points. The longitudinal stress  $\sigma$  for a point  $(y,z)$  of the section results from

$$\sigma(y, z) = N_x \frac{1}{A} + M_y \frac{\Delta z I_z - \Delta y I_{yz}}{I_y I_z - I_{yz}^2} + M_z \frac{\Delta y I_y - \Delta z I_{yz}}{I_y I_z - I_{yz}^2}$$

with  $\Delta y = y - y_s$  and  $\Delta z = z - z_s$ .

## Shear Stresses

### Lateral force

For load as a result of lateral force, the differential equation of the so-called shear warpage  $\omega_\tau$

$$\frac{\partial^2 \omega_\tau}{\partial y^2} + \frac{\partial^2 \omega_\tau}{\partial z^2} = -\frac{Q_z z}{G I_y} \quad \text{or} \quad -\frac{Q_y y}{G I_z}$$

is solved numerically with the help of the boundary element method. From this the following shear characteristics are determined:

$\tau_{Qy,1}$	Shear stress for the lateral force $Q_y = 1$ .
$\tau_{Qz,1}$	Shear stress for the lateral force $Q_z = 1$ .
$W_{qy}, W_{qz}$	Area values for calculation of the extremal shear stresses from $Q_y$ and $Q_z$ with $\tau_{Qy} = Q_y / W_{qy}$ and $\tau_{Qz} = Q_z / W_{qz}$ .

If selected in the Section dialog, the shear stresses across the section width will be averaged.

### Torsion

For load as a result of St. Venant torsion, the differential equation of the unit warpage  $\omega$

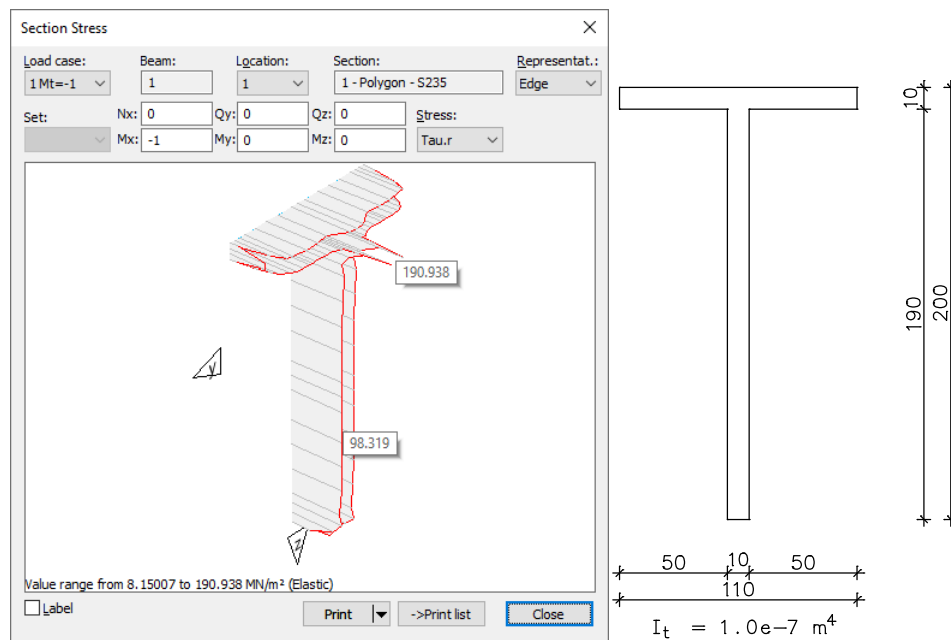
$$\frac{\partial^2 \omega}{\partial y^2} + \frac{\partial^2 \omega}{\partial z^2} = 0$$

is decisive. Its solution leads to the following characteristics:

$I_t$	Torsion moment of inertia.
$\tau_{Mx,1}$	Shear stress for the torsional moment $M_x = 1$ .
$W_t$	Moment of resistance for the calculation of the extremal shear stress from $M_x$ .

As a result, in the fillet area of rolled sections or in blunt corners of welded profiles larger torsional shear stresses occur than those according to the theory of thin-walled profiles. As an option in the section dialog, specifying the maximum sheet thickness (open profiles) or the torsion resistance moment (box profiles) allows you to limit the stresses to the maximum value according to the theory of thin-walled profiles.

The following figure shows the stress curve for a load of  $M_x = 1$  (Section Stress function):



The numerically determined value for the web in the example corresponds to the theoretical shear stress for open thin-walled sections  $\tau = M_x \cdot t / I_t = 100 \text{ MN/m}^2$ . The peak value of 190 MN/m<sup>2</sup> results from a singularity in the blunt corners.

All unit shear stresses are calculated at discrete points on the section edges. The discretization is automatically carried out by the program. The unit shear stresses are edge stresses which run along the section edge. Their sign depends on the direction of the edge. The resultant shear stress  $\tau$  from lateral force and torsion is then determined by the following equation:

$$\tau = Q_y \cdot \tau_{Qy,1} + Q_z \cdot \tau_{Qz,1} + M_x \cdot \tau_{Mx,1}$$

## Classification of Cross-Sections

The classification is performed for standard profiles and parameterized steel sections according to EN 1993-1-1, Table 5.2. To this end, the stress distribution for simultaneous stress from biaxial bending and normal force in the center line of the section parts is used. T-profiles are treated like outstand flanges according to table 5.2. Angle sections are not classified as flanges supported on one side, but according to the separate table section for angle sections of Tab. 5.2 (see Kuhlmann et al. 2016, note to Tab. 5.2). User defined rectangle sections are assigned to class 3 without further analysis. For polygonal sections and sections from the user database, the class can be specified in the cross-section dialog, whereby the classification according to Table 5.2 must be carried out in advance by the user.

A cross-section is generally classified by the most unfavorable class of its compressed section parts. The permissible exceptions described in the Chapters 5.5.2 (11), 5.5.2 (12), 6.2.1 (10) and 6.2.2.4 (1) are not used in the program.

### Example

Cross-section: HEAA 360 – S355  
 Design load:  $N = -400 \text{ kN}$ ,  $M_y = 175 \text{ kNm}$ ,  $M_z = 65 \text{ kNm}$

Strain state		Web	F1	F2	F3	F4
		$\epsilon$	0.81	0.81	0.81	0.81
		$\alpha$	0.70	1	0.66	0.78
		$\psi$	-0.45	0.58	-0.15	-0.52
		$c/t$	29	9.88	9.88	9.88
		1: $c/t \leq$	39.95	7.32	13.63	9.35
		2: $c/t \leq$	46.01	8.14	15.14	10.39
		3: $c/t \leq$	65.56	11.73	28.73	14.28
		<b>Class</b>	<b>1</b>	<b>3</b>	<b>1</b>	<b>2</b>

Under the given load, resulting from the classification of flange F1, the whole cross-section is categorized as class 3 and only the elastic check applies.

## Elastic Cross-Section Resistance

The elastic cross-section resistance is verified for all cross-sections of the classes 1 to 4. Cross-sections of class 4 are treated like class 3 if selected by the user and if the  $c/t$  ratio does not exceed the limits of class 3 increased by a factor according to Chapter 5.5.2 (9). Otherwise the check is carried out with effective cross-section properties as per EN 1993-1-5, Clause 4.3, taking into account the additional moments according to EN 1993-1-1, Eq. (6.4). The effective cross-section properties are determined under simultaneous effect of  $N_{Ed}$  and  $M_{Ed}$  without iteration. Circular hollow sections classified in class 4 cannot be checked, because the effective cross section area  $A_{eff}$  cannot be determined according to Chapter 4.3. For angle sections, the effective cross-section values are determined for the unfavorable case of a completely compressed cross-section (cf. Kuhlmann et al. 2016, note to Tab. 5.2).

### Check

The permissibility of the comparison stress is proofed with the yield criterion according to equation (6.1).

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right) \cdot \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right) + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2 \leq 1 \quad (6.1)$$

Where

- $\sigma_{x,Ed}$  is the design value of the longitudinal stress at the point of consideration.
- $\sigma_{z,Ed}$  is the design value of the transverse stress at the point of consideration.  $\sigma_{z,Ed} = 0$ .
- $\tau_{Ed}$  is the design value of the shear stress at the point of consideration.
- $f_y$  is the nominal value of the yield strength as per Table 3.1.
- $\gamma_{M0}$  is the partial safety factor for the resistance of cross-sections as per Clause 6.1 (1).

Stresses resulting from the *Special Combination* are checked against the characteristic value of the yield strength  $f_y$ , because the partial safety factor  $\gamma_{M0}$  has already been taken into account during the calculation of the internal forces.

## Plastic Cross-Section Resistance

The plastic cross-section resistance will be verified for all cross-sections of the classes 1 and 2 if selected by the user and if the elastic cross-section resistance of the contemplated set of internal forces is exceeded. For this, the following methods apply:

- Interaction equations for double symmetrical I and H profiles as well as for rectangular box sections.
- Equilibrium iteration of absorbable internal forces for the remaining cross-sections.

Optionally, the user can select to determine the plastic resistance by equilibrium iteration for **all** cross-sections.

### Interaction Equations

For double symmetrical I and H profiles with constant flange thickness as well as for rectangular box sections, the plastic cross-section resistance can be determined according to EN 1993-1-1, Chapter 6.2.3 to 6.2.10. If none of the single actions exceeds its plastic resistance, the interaction of internal forces is examined using the interaction equations given in Chapter 6.2. For this, according to Chapter 6.2.10 (2), the load bearing capacity of cross-sections stressed by bending and normal force is not reduced if the effect of lateral force does not exceed half of the plastic lateral force resistance. The following special conditions also apply:

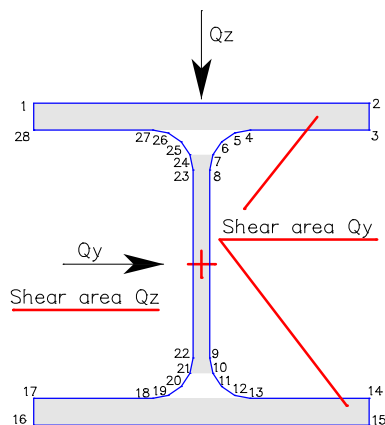
- The coefficient  $\eta$  as per Chapter 6.2.6 (3) is assumed to be 1.
- In the case of lateral force interaction according to Chapter 6.2.8 (3) and 6.2.10 (3), the reduced yield strengths for rolled I and H profiles are applied to the shear area  $A_{Vz}$  for lateral force in z direction and  $A - A_{Vz}$  for lateral force in y direction.
- The simplification according to Chapter 6.2.8 (5) is not applied.
- If the simultaneous interaction of bending, normal force and lateral force is taken into account, the factors  $a$ ,  $a_w$  and  $a_f$  from Chapter 6.2.9.1 (5) are calculated with the correspondingly reduced shear areas. This is based on the interpretation of the NABau to DIN EN 1993-1-1 of April 2017.

### Equilibrium Iteration

The verification of the plastic cross-section resistance is carried out by comparison of the absorbable internal forces (resistance) with the load internal forces of a section. Here it is to be checked that the limit internal forces in the plastic state are not exceeded. The calculation of the absorbable internal forces is performed by integration of the stresses on the section polygon and equilibrium iteration taking into account the following conditions:

- Linear-elastic ideal plastic stress-strain relationship.
- Section remains planar.
- Huber-von Mises yield criterion.

The interaction of all 6 internal forces  $N_x, Q_y, Q_z, M_x, M_y, M_z$  is to be taken into account. To ensure this, simplified assumptions with respect to the shear stress distribution from lateral force and torsion are to be made as a closed solution of the interaction problem is not available. Because in steel construction thin-walled sections are common, subareas can be defined as a good approximation and then used to dissipate the shear stresses:



The steel sections in the section library have predefined subareas which are listed in the section table of the log. The following assumptions apply:

- The lateral forces create constant shear stresses  $\tau_{xy} = Q_y/A_{qy}$  or  $\tau_{xz} = Q_z/A_{qz}$  in their corresponding subareas.
- The torsion moment  $M_x$  results in local, constant shear stress states in the subareas and the remaining area with the maximum edge shear stress  $\tau_t$  acting on the area.

For polygon sections and sections from the user database, subareas cannot be taken into account. In this case the following assumptions apply for the entire section:

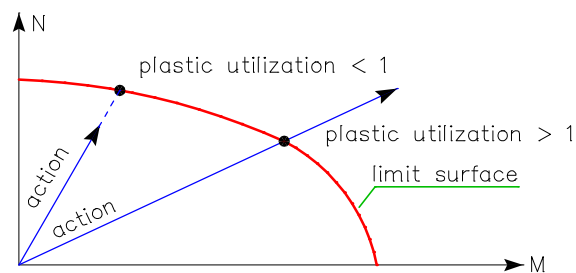
- The lateral forces create constant shear stresses  $\tau_{xy} = Q_y/W_{qy}$  or  $\tau_{xz} = Q_z/W_{qz}$ .
- The torsion moment  $M_x$  creates the constant shear stress state  $\tau_t = M_x/W_t$ .

The design values of the strengths are each reduced locally by the shear stress described above. To allow calculation of the absorbable internal forces, the remaining available strength is used for each area. The calculation of the shear characteristics is carried out for determining the section properties as described above. The exception for effects of small lateral forces according to EN 1993-1-1, Chapter 6.2.10 (2), is not utilized.

The calculation approach described delivers limit internal forces that are always on the safe side; however, in individual cases it could be that not all reserves are used.

To help estimate the degree of plastic utilization of a section, the load vector is intersected with the limit area of the 6-dimensional internal force space. This can be used to calculate a factor for plastic utilization, which is  $\leq 1$  if the limit area is not exceeded and  $> 1$  if it is exceeded.

The following illustration shows the iteration of the limit area using the example of an N-M interaction.



## Check

The check evaluates if the load internal forces exceed the limit internal forces taking into account the interaction of all internal forces in the full plastic state. In the *Special Combination*, the plastic internal forces are calculated with the nominal value  $f_y$  of the yield strength whereas  $f_y / \gamma_{M0}$  is used in all other combinations.

## Check Against Buckling and Lateral Torsional Buckling

The check for potentially unstable frameworks can be carried out according to EN 1993-1-1, Section 5.2.2 (7b) for the individual component. The program performs the buckling and the lateral torsional buckling check on the fork supported equivalent beam according to Chapter 6.3. The fixed end forces are to be determined in a static analysis of the entire structure, taking into account the second-order theory and global imperfections, and are to be applied at the equivalent beam. For the buckling length of the individual component the system length can be applied.

For the check against buckling and lateral torsional buckling on the equivalent beam the following requirements must be met:

- The calculation of the internal forces is carried out according to the theory of elasticity.
- The equivalent beam is assumed to be straight.
- Section and material are constant along the length of the beam.
- The dimensions of the section are small compared with the other dimensions.
- The mathematical curvature is linearized.
- The influence of shear deformations on the internal forces are not considered.
- The load is slowly increased to its final value and does not undergo any deviation in direction as a result of the system deformation.
- Actions in the z direction act at a distance  $z_p$  from the shear center.
- The actions in the y direction act at the shear center.
- The equivalent beam rests on a fork support at both ends.
- Sheet buckling is not included.
- The effects from torsion are not accounted for.

The check against lateral torsional buckling can be carried out for I, U and user-defined steel sections, for other sections only the check against buckling according to Chapter 6.3.1 can be carried out.

## Calculation of the resistance

Initially for each check location a cross-section classification according to Chapter 5.5 is carried out with the corresponding internal forces. Depending on the cross-section class the absorbable internal forces are determined according to Table 6.7.

For cross-sections of classes 1 and 2 the determination of the plastic internal forces is performed by integration of the stresses on the section polygon, as described above.

For cross-section class 4 the cross-section properties are determined with the effective area of the section parts under compression. If thereby a translation of the main axes of the effective section compared to the gross section occurs, furthermore the resulting additional moments ( $\Delta M_{y,Ed}$ ,  $\Delta M_{z,Ed}$ ) are determined.

## Calculation of $N_{cr}$

$N_{cr,y}$  and  $N_{cr,z}$  are the elastic flexural buckling forces about the y and z axis. They are calculated by the program with the following formulas:

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr,y}^2}$$

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr,z}^2}$$



## Calculation of the elastic critical moment $M_{cr}$ for lateral torsional buckling

The elastic critical moment  $M_{cr}$  for lateral torsional buckling can be determined for example according to Petersen (1980) with the following equation:

$$M_{cr} = M_{K_{i,y}} = \zeta \cdot N_{K_{i,z}} \cdot \left( \sqrt{c^2 + 0.25 \cdot z_p^2} + 0.5 \cdot z_p \right)$$

This formula is only applicable for double-symmetric I profiles. Additionally, for a general moment curve, the moment coefficient  $\zeta$  is quite difficult to determine. For single-symmetric profiles, it is no longer possible to determine  $M_{cr}$  using the abovementioned approach.

In order to be able to calculate any single-symmetric profile under general load, this program module contains a method for the direct determination of  $M_{cr}$ .

The solution is derived at by varying the elastic potential  $\Pi$  while neglecting the terms from the calculated bending  $w$ . For the unknown functions  $v$  and  $\varphi$  a multiple-term power series is used that fulfills the boundary conditions.

$$\Pi = \frac{1}{2} \int_0^L \left\{ \left[ G I_x + (r_y - 2 z_m) M_y(x) \eta_{cr} \right] \left( \frac{\partial \varphi}{\partial x} \right)^2 + E I_w \left( \frac{\partial^2 \varphi}{\partial x^2} \right)^2 + E I_z \left( \frac{\partial^2 v}{\partial x^2} \right)^2 + 2 M_y(x) \eta_{cr} \left( \frac{\partial^2 v}{\partial x^2} \right) \varphi + q \eta_{cr} z_p \varphi^2 \right\} dx$$

The geometrically nonlinear share comprising actions in the z direction is included via a uniformly distributed load  $q_z$ . This is chosen such that the same maximum moment of span is created as through the given action.

The resulting eigenvalue problem provides the smallest positive critical load factor  $\eta_{cr}$  and thus the required lateral torsional buckling moment  $M_{cr}$  according to Martin (1996). This is calculated for each check location.

$$M_{cr} = \eta_{cr} \cdot M_{y,d}$$

The method described offers the advantage that the user does not need to specify the moment coefficient  $\zeta$ .

## Serviceability Limit States

### Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.5.3, the following combinations are taken into account in the serviceability limit states:

- Combination for characteristic situations

$$\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i > 1} \Psi_{0,i} \cdot Q_{k,i} \quad (6.14b)$$

- Combination for frequent situations

(6.15b)

- Combination for quasi-continuous situations

$$\sum_{j \geq 1} G_{k,j} + P + \sum_{i > 1} \Psi_{2,i} \cdot Q_{k,i} \quad (6.16b)$$

### Limiting Deformations

In accordance with EN 1993-1-1, Chapter 7.2, the vertical and horizontal deformations shall be limited in consultation with the client corresponding to EN 1990, Annex A.1.4. According to A.1.4.3 (3) the quasi-continuous situation is decisive for the check.

OENORM B 1990-1, Chapter 8.2:

Under the quasi-continuous situation, the deflection under long-term loading should not exceed 1/250<sup>th</sup> of the span. A limitation to 1/500<sup>th</sup> of the span under the frequent or the quasi-continuous situation can be necessary for the special cases described in Chapter 8.2.

SS EN 1993-1-1:

In thin-walled structures, according to Article 16 and 17 the deformations under the frequent situation with reversible limit load should not exceed 1/200<sup>th</sup> of the span.

The compliance with the aforementioned limitations can be proven in the graphical and numerical deformation representation. In the folder *Node Deformations* of the database tree, the deformations of all decisive situations can be accessed.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The detailed log also lists the decisive combination internal forces of all design situations for each result location.

### Stresses

$\sigma_x, \sigma_y$	Extremal normal stresses from bending and normal force.
$\sigma_{xy}$	Extremal shear stresses from torsional moment and shear force.
$\tau_{xy}, \tau_{xz}$	Extremal shear stresses from lateral force and torsion.
$\sigma_v$	Maximum comparison stress.
$\sigma_1, \sigma_2$	Extremal principal stresses.

All stresses are given in [MN/m<sup>2</sup>].

### Utilizations

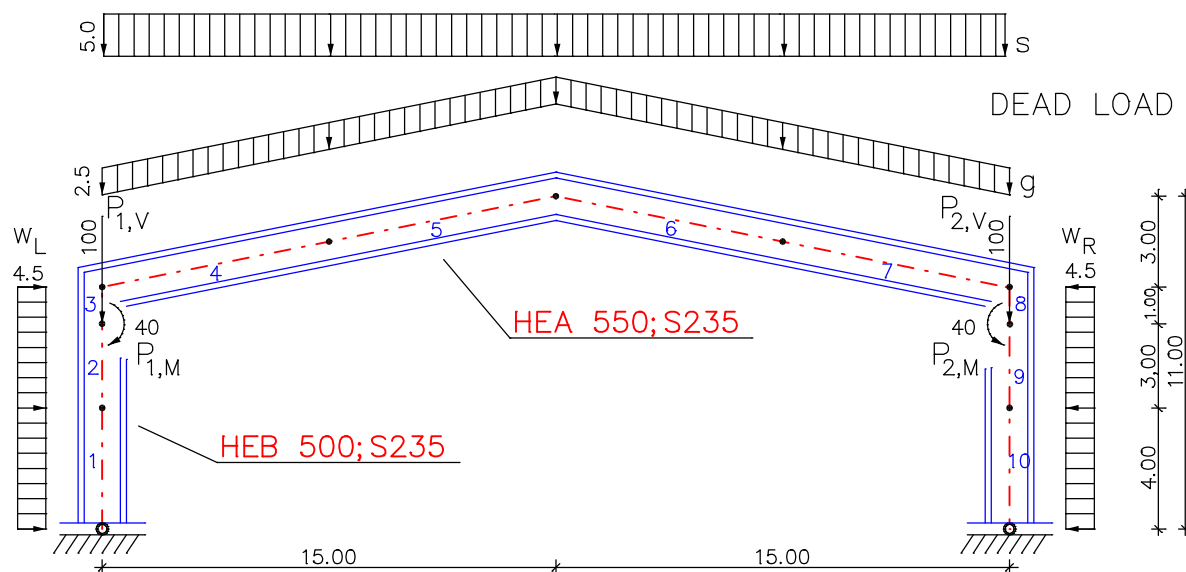
The utilization is defined as the ratio between the action  $E_d$  and, depending on the user selection, the elastic or plastic resistance  $R_d$  of a cross-section. In the folder *Stresses > Steel Checks* of the result tree the following results are available:

- Utilization of the beams for each situation.
- Maximal utilization of the beams of all situations.
- Maximal utilization of the sections of all situations.

# Examples

## Steel Checks on the Entire System

In the hall frame illustrated below a framework analysis and a stability check are carried out according to the second-order theory. The plastic cross-section resistance is verified for all cross-sections of classes 1 and 2, because exceedances occur in the elastic stress check.



Static system and dimensions

### Sum of installed loads and support reactions

LC	Label	Fx [kN]	Fz [kN]
1	Dead load	0.000	200.250
	Support reactions	-0.000	200.250
2	Snow	0.000	150.000
	Support reactions	-0.000	150.000
3	Wind left	36.000	0.000
	Support reactions	36.000	-0.000
4	Wind right	-36.000	0.000
	Support reactions	-36.000	0.000
5	Crane track left	0.000	100.000
	Support reactions	0.000	100.000
6	Crane track right	0.000	100.000
	Support reactions	-0.000	100.000
7	Crane track center:(LC5+LC6)/2	0.000	100.000
	Support reactions	-0.000	100.000
11	Th2+Predef1; My min,right;PC.1	32.400	600.338
	Support reactions	32.400	600.338
12	Th2+Predef2; My min,left;PC.1	-32.400	600.338
	Support reactions	-32.400	600.338
13	Th2; My max,center;PC.1	0.000	495.338
	Support reactions	0.000	495.338
14	Th2+Predef1; Nx min,right;PC.1	32.400	600.338
	Support reactions	32.400	600.338
15	Th2+Predef2; Nx min,left;PC.1	-32.400	600.338
	Support reactions	-32.400	600.338

**Material properties**

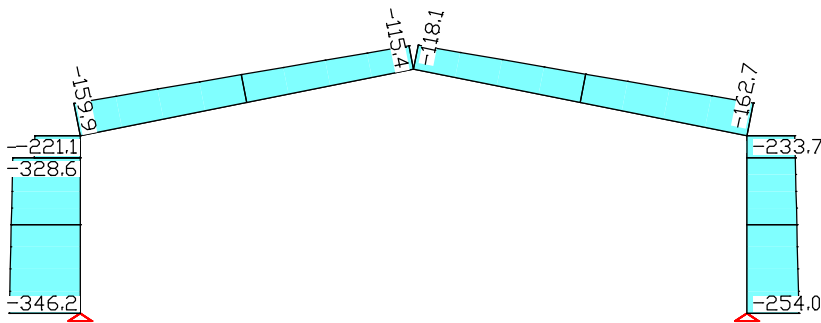
No.	Material type	EModule [MN/m <sup>2</sup> ]	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]	k <sub>bx</sub> -a	k <sub>bx</sub> -e [MN/m <sup>3</sup> ]	k <sub>bz</sub> -a	k <sub>bz</sub> -e	b <sub>x</sub>	b <sub>z</sub>
1	S235-EN	210000	1.200e-05	78.500	0	0	0	0	1.00	1.00
2	S235-EN	210000	1.200e-05	78.500	0	0	0	0	1.00	1.00

**Section properties**

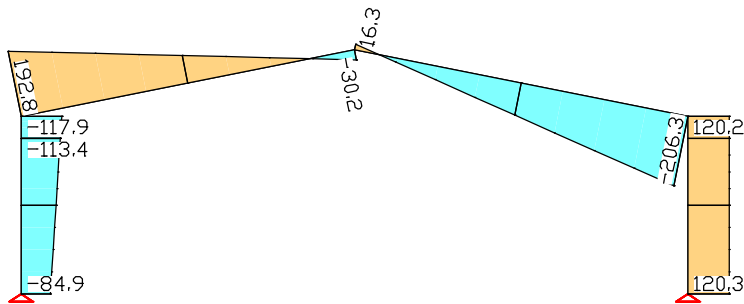
No.	1	HEB 500
		A = 2.390e-02 [m <sup>2</sup> ], I <sub>y</sub> = 1.072e-03 [m <sup>4</sup> ]
No.	2	HEA 550
		A = 2.120e-02 [m <sup>2</sup> ], I <sub>y</sub> = 1.119e-03 [m <sup>4</sup> ]

For stability checks according to the second-order theory, load groups have been composed in compliance with the combination information of the respective internal forces.

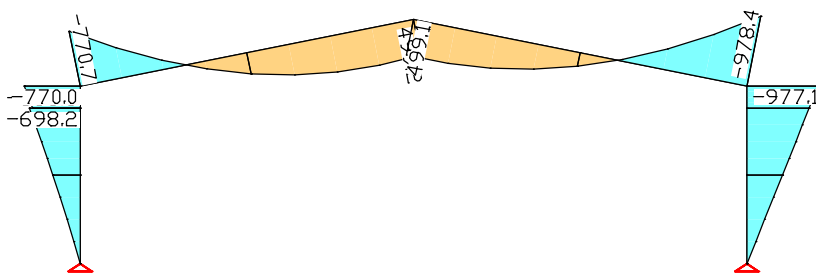
The following internal forces curves result from load case 11 composed as described before, considering a pre-deformation of l/200 to the right.



Load case 11, second-order theory, internal forces  $N_x$



Load case 11, second-order theory, internal forces  $Q_z$



Load case 11, second-order theory, internal forces  $M_y$

To perform the checks the actions must be assigned as shown in the following log. The checking program performs the necessary internal force combinations and verifies the cross-section resistance.

**Design per EN 1993-1-1:2005/A1:2014**

According to Clause 6.2.1 of the standard, for classes 1 and 2 the plastic resistance, for class 3 the elastic resist. of the cross-section is verified.

For Class 4 cross-sections the check is carried out with effective properties as per EN 1993-1-5, Clause 4.3.

The actions are combined acc. to EN 1990, Eq. (6.10), using the partial and combination factors according to EN 1990:2021.

**Designing occurs for all possible combinations of actions.**

**Terms in Printout**

Sigma.x, Sigma.v : Normal stresses, comparison stresses acc. to Eq. (6.1).  
 Tau.xy, Tau.xz : Shear stresses from lateral force Qz.  
 A : Sectional area.  
 Iy, Iz, Iyz : Moments of inertia.  
 Aqz : Shear subarea of the polygon section for Qz.  
 Wqz : Resistance momentum of the ext. shear stress of Qz.  
 y, z : Location in the sectional polygon [m].

**Stress Calculation**

The normal stresses and shear stresses are calculated for homogeneous polygon sections from normal force, bending moment and lateral force. The shear characteristics are calculated acc. to the Boundary-Element-Method. The calculation points for all stresses are edge points of the cross-section.

For the following cross-sections the shear stresses from lateral force are averaged over the section width.

Cross-section	Cross-section	Cross-section
1 HEB 500	2 HEA 550	-

**Cross-Section Classification**

The classification occurs for standard profiles and parameter cross-sections acc. to EN 1993-1-1, Table 5.2. T-Profiles are handled as outstand flanges. User-defined polygonal cross-sections are classified always into class 3.

**Plastic Cross-Section Resistance**

For double symmetrical I and H profiles with constant flange thickness as well as for rectangular box sections, the plastic cross-section resistance is determined according to EN 1993-1-1, Chapter 6.2.3 to 6.2.10. If none of the single actions exceeds its plastic resistance, the interaction of internal forces is examined. For this, according to Chapter 6.2.10 (2) the load bearing capacity is not reduced if the effect of lateral force does not exceed half of the plastic lateral force resistance. The coefficient eta as per Chapter 6.2.6 (3) is assumed to be 1.

For all further profiles, the absorbable internal forces are calculated by integration of the stresses and equilibrium iteration on the section polygon. For this, the interaction of internal forces is regarded and the exception for effects of small lateral forces as per EN 1993-1-1, Chapter 6.2.10 (2), is not utilized.

For simplification a constant shear stress  $\tau = Q_z/A_{qz}$  is assumed for the corresponding shear area during integration.

The shear characteristics are calculated with use of the Boundary-Element-Method at the section edge.

The Huber-v. Mises-Condition as per Eq. (6.1) is used as yield criterion.

**Partial Safety Factor for Steel**

	gamma.M0
Permanent and temporary comb.	1.00
Accidental combination	1.00
Special combination	1.00

**Characteristic Material Properties**

Yield stresses for steel acc. to EN 1993-1-1, Table 3.1 for  $t \leq 40$  mm [MN/m<sup>2</sup>]. Limiting stresses according to Eq. (6.19) and Eq. (6.42).

Material	Perm. C.		Accid. C.		Special C.	
	Sigma.Rd	Tau.Rd	Sigma.Rd	Tau.Rd	Sigma.Rk	Tau.Rk
S235-EN 235	235.00	135.68	235.00	135.68	235.00	135.68

**Cross-Section Properties**

Cross-section	Material	A[mm <sup>2</sup> ]	Aqz[mm <sup>2</sup> ]	Iy[m <sup>4</sup> ]	Iz[m <sup>4</sup> ]	Iyz[m <sup>4</sup> ]
1 HEB 500	S235-EN	23900	5997	1.0720e-03	1.2620e-04	0.0000e+00
2 HEA 550	S235-EN	21200	5776	1.1190e-03	1.0820e-04	0.0000e+00

**EN 1993-1-1 actions****Standard design group****G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

-----

1 Dead load

**QN - Imposed load, traffic load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
 Traffic load - category G: 30 kN < vehicle weight <= 160 kN  
 Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

Load cases 1. Variant, exclusive

- 5 Crane track left
- 6 Crane track right
- 7 Crane track center:(LC5+LC6)/2

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
 Snow load - Places in CEN member states with less than 1000 m above sea level  
 Psi.0 / Psi.1 / Psi.2 = 0.5 / 0.2 / 0

Load cases 1. Variant, exclusive

- 2 Snow

**QW - Wind load**

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures  
 Wind loads  
 Psi.0 / Psi.1 / Psi.2 = 0.6 / 0.2 / 0

Load cases 1. Variant, exclusive

- 3 Wind left
- 4 Wind right

**Fd - Design values of actions**

Load cases

- 11 Th2+Predef1; My min,right;PC.1
- 12 Th2+Predef2; My min,left;PC.1
- 13 Th2; My max,center;PC.1
- 14 Th2+Predef1; Nx min,right;PC.1
- 15 Th2+Predef2; Nx min,left;PC.1

**1. Permanent and temporary situation**

Final state

- G - Dead load
- QN - Imposed load, traffic load
- QS - Snow and ice load
- QW - Wind load

**1. Quasi-continuous situation**

Final state

- G - Dead load
- QN - Imposed load, traffic load
- QS - Snow and ice load
- QW - Wind load

**1. Special situation**

Final state

Fd - Design values of actions

**Check of the Cross-Section Resistance for Beams**

The results represent the extrema of all combinations.

Class Decisive section class according to EN 1993-1-1, Table 5.2.  
 Utilization Relation between the action Ed and the resistance Rd.  
 Plastic resistance acc. to Chapter 6.2.10 is marked with '!'.  
 PC,SC Permanent and temporary comb., Special combination

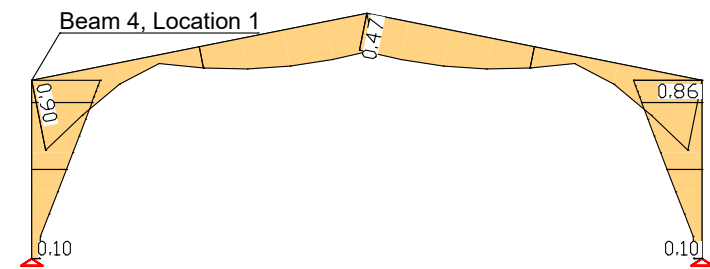
Beam	Se.	Lo.	Comb.	Class	Utilization	
					elastic	plastic
1	1	1	SC.1	1	0.14	0.10!
		2	SC.1	1	0.20	0.11!
		3	SC.1	1	0.31	0.22!
		4	SC.1	1	0.43	0.33!
		5	SC.1	1	0.55	0.44!
2	1	1	SC.1	1	0.55	0.44!
		2	SC.1	1	0.64	0.52!
		3	SC.1	1	0.73	0.60!
		4	SC.1	1	0.82	0.68!
		5	SC.1	1	0.92	0.76!

Beam	Se.	Lo.	Comb.	Class	Utilization	
					elastic	plastic
3	1	1	SC.1	1	0.89	0.76!
		2	SC.1	1	0.92	0.78!
		3	SC.1	1	0.95	0.81!
		4	SC.1	1	0.98	0.84!
		5	SC.1	1	1.01	0.86!
4	2	1	SC.1	1	1.04	0.90!
		2	SC.1	1	0.66	0.56!
		3	SC.1	1	0.33	0.27!
		4	SC.1	2	0.15	0.11!
		5	SC.1	1	0.33	0.27!
5	2	1	SC.1	1	0.33	0.27!
		2	SC.1	1	0.47	0.39!
		3	SC.1	1	0.54	0.46!
		4	SC.1	1	0.57	0.49!
		5	SC.1	1	0.55	0.47!
6	2	1	SC.1	1	0.55	0.47!
		2	SC.1	1	0.57	0.49!
		3	SC.1	1	0.54	0.46!
		4	SC.1	1	0.47	0.39!
		5	SC.1	1	0.33	0.27!
7	2	1	SC.1	1	0.33	0.27!
		2	SC.1	2	0.15	0.11!
		3	SC.1	1	0.33	0.27!
		4	SC.1	1	0.66	0.56!
		5	SC.1	1	1.04	0.90!
8	1	1	SC.1	1	1.01	0.86!
		2	SC.1	1	0.98	0.84!
		3	SC.1	1	0.95	0.81!
		4	SC.1	1	0.92	0.78!
		5	SC.1	1	0.89	0.76!
9	1	1	SC.1	1	0.92	0.76!
		2	SC.1	1	0.82	0.68!
		3	SC.1	1	0.73	0.60!
		4	SC.1	1	0.64	0.52!
		5	SC.1	1	0.55	0.44!
10	1	1	SC.1	1	0.55	0.44!
		2	SC.1	1	0.43	0.33!
		3	SC.1	1	0.31	0.22!
		4	SC.1	1	0.20	0.11!
		5	SC.1	1	0.14	0.10!

**Max. Cross-Section Utilization**

Class Decisive section class according to EN 1993-1-1, Table 5.2.  
 Utilization Relation between the action Ed and the resistance Rd.  
 x Distance from the beam startpoint [m].  
 PC,SC Permanent and temporary comb., Special combination

Cross-section	Material	Beam Loc.	x[m]	Comb.	Class	Utilization
1 HEB 500	S235-EN	3	5 1.00	SC.1	1	0.86
2 HEA 550	S235-EN	4	1 0.00	SC.1	1	0.90



Plastic utilization

An extract of the detailed listing of beam 4 is printed below. This shows that the cross-section resistance in this location is exceeded and therefore the plastic check becomes necessary.

**Check of the Cross-Section Resistance for Beams**

The results represent the extrema of all combinations.

Class Section class according to EN 1993-1-1, Table 5.2.  
 W, F, A Section part of standard profiles: Web, Flange, Angle.  
 A.eff, I.eff Effective properties for Class 4 as per EN 1993-1-5, Clause 4.3.  
 e.z Shift of the centroid of A.eff relative to the centroid of the gross cross-section to determine the additional moment as per Eq. (6.4).  
 Utilization Relation between the action Ed and the resistance Rd.  
 x Distance from the beam startpoint [m].

Beam 4: x = 0.00 m (Beam Length 7.65 m)  
 Cross-section 2: HEA 550, S235-EN, fyk=235 MN/m<sup>2</sup>, A=21200 mm<sup>2</sup>  
 Web : c = 438.0 mm, t = 12.5 mm, c/t = 35.04  
 Flange : c = 116.7 mm, t = 24.0 mm, c/t = 4.86

**1. Permanent and temporary comb. (PC.1): G+QN+QS+QW**

No set of internal forces in this combination was relevant.

1. Special combination (SC.1): Fd

Relevant values from 5 sets of internal forces

Set	Nx [kN]	Qz [kN]	My [kNm]	Class	W	F	F	F	F
2	-162.73	206.31	-978.36	1	1	1	1	1	1

Load case combination for the relevant set of internal forces

Set	Combination
2	L12

Normal Stresses [MN/m<sup>2</sup>]

Min. Sigma.x	: -243.74	Class	: 1
Sigma.x (Nx)	: -7.68	Situation	: SC.1,2
Sigma.x (My)	: -236.07	Se.point y [m]	: 0.150
		z [m]	: 0.270

Max. Sigma.x	: 228.39	Class	: 1
Sigma.x (Nx)	: -7.68	Situation	: SC.1,2
Sigma.x (My)	: 236.07	Se.point y [m]	: -0.150
		z [m]	: -0.270

Shear Stresses [MN/m<sup>2</sup>]

Ext. Tau.xy	: -5.53	Class	: 1
Tau.xy (Qz)	: -5.53	Situation	: SC.1,2
		Se.point y [m]	: -0.033
		z [m]	: 0.246

Ext. Tau.xz	: 34.05	Class	: 1
Tau.xz (Qz)	: 34.05	Situation	: SC.1,2
		Se.point y [m]	: 0.006
		z [m]	: -0.000

Comparison Stress [MN/m<sup>2</sup>]

Max. Sigma.v	: 243.90	Class	: 1
Sigma.x (Nx)	: -7.68	Situation	: SC.1,2
Sigma.x (My)	: -236.07	Se.point y [m]	: 0.045
Tau.xy (Qz)	: 5.13	z [m]	: 0.270
Tau.xz (Qz)	: 0.00		

Check of the Cross-Section Resistance and Utilisation

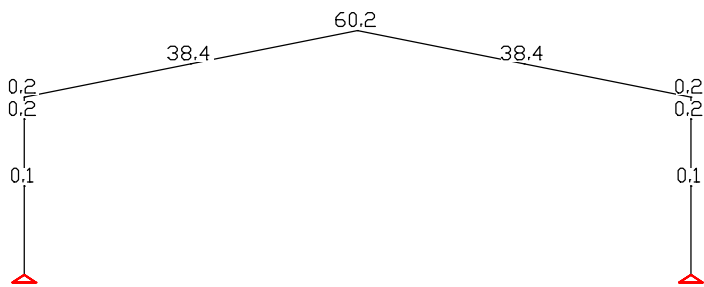
Elastic	Nx [kN]	Ed	Rd	Utilization
	-162.73	-156.79		1.04 > 1
	Qz [kN]	198.78		Class : 1
	My [kNm]	-942.65		Situation : SC.1,2

Plastic	Nx [kN]	Ed	Rd	Ch. 6.2.10
	-162.73	4982.00		Utilization: 0.90
	Qz [kN]	1139.15		Class : 1
	My [kNm]	1086.13		Situation : SC.1,2

The plastic utilization is decisive.

Limitation of deformation

The maximal deformations  $u_{z,max}$  under the quasi-continuous situation are presented in the following picture. The maximal deformation in the center of the frame is 1/498<sup>th</sup> of the span.

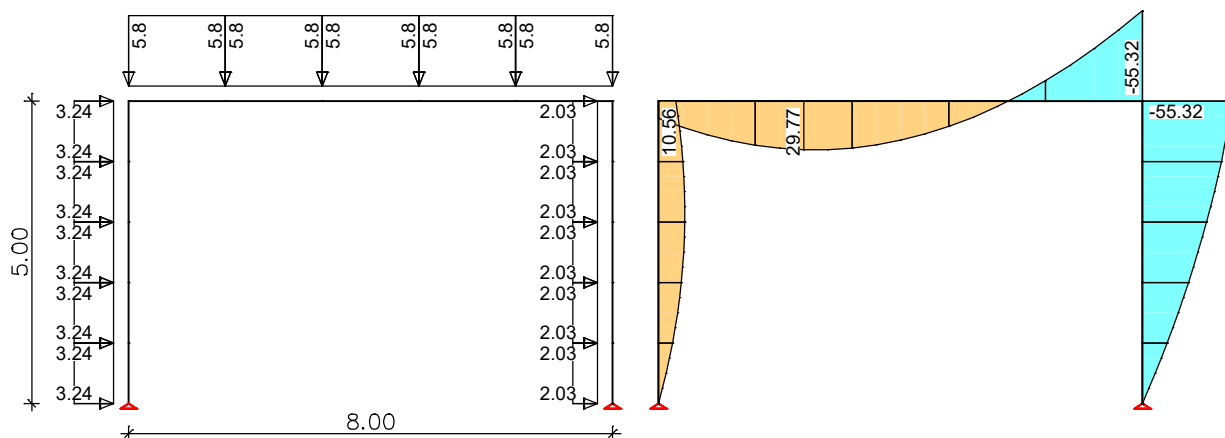


Maximal deformations  $u_{z,max}$  under the 1<sup>st</sup> quasi-continuous situation [mm].



## Lateral Torsional Buckling Check of a Two-Hinged Frame

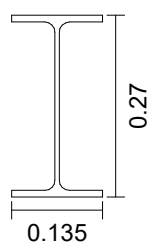
The input values correspond to example 2b on page 79 from the essay of W. Martin (1996).



### Log Printout:

#### Lateral Torsional Buckling as per EN 1993-1-1:2014-07

##### Cross section: IPE 270



$h = 270.0 \text{ mm}$   
 $b = 135.0 \text{ mm}$   
 $s = 6.6 \text{ mm}$   
 $t = 10.2 \text{ mm}$   
 $r = 15.0 \text{ mm}$

$A = 45.9 \text{ cm}^2$   
 $I_T = 16.0 \text{ cm}^4$   
 $i_y = 11.23 \text{ cm}$

$I_y = 5790.0 \text{ cm}^4$   
 $I_\omega = 70580 \text{ cm}^6$   
 $i_z = 3.02 \text{ cm}$

$I_z = 420.0 \text{ cm}^4$   
 $i_p = 11.63 \text{ cm}$

##### Material: S235-EN

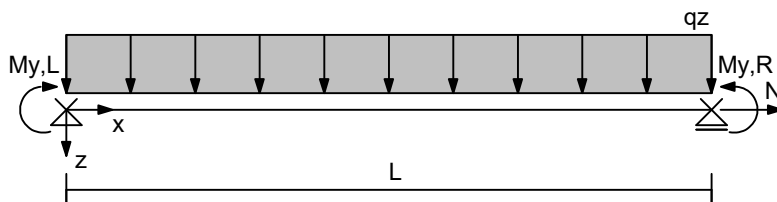
$f_{y,k} = 235 \text{ N/mm}^2$   
 $E\text{-Modulus} = 210000 \text{ N/mm}^2$   
 $G\text{-Modulus} = 81000 \text{ N/mm}^2$   
 $\gamma_{M1} = 1.00$

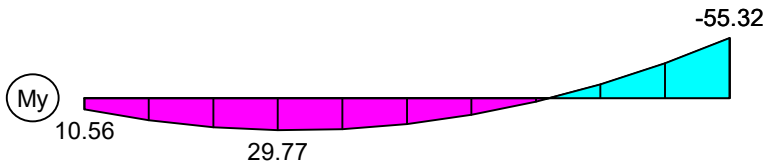
##### Load in z-direction

$L = 8.00 \text{ m}$                        $N_d = 0.00 \text{ kN}$                        $q_{z,d} = 5.80 \text{ kN/m}$   
 $M_{y,d,\text{left}} = 10.56 \text{ kNm}$                        $M_{y,d,\text{right}} = -55.32 \text{ kNm}$

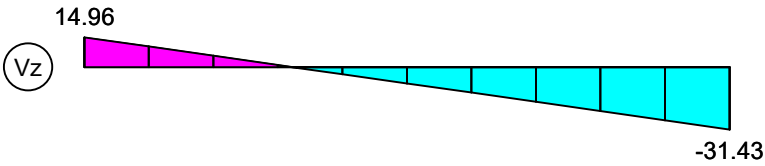
The load application point is in the shear center  $\Rightarrow z_p = 0.00 \text{ cm}$

The ends of the beam are forked supported.

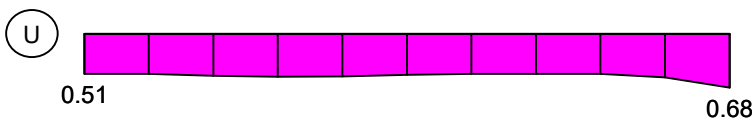




Max  $M_{y,d} = 29.77$  kNm, Min  $M_{y,d} = -55.32$  kNm



Max  $V_{z,d} = 14.96$  kN, Min  $V_{z,d} = -31.43$  kN



Maximum utilization = 0.68

**Elastic flexural buckling force  $N_{cr,y}$  about the y axis**

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr,y}^2}$$

$E = 210000$  N/mm<sup>2</sup>       $I_y = 5790.0$  cm<sup>4</sup>       $L_{cr,y} = 8.00$  m

$N_{cr,y} = 1875.07$  kN

**Elastic flexural buckling force  $N_{cr,z}$  about the z axis**

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr,z}^2}$$

$E = 210000$  N/mm<sup>2</sup>       $I_z = 420.0$  cm<sup>4</sup>       $L_{cr,z} = 8.00$  m

$N_{cr,z} = 136.02$  kN

**Critical load factor  $\eta_{cr}$  for the lateral torsional buckling moment  $M_{cr}$**

$\eta_{cr} = 1.97$

**Decisive check at location  $x = 8.00$  m**

**Design loading**

$N_{Ed} = 0.00$  kN

$V_{z,Ed} = -31.43$  kN

$M_{y,Ed} = -55.32$  kNm

$V_{y,Ed} = 0.00$  kN

$M_{z,Ed} = 0.00$  kNm

⇒ Section class 1

**Plastic resistances as per Table 6.7**

$N_{Rk} = \pm 1078.65$  kN

$M_{y,Rk} = \pm 113.74$  kNm

$M_{z,Rk} = \pm 22.78$  kNm

**Elastic critical moment  $M_{cr}$  for lateral torsional buckling**

$$M_{cr} = \eta_{cr} \cdot M_{y,d}$$

$$M_{cr} = 1.97 \cdot (-55.32) = -108.73 \text{ kNm}$$

**Reduction factor for lateral torsional buckling as per Chapter 6.3.2.3**

$$\chi_{LT} = \frac{1}{\Theta_{LT} + \sqrt{\Theta_{LT}^2 - \beta \bar{\lambda}_{LT}^2}} \leq \frac{1}{\bar{\lambda}_{LT}^2} \leq 1 \quad (6.57)$$

$$\Theta_{LT} = 0.5 \left[ 1 + \alpha_{LT} (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2 \right]$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}}$$

$$\text{with } \bar{\lambda}_{LT} = 1.02$$

$$\bar{\lambda}_{LT0} = 0.40$$

$$\beta = 0.75$$

$$\alpha_{LT} = 0.34$$

as per Tables 6.3 and 6.5

$$M_{cr} = -108.73 \text{ kNm}$$

$$W_{pl,y} = 4.8400 \text{e-4 m}^3$$

as per Table 6.7

$$\chi_{LT} = 0.69$$

Modification as per Equation (6.58)

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \leq 1 \quad (6.58)$$

$$f = 1 - 0.5 \cdot (1 - k_c) \cdot [1 - 2(\bar{\lambda}_{LT} - 0.8)^2] \leq 1$$

$$f = 0.96$$

$$k_c = 0.91$$

as per Table 6.6

$$\chi_{LT,mod} = 0.71$$

**Lateral torsional buckling check as per Chapter 6.3.2**

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1 \quad (6.54)$$

$$M_{b,Rd} = \chi_{LT} \cdot W_y \frac{f_y}{\gamma_{M1}} \quad (6.55)$$

$$\text{with } M_{Ed} = -55.32 \text{ kNm}$$

$$W_y = 4.8400 \text{e-5 m}^3$$

$$\gamma_{M1} = 1.00$$

$$f_y = 235 \text{ N/mm}^2$$

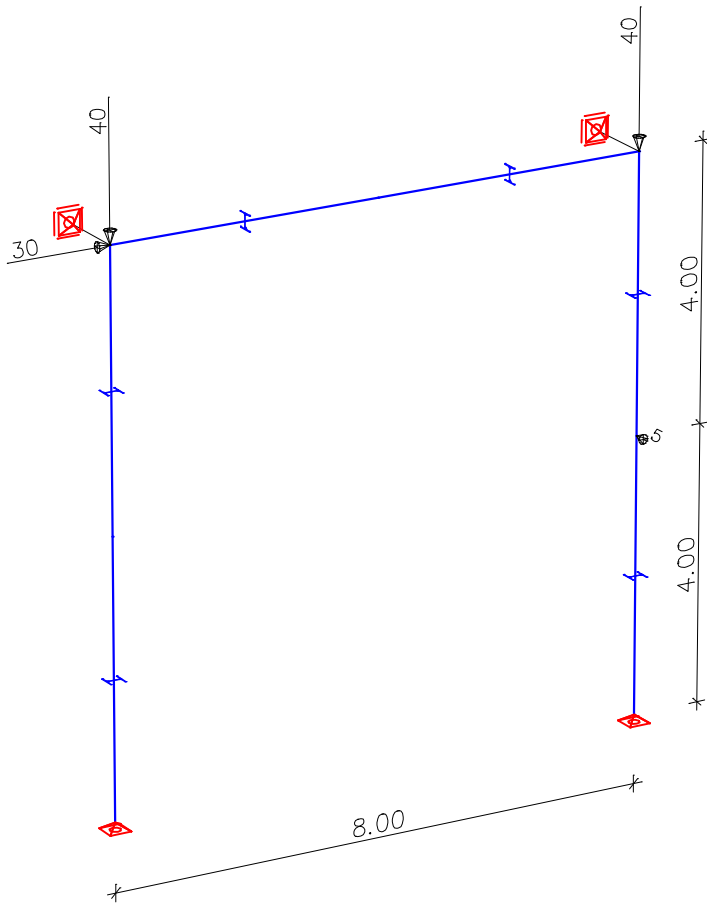
$$\chi_{LT} = \chi_{LT,mod} = 0.71$$

$$M_{Ed} / M_{b,Rd} = 0.68 \leq 1$$

**Check ok !**

## Lateral Torsional Buckling Check of a Frame Column With Two-Axis Bending and Normal Force

The input values correspond to example 7c (check with corresponding internal forces) on page 262 from the book of J. Meister (2002).

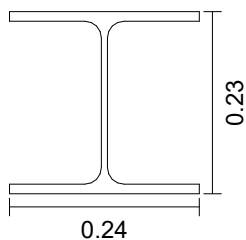


The right frame column is checked.  
 $\gamma_M = 1.00$  (simplified assumption for the example)

### Log Printout:

#### Lateral Torsional Buckling as per EN 1993-1-1:2014

Cross section: HEA 240



h = 230.0 mm  
 b = 240.0 mm  
 s = 7.5 mm  
 t = 12.0 mm  
 r = 21.0 mm

A = 76.8 cm<sup>2</sup>  
 $I_T = 41.7 \text{ cm}^4$   
 $i_y = 10.05 \text{ cm}$

$I_y = 7760.0 \text{ cm}^4$   
 $I_{\omega} = 328500 \text{ cm}^6$   
 $i_z = 6.01 \text{ cm}$

$I_z = 2770.0 \text{ cm}^4$   
 $i_p = 11.71 \text{ cm}$

**Material: S235-EN**

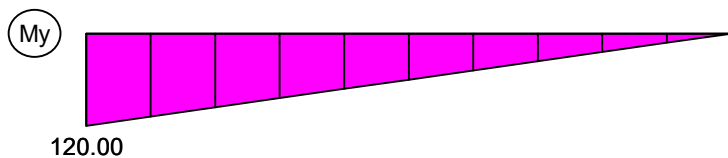
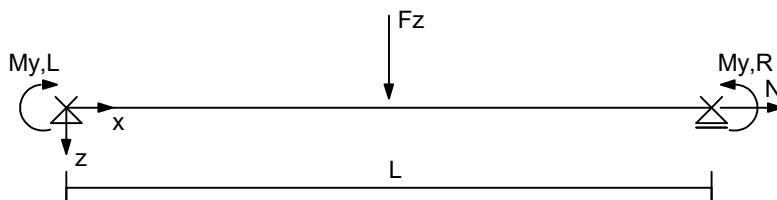
$f_{y,k} = 235 \text{ N/mm}^2$   
 $E\text{-Modulus} = 210000 \text{ N/mm}^2$   
 $G\text{-Modulus} = 81000 \text{ N/mm}^2$   
 $\gamma_{M1} = 1.00$

**Load in z-direction**

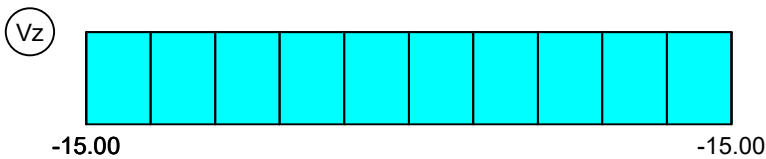
$L = 8.00 \text{ m}$                        $N_d = -70.00 \text{ kN}$                        $F_{z,d} = 0.00 \text{ kN}$   
 $M_{y,d,\text{left}} = 120.00 \text{ kNm}$                        $M_{y,d,\text{right}} = 0.00 \text{ kNm}$

The load application point is in the shear center  $\Rightarrow z_p = 0.00 \text{ cm}$

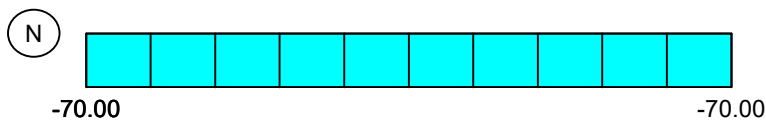
The ends of the beam are forked supported.



Max  $M_{y,d} = 120.00 \text{ kNm}$



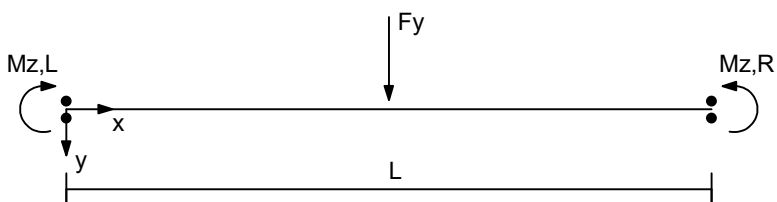
Max  $V_{z,d} = -15.00 \text{ kN}$

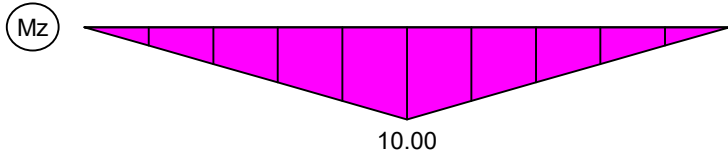


Min  $N_{,d} = -70.00 \text{ kN}$

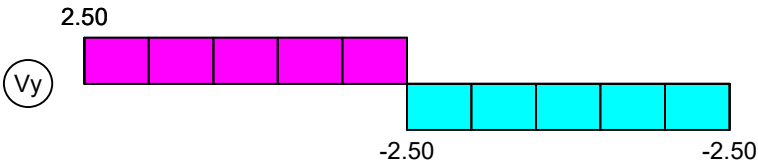
**Load in y-direction**

$L = 8.00 \text{ m}$                        $F_{y,d} = 5.00 \text{ kN}$   
 $M_{z,d,\text{left}} = 0.00 \text{ kNm};$                        $M_{z,d,\text{right}} = 0.00 \text{ kNm}$

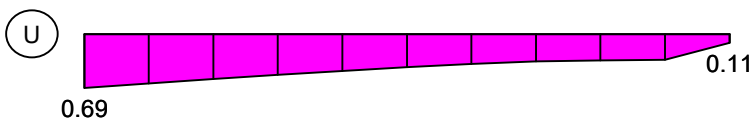




Max  $M_{z,d} = 10.00$  kNm



Max  $V_{y,d} = 2.50$  kN, Min  $V_{y,d} = -2.50$  kN



Maximum utilization = 0.69

**Elastic flexural buckling force  $N_{cr,y}$  about the y axis**

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr,y}^2}$$

$E = 210000$  N/mm<sup>2</sup>       $I_y = 7760.0$  cm<sup>4</sup>       $L_{cr,y} = 8.00$  m

$N_{cr,y} = 2513.05$  kN

**Elastic flexural buckling force  $N_{cr,z}$  about the z axis**

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr,z}^2}$$

$E = 210000$  N/mm<sup>2</sup>       $I_z = 2770.0$  cm<sup>4</sup>       $L_{cr,z} = 8.00$  m

$N_{cr,z} = 897.05$  kN

**Critical load factor  $\eta_{cr}$  for the lateral torsional buckling moment  $M_{cr}$**

$\eta_{cr} = 3.03$

**Decisive check at location  $x = 0.00$  m**

**Design loading**

$N_{Ed} = -70.00$  kN  
 $V_{z,Ed} = -15.00$  kN       $V_{y,Ed} = 2.50$  kN  
 $M_{y,Ed} = 120.00$  kNm       $M_{z,Ed} = 0.00$  kNm

⇒ Section class 1

**Plastic resistances as per Table 6.7**

$N_{Rk} = \pm 1804.80$  kN  
 $M_{y,Rk} = \pm 174.99$  kNm       $M_{z,Rk} = \pm 82.65$  kNm

**Reduction factors for buckling as per Chapter 6.3.1**

$$\chi = \frac{1}{\Theta + \sqrt{\Theta^2 - \bar{\lambda}^2}} \leq 1 \quad (6.49)$$

$$\Theta = 0.5 \left[ 1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right]$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

with  $\bar{\lambda}_y = 0.85$                        $\bar{\lambda}_z = 1.42$   
 $\alpha_y = 0.34$                                $\alpha_z = 0.49$                       as per Tables 6.1 and 6.2  
 $N_{cr,y} = 2513.05 \text{ kN}$                $N_{cr,z} = 897.05 \text{ kN}$

$$\chi_y = 0.69$$

$$\chi_z = 0.34$$

**Lateral buckling check as per Chapter 6.3.1**

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \quad (6.46)$$

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \quad (6.47)$$

with  $N_{Ed} = -70.00 \text{ kN}$                        $f_y = 235 \text{ N/mm}^2$   
 $A = 76.80 \text{ cm}^2$                                $\gamma_{M1} = 1.00$   
 $\chi_y = 0.69$                                        $\chi_z = 0.34$

$$N_{Ed} / N_{b,y,Rd} = 0.06 \leq 1$$

$$N_{Ed} / N_{b,z,Rd} = 0.11 \leq 1$$

**Check ok !**

**Elastic critical moment  $M_{cr}$  for lateral torsional buckling**

$$M_{cr} = \eta_{cr} \cdot M_{y,d}$$

$$M_{cr} = 3.03 \cdot 120.00 = 363.60 \text{ kNm}$$

**Reduction factor for lateral torsional buckling as per Chapter 6.3.2.3**

$$\chi_{LT} = \frac{1}{\Theta_{LT} + \sqrt{\Theta_{LT}^2 - \beta \bar{\lambda}_{LT}^2}} \leq \frac{1}{\bar{\lambda}_{LT}^2} \leq 1 \quad (6.57)$$

$$\Theta_{LT} = 0.5 \left[ 1 + \alpha_{LT} (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2 \right]$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}}$$

with  $\bar{\lambda}_{LT} = 0.69$                                $\bar{\lambda}_{LT0} = 0.40$   
 $\beta = 0.75$                                        $\alpha_{LT} = 0.34$                       as per Tables 6.3 and 6.5  
 $M_{cr} = 363.60 \text{ kNm}$                        $W_{pl,y} = 7.4462 \text{e-4 m}^3$               as per Table 6.7

$$\chi_{LT} = 0.87$$

Modification as per Equation (6.58)

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \leq 1 \quad (6.58)$$

$$f = 1 - 0.5 \cdot (1 - k_c) \cdot [1 - 2(\bar{\lambda}_{LT} - 0.8)^2] \leq 1$$

$$f = 0.88$$

$$k_c = 0.75$$

as per Table 6.6

$$\chi_{LT,mod} = 0.99$$

**Lateral torsional buckling check as per Chapter 6.3.2**

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1 \quad (6.54)$$

$$M_{b,Rd} = \chi_{LT} \cdot W_y \frac{f_y}{\gamma_{M1}} \quad (6.55)$$

with  $M_{Ed} = 120.00 \text{ kNm}$        $W_y = 7.4462e-4 \text{ m}^3$   
 $\gamma_{M1} = 1.00$        $f_y = 235 \text{ N/mm}^2$   
 $\chi_{LT} = \chi_{LT,mod} = 0.99$

$$M_{Ed} / M_{b,Rd} = 0.69 \leq 1$$

**Check ok !****Interaction factors as per Annex B**

Equivalent moment factors as per Table B.3

$$\psi_y = 0.00$$

$$\psi_z = 0.00$$

$$\alpha_{h,z} = 0.00$$

$$C_{my} = 0.60$$

$$C_{mz} = 0.90$$

$$C_{mLT} = 0.60$$

Interaction factors as per Table B.1 for members not susceptible to torsional deformations

For I-sections under axial compression and uniaxial bending  $M_{y,Ed}$  the value  $k_{zy} = 0$  may be assumed

$$k_{yy} = 0.62$$

$$k_{yz} = 0.63$$

$$k_{zy} = 0.00$$

$$k_{zz} = 1.04$$

**Lateral torsional buckling check as per Chapter 6.3.3**

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (6.61)$$

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (6.62)$$

with  $N_{Ed} = -70.00 \text{ kN}$        $M_{y,Ed} = 120.00 \text{ kNm}$        $M_{z,Ed} = 0.00 \text{ kNm}$   
 $N_{Rk} = -1804.80 \text{ kN}$        $\Delta M_{y,Ed} = 0.00 \text{ kNm}$        $\Delta M_{z,Ed} = 0.00 \text{ kNm}$   
 $\chi_y = 0.69$        $M_{y,Rk} = 174.99 \text{ kNm}$        $M_{z,Rk} = 82.65 \text{ kNm}$   
 $\chi_z = 0.34$        $k_{yy} = 0.62$        $k_{yz} = 0.63$   
 $\gamma_{M1} = 1.00$        $k_{zy} = 0.00$        $k_{zz} = 1.04$   
 $\chi_{LT} = \chi_{LT,mod} = 0.99$

$$0.06 + 0.43 + 0.00 = 0.49 \leq 1 \quad (6.61)$$

$$0.11 + 0.00 + 0.00 = 0.11 \leq 1 \quad (6.62)$$

**Check ok !**



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# DIN 1052 Timber Checks

## Basics

The timber checks according to DIN 1052:2008 can be used for buildings and engineering constructions made of the following materials:

- C14 to C50 for softwood as per EN 388:2016, Table 1
- D18 to D80 for hardwood as per EN 388:2016, Table 3
- GL20h to GL32h for homogeneous glued laminated timber as per EN 14080:2013, Table 5
- GL20c to GL32c for combined glued laminated timber as per EN 14080:2013, Table 4

Deviating strengths according to Tables F.5, F.7 and F.9 of the DIN 1052 are taken into account.

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with DIN 1055-100. The program will take into account the preset safety factors and combination coefficients for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements. For every set of internal forces the modification factor  $k_{\text{mod}}$  is automatically determined from the service class of the section and the decisive load-duration.

Rectangle sections and polygon sections can be used as cross-sections. Beams with the *Beam* section type are not checked as the section geometry is not known for them.

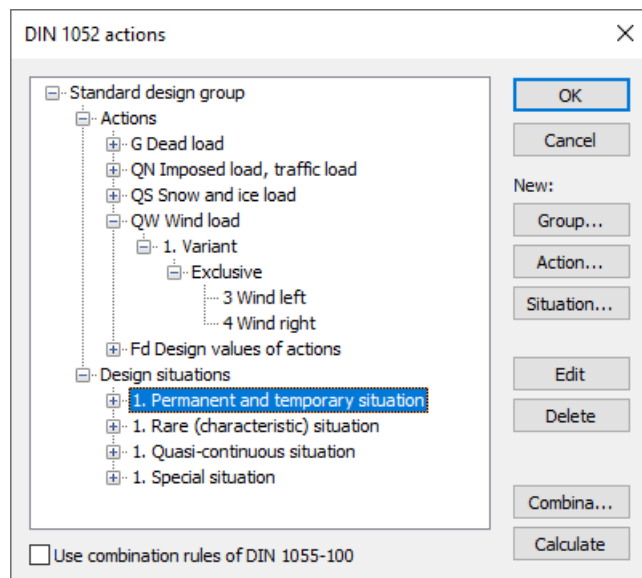
For area sections, the extremal internal forces for the defined action combinations are determined without performing a check and saved in the database for graphical representation.

## Input

### Actions and Design Situations

The load design values are calculated based on the internal forces of individual load cases and load case combinations. To do so, the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



#### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)

The assigned load cases should contain a design-relevant set of loads with combination coefficients and partial safety factors for actions and material and can be used for example for the stability check according to DIN 1052, Chapter 8.5. They are combined exclusively.

**Group...**

Open the dialog for entering a new design group. According to e.g. standard DIN 1055-3, Chapter 6.3 (4), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

**Situation...**

Open the dialog for entering new design situations.

**Edit**

Open the Edit dialog for the selected action or situation.

**Delete**

Delete the selected action or situation.

**Combinations...**

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis.

**Calculate**

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to open the checking module. Each time you open the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

**Use combination rules of DIN 1055-100**

The simplified combination rules according to DIN 1052, Eq. (1) and (2), can be used for the permanent and temporary design situation. Optionally, combining according to DIN 1055-100, Eq. (14), is allowed. For the remaining design situations, the rules of DIN 1055-100 principally apply.

## Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.


**Label**

User-defined label for the action.


**Gamma.sup, Gamma.inf**

Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$

**Combination coefficients psi for:**

Input fields for selecting the combination coefficients for variable actions. The  button allows you to view and change the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .



**Load cases**

List of possible load cases or load case combinations. You can choose an item from the list by selecting it and clicking the  button or by using drag & drop.

**Multi-select**

Load cases and combinations can be added to the actions more than once.

**Exclusive variants**

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the  or  buttons.

**Inclusive load cases**

Selected load cases and combinations that can act simultaneously. Since the number of internal forces to be checked increases exponentially, especially with a large number of load cases, it may be useful to use a load case combination instead of load cases.

**Exclusive load cases**

Selected load cases and combinations that exclude each other.

**Load-duration**

The load-duration class according to DIN 1052, Chapter 7.1.2, is used to determine the modification factor  $k_{\text{mod}}$  for the material strengths. The classes as per Table 4 are suggested.

## Partial Safety Factors

The partial safety factors of the construction materials are preset with the values as specified in DIN 1052, Table 1. In design situations resulting from earthquakes, the factors of the permanent and temporary design situation apply in accordance with DIN 4149:2005, Chapter 8.1.3 (3). For accidental and special combinations the factor 1 is assumed.

The partial safety factors for the actions are specified in the definition of the actions based on DIN 1055-100:2001, Table A.3.

## Equivalent Beam Length

For the buckling check with the equivalent beam method according to DIN 1052, Chapter 10.3, the equivalent beam lengths must be defined in the properties dialog of the concerned beams. The check is activated in the analysis settings dialog.

The screenshot shows a software dialog box titled "Properties for element 61 - Equiv. beam length". On the left is a tree view with the following items: Section, Form, Material, Bedding, EN 1995-1-1, Thermal analysis, General, Equiv. beam length (highlighted), Deformation check, and Eccentricity. The main area of the dialog contains the following fields and labels:

- Beam length:  m
- Equivalent beam length for buckling about the
- y-axis:  m
- z-axis:  m

At the bottom of the dialog are three buttons: "OK", "Cancel", and "Help".

### y-axis, z-axis

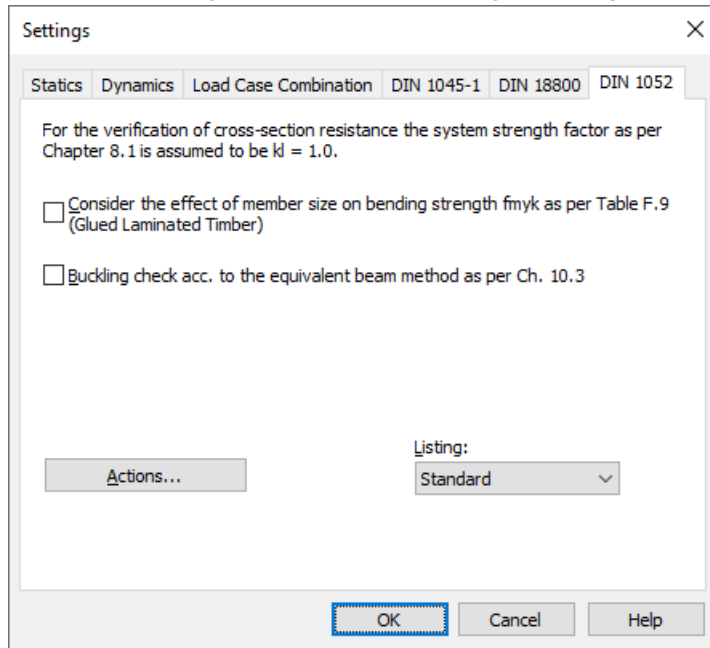
The specification of an appropriate equivalent beam length under consideration of support and load conditions is subject to the discretion and the responsibility of the user. If the value 0 is entered no check is carried out for the corresponding direction.

### Note

If a structural member with constant cross-section is divided into multiple beam elements, the buckling length decisive for the whole structural member shall be applied to all elements. A computational determination of buckling lengths and buckling eigenmodes by the FEM program is possible for selected load combinations (see description of *Load group*).

## Analysis Settings

The *DIN 1052* dialog field can be accessed using the *Settings* option of the *Analysis* menu.



### Consider the effect of member size

The bending strength of glued laminated timber beams can be increased by a factor  $k_h$  according to DIN 1052, Table F.9. The program uses this option for the strength  $f_{m,y,k}$ .

### Buckling check

The check with the equivalent beam method is carried out according to DIN 1052, Chapter 10.3. The equivalent beam length of the beams to be checked can be defined in the properties dialog.

### Actions...

Open the dialog for describing actions.

### Listing

- *No*: No log is generated by the design program.
- *Standard*: During analysis a log with a tabular output of the calculated utilizations is created.
- *Detailed*: This log differs from the standard log by additionally offering information on the analyzed internal force combinations.
- *Standard>permissible*: Standard log of the check locations at which the permissible limits are exceeded.
- *Detailed>permissible*: Detailed log of the check locations at which the permissible limits are exceeded.

## Ultimate Limit States

Depending on the user selection the timber checks are performed using the combinations of DIN 1055-100, Chapter 9.4, or the simplified rules of DIN 1052, Chapter 5.2, for the permanent and temporary design situations. For the remaining design situations the rules of DIN 1055-100 are decisive.

### Design Combinations according to DIN 1055-100

The following combinations in accordance with DIN 1055-100, Chapter 9.4, are taken into account in the ultimate limit states:

- Combination for permanent and temporary design situations

$$E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_P \cdot P_k \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \right\} \quad (14)$$

- Combination for accidental design situations

$$E \left\{ \sum_{j \geq 1} \gamma_{GA,j} \cdot G_{k,j} \oplus \gamma_{PA} \cdot P_k \oplus A_d \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (15)$$

- Combination for design situations caused by earthquakes

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \gamma_1 \cdot A_{Ed} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (16)$$

Additionally, for the results of a stability check according to the second-order theory the

- Special combination

$$E \left\{ F_{d,1} \right\}; \dots; E \left\{ F_{d,n} \right\}$$

is available. In this combination the design values of actions ( $F_d$ ) are superposed exclusively.

For each combination you can define different design situations for the construction stages and final states. Each check is performed for all situations of a combination. When conducting the check, the extreme value deriving from all combinations and situations is decisive. The partial safety factors and combination factors are defined in the Tables A.2 and A.3 of the standard.

### Design Combinations according to DIN 1052

The following simplified combinations for buildings in accordance with DIN 1052, Chapter 5.2, can be used for the permanent and temporary design situation:

$$E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus 1.5 \cdot Q_{k,1} \right\}; \dots; E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus 1.5 \cdot Q_{k,n} \right\} \quad (1)$$

$$E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus 1.35 \cdot \sum_{i \geq 1} Q_{k,i} \right\} \quad (2)$$

The most unfavorable value is decisive.



## Design Values according to the Second-Order Theory

The calculation according to the second-order theory as per Clause 8.5 is carried out as part of the internal forces calculation. For this you need to create – with the help of load groups – appropriate load cases from decisive loads (see also function *Combinations* in the action dialog).

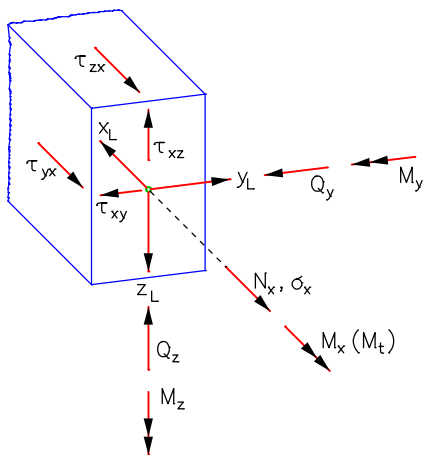
As the partial safety factors are already to be taken into account for the internal forces calculation, within the load groups the respective actions must be multiplied with their corresponding partial safety factor  $\gamma_G$  or  $\gamma_Q$  as well as with  $\gamma_M$  according to Table 1, without consideration of the load-duration.

In the check the load cases calculated according to the second-order theory are assigned to an action  $F_d$  and superposed in the *Special Combination* as an exclusive selection. The results are checked against the characteristic values of the cross-section resistances.

## Stress Determination

### Coordinate Systems

The following illustration shows the definition of the coordinate systems for internal forces, loads and stresses using the example of a beam with a rectangular profile. The section edge is defined as a closed polygon.



The longitudinal force  $N_x$  runs through the centroid, the lateral forces  $Q_y$  and  $Q_z$  run through the shear center of the section which, in this case, coincides with the centroid. The vectors of the moments  $M_x$ ,  $M_y$ ,  $M_z$  and stresses  $\sigma_x$ ,  $\tau_{xy}$ ,  $\tau_{xz}$  run parallel to the coordinate axes.

### Longitudinal Stresses

The necessary section properties for determination of the longitudinal stresses from longitudinal force and bending are determined elementarily with the segmentation method from the coordinates of the polygon.

More specifically, these are:

- $y_s, z_s$  Centroid coordinates.
- $A$  Area of the section.
- $I_y, I_z$  Moments of inertia in relation to the coordinate axes.
- $I_{yz}$  Deviation moment of inertia.
- $I_1, I_2$  Moments of inertia in relation to the main axes.
- $\varphi$  Twisting angle of the main axes.
- $W_y, W_z$  Moments of resistance for the calculation of the extremal longitudinal stresses from  $M_y, M_z$ .

For the analysis of a section, the stresses are determined at the automatically defined calculation points. The longitudinal stress  $\sigma$  for a point  $(y, z)$  of the section results from

$$\sigma(y, z) = N_x \frac{1}{A} + M_y \frac{\Delta z I_z - \Delta y I_{yz}}{I_y I_z - I_{yz}^2} + M_z \frac{\Delta y I_y - \Delta z I_{yz}}{I_y I_z - I_{yz}^2}$$

with  $\Delta y = y - y_s$  and  $\Delta z = z - z_s$ .

## Shear Stresses

### Lateral force

For load as a result of lateral force, the differential equation of the so-called shear warpage  $\omega_\tau$

$$\frac{\partial^2 \omega_\tau}{\partial y^2} + \frac{\partial^2 \omega_\tau}{\partial z^2} = -\frac{Q_z z}{G I_y} \quad \text{or} \quad -\frac{Q_y y}{G I_z}$$

is solved numerically with the help of the boundary element method. From this the following shear characteristics are determined:

$\tau_{Q_y,1}$  Shear stress for the lateral force  $Q_y = 1$ .

$\tau_{Q_z,1}$  Shear stress for the lateral force  $Q_z = 1$ .

$W_{qy}, W_{qz}$  Area values for calculation of the extremal shear stresses from  $Q_y$  and  $Q_z$  with  
 $\tau_{Q_y} = Q_y / W_{qy}$  and  $\tau_{Q_z} = Q_z / W_{qz}$ .

If selected in the Section dialog, the shear stresses across the section width will be averaged.

### Torsion

For load as a result of St. Venant torsion, the differential equation of the unit warpage  $\omega$

$$\frac{\partial^2 \omega}{\partial y^2} + \frac{\partial^2 \omega}{\partial z^2} = 0$$

is decisive. Its solution leads to the following characteristics:

$I_t$  Torsion moment of inertia.

$\tau_{M_x,1}$  Shear stress for the torsional moment  $M_x = 1$ .

$W_t$  Moment of resistance for the calculation of the extremal shear stress from  $M_x$ .

## Construction Material Properties

### Characteristic Values

For sections made of construction timber the characteristic material properties are taken from EN 338:2016, Table 1 and 3, for solid timber and from EN 14080:2013, Table 4 and 5, for glued laminated timber. In the check the partially deviating strengths according to DIN 1052, Table F.5, F.7 and F.9 are taken into account.

### Moisture Content and Load-Duration

The moisture content and the load-duration affect the strength and stiffness properties of timber according to Clause 7.1.3 (1). During design they are therefore to be taken into account for the mechanical resistance using a modification factor. The program considers this automatically depending on the user-defined properties.

### Service class

For the determination of the moisture content, structures shall be assigned to one of three service classes as described in Chapter 7.1.1. The assignment is made on the material property page of the section dialog. Therewith different service classes can be defined for specific parts of the structure.

The screenshot shows a software dialog box titled "Element Properties - Section - Material". On the left is a tree view with "Section" expanded, showing sub-items: "Form", "Material", "Bedding", "EN 1995-1-1", "Thermal analysis", and "Joints". The main area contains several input fields and dropdown menus:

- Number:** 1 (dropdown)
- Section Type:** Rectangle (dropdown)
- Material Type:** GL28H (dropdown)
- Buttons:** New, Copy, Delete (dropdown)
- Label:** (empty text box)
- E-Modulus:** 12600 [MN/m²] (text box)
- E-Modulus:** Ring direction: (empty text box)
- Nonlinear Analysis:**  Material linear
- G-Modulus:** 650 [MN/m²] (text box)
- Poisson's ratio:** (empty text box)
- Service class:** 1 (dropdown)
- Gamma** [kN/m³]: 4 (text box)
- Alpha.t** [1/K]: 5e-06 (text box)
- Raw density** [kg/m³]: 425 (text box)
- Buttons:** OK, Cancel, Help

### Load-duration

In accordance with Chapter 7.1.2 the actions are to be assigned to classes as per Table 3 to determinate the strength properties. The assignment is made within the action dialog where the classification according to Table 4 is suggested.

### Modification factor $k_{\text{mod}}$

The modification factor takes into account the effect of the moisture content and the load-duration on the material strengths. Numerical values for the factor  $k_{\text{mod}}$  are given in Table F.1 of the standard. In the check of the cross-section resistance it is used for determination of the strengths according to Eq. (3):

$$X_d = k_{\text{mod}} \cdot X_k / \gamma_M \quad (3)$$

where

$X_d$  is the design value of the strength property.

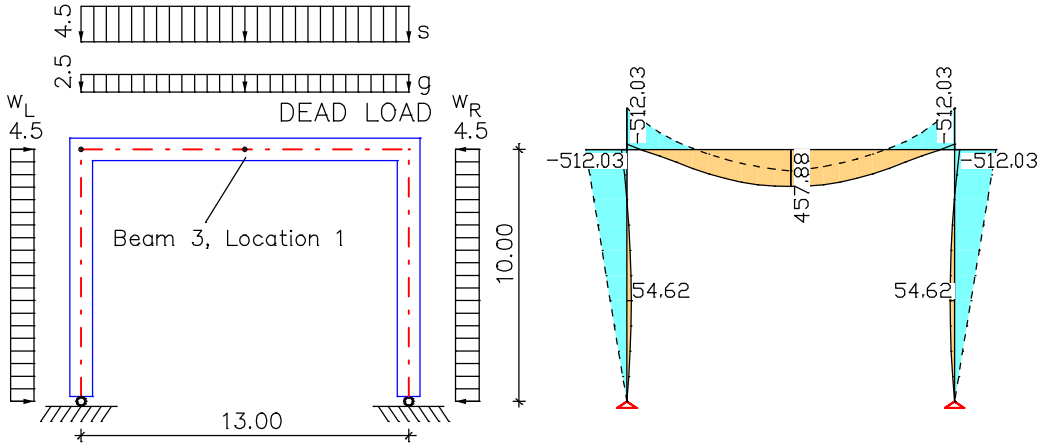
$X_k$  is the characteristic value of the strength property.

$\gamma_M$  is the partial safety factor of the material property.

If a load combination contains actions which belong to different load-duration classes, the value of  $k_{\text{mod}}$  corresponding to the action with the shortest duration should be used according to Clause 7.1.3 (1). The effect of the modification factor on the design value of the resistance can lead to the result in accordance with Clause 5.4 (1) that an action combination

becomes decisive which does not provide the maximum design value of the appropriate load. Because of that all possible combinations of load cases are analyzed in the checks.

Each set of internal forces which is analyzed provides the decisive load-duration from the involved actions. Together with the service class it determines the factor  $k_{mod}$  of the material. The following example illustrates the issue. Relevant facts in the listing are underlined.



Structure with loads Internal forces min/max My; 1. Permanent and temporary Situation

Action	Load cases	Label	Load-duration
G	1	Dead load, permanent load	Permanent
QS	2	Snow	Medium-term
QW	3	Wind from left	Short-term
	4	Wind from right	Short-term

**Location 1**

Beam 3: x = 0.00 m (Beam Length 6.50 m)  
 Section 1: Rectangle, GL32H, Class 1, h/w=900/300 mm, A=270000 mm<sup>2</sup>  
 km=0.70, khy/kht=1.00/1.07, kcr=0.67

**1. Permanent and temporary comb. (PC.1): G+QS+QW**

Relevant values from 16 sets of internal forces

Set	Nx[kN]	Qz[kN]	My[kNm]	Load-dur.	kmod
1	-29.77	-0.00	378.10	<u>Permanent</u>	<u>0.60</u>
5	-51.20	-25.96	403.77	<u>Sho.-term</u>	<u>0.90</u>

Load case combinations for the relevant sets of internal forces

Set	Combination
1	1.35* <u>L1</u>
5	1.35*L1+0.75*L2+1.50* <u>L3</u>

In the check both sets of internal forces presented in the log provide the maximum utilization for bending (set 1) and for lateral force (set 5). The set of internal forces with the maximum bending moment at the check location ( $M_y = 457.88$  kNm) is not decisive because of the effect of  $k_{mod}$  on the resistance.

**Effect of Member Size**

When the lamellas of glued laminated timber beams with  $h \leq 600$  mm are stressed from bending perpendicular to the smaller dimension ( $M_y$ ) the characteristic strength value  $f_{m,y,k}$  can be multiplied with the factor

$$k_h = \min((600/h)^{0.14}; 1.1)$$

according to Table F.9. The program considers this automatically if the user selects this option in the analysis settings. Thereby h [mm] is assumed to be the larger dimension of the section.

The option to increase the strength value in case of stress from bending perpendicular to the larger dimension ( $M_z$ ) is not used by the program.

## Cross-Section Checks

The orientation of the grains are assumed to be parallel to the beam axis. The check of tensile and compressive stresses is limited to this fiber direction. The system strength factor according to Chapter 8.1 is assumed to be  $k_1 = 1$ .

### Tension Parallel to the Grain

In accordance with Chapter 10.2.1 the following expression shall be satisfied:

$$\sigma_{t,0,d} / f_{t,0,d} \leq 1 \quad (43)$$

where

$\sigma_{t,0,d}$  is the design tensile stress along the grain.

$f_{t,0,d}$  is the design tensile strength along the grain.

### Compression Parallel to the Grain

In accordance with Chapter 10.2.3 the following expression shall be satisfied:

$$\sigma_{c,0,d} / f_{c,0,d} \leq 1 \quad (46)$$

where

$\sigma_{c,0,d}$  is the design compressive stress along the grain.

$f_{c,0,d}$  is the design compressive strength along the grain.

### Bending

In accordance with Chapter 10.2.6 the following expressions shall be satisfied:

$$\sigma_{m,y,d} / f_{m,y,d} + k_{red} \cdot \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (53)$$

$$k_{red} \cdot \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (54)$$

where

$\sigma_{m,y,d}$  and  $\sigma_{m,z,d}$  are the design bending stresses about the principal axes.

$f_{m,y,d}$  and  $f_{m,z,d}$  are the corresponding design bending strengths.

and also

$k_m = 0.7$  for rectangular sections made of solid timber, glued laminated timber and glued laminated beams with  $h/b \leq 4$ .

$k_m = 1.0$  for other cross-sections and other wood-based structural products.

### Combined Bending and Axial Tension

In accordance with Chapter 10.2.7 the following expressions shall be satisfied for the combined stresses from bending and tension:

$$\sigma_{t,0,d} / f_{t,0,d} + \sigma_{m,y,d} / f_{m,y,d} + k_{red} \cdot \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (55)$$

$$\sigma_{t,0,d} / f_{t,0,d} + k_{red} \cdot \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (56)$$

For the factor  $k_{red}$  the value given for bending applies.

## Combined Bending and Axial Compression

In accordance with Chapter 10.2.8 the following expressions shall be satisfied for the combined stresses from bending and compression:

$$(\sigma_{c,0,d}/f_{c,0,d})^2 + \sigma_{m,y,d}/f_{m,y,d} + k_{red} \cdot \sigma_{m,z,d}/f_{m,z,d} \leq 1 \quad (57)$$

$$(\sigma_{c,0,d}/f_{c,0,d})^2 + k_{red} \cdot \sigma_{m,y,d}/f_{m,y,d} + \sigma_{m,z,d}/f_{m,z,d} \leq 1 \quad (58)$$

For the factor  $k_{red}$  the value given for bending applies.

## Shear From Lateral Force

In accordance with Chapter 10.2.9 the following expression shall be satisfied:

$$\tau_d/f_{v,d} \leq 1 \quad (59)$$

The check is performed in case of biaxial bending for  $\tau_d = \sqrt{\tau_{y,d}^2 + \tau_{z,d}^2}$  and in all other cases for  $\tau_d = \max(\tau_{y,d}, \tau_{z,d})$ .

The following expression shall be satisfied for biaxial bending at rectangular sections according to Clause (5):

$$(\tau_{y,d}/f_{v,d})^2 + (\tau_{z,d}/f_{v,d})^2 \leq 1 \quad (60)$$

where

$\tau_{y,d}, \tau_{z,d}$  are the design shear stresses of the lateral forces in the y and z direction of the cross-section.

$f_{v,d}$  is the design shear strength.

The options of Clauses (2) to (4) are not used by the program.

## Torsion

In accordance with Chapter 10.2.10 the following expression shall be satisfied for torsional stresses:

$$\tau_{tor,d}/f_{v,d} \leq 1 \quad (61)$$

where

$\tau_{tor,d}$  is the design torsional stress.

$f_{v,d}$  is the design shear strength.

## Combined Shear and Torsion

The following expression shall be satisfied for the combination of shear from lateral force and torsion:

$$\tau_{tor,d}/f_{v,d} + (\tau_{y,d}/f_{v,d})^2 + (\tau_{z,d}/f_{v,d})^2 \leq 1 \quad (62)$$

## Buckling Check With Equivalent Beam Method

For the usage of the equivalent beam method the stability of structural members subjected to combined compression and bending is to be checked according to DIN 1052, Chapter 10.3. Thereby the following conditions must be satisfied:

$$(\sigma_{c,0,d} / (k_{c,y} \cdot f_{c,0,d}) + \sigma_{m,y,d} / (k_m \cdot f_{m,y,d}) + k_{red} \cdot \sigma_{m,z,d} / f_{m,z,d}) \leq 1 \quad (71)$$

$$(\sigma_{c,0,d} / (k_{c,z} \cdot f_{c,0,d}) + k_{red} \cdot \sigma_{m,y,d} / (k_m \cdot f_{m,y,d}) + \sigma_{m,z,d} / f_{m,z,d}) \leq 1 \quad (72)$$

where

$$k_c = \min \left( \frac{1}{k + \sqrt{k^2 - \lambda_{rel,c}^2}}; 1 \right) \quad \text{is the buckling factor} \quad (64)$$

$$k = 0.5 \cdot (1 + \beta_c (\lambda_{rel,c} - 0.3) + \lambda_{rel,c}^2) \quad (65)$$

$$\lambda_{rel,c} = \frac{\lambda}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \quad \text{is the relative slenderness ratio} \quad (66)$$

$\beta_c = 0.2$  for solid timber and glued laminated beams;

$\beta_c = 0.1$  for glued laminated timber and other wood-based structural products;

$\lambda = l_{ef} / i$  is the slenderness ratio;

$l_{ef}$  is the equivalent beam length;

$i$  is the radius of gyration;

$k_m$  is the tilt factor defined in Eq. (68) which is here assumed to be  $k_m = 1$ ;

$k_{red}$  is the factor defined in Chapter 10.2.6.

After the check is activated in the settings dialog it will be carried out for all beams with defined equivalent beam length.

## Serviceability Limit States

### Design Combinations

In accordance with DIN 1055-100, Chapter 10.4, the following combinations are taken into account in the serviceability limit states:

- Combination for rare (characteristic) situations

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus Q_{k,1} \oplus \sum_{i > 1} \psi_{0,i} \cdot Q_{k,i} \right\} \quad (22)$$

- Combination for frequent situations

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (23)$$

- Combination for quasi-continuous situations

$$E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \right\} \quad (24)$$

## Limiting Deformations

In accordance with Chapter 9.1 (1) the verification of serviceability can be performed by limiting the deformations. The check with the characteristic combination shall prevent damages on dividing walls and similar components; with the quasi-continuous combination the general usability and the appearance shall be ensured.

Recommended limitation values of deformations are given in Chapter 9.2 (4) whose compliance can be proven in the graphical and numerical deformation representation. In the folder *Node Deformations* of the database tree, the deformations of all decisive situations can be accessed.

## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The detailed log also lists the decisive combination internal forces of all design situations for each result location.

### Stresses

$\sigma_x, \sigma_y$	Extremal normal stresses from bending and normal force.
$\sigma_{xy}$	Extremal shear stresses from torsional moment and shear force.
$\tau_{xy}, \tau_{xz}$	Extremal shear stresses from lateral force and torsion.
$\sigma_v$	Maximum comparison stress.
$\sigma_1, \sigma_2$	Extremal principal stresses.

All stresses are given in [MN/m<sup>2</sup>].

### Utilizations

The utilization is defined as the ratio between the action  $E_d$  and the resistance  $R_d$  of a cross-section. In the folder *Stresses/Timber Checks* of the result tree the following results are available:

- Utilization of the beams for each situation.
- Maximal utilization of the beams of all situations.
- Maximal utilization of the sections of all situations.

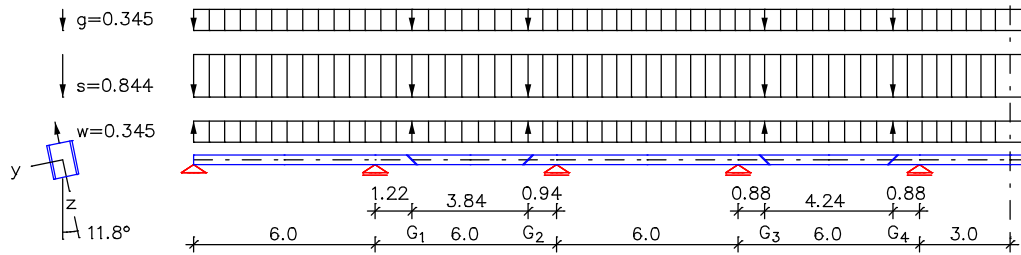


# Example Timber Checks on a Purlin With Joints

The example was chosen inspired by the book  
Gerhard Werner; Karlheinz Zimmer.

**Holzbau 2** - Dach- und Hallentragwerke nach DIN 1052 (neu 2008) und Eurocode 5.  
4., neu bearbeitete Auflage. Springer-Verlag, Berlin 2010.

A framework analysis and also timber checks are performed for the purlin illustrated below. The axes of the structure formed by a purlin with joints run parallel to the eave resp. the roof surface. Therefore the cross-section is rotated by 11.8 degrees. The inner spans of the purlin up to joint  $G_1$  are assigned to section 1 (rectangle 100/160). The outer spans of the purlin up to joint  $G_1$  are of section 2 (rectangle 120/160). Both sections are made of softwood of strength class C24 and assigned to service class 2.



Static system, dimensions [m] and loads [kN/m]

## Sum of installed loads and support reactions

LC.	Label	Fx [kN]	Fy [kN]	Fz [kN]
1	Dead load and roof cladding	0.000	0.000	9.315
	Support reactions	0.000	0.000	9.315
2	Snow load (short-term)	0.000	0.000	22.788
	Support reactions	0.000	-0.000	22.788
3	Wind load (short-term)	0.000	1.905	-9.118
	Support reactions	0.000	1.905	-9.118

Loads are to be assigned to actions like described in the following log to perform the checks. The checking program carries out the necessary internal force combinations and checks the cross-section resistance.

## Design per DIN 1052:2008 with Cor. 1:2010

The check of the cross-section resistance is performed as per Chapter 10.2.  
The system strength factor as per Chapter 8.1 is assumed to be  $k_1 = 1$ .

The action combinations are calculated according to DIN 1055-100, Chapter 9.4,  
with the Equations (14) to (16) considering the corresponding combination  
factors. All variations of cooperation of the variable actions are examined.

**Designing occurs for all possible combinations of actions.**

## Stress Calculation

The normal stresses and shear stresses are calculated for homogeneous  
polygon sections from normal force, bending moments, lateral force and torsion.  
The shear characteristics are calculated acc. to the Boundary-Element-Method.  
The calculation points for all stresses are edge points of the cross-section.

## Partial Safety Factors for Timber

	gamma.M
Permanent and temporary comb.	1.30
Accidental combination	1.00

## Characteristic Material Properties

Strength parameters as per DIN 1052:2008, Table F.5, F.7 and F.9 [MN/m<sup>2</sup>].

Material	f <sub>m</sub> k	f <sub>t0</sub> k	f <sub>c0</sub> k	f <sub>vk</sub>
C24	24.0	14.0	21.0	2.0

**Cross-Section Properties**

Cl.	Service class				
A	Sectional area [mm <sup>2</sup> ]				
d,h,b	Diameter, height, width [mm]				
Iy..Iyz	Moments of inertia [m <sup>4</sup> ]				
Cross-section	Material	Cl.	A	d,h	Iy Iz Ixz
1 Inner spans	C24	2	16000	160	3.4133e-05 3.2598e-05
2 Outer span	C24	2	19200	160	1.3333e-05 0.0000e+00
				120	4.0960e-05 4.9752e-05
					2.3040e-05 0.0000e+00

**DIN 1052 actions**

**Standard design group**

**G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load-duration: Permanent

Load cases

1 Dead load and roof cladding

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Load-duration: Short-term

Combination coefficients for: Superstructures

Snow and ice load - places to NN + 1000 m

Psi.0 / Psi.1 / Psi.2 = 0.5 / 0.2 / 0

Load cases 1. Variant, inclusive

2 Snow load (short-term)

**QW - Wind load**

Gamma.sup / gamma.inf = 1.5 / 0

Load-duration: Short-term

Combination coefficients for: Superstructures

Wind loads

Psi.0 / Psi.1 / Psi.2 = 0.6 / 0.5 / 0

Load cases 1. Variant, exclusive

3 Wind load (short-term)

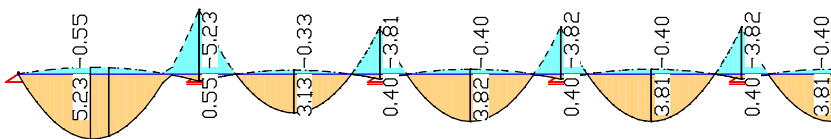
**1. Permanent and temporary situation**

Final state

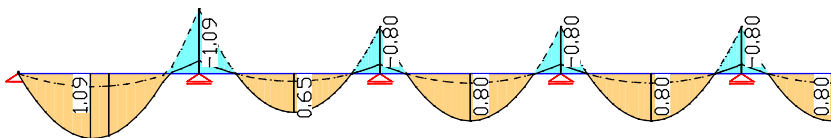
G - Dead load

QS - Snow and ice load

QW - Wind load



Bending moment min/max  $M_y$  [kNm]



Bending moment min/max  $M_z$  [kNm]

**Max. Cross-Section Utilization**

Utilization Relation between the design values of stress and strength.

Tension Tension parallel to the grain as per Eq. (43).

Compression Compression parallel to the grain as per Eq. (46).

Bending Bending as per Eq. (53) and (54).

Bend.,Tens. Bending and axial tension as per Eq. (55) and (56).

Bend.,Comp. Bending and axial compression as per Eq. (57) and (58).

Shear Shear as per Eq. (59) and (60).

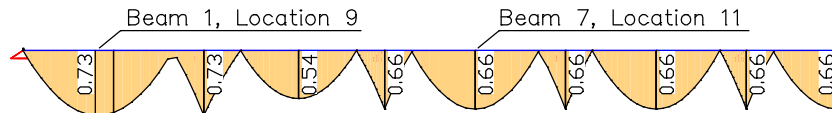
Torsion Torsion as per Eq. (61).

Shear,Tors. Shear and Torsion as per Eq. (62).

x Distance from the beam startpoint [m].

PC Permanent and temporary comb.

Cross-section	Material	Result	Beam Loc.	x[m]	Comb.	Util.	
1 Inner spans	C24	Tension					
		Compression					
		Bending	7	11	3.00	PC.1	0.66
		Bending, Tens.					
		Bending, Comp.					
		Shear	8	11	3.00	PC.1	0.35
		Torsion					
		Shear, Torsion					
2 Outer span	C24	Tension					
		Compression					
		Bending	1	9	2.40	PC.1	0.73
		Bending, Tens.					
		Bending, Comp.					
		Shear	2	11	3.00	PC.1	0.34
		Torsion					
		Shear, Torsion					



Utilization resulting from check 'Bending'

The following pages contain excerpts from the detailed check log for beam 1 at location 9.

**Check of the Cross-Section Resistance for Beams**

The results represent the extrema of all combinations.

Utilization Relation between the design values of stress and strength.  
 kmod Modif. factor for load-duration and moisture content, Eq. (3).  
 kred Reduction factor for the bending stress as per Chapter 10.2.6(1).  
 ft0d Axial tensile strength =  $kmod \cdot ft0k / \gamma.M$  [MN/m<sup>2</sup>].  
 fc0d Axial compressive strength =  $kmod \cdot fc0k / \gamma.M$  [MN/m<sup>2</sup>].  
 fmd Bending strength =  $kmod \cdot fmk / \gamma.M$  [MN/m<sup>2</sup>].  
 fvd Strength for lat. force & torsion =  $kmod \cdot fvk / \gamma.M$  [MN/m<sup>2</sup>].  
 x Distance from the beam startpoint [m].  
 y, z Location in the sectional polygon [m].

**Location 9**

Beam 1: x = 2.40 m (Beam Length 3.00 m)  
 Cross-section 2: Outer span, C24; Class 2; h/w=160/120 mm; A=19200 mm<sup>2</sup>; kred=0.70

**1. Permanent and temporary comb. (PC.1): G+QS+QW**

Relevant values from 10 sets of internal forces  
 Set Nx[kN] Qy[kN] Qz[kN] Mx[kNm] My[kNm] Mz[kNm] Load-dur. kmod  
 3 : 0.00 0.03 0.15 -0.00 5.23 1.09 Sho.-term 0.90

Load case combination for the relevant set of internal forces  
 Set Combination  
 3 : 1.35\*L1+1.50\*L2

**Check for Bending**

Max. Utilization as per Equation (53) and (54) : 0.73  
 cor. Sigma.myd / fmd : -10.217 / 16.615 : 0.61  
 Sigma.mzd / fmd : -2.846 / 16.615 : 0.17  
 S.point y / z [m] : 0.000 / 0.000  
 Situation / Set : PC.1 / 3

**Check for Shear**

Max. Utilization as per Equation (59) and (60) : 0.01  
 cor. Tau.yd / fvd : 0.002 / 1.385 : 0.00  
 Tau.zd / fvd : 0.011 / 1.385 : 0.01  
 Tau.d / fvd : 0.012 / 1.385 : 0.01  
 S.point y / z [m] : 0.060 / 0.080  
 Situation / Set : PC.1 / 3

## References

DIN 1052:2008-12

Entwurf, Berechnung und Bemessung von Holzbauwerken –  
Allgemeine Bemessungsregeln und Bemessungsregeln für den Hochbau.  
(Design of Timber Structures – Common Rules and Rules for Buildings).  
Beuth Verlag GmbH, Berlin 2008.

DIN 1052 Berichtigung 1:2010-05 (Corrigendum 1:2010-05)

Entwurf, Berechnung und Bemessung von Holzbauwerken –  
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DIN 1055-100:2001

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Teil 100: Grundlagen der Tragwerksplanung, Sicherheitskonzept und Bemessungsregeln.  
(Actions on Structures. Part 100: Basis of Design, Safety Concept and Design Rules).  
Beuth Verlag GmbH, Berlin 2003.

EN 338:2016

Structural timber – Strength classes  
Publisher: CEN European Committee for Standardization, Brussels.  
Beuth Verlag, Berlin 2016.

EN 14080:2013

Timber structures – Glued laminated timber and glued solid timber – Requirements.  
Publisher: CEN European Committee for Standardization, Brussels.  
Beuth Verlag, Berlin 2013.

Werner, G.; Zimmer, K.

Holzbau 2 – Dach- und Hallentragwerke nach DIN 1052 (neu 2008) und Eurocode 5.  
(Timber Structures 2 – Roof and Hall Structures According to DIN 1052 (new 2008) and Eurocode 5).  
4., neu bearbeitete Auflage. (4th newly reworked edition).  
Springer Verlag, Berlin 2010.

# EN 1995-1-1 Timber Checks

## Basics

The timber checks according to EN 1995-1-1 (Eurocode 5) can be used for buildings and engineering constructions for the design at normal temperature and under fire conditions with observance of the following standards:

- EN 1995-1-1:2014 and EN 1995-1-2:2009 (Basic document)
- DIN EN 1995-1-1/NA:2013-08 and DIN EN 1995-1-2/NA:2010-12 (Germany)
- OENORM B 1995-1-1:2015-06 and OENORM B 1995-1-2:2011-09 (Austria)
- SS EN 1995-1-1/BFS:2019 and SS EN 1995-1-2/BFS:2019 (EKS 11, Sweden)
- BS EN 1995-1-1/NA:2009-10 and BS EN 1995-1-2/NA:2006-10 (Great Britain)

The desired rule is selected in the *Settings* dialog in the *Analysis* menu. When selecting the material the following alternatives are available:

- C14 to C50 for softwood as per EN 388:2016, Table 1
- D18 to D80 for hardwood as per EN 388:2016, Table 3
- GL20h to GL32h for homogeneous glued laminated timber as per EN 14080:2013, Table 5
- GL20c to GL32c for combined glued laminated timber as per EN 14080:2013, Table 4
- GL75 for glued laminated timber made of construction beech (BauBuche) as per ETA-14/0354:2018, Tables 2 and 3
- In Germany for glued laminated timber the application rules of DIN 20000-3:2015 are decisive.

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with EN 1991-1-1:2002/AC:2009 and EN 1991-1-2:2002/AC:2009 (Eurocode 1). The program will take into account the preset safety factors and combination coefficients defined in EN 1990:2021 (Eurocode 0) for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements. For every set of internal forces the modification factor  $k_{mod}$  is automatically determined from the service class of the section and the decisive load-duration.

Rectangle sections and polygon sections can be used as cross-sections. Beams with the *Beam* section type are not checked as the section geometry is not known for them.

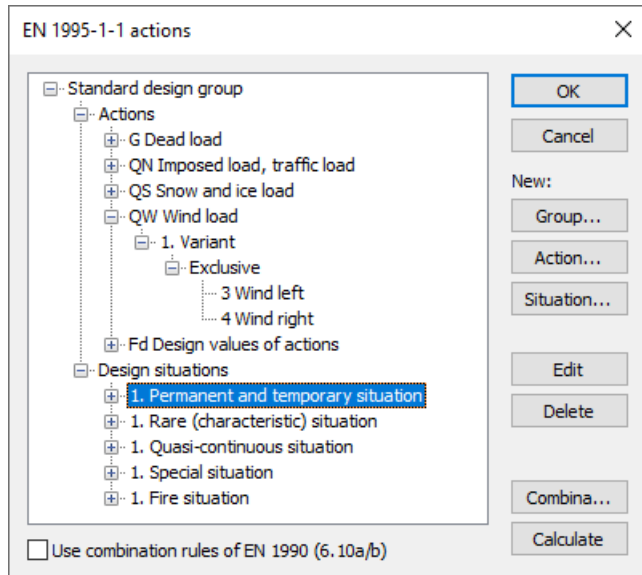
For area sections, the extremal internal forces for the defined action combinations are determined without performing a check and saved in the database for graphical representation.

The EN 1995-1-1 and EN 1995-1-2 guidelines are primarily cited for the following descriptions. Reference to the relevant national settings is only made if they contain different or complementary rules. The passages in question are marked by a vertical line left of the text.

# Input

## Actions and Design Situations

The load design values are calculated based on the internal forces of individual load cases and load case combinations. To do so, the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations. The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



### Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)

The assigned load cases should contain a design-relevant set of loads with combination coefficients and partial safety factors for actions and material such as for example a load group for the stability check according to EN 1995-1-1, Clause 2.2.2 (1)P. The selected load cases are combined exclusively.

### Group...

Open the dialog for entering a new design group. According to e.g. standard EN 1991-1-1, Chapter 6.2.2 (2), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

### Situation...

Open the dialog for entering new design situations.

### Edit

Open the Edit dialog for the selected action or situation.

### Delete

Delete the selected action or situation.

### Combinations...

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis.

The following example shows the total variants of the *permanent and temporary situation* according to Eq. (6.10) to be examined with the load cases (L1...L6) involved and their weighting factors.

Actions	Load cases	$\gamma_{sup}$	$\gamma_{inf}$	$\Psi_0$
Dead load	1	1.35	1.0	-
Imposed load, traffic load	2, 3 (inclusive)	1.5	0	0.7
Wind load	4	1.5	0	0.6
$F_d$ Design values of actions	5, 6	1.0	1.0	-

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
1	1.35					
2	1.35	1.50	1.50	0.90		
3	1.35	1.50	1.50			
4	1.35	1.50		0.90		
5	1.35	1.50				
6	1.35		1.50	0.90		
7	1.35		1.50			
8	1.35	1.05	1.05	1.50		
9	1.35	1.05		1.50		
10	1.35		1.05	1.50		
11	1.35			1.50		
12	1.00					

OK Cancel Load group...

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
13	1.00	1.50	1.50	0.90		
14	1.00	1.50	1.50			
15	1.00	1.50		0.90		
16	1.00	1.50				
17	1.00		1.50	0.90		
18	1.00		1.50			
19	1.00	1.05	1.05	1.50		
20	1.00	1.05		1.50		
21	1.00		1.05	1.50		
22	1.00			1.50		
23					1.00	
24						1.00

OK Cancel Load group...

### Calculate

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to open the checking module. Each time you open the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

### Use combination rules of EN 1990 (6.10a/b)

Optionally the Eq. (6.10a/b) are used for the combination of the permanent and temporary situation, otherwise Eq. (6.10).

## Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.

### Label

User-defined label for the action.

### Gamma.sup, Gamma.inf

Partial safety factors  $\gamma_{sup}$  and  $\gamma_{inf}$

SS EN 1990:

The program suggests the partial safety factors as they result in accordance with Section A, Article 11, for safety class 3 from  $\gamma_d \cdot \gamma_{sup}$  with the reduction factor  $\gamma_d = 1.0$  as per Article 14. If required, lower safety classes can be taken into account entering lower values.

### Combination coefficients psi for:

Input fields for selecting the combination coefficients for variable actions. The button allows you to view and change the selected combination coefficients  $\psi_0$ ,  $\psi_1$  and  $\psi_2$ .

### Load cases

List of possible load cases or load case combinations. You can choose an item from the list by selecting it and clicking the button or by using drag & drop.

### Multi-select

Load cases and combinations can be added to the actions more than once.

### Exclusive variants

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the or buttons.

### Inclusive load cases

Selected load cases and combinations that can act simultaneously. Since the number of internal forces to be checked increases exponentially, especially with a large number of load cases, it may be useful to use a load case combination instead of load cases.

### Exclusive load cases

Selected load cases and combinations that exclude each other.

### Load-duration

The load-duration class according to EN 1995-1-1, Chapter 2.3.1.2, is used to determine the modification factor  $k_{mod}$  for the material strengths. The classes as per Table 2.2 with the national decisive values are suggested.



## Fire Exposures

The definition of fire exposures for the check method with reduced cross-section according to EN 1995-1-2, Chapter 4.2.2, is made in the section dialog. The check of the cross-section resistance can be activated in the analysis settings dialog and additionally requires the definition of a design combination for fire conditions.

### Sides exposed to fire

Determination of the sides with fire exposures. The z-axis of the beam points to the bottom and the y-axis to the right. Round sections are exposed along their whole perimeter.

### Duration $t$

Duration  $t$  of the fire exposure for the calculation of the notional charring depth  $d_{char,n}$  according to Eq. (3.2). In accordance with Chapter 3.4.2 unprotected surfaces throughout the time of fire exposure are assumed.

$$d_{char,n} = \beta_n \cdot t \quad (3.2)$$

### Charring rate $\beta_n$

Design value of the notional charring rate in accordance with Chapter 3.4.2 (2) for the calculation of the charring depth according to Eq. (3.2). The value given in Table 3.1 is preset.

### Note

Fire exposures for the check with the advanced calculation method as per EN 1995-1-2, Chapter 4.4, are entered at the dialog page 'Thermal analysis'. A description concerning this can be found in the manual section 'Structural Analysis for Fire Scenarios'.

## Partial Safety Factors

The partial safety factors of the construction materials are preset with the nationally applicable values as specified in EN 1995-1-1, Table 2.3. In design situations resulting from earthquakes, the factors of the permanent and temporary design situation apply in accordance with EN 1998-1:2004, Chapter 5.2.4 (2). For accidental and special combinations as well as for fire scenarios the factor 1 is assumed. The partial safety factors for the actions are specified in the definition of the actions based on EN 1990, Table A.1.2(B).

## Equivalent Beam Length

For the buckling check with the equivalent beam method according to EN 1995-1-1, Chapter 6.3.2, the equivalent beam lengths must be defined in the properties dialog of the concerned beams. The check is activated in the analysis settings dialog.

The screenshot shows a software dialog box titled "Properties for element 61 - Equiv. beam length". On the left is a tree view with the following items: Section, Form, Material, Bedding, EN 1995-1-1, Thermal analysis, General, Equiv. beam length (highlighted), Deformation check, and Eccentricity. The main area of the dialog contains the following fields and labels:

- Beam length:  m
- Equivalent beam length for buckling about the
- y-axis:  m
- z-axis:  m

At the bottom of the dialog are three buttons: "OK", "Cancel", and "Help".

### y-axis, z-axis

The specification of an appropriate equivalent beam length under consideration of support and load conditions is subject to the discretion and the responsibility of the user. If the value 0 is entered no check is carried out for the corresponding direction.

### Note

If a structural member with constant cross-section is divided into multiple beam elements, the buckling length decisive for the whole structural member shall be applied to all elements. A computational determination of buckling lengths and buckling eigenmodes by the FEM program is possible for selected load combinations (see description of *Load group*).

## Deformation Check

The specifications for the deformation check according EN 1995-1-1, Chapter 2.2.3, are made in the properties dialog of the beams. The check is activated in the analysis setting dialog.

Properties for element 1 - Deformation check

Section

- Form
- Material
- Bedding
- EN 1995-1-1
  - Thermal analysis
  - General
    - Equiv. beam length
    - Deformation check
    - Eccentricity

Ref. length: L =  m

Check direction:

Perm. instant. deflection: w.inst = L /

Perm. final deflection: w.fin = L /

OK Cancel Help

### Reference length

The reference length is the length of a field (in the case of a continuous girder, the distance between the supports) and should be the same for all beams of this field. It is needed for the calculation of the permissible deflection according to Chapter 7.2 (2). If  $L = 0$  is specified, no deformation check is performed for this beam.

### Check direction

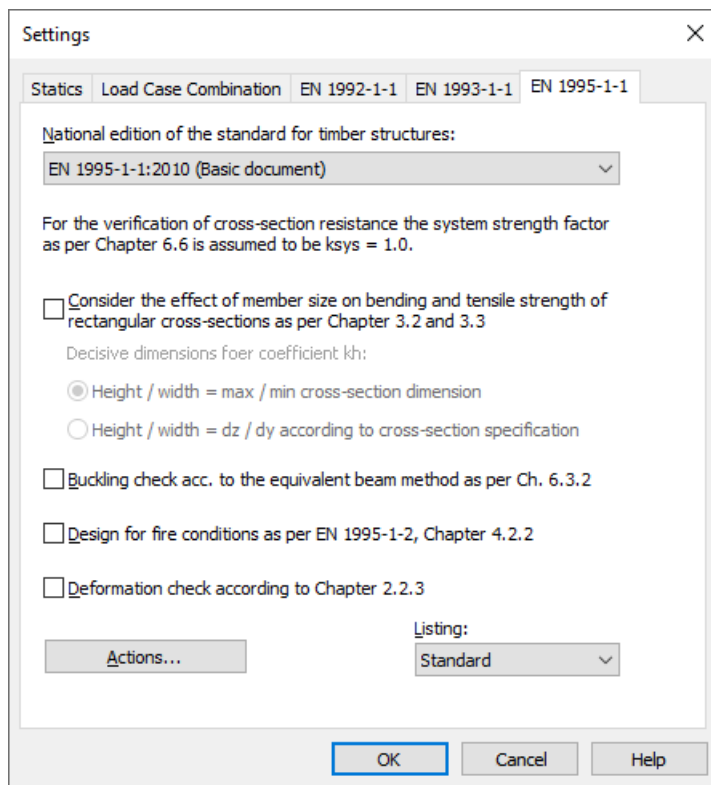
The deformation check can be performed for the local directions  $y$  and  $z$  or for the resulting direction  $r$ .

### Perm. instantaneous deflection, Perm. final deflection

User-defined limits for the deflection of bending beams according to Chapter 7.2 (2), Table 7.2.

## Analysis Settings

The *EN 1995-1-1* dialog field can be accessed using the *Settings* option of the *Analysis* menu.



### National edition of the standard

The edition you select will be used for all subsequent entries and calculations.

### Consider the effect of member size

The bending and tensile strength of solid timber and glued laminated timber can be increased by the factor  $k_h$  depending on the cross-section dimensions according to the rules of EN 1995-1-1, Chapters 3.2 and 3.3. The decisive dimension for the calculation of the factor  $k_h$  is selected by the user.

### Buckling check

The check with the equivalent beam method is carried out according to EN 1995-1-1, Chapter 6.3.2. The equivalent beam length of the beams to be checked can be defined in the properties dialog.

### Design for fire conditions

Rectangular and circular cross-sections are checked for the entered fire situation using the reduced cross-section according to EN 1995-1-2, Chapter 4.2.2. The fire exposure is described in the section dialog.

### Deformation check

The check according to EN 1995-1-1, Chapter 2.2.3 is performed at the calculation locations of the beams. A sufficient number of calculation locations should be considered. The specifications for the check are defined in the corresponding properties dialog of the beams.

### Actions...

Open the dialog for describing actions.

### Listing

- *No*: No log is generated by the design program.
- *Standard*: During analysis a log with a tabular output of the calculated utilizations is created.
- *Detailed*: This log differs from the standard log by additionally offering information on the analyzed internal force combinations.
- *Standard>permissible*: Standard log of the check locations at which the permissible limits are exceeded.
- *Detailed>permissible*: Detailed log of the check locations at which the permissible limits are exceeded.

# Ultimate Limit States

## Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.4.3, the following combinations are taken into account in the ultimate limit states:

- For the combination of the permanent and temporary design situation either Equation (6.10) or the most unfavorable equation from (6.10a) and (6.10b) is permitted.

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10)$$

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10a)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10b)$$

For the coefficient  $\xi$  the value of  $\xi = 0.85$  results from Table A.1.2(B).

DIN EN 1990, OENORM B 1990:

Equation (6.10) is used for the combination.

SS EN 1990:

Equations (6.10a) and (6.10b) apply with following modifications:

$$\sum_{j \geq 1} \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P \quad (6.10aSS)$$

$$\sum_{j \geq 1} \xi_j \cdot \gamma_d \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_d \cdot \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_d \cdot \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (6.10bSS)$$

Assuming reliability class 3, factor  $\gamma_d$  is set to 1. (see Section A, Article 11 and 14). The coefficient  $\xi$  is set to the value of  $\xi = 0.89$ .

BS EN 1990:

The coefficient  $\xi$  in Equation (6.10b) is set to the value of  $\xi = 0.925$ .

- Combination for accidental design situations

$$\sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.11b)$$

$\psi_{1,1} \cdot Q_{k,1}$  is used by the program for this combination.

OENORM B 1990-1:

$\psi_{2,1} \cdot Q_{k,1}$  is decisive.

- Combination for design situations caused by earthquakes

$$\sum_{j \geq 1} G_{k,j} + P + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.12b)$$

- Combination for design situations under fire conditions acc. to EN 1991-1-2 (Eurocode 1), Ch. 4.2.1

$$\sum_{j \geq 1} G_{k,j} + P + (\psi_{1,1} \text{ or } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i}$$

In accordance with EN 1991-1-2, Clause 4.3.1 (2), the value  $\psi_{2,1} \cdot Q_{k,1}$  is used by the program.

DIN EN 1991-1-2:

For wind actions  $\psi_{1,1} \cdot Q_{k,1}$ , otherwise  $\psi_{2,1} \cdot Q_{k,1}$  is to be used.

SS EN 1991-1-2, BS EN 1991-1-2:

The value  $\psi_{1,1} \cdot Q_{k,1}$  is decisive.

Additionally, for the results of a stability check according to the second-order theory the

- Special combination

$$F_{d,1}; \dots; F_{d,n}$$

is available. In this combination the design value of actions ( $F_{d,i}$ ) are superposed exclusively.

For each combination you can define different design situations for the construction stages and final states. Each check is performed for all situations of a combination.

## Design Values According to the Second-Order Theory

The calculation according to the second-order theory as per Clause 2.2.2 (1)P is carried out as part of the internal forces calculation considering the cross-section properties at normal temperature. For this you need to create – with the help of load groups – appropriate load cases from decisive loads (see also function *Combinations* in the action dialog).

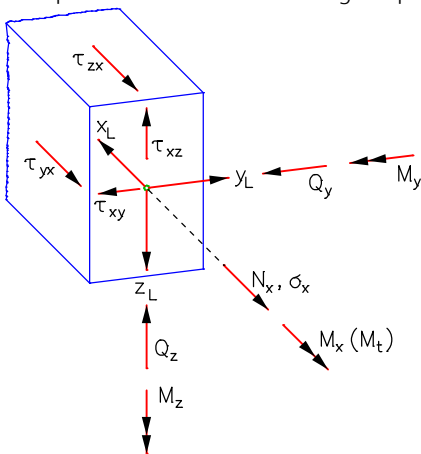
As the partial safety factors are already to be taken into account for the internal forces calculation, within the load groups the respective actions must be multiplied with their corresponding partial safety factor  $\gamma_G$  or  $\gamma_Q$  as well as with  $\gamma_M$  according to Table 2.3, without consideration of the load-duration.

In the check the load cases calculated according to the second-order theory are assigned to an action  $F_d$  and superposed in the *Special Combination* as an exclusive selection. The results are checked against the characteristic values of the cross-section resistances.

## Stress Determination

### Coordinate Systems

The following illustration shows the definition of the coordinate systems for internal forces, loads and stresses using the example of a beam with a rectangular profile. The section edge is defined as a closed polygon.



The longitudinal force  $N_x$  runs through the centroid, the lateral forces  $Q_y$  and  $Q_z$  run through the shear center of the section which, in this case, coincides with the centroid. The vectors of the moments  $M_x$ ,  $M_y$ ,  $M_z$  and stresses  $\sigma_x$ ,  $\tau_{xy}$ ,  $\tau_{xz}$  run parallel to the coordinate axes.

## Longitudinal Stresses

The necessary section properties for determination of the longitudinal stresses from longitudinal force and bending are determined elementarily with the segmentation method from the coordinates of the polygon.

More specifically, these are:

$y_s, z_s$	Centroid coordinates.
$A$	Area of the section.
$I_y, I_z$	Moments of inertia in relation to the coordinate axes.
$I_{yz}$	Deviation moment of inertia.
$I_1, I_2$	Moments of inertia in relation to the main axes.
$\varphi$	Twisting angle of the main axes.
$W_y, W_z$	Moments of resistance for the calculation of the extremal longitudinal stresses from $M_y, M_z$ .

For the analysis of a section, the stresses are determined at the automatically defined calculation points. The longitudinal stress  $\sigma$  for a point  $(y, z)$  of the section results from

$$\sigma(y, z) = N_x \frac{1}{A} + M_y \frac{\Delta z I_z - \Delta y I_{yz}}{I_y I_z - I_{yz}^2} + M_z \frac{\Delta y I_y - \Delta z I_{yz}}{I_y I_z - I_{yz}^2}$$

with  $\Delta y = y - y_s$  and  $\Delta z = z - z_s$ .

## Shear Stresses

### Lateral force

For load as a result of lateral force, the differential equation of the so-called shear warpage  $\omega_\tau$

$$\frac{\partial^2 \omega_\tau}{\partial y^2} + \frac{\partial^2 \omega_\tau}{\partial z^2} = -\frac{Q_z z}{G I_y} \quad \text{or} \quad -\frac{Q_y y}{G I_z}$$

is solved numerically with the help of the boundary element method. From this the following shear characteristics are determined:

$\tau_{Q_y,1}$	Shear stress for the lateral force $Q_y = 1$ .
$\tau_{Q_z,1}$	Shear stress for the lateral force $Q_z = 1$ .
$W_{qy}, W_{qz}$	Area values for calculation of the extremal shear stresses from $Q_y$ and $Q_z$ with $\tau_{Q_y} = Q_y / W_{qy}$ and $\tau_{Q_z} = Q_z / W_{qz}$ .

If selected in the Section dialog, the shear stresses across the section width will be averaged.

### Torsion

For load as a result of St. Venant torsion, the differential equation of the unit warpage  $\omega$

$$\frac{\partial^2 \omega}{\partial y^2} + \frac{\partial^2 \omega}{\partial z^2} = 0$$

is decisive. Its solution leads to the following characteristics:

$I_t$	Torsion moment of inertia.
$\tau_{M_x,1}$	Shear stress for the torsional moment $M_x = 1$ .
$W_t$	Moment of resistance for the calculation of the extremal shear stress from $M_x$ .

## Construction Material Properties

### Characteristic Values

For sections made of construction timber the characteristic material properties are taken from EN 338:2016, Tables 1 and 3, for solid timber and from EN 14080:2013, Tables 4 and 5, for glued laminated timber.

DIN EN 1995-1-1:

In accordance with Chapter 3.3 (NA.10) for all strength classes of glued laminated timber made of softwood the characteristic shear strength is set to  $f_{v,k} = 3.5 \text{ N/mm}^2$ .

OENORM B 1995-1-1:

In accordance with Clause 6.1.7 (2) a shear strength of  $f_{v,k} = 2.3 \text{ N/mm}^2$  is assumed for solid timber made of softwood and  $f_{v,k} = 2.5 \text{ N/mm}^2$  for glued laminated timber made of softwood.

ETA-14/0354:

Glued laminated timber made of construction beech (BauBuche) GL75 is approved for service classes 1 and 2. The characteristic values according to Tables 2 and 3 of the ETA apply.

### Moisture Content and Load-Duration

The moisture content and the load-duration affect the strength and stiffness properties of timber according to Clause 2.3.1.1 (2) P. During design they are therefore to be taken into account for the mechanical resistance using a modification factor. The program considers this automatically depending on the user-defined properties. When performing the check for fire conditions with reduced cross-section, the modification factor is assumed to be 1.0 according to EN 1995-1-2, Clause 4.2.2 (5).

### Service class

For the determination of the moisture content, structures shall be assigned to one of three service classes as described in Chapter 2.3.1.3. The assignment is made on the material property page of the section dialog. Therewith different service classes can be defined for specific parts of the structure.

The screenshot shows a software dialog box titled "Element Properties - Section - Material". On the left is a tree view with "Section" expanded, showing sub-items: "Form", "Material", "Bedding", "EN 1995-1-1", "Thermal analysis", and "Joints". The main area contains the following fields and controls:

- Number:** 1 (dropdown)
- Section Type:** Rectangle (dropdown)
- Material Type:** GL28H (dropdown)
- Buttons: New, Copy, Delete
- Label:** (empty text box)
- E-Modulus:** 12600 [MN/m²] (input box)
- E-Modulus:** Ring direction: (input box)
- G-Modulus:** 650 [MN/m²] (input box)
- Poisson's ratio:** (input box)
- Service class:** 1 (dropdown)
- Nonlinear Analysis:**  Material linear
- Gamma [kN/m³]:** 4 (input box)
- Alpha.t [1/K]:** 5e-06 (input box)
- Raw density [kg/m³]:** 425 (input box)
- Buttons: OK, Cancel, Help

### Load-duration

In accordance with Chapter 2.3.1.2 the actions are to be assigned to classes as per Table 2.1 to determinate the strength properties. The assignment is made within the action dialog where the classification according to Table 2.2 is suggested.



**Modification factor  $k_{mod}$**

The modification factor takes into account the effect of the moisture content and the load-duration on the material strengths. Numerical values for the factor  $k_{mod}$  are given in Table 3.1 of the standard. In the check of the cross-section resistance it is used for determination of the resistance according to Eq. (2.17):

$$R_d = k_{mod} \cdot R_k / \gamma_M \tag{2.17}$$

where

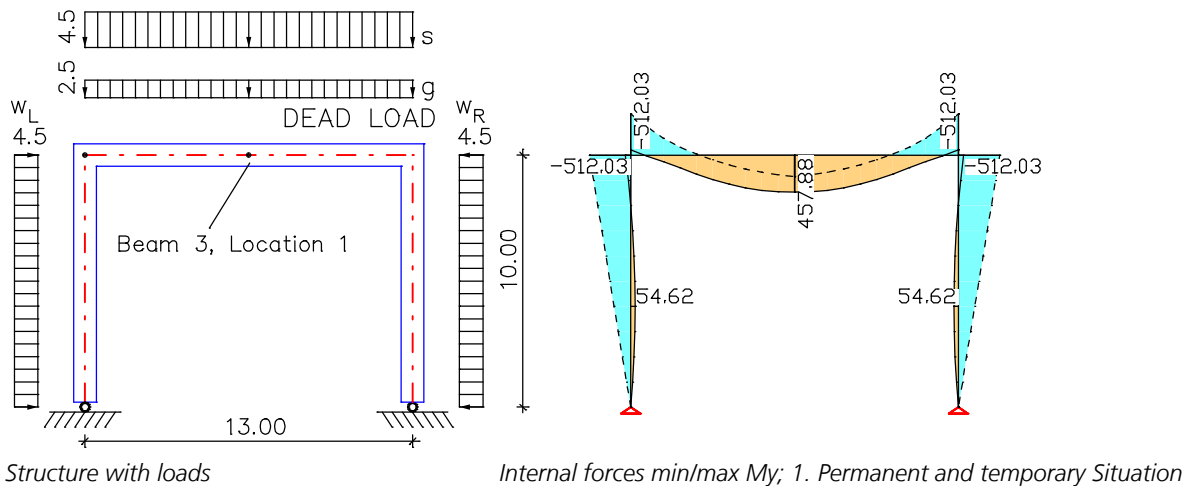
$R_d$  is the design value of the resistance (load-bearing capacity).

$R_k$  is the characteristic value of the resistance.

$\gamma_M$  is the partial safety factor of the material property.

If a load combination contains actions which belong to different load-duration classes, the value of  $k_{mod}$  corresponding to the action with the shortest duration should be used according to Clause 3.1.3 (2). The effect of the modification factor on the design value of the resistance can lead to the result that an action combination becomes decisive which does not provide the maximum design value of the appropriate load. Because of that all possible combinations of load cases are analyzed in the checks.

Each set of internal forces which is analyzed provides the decisive load-duration from the involved actions. Together with the service class it determines the factor  $k_{mod}$  of the material. The following example illustrates the issue. Relevant facts in the listing are underlined.



Action	Load cases	Label	Load-duration
G	1	Dead load, permanent load	Permanent
QS	2	Snow	Medium-term
QW	3	Wind from left	Short-term
	4	Wind from right	Short-term

**Location 1**

Beam 3:  $x = 0.00$  m (Beam Length 6.50 m)  
 Section 1: Rectangle, GL32H, Class 1,  $h/w=900/300$  mm,  $A=270000$  mm<sup>2</sup>  
 $k_m=0.70$ ,  $k_{hy}/k_{ht}=1.00/1.07$ ,  $k_{cr}=0.67$

**1. Permanent and temporary comb. (PC.1): G+QS+QW**

Relevant values from 16 sets of internal forces

Set	Nx [kN]	Qz [kN]	My [kNm]	Load-dur.	$k_{mod}$
1	-29.77	-0.00	378.10	<u>Permanent</u>	<u>0.60</u>
5	-51.20	-25.96	403.77	<u>Sho.-term</u>	<u>0.90</u>

Load case combinations for the relevant sets of internal forces

Set	Combination
1	$1.35 * \underline{L1}$
5	$1.35 * \underline{L1} + 0.75 * \underline{L2} + 1.50 * \underline{L3}$

In the check both sets of internal forces presented in the log provide the maximum utilization for bending (set 1) and for lateral force (set 5). The set of internal forces with the maximum bending moment at the check location ( $M_y = 457.88$  kNm) is not decisive because of the effect of  $k_{mod}$  on the resistance.

## Effect of Member Size

The effect of member size on strength can be taken into account for solid timber and glued laminated timber according to Chapter 3.2 and 3.3. The program considers this automatically if the user selects this option in the analysis settings.

If the decisive dimension of the cross-section  $h$  [mm] falls below the material-specific reference value the characteristic bending strength  $f_{m,k}$  resp. the tensile strength  $f_{t,k}$  can be increased by a factor  $k_h$ . According to the selection in the analysis settings, the decisive dimension for the bending strength is assumed either with the largest cross-sectional dimension (standard) or with the dimension in the  $z$  direction ( $d_z$ ) specified by the cross-section. For the tensile strength, either the smallest cross-sectional dimension (standard) or the dimension in  $y$  direction ( $d_y$ ) is used.

### Solid timber

For solid timber with rectangular cross-section and a characteristic density  $\rho_k \leq 700 \text{ kg/m}^3$  the reference value is 150 mm according to Chapter 3.2 and the factor is

$$k_h = \min((150/h)^{0.2}; 1.3) \quad (3.1)$$

DIN EN 1995-1-1:

For the tensile strength the largest section dimension according to NCI of Chapter 3.2 (3) is always decisive.

### Glued laminated timber

For glued laminated timber with rectangular cross-section the reference value is 600 mm according to Chapter 3.3 and the factor is

$$k_h = \min((600/h)^{0.1}; 1.1) \quad (3.2)$$

DIN EN 1995-1-1:

For the tensile strength the largest section dimension according to NCI of Chapter 3.3 (3) is always decisive. An increase of the bending strength as per Eq. (3.2) is only applied for bending stress perpendicular to the smaller dimension ( $M_y$ ) according to Section (NA.9). The option to increase the strength value in case of stress from bending perpendicular to the larger dimension ( $M_z$ ) according to Chapter (NA.6) is not used by the program.

ETA-14/0354:

For glued laminated timber made of construction beech (BauBuche) GL75, the strengths are bindingly specified in Table 3 of the approval, taking into account the component size. Chapter 3.3 of the standard is therefore not applied in the program.

## Design Method for Fire Conditions

Structural fire design is carried out according to the reduced cross-section method as described in EN 1995-1-2, Chapter 4.2.2. The corresponding property page of the section dialog allows to define fire exposures for the affected rectangular and round cross-sections. The check of the cross-section resistance can be activated in the analysis settings dialog and additionally requires entering a design situation for fire conditions.

The residual cross-section is determined by reducing the initial cross-section about the charring depth  $d_{ef}$  according to Eq. (4.1) and is verified to fulfill the condition for bracing members in Clause 4.3.5 (2).

$$d_{ef} = d_{char,n} + k_0 \cdot d_0 \quad (4.1)$$

$$d_{char,n} = \beta_n \cdot t \quad (3.2)$$

where

$$d_0 = 7 \text{ mm};$$

$k_0$  is the factor according to Table 4.1;

$d_{char,n}$  is the notional design charring depth according to Eq. (3.2);

$\beta_n$  is the notional charring rate according to Chapter 3.4.2 (2).

Under fire conditions the cross-section checks described below use the strengths defined in EN 1995-1-2, Eq. (2.1).

### Note:

Alternatively to the described method with reduced cross-section, the check can be performed in accordance with EN 1995-1-2, Chapter 4.4, using the 'Advanced calculation method'. For more information on this, refer to section 'Structural Analysis for Fire Scenarios' of the manual.

## Cross-Section Checks

In accordance with EN 1995-1-1, Chapter 6.1.1 the orientation of the grains are assumed to be parallel to the beam axis. The check of tensile and compressive stresses is limited to this fiber direction. The system strength factor according to Chapter 6.6 is assumed to be  $k_{\text{sys}} = 1$ .

### Design Values of Strengths

#### Normal temperature

The strengths at normal temperature result from EN 1995-1-1, Eq. (2.14):

$$f_d = k_{\text{mod}} \cdot f_k / \gamma_M \quad (2.14)$$

where

- $f_d$  is the design value of strength at normal temperature;
- $k_{\text{mod}}$  is the modification factor for load duration and moisture content as per Chapter 3.1.3;
- $f_k$  is the characteristic strength according to EN 338 and EN 14080;
- $\gamma_M$  is the partial safety factor for timber according to Table 2.3.

#### Fire conditions

The strengths in fire are defined in EN 1995-1-2, Eq. (2.1):

$$f_{d,fi} = k_{\text{mod},fi} \cdot f_{20} / \gamma_{M,fi} \quad (2.1)$$

where

- $f_{d,fi}$  is the design strength in fire;
- $k_{\text{mod},fi}$  is the modification factor for fire, which is assumed to be  $k_{\text{mod},fi} = 1$  for the check with reduced cross-section according to 4.2.2 (5);
- $f_{20}$  is the 20% fractile of a strength property at normal temperature with  $f_{20} = k_{fi} \cdot f_k$  acc. to Eq. (2.4);
- $k_{fi}$  is the factor given in table 2.1;
- $f_k$  is the characteristic strength according to EN 338 and EN 14080;
- $\gamma_{M,fi}$  is the partial safety factor for timber in fire which is assumed to be  $\gamma_{M,fi} = 1$ .

### Tension Parallel to the Grain

In accordance with Chapter 6.1.2 the following expression shall be satisfied:

$$\sigma_{t,0,d} \leq f_{t,0,d} \quad (6.1)$$

where

- $\sigma_{t,0,d}$  is the design tensile stress along the grain.
- $f_{t,0,d}$  is the design tensile strength along the grain.

### Compression Parallel to the Grain

In accordance with Chapter 6.1.4 the following expression shall be satisfied:

$$\sigma_{c,0,d} \leq f_{c,0,d} \quad (6.2)$$

where

- $\sigma_{c,0,d}$  is the design compressive stress along the grain.
- $f_{c,0,d}$  is the design compressive strength along the grain.

## Bending

In accordance with Chapter 6.1.6 the following expressions shall be satisfied:

$$\sigma_{m,y,d} / f_{m,y,d} + k_m \cdot \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (6.11)$$

$$k_m \cdot \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (6.12)$$

where

$\sigma_{m,y,d}$  and  $\sigma_{m,z,d}$  are the design bending stresses about the principal axes.

$f_{m,y,d}$  and  $f_{m,z,d}$  are the corresponding design bending strengths.

The factor  $k_m$  makes allowance for the inhomogeneities of the material in a cross-section and is assumed as follows according to Clause 6.1.6 (2):

$k_m = 0.7$  for rectangular sections made of solid timber, glued laminated timber and laminated veneer lumber.

$k_m = 1.0$  for other cross-sections and other wood-based structural products.

## Combined Bending and Axial Tension

In accordance with Chapter 6.2.3 the following expressions shall be satisfied for the combined stresses from bending and tension:

$$\sigma_{t,0,d} / f_{t,0,d} + \sigma_{m,y,d} / f_{m,y,d} + k_m \cdot \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (6.17)$$

$$\sigma_{t,0,d} / f_{t,0,d} + k_m \cdot \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (6.18)$$

For the factor  $k_m$  the value given for bending applies.

## Combined Bending and Axial Compression

In accordance with Chapter 6.2.4 the following expressions shall be satisfied for the combined stresses from bending and compression:

$$(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} + k_m \cdot \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (6.19)$$

$$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \cdot \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d} \leq 1 \quad (6.20)$$

For the factor  $k_m$  the value given for bending applies.

## Shear

In accordance with Chapter 6.1.7 the following expression shall be satisfied:

$$\tau_d \leq f_{v,d} \quad (6.13)$$

The check is performed in case of biaxial bending for  $\tau_d = \sqrt{\tau_{y,d}^2 + \tau_{z,d}^2}$  and in all other cases for  $\tau_d = \max(\tau_{y,d}; \tau_{z,d})$ .

where

$\tau_{y,d}$ ,  $\tau_{z,d}$  are the design shear stresses of the lateral forces in the y and z direction of the cross-section.

$f_{v,d}$  is the design shear strength.

DIN EN 1995-1-1:

The following expression shall be satisfied for biaxial bending at rectangular sections:

$$(\tau_{y,d} / f_{v,d})^2 + (\tau_{z,d} / f_{v,d})^2 \leq 1 \quad (NA.55)$$

The effect of cracks on structural members stressed by bending shall be taken into account using the effective width  $b_{ef}$  of the member according to Clause 6.1.7 (2):

$$b_{ef} = k_{cr} \cdot b \quad (6.13a)$$

The reduction factor  $k_{cr}$  is assumed as follows:

$k_{cr} = 0.67$  for solid timber and glued laminated timber

$k_{cr} = 1.0$  for other wood-based structural products

DIN EN 1995-1-1:

The following factors apply:

$k_{cr} = 0.67$  for solid timber made of hardwood

$k_{cr} = 2.0 / f_{v,k}$  for solid timber and glued laminated beams made of softwood

$k_{cr} = 2.5 / f_{v,k}$  for glued laminated timber with  $f_{v,k}$  in N/mm<sup>2</sup>

For beams made from softwood timber, the values for  $k_{cr}$  may be increased by 30 % in areas at least 1.50 m from the end-grain of the wood.

OENORM B 1995-1-1:

The following factors apply:

$k_{cr} = 0.67$  for solid timber and glued laminated timber made of hardwood

$k_{cr} = 1.00$  for other wood-based structural products made of hardwood

$k_{cr} = 1.00$  for solid timber, glued laminated timber and other wood-based structural products made of softwood.

This value is applied for solid timber on the assumption of a characteristic shear strength  $f_{v,k} = 2.3$  N/mm<sup>2</sup> and for glued laminated timber on the assumption of  $f_{v,k} = 2.5$  N/mm<sup>2</sup>.

SS EN 1995-1-1:

The following factors apply:

$k_{cr} = 0.67$  for solid timber and glued laminated timber fully or partially exposed to precipitation and solar radiation.

The program assumes these prerequisites for service class 3.

$k_{cr} = 3.0 / f_{v,k}$  for solid timber and glued laminated timber in all other cases.

## Torsion

In accordance with Chapter 6.1.8 the following expression shall be satisfied for torsional stresses:

$$\tau_{\text{tor,d}} \leq k_{\text{shape}} \cdot f_{\text{v,d}} \quad (6.14)$$

with

$$k_{\text{shape}} = \begin{cases} 1.2 & \text{for a circular cross-section} \\ \min(1 + 0.05 \cdot h/b; 1.3) & \text{for a rectangular cross-section} \end{cases} \quad (6.15)$$

where

$\tau_{\text{tor,d}}$  is the design torsional stress.

$f_{\text{v,d}}$  is the design shear strength.

$k_{\text{shape}}$  is a factor depending on the shape of the cross-section.

$h$  is the larger cross-sectional dimension.

$b$  is the smaller cross-sectional dimension.

## Combined Shear and Torsion

DIN EN 1995-1-1 und OENORM B 1995-1-1:

The following expression shall be satisfied for the combination of shear from lateral force and torsion:

$$\tau_{\text{tor,d}} / (k_{\text{shape}} \cdot f_{\text{v,d}}) + (\tau_{\text{y,d}} / f_{\text{v,d}})^2 + (\tau_{\text{z,d}} / f_{\text{v,d}})^2 \leq 1 \quad (NA.56)$$

(NA.6.15-E1)

For the factor  $k_{\text{shape}}$  the value given for torsion applies.

## Buckling Check With Equivalent Beam Method

For the usage of the equivalent beam method the stability of structural members subjected to combined compression and bending is to be checked according to EN 1995-1-1, Chapter 6.3.2. Thereby the relative slenderness ratio should be taken as:

$$\lambda_{\text{rel},y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad (6.21)$$

and

$$\lambda_{\text{rel},z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad (6.22)$$

where

$\lambda_y, \lambda_{\text{rel},y}$  are the slenderness ratios for buckling about the y-axis;

$\lambda_z, \lambda_{\text{rel},z}$  are the slenderness ratios for buckling about the z-axis;

$\lambda = l_{\text{ef}} / i$

$l_{\text{ef}}$  is the equivalent beam length;

$i$  is the radius of gyration.

In accordance with 6.3.2 (2) if  $\lambda_{\text{rel},y} \leq 0.3$  or  $\lambda_{\text{rel},z} \leq 0.3$  the compression and bending stresses are checked according to Eq. (6.19) or (6.20). Otherwise the following equations are used:

$$(\sigma_{c,0,d} / (k_{c,y} \cdot f_{c,0,d}) + \sigma_{m,y,d} / (k_m \cdot f_{m,y,d}) + k_m \cdot \sigma_{m,z,d} / f_{m,z,d}) \leq 1 \quad (6.23)$$

$$(\sigma_{c,0,d} / (k_{c,z} \cdot f_{c,0,d}) + k_m \cdot \sigma_{m,y,d} / (k_m \cdot f_{m,y,d}) + \sigma_{m,z,d} / f_{m,z,d}) \leq 1 \quad (6.24)$$

where

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel},y}^2}} \quad (6.25)$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{\text{rel},z}^2}} \quad (6.26)$$

$$k_y = 0.5 \cdot (1 + \beta_c (\lambda_{\text{rel},y} - 0.3) + \lambda_{\text{rel},y}^2) \quad (6.27)$$

$$k_z = 0.5 \cdot (1 + \beta_c (\lambda_{\text{rel},z} - 0.3) + \lambda_{\text{rel},z}^2) \quad (6.28)$$

$\beta_c$  is a factor for imperfections defined in Section 10;

$\beta_c = 0.2$  for solid timber; (6.29)

0.1 for glued laminated timber and LVL;

$k_m$  is a factor according to 6.1.6 (2).

After the check is activated in the settings dialog it will be carried out for all beams with defined equivalent beam length.

## Serviceability Limit States

### Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.5.3, the following combinations are taken into account in the serviceability limit states:

- Combination for characteristic situations

$$\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i > 1} \psi_{0,i} \cdot Q_{k,i} \quad (6.14b)$$

- Combination for frequent situations

$$\sum_{j \geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.15b)$$

- Combination for quasi-continuous situations

$$\sum_{j \geq 1} G_{k,j} + P + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \quad (6.16b)$$

### Limiting Deformations

In accordance with EN 1995-1-1, Chapter 7.2, the deformations of beams should not exceed the values in Table 7.2.

DIN EN 1995-1-1:

The limits given in table NA.13 apply.

OENORM B 1995-1-1:

The limits given in table NA.7.2 apply.

SS EN 1995-1-1:

In accordance with Article 8 the limitation values for the deflection are determined in every individual case depending on the respective circumstances.

The deformation check is selected in the analysis settings. The instantaneous deformation is calculated according to Chapter 2.2.3 (2) for the characteristic design situation. Deviating from the procedure in EN 1995-1-1, the final deformation is always calculated according to DIN EN 1995-1-1, Chapter 2.2.3 (NA.8), Equation (NA.1). It is based on the quasi-continuous design situation.

$$u_{\text{net,fin}} = \left( u_{\text{inst,G}} + \sum_{i \geq 1} \psi_{2,i} \cdot u_{\text{inst,Q,i}} \right) \cdot (1 + k_{\text{def}}) - u_c \quad (\text{NA.1})$$

OENORM B 1995-1-1:

The equation (NA.7.2) corresponding to the above equation is used.

For this purpose, the characteristic and quasi-continuous design situations are to be defined in the action dialog. The deformation coefficient  $k_{\text{def}}$  is determined automatically according to EN 1995-1-1, Table 3.2, depending on the timber material and the service class, defined in the cross section. An elevation of the component is not considered.

The deformations can be found in the folder *Beam deformations > Timber checks* of the result tree.



## Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The detailed log also lists the decisive combination internal forces of all design situations for each result location.

### Stresses

$\sigma_x, \sigma_y$	Extremal normal stresses from bending and normal force.
$\sigma_{xy}$	Extremal shear stresses from torsional moment and shear force.
$\tau_{xy}, \tau_{xz}$	Extremal shear stresses from lateral force and torsion.
$\sigma_v$	Maximum comparison stress.
$\sigma_1, \sigma_2$	Extremal principal stresses.

All stresses are given in [MN/m<sup>2</sup>].

### Utilizations

The utilization is defined as the ratio between the action  $E_d$  and the resistance  $R_d$  of a cross-section. In the folder *Stresses/Timber Checks* of the result tree the following results are available:

- Utilization of the beams for each situation.
- Maximal utilization of the beams of all situations.
- Maximal utilization of the sections of all situations.

# Examples

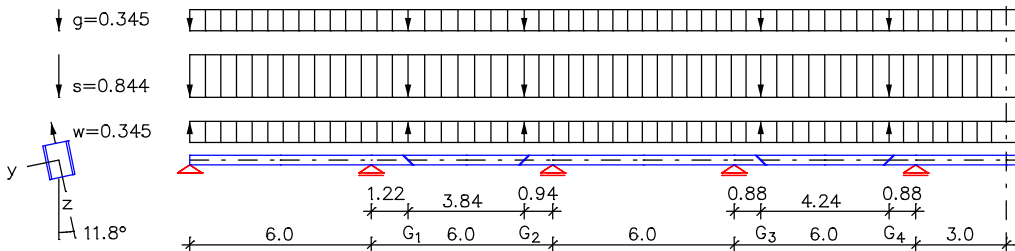
## Timber Checks on a Purlin With Joints

The example was chosen inspired by the book

Gerhard Werner; Karlheinz Zimmer.

**Holzbau 2** - Dach- und Hallentragwerke nach DIN 1052 (neu 2008) und Eurocode 5. 4., neu bearbeitete Auflage. Springer-Verlag, Berlin 2010.

A framework analysis and also timber checks are performed for the purlin illustrated below. The axes of the structure formed by a purlin with joints run parallel to the eave resp. the roof surface. Therefore the cross-section is rotated by 11.8 degrees. The inner spans of the purlin up to joint  $G_1$  are assigned to section 1 (rectangle 100/160). The outer spans of the purlin up to joint  $G_1$  are of section 2 (rectangle 120/160). Both sections are made of softwood of strength class C24 and assigned to service class 2.



Static system, dimensions [m] and loads [kN/m]

### Sum of installed loads and support reactions

LC.	Label	Fx [kN]	Fy [kN]	Fz [kN]
1	Dead load and roof cladding	0.000	0.000	9.315
	Support reactions	0.000	0.000	9.315
2	Snow load (short-term)	0.000	0.000	22.788
	Support reactions	0.000	-0.000	22.788
3	Wind load (short-term)	0.000	1.905	-9.118
	Support reactions	0.000	1.905	-9.118

Loads are to be assigned to actions like described in the following log to perform the checks. The checking program carries out the necessary internal force combinations and checks the cross-section resistance and the deformations.

### Design per EN 1995-1-1:2014

The check of the cross-section resistance is based on EN 1995-1-1, Chapter 6.1. The system strength factor as per Chapter 6.6 is assumed to be  $k_{sys} = 1$ . The effect of member size on strength is considered as per Chapter 3.2 and 3.3. The maximum cross-section dimension for the bending strength and the minimum for the tensile strength are used as the decisive dimension for the factor  $k_h$ .

The actions are combined acc. to EN 1990, Eq. (6.10), using the partial and combination factors according to EN 1990:2021.

**All variations of cooperation of the variable actions are examined.**

### Stress Calculation

The normal stresses and shear stresses are calculated for homogeneous polygon sections from normal force, bending moments, lateral force and torsion. The shear characteristics are calculated acc. to the Boundary-Element-Method. The calculation points for all stresses are edge points of the cross-section.

### Partial Safety Factors for Timber

	gamma.M
Permanent and temporary comb.	1.30 (Solid Timber) 1.25 (Glued Laminated Timber)
Accidental combination	1.00

### Characteristic Material Properties

Strength and density as per EN 338:2016 and EN 14080:2013 [MN/m<sup>2</sup>, kg/m<sup>3</sup>].

Material	f <sub>m</sub> k	f <sub>t</sub> 0k	f <sub>c</sub> 0k	f <sub>v</sub> k	ρ <sub>h</sub> 0k
C24	24.0	14.5	21.0	4.0	350.0

**Cross-Section Properties**

Cl.	Service class						
A	Sectional area [mm <sup>2</sup> ]						
d,h,b	Diameter, height, width [mm]						
Iy..Iyz	Moments of inertia [m <sup>4</sup> ]						
Cross-section	Material	Cl.	A	d, h	Iy	Iz	Ix
1 Inner spans	C24	2	16000	160	3.4133e-05	3.2598e-05	3.2598e-05
				100	1.3333e-05	0.0000e+00	0.0000e+00
2 Outer span	C24	2	19200	160	4.0960e-05	4.9752e-05	4.9752e-05
				120	2.3040e-05	0.0000e+00	0.0000e+00

**EN 1995-1-1 actions**

**Standard design group**

**G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load-duration: Permanent

Load cases

1 Dead load and roof cladding

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Load-duration: Short-term

Combination coefficients for: Superstructures

Snow load - Places in CEN member states with less than 1000 m above sea level

Psi.0 / Psi.1 / Psi.2 = 0.5 / 0.2 / 0

Load cases 1. Variant, inclusive

2 Snow load (short-term)

**QW - Wind load**

Gamma.sup / gamma.inf = 1.5 / 0

Load-duration: Short-term

Combination coefficients for: Superstructures

Wind loads

Psi.0 / Psi.1 / Psi.2 = 0.6 / 0.2 / 0

Load cases 1. Variant, exclusive

3 Wind load (short-term)

**1. Permanent and temporary situation**

Final state

G - Dead load  
 QS - Snow and ice load  
 QW - Wind load

**1. Rare (characteristic) situation**

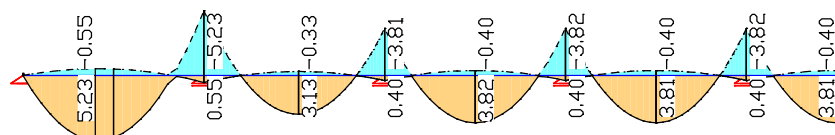
Final state

G - Dead load  
 QS - Snow and ice load  
 QW - Wind load

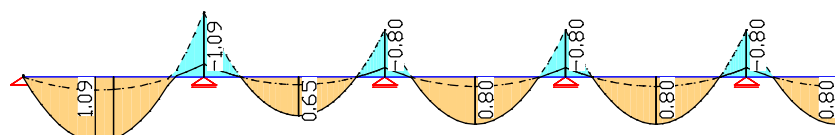
**1. Quasi-continuous situation**

Final state

G - Dead load  
 QS - Snow and ice load  
 QW - Wind load



Bending moment min/max  $M_y$  [kNm]

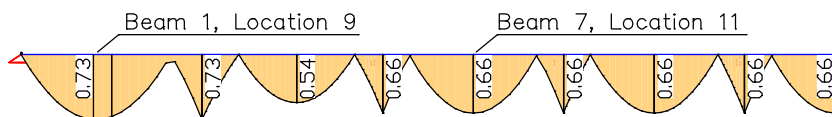


Bending moment min/max  $M_z$  [kNm]

**Max. Cross-Section Utilization**

Utilization Relation between the design values of stress and strength.  
 Tension Tension parallel to the grain as per Eq. (6.1).  
 Compression Compression parallel to the grain as per Eq. (6.2).  
 Bending Bending as per Eq. (6.11) and (6.12).  
 Bend.,Tens. Bending and axial tension as per Eq. (6.17) and (6.18).  
 Bend.,Comp. Bending and axial compression as per Eq. (6.19) and (6.20).  
 Shear Shear as per Eq. (6.13).  
 Torsion Torsion as per Eq. (6.14).  
 x Distance from the beam startpoint [m].  
 PC Permanent and temporary comb.

Cross-section	Material	Result	Beam Loc.	x[m]	Comb.	Util.	
1 Inner spans	C24	Tension					
		Compression					
		Bending	7	11	3.00	PC.1	0.66
		Bending, Tens.					
		Bending, Comp.					
		Shear	8	11	3.00	PC.1	0.26
		Torsion					
2 Outer span	C24	Tension					
		Compression					
		Bending	1	9	2.40	PC.1	0.73
		Bending, Tens.					
		Bending, Comp.					
		Shear	2	11	3.00	PC.1	0.26
		Torsion					



Utilization by check 'Bending'

The following pages contain excerpts from the detailed check log for beam 1 at location 9.

**Check of the Cross-Section Resistance for Beams**

The results represent the extrema of all combinations.

Utilization Relation between the design values of stress and strength.  
 kmod Modif. factor for load-duration and moisture content, Eq. (2.17).  
 km Reduction factor for the bending stress as per Chapter 6.1.6(2).  
 khy, khz Increase factors for the bending strength as per Eq. (3.1), (3.2).  
 kht Increase factor for the tensile strength as per Eq. (3.1), (3.2).  
 kcr Reduction factor for the shear strength as per Eq. (6.13a).  
 kshape Increase factor for the shear strength as per Eq. (6.14).  
 ft0d Axial tensile strength =  $kmod \cdot ft0k / \gamma.M$  [MN/m<sup>2</sup>].  
 fc0d Axial compressive strength =  $kmod \cdot fc0k / \gamma.M$  [MN/m<sup>2</sup>].  
 fmd Bending strength =  $kmod \cdot fmk / \gamma.M$  [MN/m<sup>2</sup>].  
 fvd Strength for lat. force & torsion =  $kmod \cdot fvk / \gamma.M$  [MN/m<sup>2</sup>].  
 x Distance from the beam startpoint [m].  
 y, z Location in the sectional polygon [m].

**Location 9**

Beam 1: x = 2.40 m (Beam Length 3.00 m)  
 Cross-section 2: Outer span, C24; Class 2; h/b=160/120 mm; A=19200 mm<sup>2</sup>  
 km=0.70; khy/khz/kht=1.00/1.00/1.05; kcr=0.67; kshape=1.07

**1. Permanent and temporary comb. (PC.1): G+QS+QW**

Relevant values from 10 sets of internal forces

Set	Nx[kN]	Qy[kN]	Qz[kN]	Mx[kNm]	My[kNm]	Mz[kNm]	Load-dur.	kmod
3	0.00	0.03	0.15	-0.00	5.23	1.09	Sho.-term	0.90

Load case combination for the relevant set of internal forces  
 Set Combination  
 3 : 1.35\*L1+1.50\*L2

**Check for Bending**

Max. Utilization as per Equation (6.11) and (6.12) : 0.73  
 cor. Sigma.myd / (khy·fmd) : -10.217 / 16.615 : 0.61  
 Sigma.mzd / (khz·fmd) : -2.846 / 16.615 : 0.17  
 S.point y / z [m] : 0.000 / 0.000  
 Situation / Set : PC.1 / 3

**Check for Shear**

Max. Utilisation as per Equation (6.13) : 0.01  
 cor. Tau.yd / (kcr·fvd) : 0.002 / 1.855 : 0.00  
 Tau.zd / (kcr·fvd) : 0.011 / 1.855 : 0.01  
 Tau.d / (kcr·fvd) : 0.012 / 1.855 : 0.01  
 S.point y / z [m] : 0.060 / 0.080  
 Situation / Set : PC.1 / 3

Following is the detailed log of deformations to be detected for beam 1, location 10, the location with the maximum

instantaneous and final deformation.

**Check of beam deformations**

The results represent the extrema of all combinations.

```

Class      Service class
kdef      Deformation factor acc. to Table 3.2.
L         User-defined length of the field (Figure 7.1) [m].
w.inst,   User-defined permitted deflection for the instantaneous, final
w.fin     deformation check.
Utilis.   Ratio of instantaneous, final deformation to permitted deflection.
Direction Direction of deformation for the check acc. to user input.
          y; z; y,z Local y, z direction or y and z direction.
          r         Direction of the resulting deformation.
u.max     Maximum deformation in selected check direction [mm].
u.per     Permitted deflection calculated with L and w.inst or w.fin [mm].
Sit./Set  Decisive situation and set of deformations for the check.
    
```

**Beam 1**

**Location 10**

```

Beam 1: x = 2.70 m (Beam Length 3.00 m)
Cross-section 2: Endfeld, C24; Class 2; kdef=0.80; L=6.00 m;
w.inst=L/300; w.fin=L/250; Direction=r;
    
```

**1. Characteristic (rare) combination (CC.1): G+QS+QW**

```

Decisive local Deformation
Set      uy[mm]  uz[mm]  ur[mm]
3       :    9.6    25.9    27.6
    
```

```

Load case combination of decisive set of deformations
Set      Combination
3       : L1+L2
    
```

**1. Quasi-continuous combination (QC.1): G+QS+QW**

```

Decisive local Deformation
Set      uy[mm]  uz[mm]  ur[mm]
1       :    5.0    13.5    14.4
    
```

```

Load case combination of decisive set of deformations
Set      Combination
1       : L1
    
```

**Check of Beam Deformations**

```

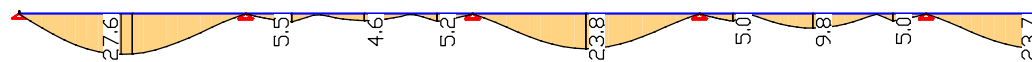
Instantaneous deformation [mm]:
Direct. Sit./Set  u.max  u.per  Utilis.
r       CC.1 / 3    27.6   20.0   1.38 >1
    
```

```

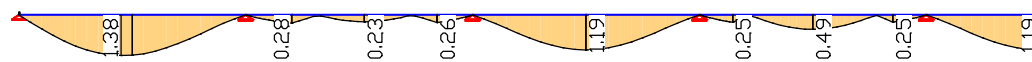
Final deformation [mm]:
Direct. Sit./Set  u.max  u.per  Utilis.
r       QC.1 / 1    14.4   24.0   0.60
    
```

The check of the instantaneous deformations exceeds the permissible deflection at several points. The literature example refers to DIN 1052, where only the deformations from traffic loads have to comply with the limit of L/300 when determining the initial deflection (load case 2, beam 1, location 10:  $u_r = 19.61 \text{ mm} < 20 \text{ mm}$ ).

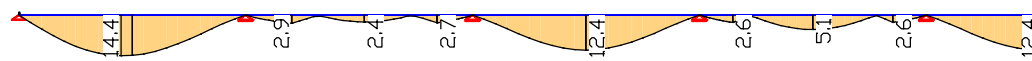
In the case of EN 1995-1-1, the deformation must be taken from the characteristic situation, i.e. in addition the deformation from permanent loads (load case 1, beam 1, location 10:  $u_r = 8.02 \text{ mm}$ ). By choosing a larger purlin cross-section this check can be satisfied.



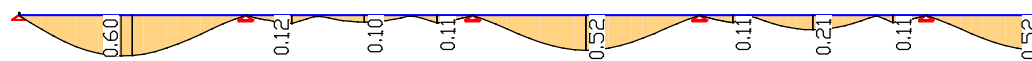
Instantaneous deformation in resulting direction [mm]



Utilization instantaneous deformation / perm. deflection



Final deformation in resulting direction [mm]

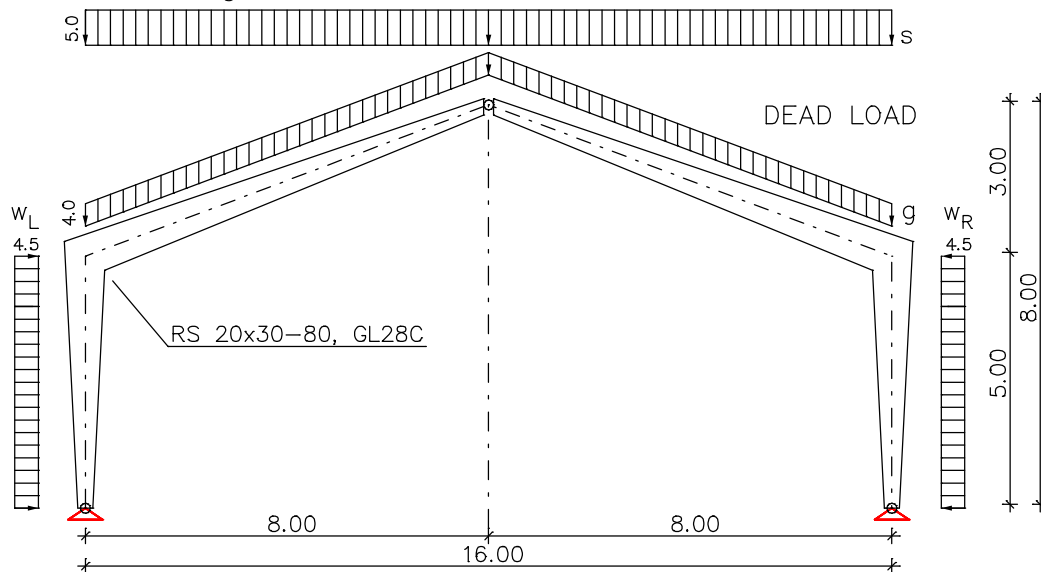


Utilization final deformation / perm. deflection

## Three-Hinged Frame at Normal Temperature and Under Fire Conditions

This example shows the generation of a plane three-hinged frame made of timber with different cross-section heights at the beginning and the end of the beams and columns with corresponding loads. It also shows the process of the following calculations:

- Framework analysis according to the second-order theory
- Checks according to EN 1995-1-1 and EN 1995-1-2



Static system, dimensions [m] and loads [kN/m]

### Cross-sections

At the hinges	Rectangle 20x30, GL28C, Service class 2
In the frame corners	Rectangle 20x80, GL28C, Service class 2
Settings for the check under fire conditions	
Fire exposure	from three sides (left / right / bottom)
Duration of fire	30 min
Charring rate	0.7 mm/min

### Load

Load case 1:	Permanent Loads, dead load and additional load 4.0 kN/m (see Fig.)
Load case 2:	Snow, projective line load 5.0 kN/m (load duration: medium-term) (see Fig.)
Load case 3, 4:	Wind, line load on columns 4.5 kN/m (load duration: short-term) (see Fig.)
Load case 11:	Stability check at normal temperature (see below)

### Stability check at normal temperature

This load case has to be calculated according to the second-order theory. For the check in accordance with EN 1995-1-1 the results are checked against the characteristic material properties without further partial safety factors.

Load case 11 is described by the load type load group with the following settings:

#### Theory

Geometrical second-order theory

#### Selected load cases

Load case 1 with factor 1.35 ( $\gamma_G$ )

Load case 2 with factor 1.5 ( $\gamma_Q$ )

#### Additional global load factor

The actions have to be multiplied with the factors  $\gamma_G$ ,  $\gamma_Q$  and  $\gamma_M = 1.25$ , because the safety factors have to be considered during the internal force calculation.

#### Predeformation

Is not taken into account in this example.

### Design overview

To perform the checks the actions have to be assigned to the situations

- Permanent and temporary situation at normal temperature
- Special situation for the stability check at normal temperature
- Fire situation

as they are printed out in the following listing. The check program carries out the necessary internal force combinations and verifies the cross-section resistance.

#### Design per EN 1995-1-1:2014 and EN 1995-1-2:2009

The check of the cross-section resistance is based on EN 1995-1-1, Chapter 6.1. The system strength factor as per Chapter 6.6 is assumed to be  $k_{sys} = 1$ . The effect of member size on strength is considered as per Chapter 3.2 and 3.3. The maximum cross-section dimension for the bending strength and the minimum for the tensile strength are used as the decisive dimension for the factor  $k_h$ .

For fire conditions unprotected surfaces throughout the time of fire exposure are assumed acc. to EN 1995-1-2, Chapter 3.4.2. The check is performed with reduced cross-sections as per Chapter 4.2.2 of the standard.

The actions are combined acc. to EN 1990, Eq. (6.10), using the partial and combination factors according to EN 1990:2021 and 1991-1-2:2009. All variations of cooperation of the variable actions are examined.

**Designing occurs for all possible combinations of actions.**

#### Stress Calculation

The normal stresses and shear stresses are calculated for homogeneous polygon sections from normal force, bending moment and lateral force. The shear characteristics are calculated acc. to the Boundary-Element-Method. The calculation points for all stresses are edge points of the cross-section.

#### Partial Safety Factors for Timber

	gamma.M
Permanent and temporary comb.	1.30 (Solid Timber)
	1.25 (Glued Laminated Timber)
Accidental combination	1.00
Special combination	1.00
Fire combination	1.00

#### Characteristic Material Properties

Strength and density as per EN 338:2016 and EN 14080:2013 [MN/m<sup>2</sup>, kg/m<sup>3</sup>].

Material	f <sub>mk</sub>	f <sub>t0k</sub>	f <sub>c0k</sub>	f <sub>vk</sub>	ρ <sub>hok</sub>
GL28C	28.0	19.5	24.0	3.5	390.0

#### Cross-Section Properties under Normal Temperature

Cl. Service class  
 A Sectional area [mm<sup>2</sup>]  
 d,h,b Diameter, height, width [mm]  
 I<sub>y</sub>..I<sub>yz</sub> Moments of inertia [m<sup>4</sup>]

Cross-section	Material	Cl.	A	d,h	b	I <sub>y</sub>	I <sub>yz</sub>
3 Rectangle	GL28C	2	62500	313	200	5.0863e-04	0.0000e+00
4 Rectangle	GL28C	2	67500	338	200	6.4072e-04	0.0000e+00
5 Rectangle	GL28C	2	72500	363	200	7.9391e-04	0.0000e+00
6 Rectangle	GL28C	2	77500	388	200	9.6976e-04	0.0000e+00
7 Rectangle	GL28C	2	82500	413	200	1.1698e-03	0.0000e+00
8 Rectangle	GL28C	2	87500	438	200	1.3957e-03	0.0000e+00
9 Rectangle	GL28C	2	92500	463	200	1.6489e-03	0.0000e+00
10 Rectangle	GL28C	2	97500	488	200	1.9310e-03	0.0000e+00
11 Rectangle	GL28C	2	102500	513	200	2.2435e-03	0.0000e+00
12 Rectangle	GL28C	2	107500	538	200	2.5881e-03	0.0000e+00
13 Rectangle	GL28C	2	112500	563	200	2.9663e-03	0.0000e+00
14 Rectangle	GL28C	2	117500	588	200	3.3797e-03	0.0000e+00
15 Rectangle	GL28C	2	122500	613	200	3.8297e-03	0.0000e+00
16 Rectangle	GL28C	2	127500	638	200	4.3181e-03	0.0000e+00
17 Rectangle	GL28C	2	132500	663	200	4.8463e-03	0.0000e+00
18 Rectangle	GL28C	2	137500	688	200	5.4159e-03	0.0000e+00
19 Rectangle	GL28C	2	142500	713	200	6.0284e-03	0.0000e+00
20 Rectangle	GL28C	2	147500	738	200	6.6855e-03	0.0000e+00
21 Rectangle	GL28C	2	152500	763	200	7.3887e-03	0.0000e+00
22 Rectangle	GL28C	2	157500	788	200	8.1396e-03	0.0000e+00

**Cross-Section Properties under Fire Conditions**

Cl. Service class  
 Ar Area of the residual cross-section as per EN 1995-1-2, Chapter 3.4 [mm<sup>2</sup>]  
 dr,hr,br Diameter, height, width of the residual cross-section [mm]  
 Iy..Iyz Moments of inertia [m<sup>4</sup>]  
 t Duration of the fire exposure as per EN 1995-1-2, Eq. (3.2) [min]  
 βn Design notional charring rate as per EN 1995-1-2, Table 3.1 [mm/min]  
 Side Side exposed to fire: (L)eft, (R)ight, (T)op, (B)ottom, (A)ll sides

Cross-section	Material	Cl.	Ar	dr,hr br	Iy Iz	Iyz	t	βn	Side
3 Rectangle	GL28C	2	40968	285 144	2.7633e-04 7.0793e-05	0.0000e+00	30 0.70	30	LRB
4 Rectangle	GL28C	2	44568	310 144	3.5576e-04 7.7014e-05	0.0000e+00	30 0.70	30	LRB
5 Rectangle	GL28C	2	48168	334 144	4.4913e-04 8.3234e-05	0.0000e+00	30 0.70	30	LRB
6 Rectangle	GL28C	2	51768	360 144	5.5754e-04 8.9455e-05	0.0000e+00	30 0.70	30	LRB
7 Rectangle	GL28C	2	55368	384 144	6.8213e-04 9.5676e-05	0.0000e+00	30 0.70	30	LRB
8 Rectangle	GL28C	2	58968	410 144	8.2403e-04 1.0190e-04	0.0000e+00	30 0.70	30	LRB
9 Rectangle	GL28C	2	62568	435 144	9.8435e-04 1.0812e-04	0.0000e+00	30 0.70	30	LRB
10 Rectangle	GL28C	2	66168	459 144	1.1642e-03 1.1434e-04	0.0000e+00	30 0.70	30	LRB
11 Rectangle	GL28C	2	69768	484 144	1.3648e-03 1.2056e-04	0.0000e+00	30 0.70	30	LRB
12 Rectangle	GL28C	2	73368	509 144	1.5871e-03 1.2678e-04	0.0000e+00	30 0.70	30	LRB
13 Rectangle	GL28C	2	76968	535 144	1.8324e-03 1.3300e-04	0.0000e+00	30 0.70	30	LRB
14 Rectangle	GL28C	2	80568	560 144	2.1018e-03 1.3922e-04	0.0000e+00	30 0.70	30	LRB
15 Rectangle	GL28C	2	84168	585 144	2.3963e-03 1.4544e-04	0.0000e+00	30 0.70	30	LRB
16 Rectangle	GL28C	2	87768	609 144	2.7171e-03 1.5166e-04	0.0000e+00	30 0.70	30	LRB
17 Rectangle	GL28C	2	91368	635 144	3.0653e-03 1.5788e-04	0.0000e+00	30 0.70	30	LRB
18 Rectangle	GL28C	2	94968	660 144	3.4421e-03 1.6410e-04	0.0000e+00	30 0.70	30	LRB
19 Rectangle	GL28C	2	98568	685 144	3.8486e-03 1.7033e-04	0.0000e+00	30 0.70	30	LRB
20 Rectangle	GL28C	2	102168	710 144	4.2859e-03 1.7655e-04	0.0000e+00	30 0.70	30	LRB
21 Rectangle	GL28C	2	105768	734 144	4.7551e-03 1.8277e-04	0.0000e+00	30 0.70	30	LRB
22 Rectangle	GL28C	2	109368	760 144	5.2573e-03 1.8899e-04	0.0000e+00	30 0.70	30	LRB

All residual cross-sections fulfill Clause 4.3.5(2) of EN 1995-1-2.

**EN 1995-1-1 actions**

**Standard design group**

**G - Dead load**

Gamma.sup / gamma.inf = 1.35 / 1

Load-duration: Permanent

Load cases

-----  
 1 Dead load

**QS - Snow and ice load**

Gamma.sup / gamma.inf = 1.5 / 0

Load-duration: Medium-term

Combination coefficients for: Superstructures

Snow load - Places in CEN member states with less than 1000 m above sea level

Psi.0 / Psi.1 / Psi.2 = 0.5 / 0.2 / 0

Load cases 1. Variant, inclusive

-----  
 2 Snow

**QW - Wind load**

Gamma.sup / gamma.inf = 1.5 / 0

Load-duration: Short-term

Combination coefficients for: Superstructures

Wind loads

Psi.0 / Psi.1 / Psi.2 = 0.6 / 0.2 / 0

Load cases 1. Variant, exclusive

-----  
 3 Wind left  
 4 Wind right



**Fd - Design values of actions**

Load-duration: Medium-term

Load cases

-----  
11 PT Th.2

**1. Permanent and temporary situation**

Final state

G - Dead load  
QS - Snow and ice load  
QW - Wind load

**1. Special situation**

Final state

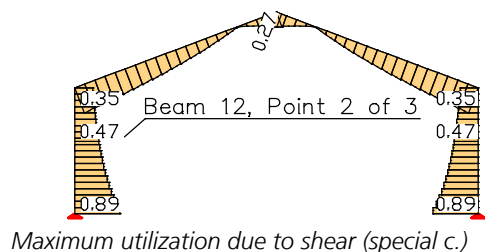
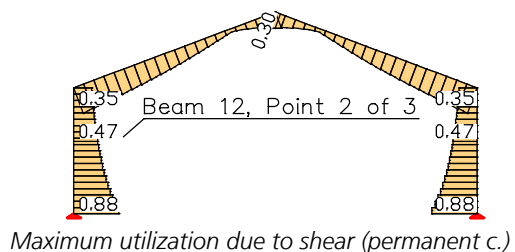
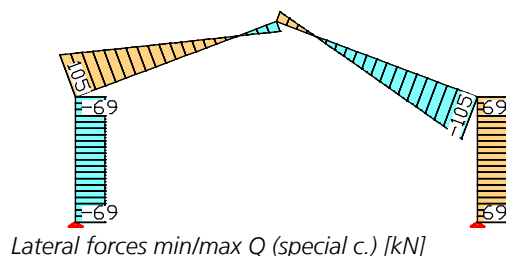
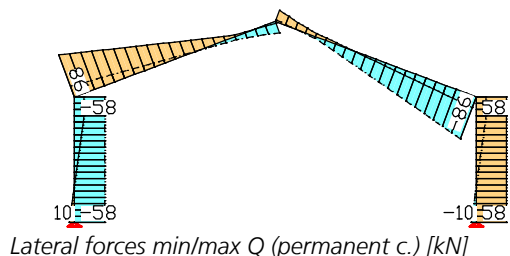
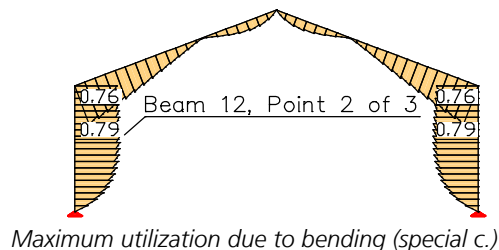
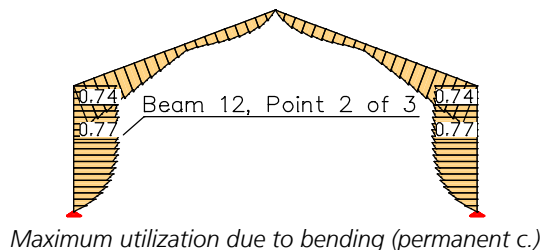
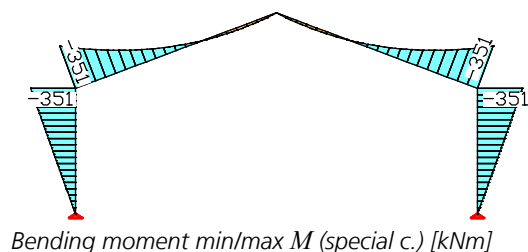
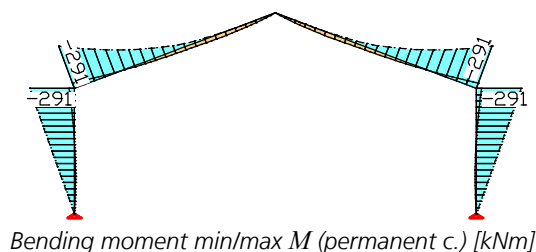
Fd - Design values of actions

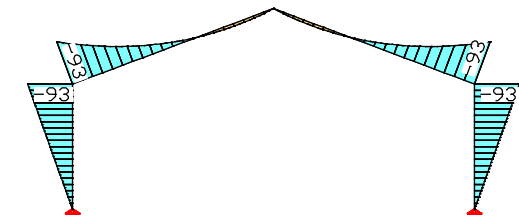
**1. Fire situation**

Final state

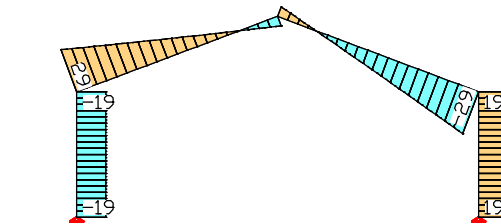
G - Dead load  
QS - Snow and ice load  
QW - Wind load

**Results**

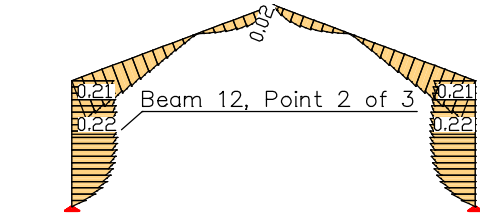




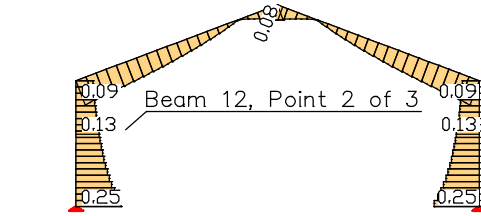
Bending moment min/max M (fire c.) [kNm]



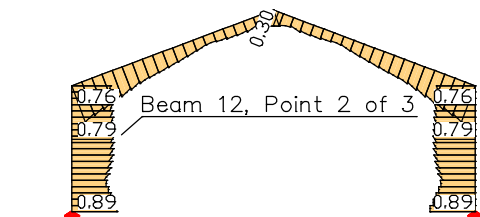
Lateral forces min/max Q (fire c.) [kN]



Maximum utilization due to bending (fire c.)



Maximum utilization due to shear (fire c.)



Maximum utilization due to all checks

Subsequently an additional extract of the detailed listing for beam 12, location 2 is printed.

**Check of the Cross-Section Resistance for Beams under Normal Temperature**

The results represent the extrema of all combinations.

Utilization Relation between the design values of stress and strength.  
 kmod Modif. factor for load-duration and moisture content, Eq. (2.17).  
 km Reduction factor for the bending stress as per Chapter 6.1.6(2).  
 khy Increase factor for the bending strength as per Eq. (3.1), (3.2).  
 kht Increase factor for the tensile strength as per Eq. (3.1), (3.2).  
 kcr Reduction factor for the shear strength as per Eq. (6.13a).  
 ft0d Axial tensile strength =  $kmod \cdot ft0k / \gamma.M$  [MN/m<sup>2</sup>].  
 fc0d Axial compressive strength =  $kmod \cdot fc0k / \gamma.M$  [MN/m<sup>2</sup>].  
 fmd Bending strength =  $kmod \cdot fmk / \gamma.M$  [MN/m<sup>2</sup>].  
 fvd Shear strength for lateral force =  $kmod \cdot fvk / \gamma.M$  [MN/m<sup>2</sup>].  
 x Distance from the beam startpoint [m].  
 y, z Location in the sectional polygon [m].

**Location 2**

Beam 12: x = 0.13 m (Beam Length 0.25 m)  
 Cross-section 14: Rectangle, GL28C; Class 2; h/b=588/200 mm; A=117500 mm<sup>2</sup>  
 km=0.70; khy/kht=1.00/1.10; kcr=0.67

**1. Permanent and temporary comb. (PC.1): G+QS+QW**

No set of internal forces in this combination was relevant.

**1. Special combination (SC.1): Fd**

Relevant values from 1 set of internal forces  
 Set Nx[kN] Qz[kN] My[kNm] Load-dur. kmod  
 1 : -140.38 -69.29 -204.14 Med.-term 0.80

Load case combination for the relevant set of internal forces  
 Set Combination  
 1 : L11

**Check for Compression parallel to the Grain**

Max. Utilization as per Equation (6.2) : 0.06  
 cor. Sigma.c0d / fc0d : -1.195 / 19.200 : 0.06  
 Situation / Set : SC.1 / 1

**Check for Bending**

Max. Utilization as per Equation (6.11) and (6.12) : 0.79  
 cor. Sigma.myd / (khy.fmd) : 17.743 / 22.447 : 0.79  
 S.point y / z [m] : 0.000 / 0.000  
 Situation / Set : SC.1 / 1

**Check for Bending and Axial Compression**

```

Max. Utilization as per Equation (6.19) and (6.20) : 0.79
cor. Sigma.c0d / fc0d : -1.195 / 19.200 : 0.06
Sigma.myd / (khy·fmd) : 17.743 / 22.447 : 0.79
S.point y / z [m] : 0.000 / 0.000
Situation / Set : SC.1 / 1

```

**Check for Shear**

```

Max. Utilization as per Equation (6.13) : 0.47
cor. Tau.yd / (kcr·fvd) : 0.000 / 1.876 : 0.00
Tau.zd / (kcr·fvd) : -0.885 / 1.876 : 0.47
S.point y / z [m] : 0.200 / 0.294
Situation / Set : SC.1 / 1

```

**Check of the Cross-Section Resistance for Beams under Fire Conditions**

The results represent the extrema of all combinations.

Utilization Relation between the design values of stress and strength.

```

kmod,fi Mod. factor for fire conditions as per EN 1995-1-2, Eq. (2.1).
kfi Factor as per EN 1995-1-2, Table 2.1.
km Reduction factor for the bending stress as per Chapter 6.1.6(2).
khy Increase factor for the bending strength as per Eq. (3.1), (3.2).
kht Increase factor for the tensile strength as per Eq. (3.1), (3.2).
kcr Reduction factor for the shear strength as per Eq. (6.13a).
ft0d,fi Axial tensile strength = kmod,fi · kfi · ft0k / gamma.M,fi.
fc0d,fi Axial compressive strength = kmod,fi · kfi · fc0k / gamma.M,fi.
fmd,fi Bending strength = kmod,fi · kfi · fmk / gamma.M,fi.
fvd,fi Shear strength for lateral force = kmod,fi · kfi · fvk / gamma.M,fi.
x Distance from the beam startpoint [m].
y, z Location in the sectional polygon [m].

```

**Location 2**

Beam 12: x = 0.13 m (Beam Length 0.25 m)  
 Cross-section 14: Rectangle, GL28C, Class 2; hr/br=560/144 mm; Ar=80568 mm<sup>2</sup>  
 kmod,fi=1.00; kfi=1.15; km=0.70; khy/kht=1.01/1.10; kcr=0.67

**1. Fire combination (FC.1): G+QS+QW**

Relevant values from 1 set of internal forces

```

Set Nx[kN] Qz[kN] My[kNm]
1 : -38.74 -18.56 -53.37

```

Load case combination for the relevant set of internal forces

```

Set Combination
1 : L1

```

**Check for Compression parallel to the Grain**

```

Max. Utilization as per Equation (6.2) : 0.02
cor. Sigma.c0d / fc0d,fi : -0.481 / 27.600 : 0.02
Situation / Set : FC.1 / 1

```

**Check for Bending**

```

Max. Utilization as per Equation (6.11) and (6.12) : 0.22
cor. Sigma.myd / (khy·fmd,fi) : 7.104 / 32.426 : 0.22
S.point y / z [m] : 0.000 / 0.000
Situation / Set : FC.1 / 1

```

**Check for Bending and Axial Compression**

```

Max. Utilization as per Equation (6.19) and (6.20) : 0.22
cor. Sigma.c0d / fc0d,fi : -0.481 / 27.600 : 0.02
Sigma.myd / (khy·fmd,fi) : 7.104 / 32.426 : 0.22
S.point y / z [m] : 0.000 / 0.000
Situation / Set : FC.1 / 1

```

**Check for Shear**

```

Max. Utilization as per Equation (6.13) : 0.13
cor. Tau.yd / (kcr·fvd,fi) : 0.000 / 2.697 : 0.00
Tau.zd / (kcr·fvd,fi) : -0.346 / 2.697 : 0.13
S.point y / z [m] : 0.144 / 0.280
Situation / Set : FC.1 / 1

```

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# Stress in Structure

## Basics

This module is used to determine and check the permissibility of the stresses in homogenous beam sections with polygonal bordering according to the theory of elasticity.

The checks are carried out subsequent to the static calculation for the internal forces determined in the load cases or load case combinations on the entire system on cross-sections made of the following materials:

- Structural steel *S235* and *S355* according to *DIN 18800*
- Structural timber *NH1*, *NH2*, *NH3*, *BSH1*, *BSH2* according to *DIN 1052:1988*
- Concrete *B25*, *B35*, *B45*, *B55* according to *DIN 1045:1988*
- free material

The predefined steel sections, project-specific parameter and polygon sections or sections from the user database can be used as sections. Beams with the section form *Beam* and *Tension member* are not checked.

The input data for the stress and section analysis are provided using the database.

## Input

### Settings for Stress Determination

A precondition for the performance of the check is a previous framework analysis for determining the internal forces and, if necessary, a subsequent load case combination.

Load case or combination	-1
Load factor	1.00
Load case type	H
Listing	0

#### *Load case or combination*

Number of the load case for which the stress checks are to be carried out. A negative number indicates that a load case combination with the corresponding number is to be used.

#### *Load factor*

Factor applied to the load case or combination.

#### *Load Case Class*

*H* or *HZ* for determining stress exceeding with the permissible stresses defined for the load case classes.

#### *Listing type*

- 0: Short log
- 1: Long log
- 2: Short log, only the results for beams with stress exceeding are printed.
- 3: As 2, but long log

## Permissible Stresses

Permissible stresses [MN/m <sup>2</sup> ] [SPGZUL]					
Load case	H		HZ		
	S235	S355	S235	S355	
Steel					
permissible sigma	160,0	240,0	180,0	270,0	
permissible tau	92,0	139,0	104,0	156,0	
Wood load case H	NH1	NH2	NH3	BSH1	BSH2
Bending perm. sigma.b	13,0	10,0	7,0	14,0	11,0
Tension perm. sigma.z	10,5	8,5	0,0	10,5	8,5
Press. perm. sigma.d	11,0	8,5	6,0	11,0	8,5
Shear perm. tau	0,9	0,9	0,9	1,2	1,2
different materials	frei	B25	B35	B45	B55
Tension perm. sigma.z	3,5	3,5	4,0	4,5	5,0
Press. perm. sigma.d	12,0	12,0	15,0	18,0	20,0
Shear perm. tau	5,5	5,5	7,0	8,0	9,0

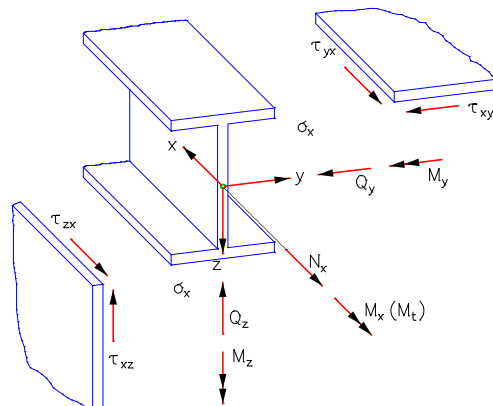
The limits given here are decisive for labeling stress exceeding.

## Checks

### Stress Determination

#### Coordinate Systems

The following illustration shows the definition of the coordinate systems for internal forces, loads and stresses using the example of a beam with an I profile. The section edge is defined as a closed polygon.



The longitudinal force  $N_x$  runs through the centroid, the lateral forces  $Q_y$  and  $Q_z$  run through the shear center of the section which, in this case, coincides with the centroid. The vectors of the moments  $M_x$ ,  $M_y$ ,  $M_z$  and stresses  $\sigma_x$ ,  $\tau_{xy}$ ,  $\tau_{xz}$  run parallel to the coordinate axes.

## Longitudinal Stresses

The necessary section properties for determination of the longitudinal stresses from longitudinal force and bending are determined elementarily with the by-pass method from the coordinates of the traverse.

More specifically, these are:

$y_s, z_s$  Centroid coordinates.

$A$  Area of the section.

$I_y, I_z$  Moments of inertia in relation to the coordinate axes.

$I_{yz}$  Deviation moment of inertia.

$I_1, I_2$  Moments of inertia in relation to the main axes.

$\varphi$  Twisting angle of the main axes.

$W_y, W_z$  Moments of resistance for the calculation of the extremal longitudinal stresses from  $M_y, M_z$ .

For the analysis of a section, the stresses are determined at the automatically defined calculation points. The longitudinal stress  $\sigma$  for a point  $(y,z)$  of the section results from

$$\sigma(y, z) = N_x \frac{1}{A} + M_y \frac{\Delta z I_z - \Delta y I_{yz}}{I_y I_z - I_{yz}^2} + M_z \frac{\Delta y I_y - \Delta z I_{yz}}{I_y I_z - I_{yz}^2}$$

with  $\Delta y = y - y_s$  and  $\Delta z = z - z_s$ .



## Shear Stresses

### Lateral force

For load as a result of lateral force, the differential equation of the so-called shear warpage  $\omega_\tau$

$$\frac{\partial^2 \omega_\tau}{\partial y^2} + \frac{\partial^2 \omega_\tau}{\partial z^2} = -\frac{Q_z z}{G I_y} \quad \text{or} \quad -\frac{Q_y y}{G I_z}$$

is solved numerically with the help of the boundary element method. From this the following shear characteristics are determined:

$\tau_{Q_y,1}$  Shear stress for the lateral force  $Q_y = 1$ .

$\tau_{Q_z,1}$  Shear stress for the lateral force  $Q_z = 1$ .

$W_{qy}, W_{qz}$  Area values for calculation of the extremal shear stresses from  $Q_y$  and  $Q_z$  with

$$\tau_{Q_y} = Q_y / W_{qy} \quad \text{and} \quad \tau_{Q_z} = Q_z / W_{qz}$$

If selected in the Section dialog, the shear stresses across the section width will be averaged.

### Torsion

For load as a result of St. Venant torsion, the differential equation of the unit warpage  $\omega$

$$\frac{\partial^2 \omega}{\partial y^2} + \frac{\partial^2 \omega}{\partial z^2} = 0$$

is decisive. Its solution leads to the following characteristics:

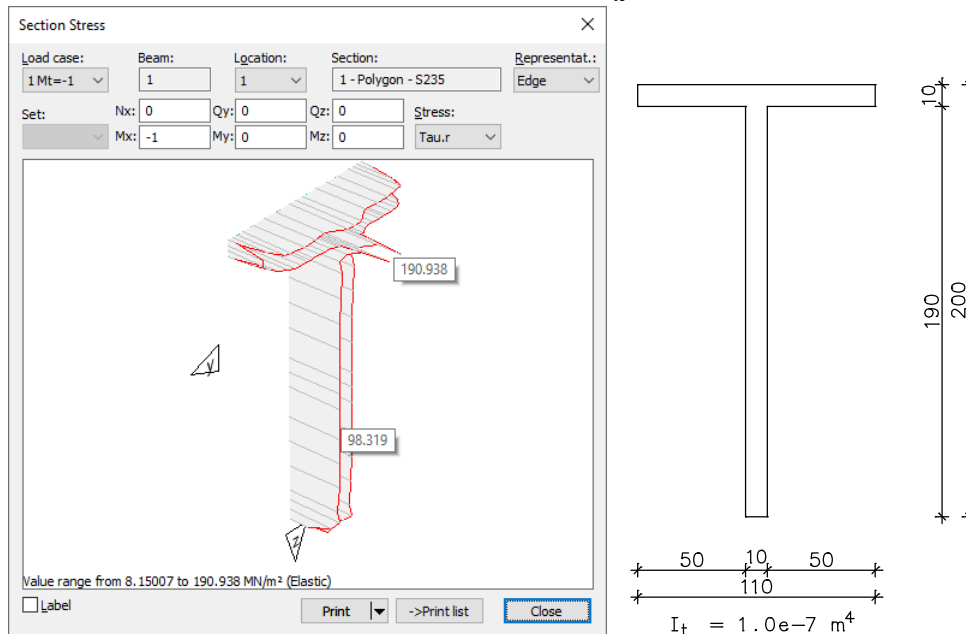
$I_t$  Torsion moment of inertia.

$\tau_{M_x,1}$  Shear stress for the torsional moment  $M_x = 1$ .

$W_t$  Moment of resistance for the calculation of the extremal shear stress from  $M_x$ .

As a result, in the fillet area of rolled sections or in blunt corners of welded profiles larger torsional shear stresses occur than those according to the theory of thin-walled profiles. As an option in the section dialog, specifying the maximum sheet thickness (open profiles) or the torsion resistance moment (box profiles) allows you to limit the stresses to the maximum value according to the theory of thin-walled profiles.

The following figure shows the stress curve for a load of  $M_x = 1$  (Section Stress function):



The numerically determined value for the web in the example corresponds to the theoretical shear stress for open thin-walled sections  $\tau = M_x \cdot t / I_t = 100 \text{ MN/m}^2$ . The peak value of 190 MN/m<sup>2</sup> results from a singularity in the blunt corners.

All unit shear stresses are calculated at discrete points on the section edges. The discretization is automatically carried out by the program. The unit shear stresses are edge stresses which run along the section edge. Their sign depends on the direction of the edge. The resultant shear stress  $\tau$  from lateral force and torsion is then determined by the following equation:

$$\tau = Q_y \cdot \tau_{Q_y,1} + Q_z \cdot \tau_{Q_z,1} + M_x \cdot \tau_{M_x,1}$$

## Check for Steel Sections

For sections with the material designation *S235* or *S355*, stress exceeding is marked by the program. The permissible stresses are considered to be observed if the following conditions are fulfilled:

$$|\sigma| \leq perm. \sigma$$

$$\leq 1.1 perm. \sigma \text{ if in each case } \begin{cases} |\sigma_{Nx} + \sigma_{My}| \leq 0.8 perm. \sigma \\ |\sigma_{Nx} + \sigma_{Mz}| \leq 0.8 perm. \sigma \end{cases}$$

$$|\tau| \leq perm. \tau$$

$$\sigma_v = \sqrt{(\sigma^2 + 3\tau^2)} \leq perm. \sigma$$

$$\leq 1.1 perm. \sigma \quad \text{for uniaxial bending without torsion and without normal force.}$$

The stresses *perm. σ* and *perm. τ* used depend on the material quality and the load case class. They are provided in the program printout.

## Check for Wood Sections

For sections with the material designation *NH1*, *NH2*, *NH3*, *BSH1* or *BSH2*, stress exceeding is marked by the program. The permissible stresses are considered to be observed if the following conditions are fulfilled:

$$\text{Tension, } \sigma_{Nx} > 0: \quad |\sigma_{Nx} + (\sigma_{My} + \sigma_{Mz}) \cdot perm. \sigma_z / perm. \sigma_b| \leq perm. \sigma_z$$

$$\text{Compression, } \sigma_{Nx} < 0: \quad |\sigma_{Nx} + (\sigma_{My} + \sigma_{Mz}) \cdot perm. \sigma_d / perm. \sigma_b| \leq perm. \sigma_d$$

$$\text{Bending, } \sigma_{Nx} = 0: \quad |\sigma_{My} + \sigma_{Mz}| \leq perm. \sigma_b$$

$$\text{Shear} \quad |\tau| \leq perm. \tau$$

The stresses *perm. σ<sub>z</sub>*, *perm. σ<sub>d</sub>*, *perm. σ<sub>b</sub>* and *perm. τ* used depend on the material quality and the load case class. They are provided in the program printout.

## Results

### Stresses

$\sigma_x$	Extremal normal stresses from bending and normal force.
$\tau$	Extremal shear stresses from lateral force and torsion.
$\sigma_v$	Maximum comparison stress.
$\sigma_{x.top}$	Maximum normal stresses in the section points that lie on the top edge of the enclosing rectangle.
$\sigma_{x.bottom}$	Maximum normal stresses in the section points that lie on the bottom edge of the enclosing rectangle.

All stresses are given in [MN/m<sup>2</sup>] and can be displayed graphically or in tabular form. In addition, a table with the maximum utilization of the sections is available. If the permissible limits are exceeded, this is recorded in the log.

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# Dynamics

Use this extension module to directly perform dynamic analyses for existing FEM-projects with the following features:

- Dynamic analysis of 3D and 2D beam, cable and shell structures as well as solid models of any geometry or load.
- Determination of eigenvalues and eigenvectors using subspace iteration. User-defined adjustment of the size of the iteration vector space and the iteration accuracy.
- Representation of point and distributed mass distributions. Simultaneous system load by exciters of different frequencies.
- Solution of the equations of motion in modal coordinates with subsequent mode superposition for periodic or instationary load-time functions. You can specify the integration range and the size of the time steps. For each time step, all deformation and load values are available. These can be passed on to post-processing programs for load case combination, design or stress determination.
- Direct integration of the equations of motion for any load-time function. Rayleigh's system damping and point dampers can be represented through the definition of a complete damping matrix. This analysis method can also be used for systems with non-linear properties such as failure of tension supports, non-linear springs, 2nd or 3rd order theory, cable dynamics and dynamic element collapse. This analysis method can also be used for systems with non-linear properties such as failure of tension supports, non-linear springs, 2nd or 3rd order theory, cable dynamics and dynamic element collapse.
- Determination of the system response ( $U_{re}$ ,  $U_{im}$ ,  $abs(U)$ ,  $abs(DU)$ ,  $abs(DDU)$ ,  $phase$ ) to periodic excitation for a chosen stress-frequency range or for an excitation spectrum. Soil acceleration as well as direct structure loads can be taken into account.
- Evaluation of the response spectra according to EN 1998-1 (Eurocode 8), DIN EN 1998-1:2021, DIN 4149:1981, DIN 4149:2005, OENORM B 4015 or SIA 261 and superposition of the internal forces and displacements according to statistical laws. Application of user-defined response spectra.
- Determination of static horizontal loads and calculation of the corresponding equivalent mass  $m^*$  and of the transformation coefficient  $\Gamma$  for a nonlinear pushover analysis according to EN 1998-1:2013 or DIN EN 1998-1:2021.

The dynamics settings as well as the contents of the log are controlled via the *Settings* option of the *Analysis* menu. Point masses are usually defined in the graphical view. Other inputs related to dynamics are performed in the database.

## Basics

### Equations of Motion

The general equations of motion for a finite element system look as follows in matrix form:

$$M \cdot \ddot{u}(t) + C \cdot \dot{u}(t) + K \cdot u(t) = p(t) \quad (1)$$

inertial forces                  damping forces                  elastic forces                  excitation forces

with

$M$                   Mass matrix  
 $C$                   Damping matrix  
 $K$                   Stiffness matrix  
 $u(t)$               Deformation vector

$\dot{u}(t) = \frac{\partial u}{\partial t}$               Velocity vector

$\ddot{u}(t) = \frac{\partial^2 u}{\partial t^2}$               Acceleration vector

$p(t)$                   Load vector

#### Load vector

This vector is composed of the following components:

$$p(t) = (p - M \ddot{u}_k) f(t)$$

with

$p$                   Load vector  
 $\ddot{u}_k$                   Vector of the node acceleration  
 $f(t)$                   Time function of the load

All force and displacement values are functions of time  $t$ . Equation (1) describes a system of linked linear differential equations which normally must be solved collectively. Ignoring inertial and damping forces and assuming a time-independent load results in the static case of application.

### Integration of the Equations of Motion

The equations of motion are integrated based on the general integration scheme defined by Newmark and Wilson. This provides reliable results regardless of the size selected for the time step.

The process is based on the following assumptions:

$$\dot{u}(t + \Delta t) = \dot{u}(t) + \frac{1}{2} [\ddot{u}(t) + \ddot{u}(t + \Delta t)] \Delta t \quad (2)$$

$$u(t + \Delta t) = u(t) + \dot{u}(t) \Delta t + \frac{1}{4} [\ddot{u}(t) + \ddot{u}(t + \Delta t)] \Delta t^2 \quad (3)$$

In addition, the equation of motion (1) is used to calculate displacements, velocities and accelerations at the time  $t + \Delta t$ . The time step solution is carried out according to the following scheme:

Analysis specifications:

1. Determination of mass, damping and stiffness matrices.
2. Specification of  $u(t)$ ,  $\dot{u}(t)$ ,  $\ddot{u}(t)$ .
3. Selection of the time step  $\Delta t$ .

Calculation of the integration constants:

4. Calculation of the effective stiffness matrix and triangulation.

Time step integration:

5. Calculation of the effective loads at time  $t + \Delta t$ .
6. Determination of the displacements at time  $t + \Delta t$  by means of reverse resolution.
7. Determination of the velocities and accelerations at time  $t + \Delta t$  from equations (2) and (3).

## Undamped Natural Frequency Problem

If the damping and excitation forces are ignored in equation (1), the result is:

$$M \ddot{u}(t) + K u(t) = 0 \quad (4)$$

The approach  $u(t) = x e^{i\omega t}$  yields the eigenvalue problem

$$(-\omega^2 M + K) x = 0 \quad (5)$$

with  $\omega$  as the angular frequency

and, using a different notation:

$$K \Phi = M \Phi \lambda \quad (6)$$

In this case:

$\lambda$  Matrix of the eigenvalues

$$\lambda = \begin{bmatrix} \omega_1^2 & & & \\ & \omega_2^2 & & \\ & & \ddots & \\ & & & \omega_i^2 \end{bmatrix}$$

$\Phi$  Matrix of the eigenvectors

The eigenvalue problem at hand is solved using the method of subspace iteration. Using this approach, the subspace of the eigenvectors in question is iterated simultaneously. The eigenvectors have the following transformation properties with respect to the mass and stiffness matrix:

$$\Phi^T K \Phi = \Phi^T M \Phi \lambda \quad (7)$$

$$\Phi^T M \Phi = I \quad (8)$$

In order to solve then eigenvalue problem, a stiffness matrix according to the linear elasticity theory is needed. Therefore nonlinear effects (e.g. compression beams, tension beams, nonlinear springs, loss of tensile support, cable) are not taken into account.

The eigenvalue analysis determines the eigenvalues and eigenvectors for cable structures on the basis of a prescribed cable force state.

A singularity of the Jacobi matrix can occur for dynamic analyses and causes the calculation to terminate.

```
Calculating projection of the stiffness matrix
Calculating projection of the mass matrix
Solving subspace
```

```
** Errors **
```

```
FEM-F-9915: Error in jacobi-iteration, subspace is singular
```

```
Abort of program, 0 Warning(s), 1 Error(s).
```

In most cases, one of the following erroneous inputs is the cause for this:

- Unrealistically high values were input for the element stiffnesses.
- For beams with polygon section no realistic torsional moment of inertia was provided; instead, the default  $I_x = 10^{-6} \text{ m}^4$  was used.

In general, be aware that when performing dynamic analyses usable values for the resonance frequency of a structure can only be determined if realistic stiffnesses are used.

## Modal Analysis

The equations (7) and (8) can be used to perform a transformation of the equations of motion to the space of eigenvectors. Taking

$$u(t) = \Phi x(t) \quad (9)$$

and performing a left-handed multiplication of the equation of motion (1) with  $\Phi^T$  yields:

$$\ddot{x}(t) + \Phi^T C \Phi \dot{x}(t) + \lambda x(t) = \Phi^T p(t) \quad (10)$$

This leads to a system of equations (10) in modal coordinates whose terms are unlinked, with the exception of the damping term.

If the following approach is used for the damping matrix

$$C = \alpha M + \beta K, \quad (11)$$

then, using the equations (7) and (8), the equation of motion is unlinked with regard to:

$$\ddot{x}_i(t) + 2\omega_i \xi_i \dot{x}_i(t) + \omega_i^2 x_i(t) = r_i(t) \quad (12)$$

with

$\omega$	Angular frequency
$\xi_i = D$ or $(\alpha + \beta \omega_i^2) / 2\omega_i$	Modal damping measure
$D$	Lehr's damping measure
$r_i(t) = \varphi_i^T p(t)$	Modal load vector
$\varphi_i$	i-th eigenvector
$i$	Index of the eigenvalues, eigenvectors

The integration of the equations (12) can be carried out either analytically or with a numerical integration scheme.

The approach (11) is generally applicable for structures with natural damping only. But it is not adequate for realistically modeling point dampers.

An essential advantage of the modal analysis is that in general the complete eigenvector space does not need to be taken into account. Instead, due to the natural damping that is always present for buildings, only the first  $n$  eigenmodes ( $n \ll n_u$ ) have a significant impact on the structure response. This greatly reduces the complexity of the analysis when compared with direct integration of equations of motion.

## Direct Integration of the Equations of Motion

The direct integration of the equations of motion makes it possible to determine the complete dynamic response for each time step. The accuracy of the solution is directly dependent on the size of the time step. This should be selected with respect to the frequency content  $\omega_p$  of the load to

$$\Delta t \leq T_p / 10 \quad (13)$$

with  $T_p$  oscillation time of the excitation frequency [s]. Higher frequencies are filtered out by the integration.

During integration the equations (1) through (3) at time  $t + \Delta t$  are used taking under consideration appropriate initial conditions.

To determine the respective displacement response, each integration step involves, among other things, the calculation of an effective load vector including the inertial and damping forces with subsequent solving of the entire equation system.

Because direct integration is very complex in terms of the calculation and requires a large amount of memory, it should only be used for special dynamics problems, for instance for the analysis of structures with viscous dampers.

Even dynamic analyses of cable structures are possible with the help of direct time step integration. In each time step a nonlinear equilibrium iteration is carried out that also takes into account the cable inertial and damping forces.

## Stationary Response

When dealing with a periodic excitation force, after a certain transient phase the particulate solution part of the differential equations alone represents the system response. The homogeneous solution part converges to zero due to the system damping. Because the system response follows the excitation force function in the steady state, time integration for a single excitation frequency is not particularly useful. However, determining the respective maximum amplitudes as a function of the corresponding excitation frequencies yields important information.

Applying a cosine excitation force yields:

$$p(t) = p^* \cos(\omega_p \cdot t + \varphi) = p_{\text{re}}^* \cos(\omega_p \cdot t) - p_{\text{im}}^* \sin(\omega_p \cdot t) \quad (14)$$

with

- $p^*$  Force amplitude vector (load vector)
- $\omega_p$  Angular frequency of the excitation with  $\omega_p = 2\pi \cdot f_p$
- $\varphi$  Phase shift of the excitation

A corresponding particulate solution approach results in:

$$u(t) = u^* \cos(\omega_p \cdot t + \psi) = u_{\text{re}}^* \cos(\omega_p \cdot t) - u_{\text{im}}^* \sin(\omega_p \cdot t) \quad (15)$$

with

- $u^*$  Deformation amplitude vector  
(maximum displacement in steady state)
- $\psi$  Phase shift of the response

Inserting this into the equations of motion yields the system of equations for the real and imaginary parts of the system response as a function of the excitation frequency.

$$\begin{aligned} (-\omega_p^2 M + K) u_{\text{re}}^* - \omega_p C u_{\text{im}}^* &= p_{\text{re}}^* \\ \omega_p C u_{\text{re}}^* + (-\omega_p^2 M + K) u_{\text{im}}^* &= p_{\text{im}}^* \end{aligned} \quad (16)$$

For modal coordinates this once again results in unlinked equations.

## System Excitation Caused by Soil Acceleration

A load resulting from soil acceleration can be replaced with a load of the unsupported structure nodes by transforming the reference system.

The effective structure load is described by the following load vector:

$$p(t)_{\text{eff}} = p_{\text{eff}} f(t) = -M (T_s \ddot{u}_b) f(t) = -M \ddot{u}_k f(t) \quad (17)$$

with

- $f(t)$  Excitation as a function of time
- $\ddot{u}_b$  Vector of the soil acceleration
- $T_s$  Transformation matrix
- $\ddot{u}_k$  Vector of the node acceleration

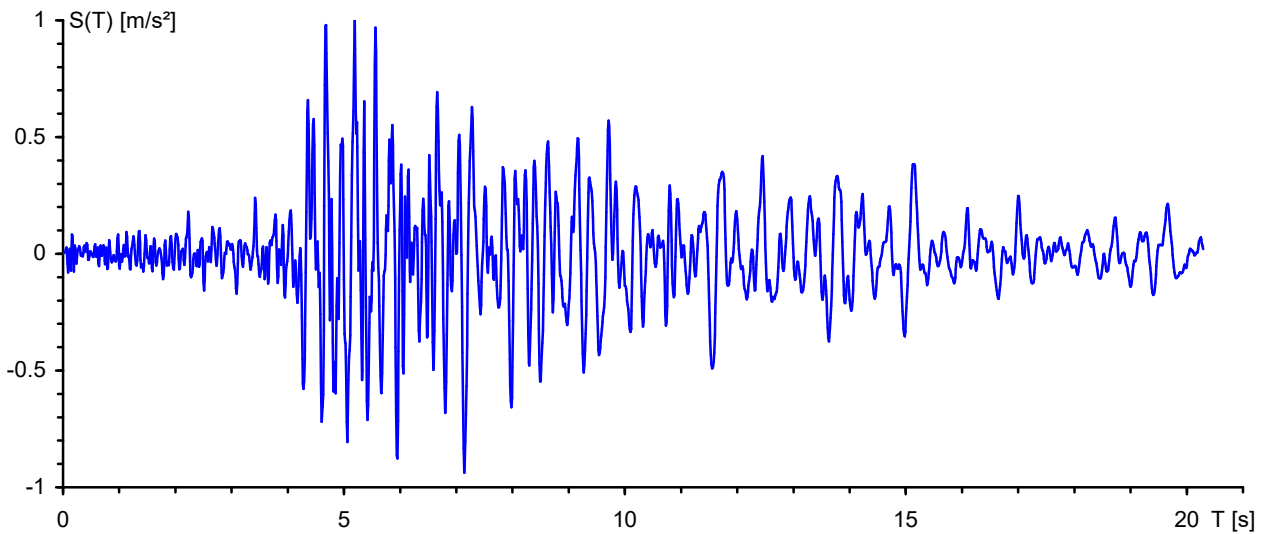
This relationship ignores damping forces resulting from the soil acceleration as well as the link between the supported and free elements of the mass matrix. These influences are, however, of low significance when compared with the inertial forces.

The transformation matrix  $T_s$  expresses the static influence of the column displacement on the other system nodes. For structures supported in a statically determined fashion or for stiff soil layers, this is described by strictly kinematic relationships.

The deformations and velocities are calculated in the accompanying coordinate system. The accelerations relate to absolute coordinates.

## Response Spectrum Method

The response spectrum method is used to calculate the deformations and internal forces of structures stressed by earthquakes. In this case, instead of analyzing the support point excitation of the system, the maximum response of the significant eigenmodes is calculated with the help of an acceleration response spectrum. This can be taken from a design standard or can be determined through integration of the equation of motion for special cases.



Soil acceleration from earthquakes standardized to  $\ddot{u}_b = 1.0 \text{ m/s}^2$

The modified equation of motion (12) in modal coordinates:

$$\ddot{x}_i(t) + 2\omega_i \xi_i \dot{x}_i(t) + \omega_i^2 x_i(t) = -\varphi_i^T M^{-1} t_s \ddot{u}_b(t) = -\beta_i \ddot{u}_b(t) \quad (18)$$

The solution of which is derived by solving:

$$x_i(t) = \beta_i S_d^*(t) \quad (19)$$

by using the DUHAMEL integral

$$S_d^*(t) = \frac{1}{\omega_i \sqrt{1-\xi_i^2}} \int_0^t \ddot{u}_b(\tau) e^{-\xi_i \omega_i (t-\tau)} \sin[\omega_i \sqrt{1-\xi_i^2} (t-\tau)] d\tau \quad (20)$$

In this case

$\beta_i$  Fraction factor

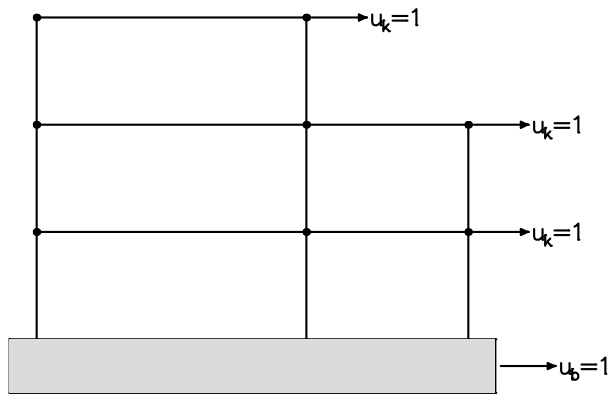
$\omega_i$  Angular frequency of the  $i$ -th eigenmode

$\xi_i$  Lehr's damping measure of the  $i$ -th eigenmode

$\ddot{u}_b(t)$  Soil acceleration from earthquake

The vector  $t_s$  expresses the proportionate node displacements that result from applying a unit deformation of the supported system nodes. The program assumes that these are the kinematic displacements of the entire system (see 'System excitation from soil acceleration').



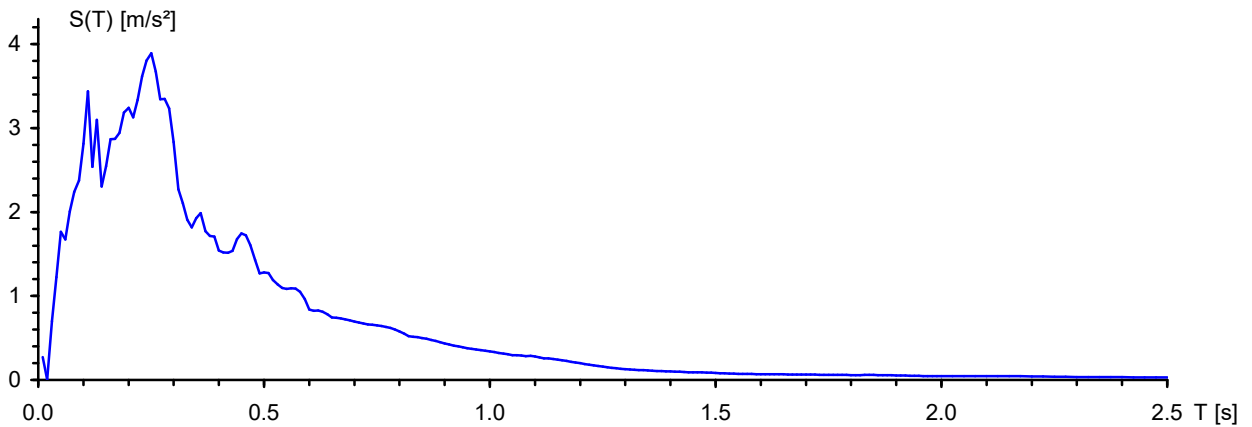


The ordinates of the displacement, velocity and acceleration response spectra result from the maximum ordinates of  $S_d^*(t)$  for different oscillation times.

$$S_d = S_v(1/\omega_i) = S_a(1/\omega_i^2) = \max | S_d^*(t) | \tag{21}$$

These are generally standardized and smoothed (see also the response spectra of the various standards).

The following illustration shows an unsmoothed, elastic acceleration response spectrum calculated on the basis of the soil acceleration shown above. A damping of 5% was used.



The static pseudo loads of the corresponding eigenmodes required for further calculation are determined from the acceleration response spectrum in the following manner:

$$H_{E,i} = -\beta_i S_{a,i} M \varphi_i \tag{22}$$

Deformations and internal forces result for each eigenmode that can optionally be statistically superposed before being used further.

An important parameter for evaluating whether sufficient modal components have been considered is the so-called effective modal mass. It results to the following under consideration of (8):

$$M_{\text{eff}} = \sum_i \beta_i^2 \tag{23}$$

If all eigenmodes are considered, this corresponds to the total mass of the analyzed structure. The general requirement is that the effective modal mass should amount to at least 90% of the total mass (see SIA 261, EC 8, etc.). This requirement is already fulfilled for conventional buildings by considering the initial significant eigenmodes.

For structures with many torsion-flexible beams and rotatory mass distribution or quasi-stiff partial systems, meeting this requirement can be problematic due to the fact that a large number of eigenmodes must be calculated. Therefore, you should consider changing the calculation model or the selected mass distribution in this case.

The additional masses which are not included in the net mass of the structure as well as the masses due to fraffic which are to be observed according to the standard, should be specified by the user using appropriate *Masses from loads* or *Point masses*.

## ZPA method

The ZPA method (Zero Period Acceleration) is one way of taking missing effective masses into account. It is also called static correction. Here additional static load cases are calculated, which capture the missing effective masses with a spectral value to be specified. Assuming that the higher-order modes that are not detected have no dynamic amplification, this method can be useful.

In general, the static correction load vector  $p_c$  can be calculated as follows for each direction of acceleration:

$$p_c = p - \sum_i r_i (M \cdot \varphi_i) \quad (24)$$

With the modal load of  $r_i$  according to equation 12 and the approach from equation 18 for the earthquake load follows:

$$r_i(t) = \varphi_i^T \cdot p(t) = -\varphi_i^T \cdot M \cdot \ddot{u}_b(t) \cdot t_s = -\beta_i \cdot \ddot{u}_b(t)$$

The acceleration  $\ddot{u}_b(t)$  is defined due to a selected spectral value  $S_{a,ZPA}$  (usually  $S_a(T=0)$ ) of the response spectrum. From equation 24 results:

$$p_c = -M \cdot \ddot{u}_b \cdot t_s + \sum_i \beta_i \cdot \ddot{u}_b \cdot (M \cdot \varphi_i) = S_{a,ZPA} \cdot M \cdot \left( \sum_i (\beta_i \cdot \varphi_i) - t_s \right) = S_{a,ZPA} \cdot M \cdot \psi$$

The expression  $\psi$  in the equation can be seen as a kind of eigenmode of the mass portion, which is not yet taken into account. The effective masses of the ZPA portion of all  $k$  nodes in the mass matrix are:

$$M_{eff,ZPA} = \sum_k (M_k \cdot \psi_k)$$

The results of the additional static load cases (correction loads) are also treated like other eigenmodes in the statistical superposition. The SRSS method is used as the combination rule, because there is no resonant frequency for the ZPA portion.

## Response Spectrum EN 1998-1:2013 (Design Spectrum)

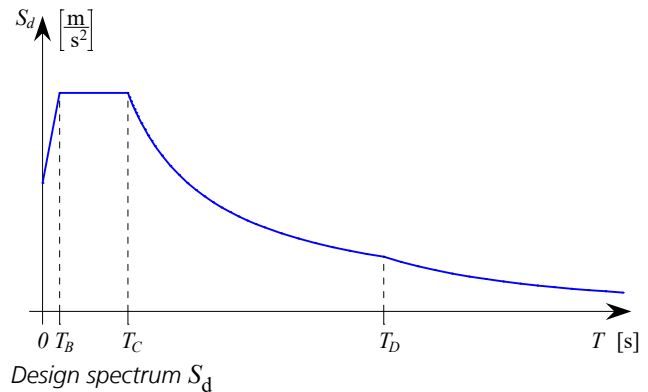
The ordinate values  $S_d$  required for the design spectrum according to the basic standard EN 1998-1 (EC8) result from the following equations:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot S \left[ \frac{2}{3} + \frac{T}{T_B} \left( \frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases}$$

$$T_D \leq T: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases}$$



The following parameters are required to determine the design spectrum:

- $\gamma_I$  Importance factor according to Section 4.2.5.
- $a_g$  Design value of the soil acceleration for subsoil class A with  $a_g = \gamma_I \cdot a_{gR}$ . The reference peak soil acceleration  $a_{gR}$  is to be specified by the user in the setting dialog ( $a_x, a_y, a_z$ ). The factor for the vertical soil acceleration must be considered by the user according to Table 3.4. For spectra of type 1 the vertical soil acceleration is to be reduced to 90 % and for spectra of type 2 to 45 % of the horizontal soil acceleration.
- $S, T_B, T_C, T_D$  Parameters according to Section 3.2.2. The design spectrum for the vertical earthquake action is internally calculated according to Table 3.4 with the parameters  $S=1.0, T_B=0.05, T_C=0.15$  and  $T_D=1.0$ .
- $q$  Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to the standard. For the vertical earthquake action,  $q$  is set to 1.0 in the calculation.
- $\beta$  Coefficient for the lower limit value for the horizontal design spectrum with  $\beta = 0.2$ .

### Statistical Combination Method

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. If the following requirement is fulfilled, then according to the standard the SRSS method is selected as superposition specification during automatic assignment, otherwise the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

### Response Spectrum DIN EN 1998-1:2021 (Design Spectrum)

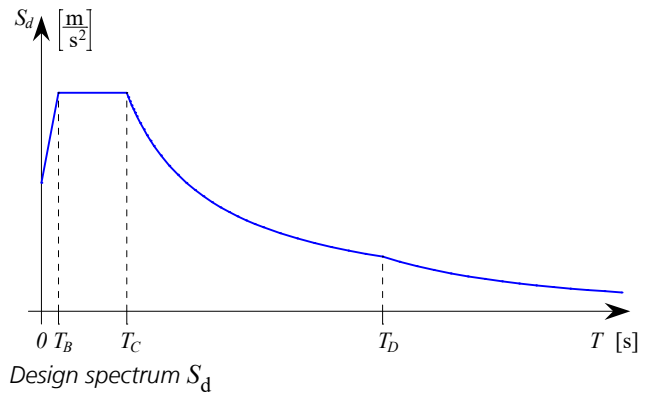
The ordinate values  $S_d$  required for the design spectrum according to DIN EN 1998-1 result from the following equations:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot S \left[ \frac{2}{3} + \frac{T}{T_B} \left( \frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases}$$

$$T_D \leq T: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases}$$



The following parameters are required to determine the design spectrum:

$\gamma_I$  Importance factor according to Table NA.5.

$a_g$  Design value of the soil acceleration for subsoil circumstance A-R with  $a_g = \gamma_I \cdot a_{gR}$  and  $a_{gR} = S_{aP,R} / 2.5$  with  $S_{aP,R}$  according to Figure NA.1 of the standard. The reference peak soil acceleration  $a_{gR}$  is to be specified by the user in the setting dialog ( $a_x, a_y, a_z$ ). The factor for the vertical soil acceleration must be considered by the user according to Table NA.3. According to this the vertical soil acceleration is to be reduced to 70 % of the horizontal soil acceleration.

$S, T_B, T_C, T_D$  Parameters according to NDP by 3.2.2.1(4), 3.2.2.2(2)P. Any deviations according to NA.E.1 and NA.E.2 for higher return periods shall be indicated by the user accordingly. The design spectrum for the vertical earthquake action is internally calculated according to Table NA.3 with the parameters  $S=1.0, T_B=0.05, T_C=0.20$  and  $T_D=1.20$ .

$q$  Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to the standard. For the vertical earthquake action,  $q$  is set to 1.0 in the calculation.

$\beta$  Coefficient for the lower limit value for the horizontal design spectrum with  $\beta = 0.0$ .

### Statistical Combination Method

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. If the following requirement is fulfilled, then according to the standard the SRSS method is selected as superposition specification during automatic assignment, otherwise the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

### Response Spectrum DIN 4149:2005

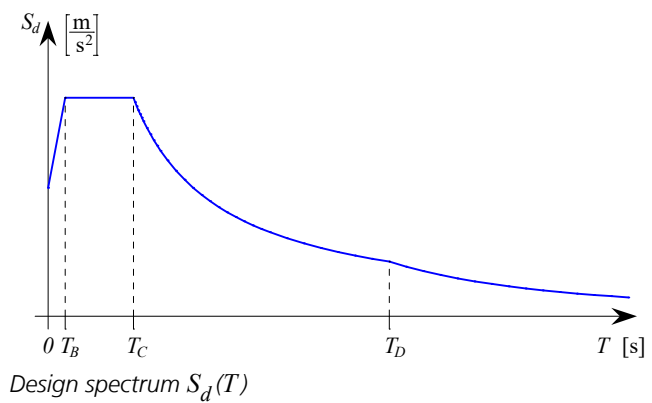
The ordinates of the design spectrum  $S_d(T)$  are determined by the following expressions:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \left[ 1 + \frac{T}{T_B} \left( \frac{\beta_0}{q} - 1 \right) \right]$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \cdot \frac{\beta_0}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \cdot \frac{\beta_0}{q} \cdot \frac{T_C}{T}$$

$$T_D \leq T: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \cdot \frac{\beta_0}{q} \cdot \frac{T_C T_D}{T^2}$$



The following parameters are required to determine the design spectrum:

- $T$  Vibration period ( $T = 1 / f$ ).
- $a_g$  Design value of the soil acceleration as a function of the earthquake zone according to DIN 4149, Table 2. It is to be specified by the user in the setting dialog ( $a_x, a_y, a_z$ ). The factor of 0.7 for the vertical soil acceleration according to Section 5.4.1 (4) must be considered by the user.
- $\gamma_I$  Importance coefficient according to Table 3.
- $\beta_0$  Amplification factor of the spectral acceleration with the reference value  $\beta_0 = 2.5$  for 5% viscous damping.
- $S, T_B, T_C, T_D$  Parameters according to Table 4 or 5.
- $q$  Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to Sections 8 through 12. For the vertical earthquake action,  $q$  is set to 1.0 in the calculation according to Chapter 8.3.3.2.2.

### Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. If the following requirement is fulfilled for all of the eigenmodes to be analyzed, then, according to the standard the SRSS method is selected as superposition specification during automatic assignment; otherwise, the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

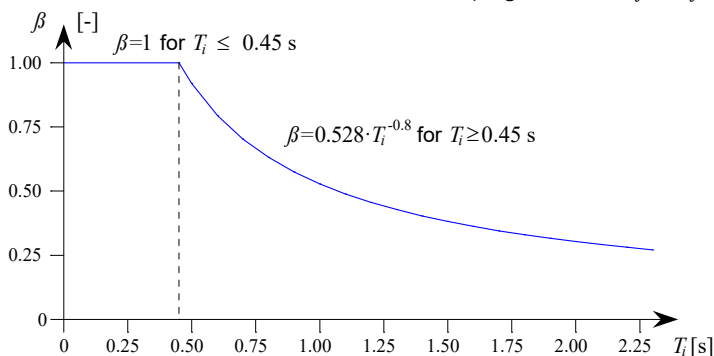
### Response Spectrum DIN 4149:1981

The ordinate values of the design spectrum  $S_{d,i}$  of the  $i$ -th eigenmode required for the response spectrum according to DIN 4149:1981 result from the following equation:

$$S_{d,i} = \beta a_0 \chi \alpha \tag{27}$$

with

$\beta = \beta(T_i)$  Coefficient of the standardized response spectrum (DIN 4149:1981, Part 1, Figure 2). An possible increase of the coefficient due to insufficient damping ( $D < 0.05$ :  $\beta^* = \beta \cdot 1.3$ ) can be determined, if necessary.



$T_i$	Vibration period of the $i$ -th resonant frequency.
$a_0$	Soil acceleration [ $m/s^2$ ] as a function of the earthquake zone (Section 7.2.1). It is to be specified by the user in the setting dialog ( $a_x, a_y, a_z$ ). The factor of 0.5 for the vertical soil acceleration must be considered by the user (Section 7.3).
$\chi$	Factor for the subsoil (Section 7.2.2).
$\alpha$	Reduction factor as a function of the subsoil class and the earthquake zone (Section 7.2.3).

**Statistical Combination Methods**

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. DIN 4149:1981 stipulates the SRSS method.

**Response Spectrum OENORM B 4015**

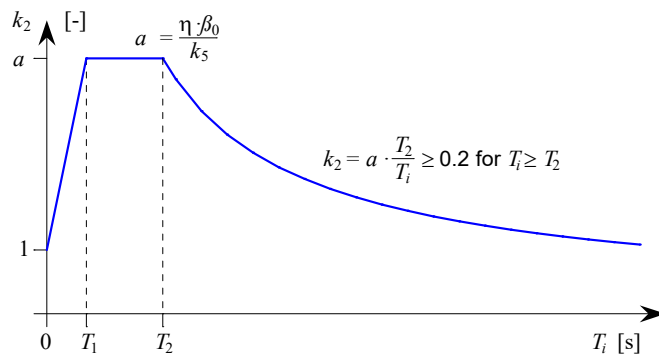
The seismic coefficients  $k_{E,i}$  of the  $i$ -th eigenmode required according to OENORM B 4015 result from the following equation:

$$k_{E,i} = \varepsilon \cdot k_1 \cdot k_2 \cdot k_{3,i} \cdot k_4$$

with

$\varepsilon$	Earthquake coefficient as a function of the soil acceleration $a_0$ and the gravitational acceleration $g$ with $\varepsilon = a_0/g$ . Make sure to use a factor of 0.67 for the effective vertical soil acceleration.
$k_1$	Foundation coefficient (Table 9).
$k_2$	Building reaction coefficient.
$k_{3,i}$	Distribution coefficient.
$k_4$	Factor for taking the building type into account (in general 1, for bridges according to Section 10.2).

For determining the building reaction coefficient  $k_2$  the following parameters are required:



$T_i$	Vibration period of the $i$ -th resonant frequency.
$T_1, T_2$	Reaction parameter [s] (Table 9).
$\eta$	Factor for determining the building damping with $\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$
$\xi$	Lehr's damping measure.
$\beta_0$	Reaction parameter as a function of the subsoil (Table 9).
$k_5$	Building ductility (ratio of the limit deformation and the elastic deformation, Table 12). If the value is greater than 1.5, $\eta$ is assumed to be 1 or greater.

**Statistical Combination Methods**

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method: If the following condition is fulfilled, according to OENORM B 4015 the SRSS method is selected as the superposition specification during automatic assignment, otherwise the CQC method is selected.

$$f_i / f_j < 1/(1 + 10\xi) \text{ with } f_i \leq f_j \text{ and } j = 1 \text{ to } n$$

**Response Spectrum SIA 261**

The ordinate values  $S_d$  required for the response spectrum according to SIA 261 result from the following equations:

$0 \leq T \leq T_B:$  
$$S_d = \gamma_F \frac{a_{gd}}{g} S \left[ 0.67 + \left( \frac{2.5}{q} - 0.67 \right) \frac{T}{T_B} \right]$$

$T_B \leq T \leq T_C:$  
$$S_d = 2.5 \gamma_F \frac{a_{gd}}{g} \frac{S}{q}$$

$T_C \leq T \leq T_D:$  
$$S_d = 2.5 \gamma_F \frac{a_{gd}}{g} S \frac{T_C}{T q}$$

$T_D \leq T:$  
$$S_d = 2.5 \gamma_F \frac{a_{gd}}{g} S \frac{T_C T_D}{T^2 q} \geq 0.1 \gamma_F \frac{a_{gd}}{g}$$

Design spectrum  $S_d$

The following parameters are required to determine the design spectrum:

- $\gamma_f$  Importance factor according to Table 25.
- $a_{gd}$  Soil acceleration according to Section 16.2.1.2 as a function of the earthquake zone. The factor of 0.7 for the vertical soil acceleration according to Section 16.2.4.2 must be considered by the user.
- $g$  Gravitational acceleration.
- $S, T_B, T_C, T_D$  Reaction parameter according to Table 24.
- $q$  Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to the standards SIA 262 to 267. For the vertical earthquake action,  $q$  is set to 1.5 in the calculation.

**Statistical Combination Method**

As the response spectrum only records maximum amplitudes, a superposition of the displacements and internal forces is carried out according to the SRSS method.

**Horizontal Elastic Response Spectrum EN 1998-1:2013**

The ordinate values  $S_e$  required for the horizontal elastic response spectrum according to the basic standard EN 1998-1 (EC8) result from the following equations:

$0 \leq T \leq T_B:$  
$$S_e(T) = a_g \cdot S \left[ 1 + \frac{T}{T_B} (\eta \cdot 2.5 - 1) \right]$$

$T_B \leq T \leq T_C:$  
$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$$

$T_C \leq T \leq T_D:$  
$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[ \frac{T_C}{T} \right]$$

$T_D \leq T \leq 4 s:$  
$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[ \frac{T_C T_D}{T^2} \right]$$

Elastic response spectrum  $S_e$

The following parameters are required to determine the horizontal elastic response spectrum needed for the pushover analysis:

$\gamma_I$	Importance factor according to Section 4.2.5.
$a_g$	Design value of the soil acceleration for subsoil class A with $a_g = \gamma_I \cdot a_{gR}$ . The reference peak soil acceleration $a_{gR}$ is to be specified by the user in the setting dialog ( $a_x, a_y$ ).
$S, T_B, T_C, T_D$	Reaction parameter according to Section 3.2.2.
$\eta$	Damping correction coefficient with the reference value $\eta = 1$ for 5 % viscous damping.
	$\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$
$\xi$	Lehr's damping measure.

## Horizontal Elastic Response Spectrum DIN EN 1998-1:2021

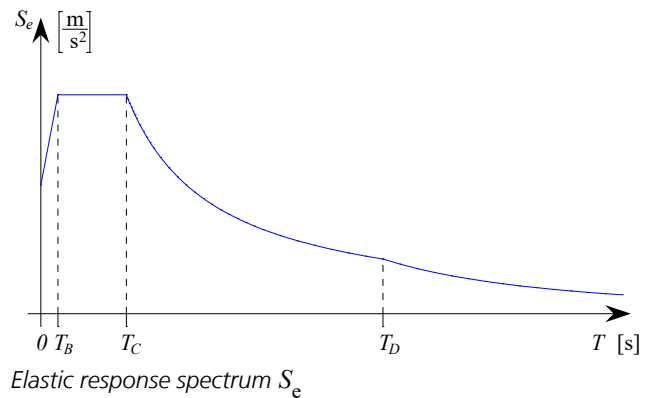
The ordinate values  $S_e$  required for the horizontal elastic response spectrum according to DIN EN 1998-1 result from the following equations:

$$0 \leq T \leq T_B: \quad S_e(T) = a_g \cdot S \left[ 1 + \frac{T}{T_B} (\eta \cdot 2.5 - 1) \right]$$

$$T_B \leq T \leq T_C: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$$

$$T_C \leq T \leq T_D: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[ \frac{T_C}{T} \right]$$

$$T_D \leq T \leq 4 \text{ s}: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[ \frac{T_C T_D}{T^2} \right]$$

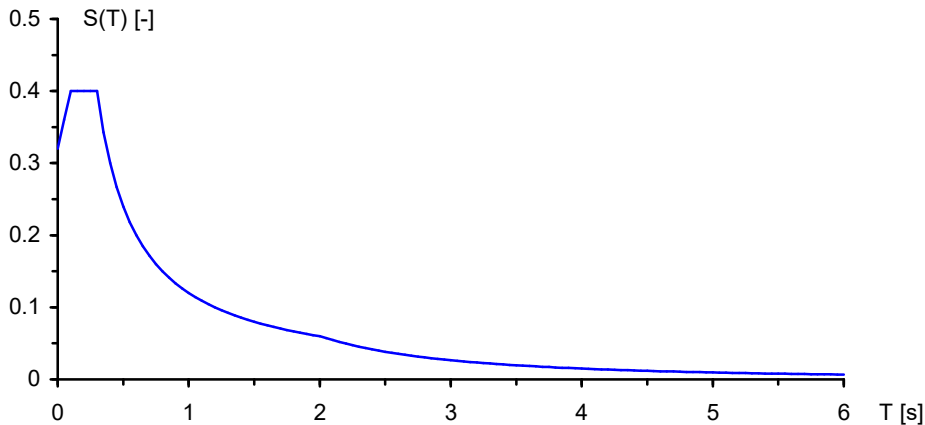


The following parameters are required to determine the horizontal elastic response spectrum needed for the pushover analysis:

$\gamma_I$	Importance factor according to Table NA.5.
$a_g$	Design value of the soil acceleration for subsoil class A with $a_g = \gamma_I \cdot a_{gR}$ and $a_{gR} = S_{aP,R} / 2.5$ with $S_{aP,R}$ according to Figure NA.1 of the standard. The reference peak soil acceleration $a_{gR}$ is to be specified by the user in the setting dialog ( $a_x, a_y$ ).
$S, T_B, T_C, T_D$	Parameters according to NDP by 3.2.2.1(4), 3.2.2.2(2)P. Any deviations according to NA.E.1 and NA.E.2 for higher return periods shall be indicated by the user accordingly.
$\eta$	Damping correction coefficient with the reference value $\eta = 1$ for 5 % viscous damping.
	$\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$
$\xi$	Lehr's damping measure.

## Alternative Response Spectrum

The curve of the alternative response spectrum can be defined freely. The alternative acceleration response spectrums ( $S_{a,x}$ ,  $S_{a,y}$ ,  $S_{a,z}$ ) are the arithmetic product of the specified soil accelerations ( $a_x$ ,  $a_y$ ,  $a_z$ ) and the user-defined standardized response spectrums (*standard* or *vertical*).



Example of an standardized alternative response spectrum

### Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method.

### Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to one of the following two statistical laws:

$$E = \sqrt{\sum_{i=1}^n E_i^2} \quad (\text{SRSS method})$$

$$E = \sqrt{\sum_{i=1}^n \sum_{j=1}^n E_i \cdot \rho_{ij} \cdot E_j} \quad (\text{CQC method})$$

with

$E_i$  Displacements and internal forces of the  $i$ -th eigenmodes.

$n$  Number of eigenmodes.

$$\rho_{ij} = \frac{8\xi^2 \cdot (1+r) \cdot r^{1.5}}{(1-r^2)^2 + 4\xi^2 \cdot r(1+r)^2}$$

$r = f_i / f_j = T_j / T_i$  Frequency ratio.

$\xi$  Lehr's damping measure.

The CQC method is only applicable if  $\xi > 0$ .

In the standard case the superposition specification is chosen automatically depending on the standard. If the following requirement, e.g. according to DIN 4149, is fulfilled for all of the eigenmodes to be analyzed, then, according to the standard the SRSS method is selected as superposition specification during automatic assignment; otherwise, the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

Given a symmetrical 3D structure, two congruent oscillation times often result ( $T_i = T_j$ ). Therefore, the automatic selection of the superposition specification always chooses the CQC method for such systems. If you wish to use a different method, you must select it manually.



## Earthquake Action

Within the checks in the ultimate limit state of the different standards an earthquake action ( $A_{Ed}$ ) is to be taken into account.

$$\sum_{j \geq 1} G_{k,j} \cdot P \cdot A_{Ed} \cdot \sum_{i \geq 1} \Psi_{2,i} \cdot Q_{k,i}$$

In general, to check the horizontal earthquake action two directions of the building section that are orthogonal to one another are analyzed.

These are to be combined in the form:

$$(E_{Edx} \oplus 0.3 \cdot E_{Edy}) \text{ or } (0.3 \cdot E_{Edx} \oplus E_{Edy})$$

Here the internal forces  $E_{Edx}$  or  $E_{Edy}$  can be determined through statistical superposition. The two following combinations illustrate this procedure.

### Load case combination 1, Edx "+" 0.3 Edy

1. Variable exclusive action	Factor
Response spectrum overlaid (x)	1.000
Response spectrum overlaid (x)	-1.000
2. Variable exclusive action	Factor
Response spectrum overlaid (y)	0.300
Response spectrum overlaid (y)	-0.300

### Load case combination 2, 0.3 Edx "+" Edy

1. Variable exclusive action	Factor
Response spectrum overlaid (x)	-0.300
Response spectrum overlaid (x)	0.300
2. Variable exclusive action	Factor
Response spectrum overlaid (y)	-1.000
Response spectrum overlaid (y)	1.000

The random torsion actions according to DIN 4149:2005, Section 6.2.3.3 or EN 1998-1, Section 4.3.2 are not included in the static pseudo loads that are calculated by the program. If necessary, you can specify them by providing additional load cases for the earthquake actions.

### Characteristic value of the sensitivity to story drift

The analysis of the load-bearing capacity according to DIN 4149:2005, Section 7.2.2 or EN 1998-1, Section 4.4.2.2, involves checking the characteristic value of the sensitivity to story drift (see example *Earthquake*). If this value exceeds 0.1, the reactions from the earthquake action should be increased, if necessary. Alternatively, you can carry out a calculation according to the second-order theory. The value of  $\theta$  may not exceed 0.3. This value cannot be automatically calculated by the program.

## Pushover Analysis EN 1998-1:2013 and DIN EN 1998-1:2021

The pushover analysis is a nonlinear method for the verification of earthquake safety of structures and can be used alternatively to the response spectrum method. Here, the displacement of the control node  $d_n$  is compared with the total earthquake force at foundation level  $F_b$  (capacity curve), which occurs under constant vertical loads and monotonically increasing horizontal loads. The method is suitable for largely regular structures whose basic eigenmode in the direction considered is not determined by torsional vibrations (Section 4.3.3.4.2.7, EN 1998-1). The procedure is described in Section 4.3.3.4.2 and Annex B, EN 1998-1. The method explained there is based on the N2 method developed by Fajfar et al.

Essentially, the implemented method is divided into two calculation steps:

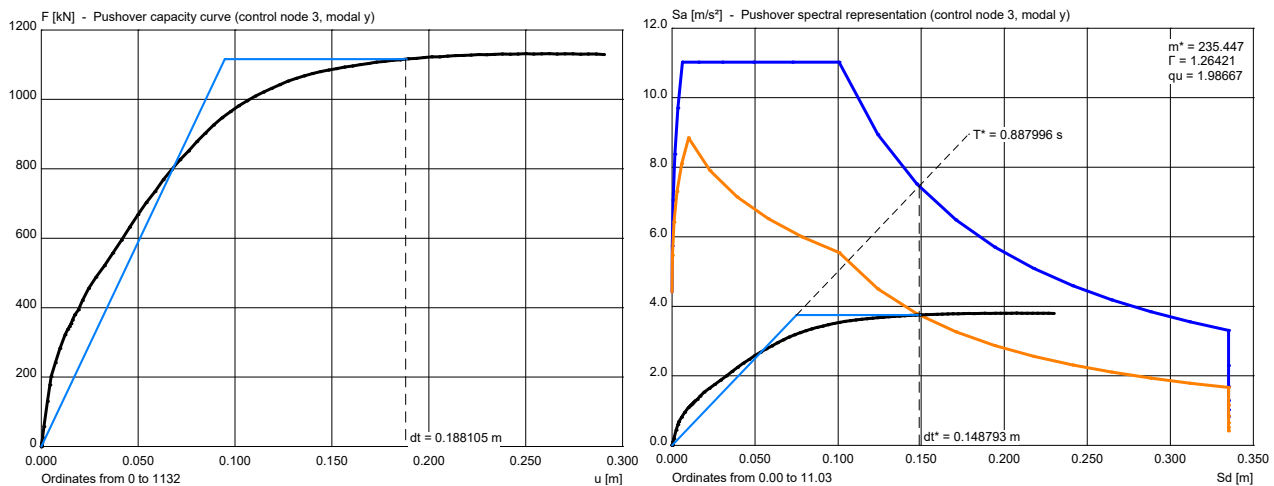
### 1. Dynamics:

Selection of the control node according to Section 4.3.3.4.2.3, definition of the elastic response spectrum, input of the soil accelerations, determination of the dynamic eigenmodes  $\phi_i$  and their normalization (displacement of the control node  $\phi_n = 1$ ), provision of selected horizontal load distributions as well as the calculation of the associated equivalent mass  $m^*$ , the transformation coefficient  $\Gamma$  and the ductility coefficient  $q_u$ . The basic modal form considered for the modal horizontal load distribution corresponds to the eigenmode with the largest effective modal mass in the desired direction.

### 2. Nonlinear structural analysis:

Definition of vertical and horizontal load cases with associated load time function, input of load groups for the distributions (*modal  $\pm x$ , constant  $\pm x$*  and possibly *modal  $\pm y$ , constant  $\pm y$* ), nonlinear system analysis with the Newton time step or the arc length method.

After the two calculation steps, the capacity curve in the multi-mass system and in the spectral representation are available. Their linearization is carried out iteratively in accordance with Section B.5, EN 1998-1.



By evaluating the elastic or inelastic response spectrum, this results in the target displacement  $d_t$ , which is also shown. To verify the earthquake stability the calculated target displacement should be held by an appropriate ductility of the structure.

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## Dynamic Train Crossing

This option offers a way to analyze the dynamic stress for any beam and shell structure based on predefined trains such as the *ICE*, *Thalys*, etc. or user-defined train loads.

### Tracks

The tracks are described with the load type '*Dynamic train load*' by entering any polygon on the structure. Multiple tracks can be considered simultaneously, allowing you, for example, to analyze the effect trains traveling in opposite directions have on each other.

### Train definition

The load model of the typical trains running on high-speed lines are stored in the program. Additionally, user-defined train loads can be added. Further default parameters are the speed and the departure time for each train. Every train is assigned to one track. You can define the intervals between the different trains by varying the departure times.

### Analysis

The dynamic train crossing can be analyzed as part of a direct or modal time-step integration. You can freely define the number and duration of the time steps. The vibration behavior and the desired route of the train are taken into consideration.

### Results

Results are calculated for the deformations, velocities and accelerations, as well as the internal forces for every time step. These can be represented individually or as time-step diagrams. Additionally, the deformation can be dynamically animated in the Systemviewer. Exclusive combinations of the results are available for further analysis.

# Input

The tabular input is made in the *Dynamics* folder of the database.

## Analysis Settings

This is where the desired calculation mode and the corresponding control values are set.

### Analysis mode

- **Eigenvalues**
  - Eigenvalues
  - Effect of the masses
- **Stationary response**
  - Eigenvalues
  - Effect of the masses
  - Consider the following load cases
  - Consider defined node accelerations
- **Response spectrum**
  - Eigenvalues
  - Effect of the masses
  - Soil acceleration
  - Coefficients as per EN 1998-1:2013, DIN EN 1998-1:2021, DIN 4149:2005, DIN 4149:1981, OENORM B 4015, SIA 261 (design spectrum)
  - Alternative response spectrum
  - Store static pseudo loads
- **Pushover analysis**
  - Eigenvalues
  - Effect of the masses
  - Soil acceleration
  - Coefficients as per EN 1998-1:2013, DIN EN 1998-1:2021 (horizontal elastic response spectrum)
  - Vertical distribution of horizontal loads (Modal x, Modal y, Constant x, Constant y)
  - Control node (Node)
- **Modal time step integration**
  - Eigenvalues
  - Effect of the masses
  - Zeitschrittintegration
  - Consider the following load cases
  - Consider defined node accelerations
- **Direct time step integration**
  - Effect of the masses
  - Zeitschrittintegration
  - Consider the following load cases
  - Consider defined node accelerations

The screenshot shows the 'Settings' dialog box for the 'Response spectrum' analysis mode. The 'Dynamics' tab is selected. The 'Analysis mode' is set to 'Response spectrum DIN EN 1998-1'. The 'Eigenvalues' section has 'Number' set to 5, 'Sub space' to 10, and 'Accuracy' to 1%. The 'Effect of the masses' section has checkboxes for 'In direction' (X, Y, Z) and 'Around axis' (X, Y, Z), all checked. The 'Soil accelerations' section has 'ax' and 'ay' set to 1.023 [m/s²] and 'az' set to 0. The 'Coefficients of DIN EN 1998-1:2021' section has 'Subsoil circumstances' set to 'A-R', 'Spectral accel.' set to 1, 'Sap,R' set to 0.1, and 'TC' set to 0.2. The 'Lehr's damping measure' is 0.05, 'Importance coefficient gamma I' is 1.2, and 'Behavior coefficient (horiz.) q' is 1.5. The 'Combination method' is 'Standard'. The 'Store static pseudo loads' checkbox is unchecked. Buttons for 'OK', 'Cancel', and 'Help' are at the bottom.

The screenshot shows the 'Settings' dialog box for the 'Direct time step integration' analysis mode. The 'Dynamics' tab is selected. The 'Analysis mode' is set to 'Direct time step integration'. The 'Eigenvalues' section has 'Number' set to 10, 'Sub space' to 18, and 'Accuracy' to 1%. The 'Effect of the masses' section has checkboxes for 'In direction' (X, Y, Z) and 'Around axis' (X, Y, Z), all checked. The 'Time step integration' section has 'Time steps' set to 6000, 'Duration' set to 0.002 [s], and 'Results all n steps' set to 2. The 'Static initial condition (LC)' is set to '600 Engine and compressor Py'. The 'Type of load-time function' is 'Load related', 'Instationary load-time function' is set to a dropdown, 'Load frequency' is 0 [Hz], and 'Phase shift' is 0 [rad]. The 'Consider the following load cases' section has a list containing '600 Engine and compressor Py +Pz', '501 Engine Pz', '502 Compressor Pz', '511 Engine Py', and '512 Compressor Py'. The 'Consider defined node accelerations' checkbox is unchecked. Buttons for 'OK', 'Cancel', and 'Help' are at the bottom.

## Eigenvalues

*Number of eigenvalues:* Up to 999 eigenvalues and vectors can be calculated.

*Max. vector space:* Size of the vector space to be used during the eigenvalue iteration. The number of iteration vectors must always be greater than or equal to the number of the selected eigenvalues.

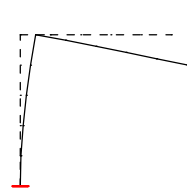
*Iteration accuracy:* Size of the maximum relative error [%].

*Stiffness according 2nd order (LC):* Calculation of eigenvalues with a stiffness matrix, which take into account the geometrical nonlinear part of the second-order theory. Therefore the internal forces of the statical load case, which is selected here, are used.

## Effect of the Masses

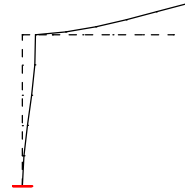
Selection of the mass portions implemented in the mass matrix.

All masses



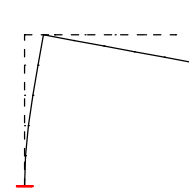
Eigenmode 1: 6.43 Hz

All masses



Eigenmode 2: 17.48 Hz

Masses only in x-direction



Eigenmode 1: 8.56 Hz

**Attention:** With the elimination of mass portions maybe differing eigenmodes and eigenfrequencies result!

## Soil Acceleration

$a_x, a_y$ : Vector with the standard value of the horizontal acceleration of the soil as a function of the earthquake zone [ $m/s^2$ ].

$a_z$ : Principle value of the vertical acceleration of the soil. In general this does not need to be considered.

*Apply components separately:* Static pseudo load cases are applied separately for each direction.

*Reference system angle Alpha:* Rotation of the soil accelerations  $a_x$  and  $a_y$  about the global z axis [°].

*ZPA-Method:* Use of the ZPA method.

*ZPA-T:* Oscillation time  $T$  [s] to calculate the spectral value  $S_{a,ZPA}$ .

## Coefficients as per EN 1998-1:2013 (Design Spectrum)

*Ground type:* Define the parameter values according to the basic standard EN 1998-1, Table 3.2 or 3.3.

*Parameters  $S, T_B, T_C, T_D$ :* Coefficients for determining the design spectrum for the horizontal earthquake action. The parameters are suggested in accordance with the ground type and the type of the response spectrum. The design spectrum for the vertical earthquake action is internally calculated according to Table 3.4 with the coefficients  $S=1.0, T_B=0.05, T_C=0.15$  and  $T_D=1.0$ .

*Lehr's damping measure:* Measure  $\xi$  for determining the factor that accounts for building damping  $\eta$ . For the design spectrum according to Section 3.2.2.5 5% viscous damping is assumed.

*Importance factor  $\gamma_I$ :* Factor according to Section 4.2.5.

*Behavior coefficient (horiz.)  $q$ :* The capacity of a structure to resist the earthquake action in the nonlinear zone with post-limit stiffness can be accounted for with the behavior coefficient  $q$ . It is used for the horizontal earthquake action. For vertical earthquake actions  $q$  is internally set to 1.0 by the program.

The button  opens a dialog which shows the related spectral acceleration  $S_d(T) / a_g$  (with  $a_g = \gamma_I \cdot a_{gR}$ ).

*Combination method:*

- *Standard:* The program automatically selects one of the two following combination methods (see *Basics > Response spectrum EN 1998-1:2013*).
- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

## Coefficients as per DIN EN 1998-1:2021 (Design Spectrum)

*Subsoil circumstances 475 years:* Define the parameter values according to DIN EN 1998-1, Table NA.1 or NA.2.

*Parameters  $S$ ,  $T_B$ ,  $T_C$ ,  $T_D$ :* Coefficients for determining the design spectrum for the horizontal earthquake action. The parameters are suggested in accordance with the subsoil circumstances and the spectral acceleration  $S_{aP,R}$ . The design spectrum for the vertical earthquake action is internally calculated according to Table NA.3 with the coefficients  $S=1.0$ ,  $T_B=0.05$ ,  $T_C=0.20$  and  $T_D=1.20$ .

*Lehr's damping measure:* Measure  $\xi$  for determining the factor that accounts for building damping  $\eta$ . For the design spectrum according to Section 3.2.2.5 5% viscous damping is assumed.

*Importance factor  $\gamma_I$ :* Factor according to Table NA.5.

*Behavior coefficient (horiz.)  $q$ :* The capacity of a structure to resist the earthquake action in the nonlinear zone with post-limit stiffness can be accounted for with the behavior coefficient  $q$ . It is used for the horizontal earthquake action. For vertical earthquake actions  $q$  is internally set to 1.0 by the program.

The button  opens a dialog which shows the related spectral acceleration  $S_d(T) / a_g$  (with  $a_g = \gamma_I \cdot a_{gR}$ ).

*Combination method:*

- *Standard:* The program automatically selects one of the two following combination methods (see *Basics > Response spectrum DIN EN 1998-1:2021*).
- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

## Coefficients as per DIN 4149:2005


*Subsoil circumstances:* Define the subsoil parameters according to DIN 4149:2005, Table 4 or 5.

*Subsoil parameters  $S$ ,  $T_B$ ,  $T_C$ ,  $T_D$ :* Coefficients for determining the design spectrum.

*Lehr's damping measure:* Measure  $\xi$  for determining the damping correction coefficient  $\eta$ . For the design spectrum according to Section 5.4.3, 5% viscous damping is assumed.

*Importance coefficient  $\gamma_I$ :* Factor according to Table 3.

*Behavior coefficient (horiz.)  $q$ :* Coefficient to take into account the energy dissipation capacity of a structure according to Sections 8 to 12. For vertical earthquake actions  $q$  is internally set to 1.0 by the program.

The button  opens a dialog which shows the related spectral acceleration  $S_d(T) / a_g$ .

*Combination method:*

- *Standard:* The program automatically selects one of the two following combination methods (see *Basics > Response spectrum DIN 4149:2005*).
- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.


## Coefficients as per DIN 4149:1981

*Subsoil factor  $\kappa$ :* Factor to account for the subsoil (DIN 4149:1981, Section 7.2.2).

*Factor for regard of the building class  $\alpha$ :* Reduction factor as a function of the subsoil class and the earthquake zone (Section 7.2.3).

*Increase coefficient  $B$  by 30%:* The coefficient  $\beta$  (Figure 2) is increased by 30%. According to Section 8.1 this is necessary for structures with a damping measure  $D < 0.05$ .

*Lehr's damping measure:* Damping factor of the structure for calculation of the interaction factor when combining the modal components according to the CQC method.

The button  opens a dialog which shows the related spectral acceleration  $\beta(T) \cdot \kappa \cdot \alpha$ .

*Combination method:*

- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

## Coefficients as per OENORM B 4015

*Type of the subsoil:* Define the foundation coefficient and the reaction parameters according to OENORM B 4015, Table 9.

*Foundation coefficient  $k_1$ :* Value according to Table 9.

*Reaction parameters  $T_1, T_2, B_0$ :* Coefficients for determining the building reaction coefficient  $k_2$ .

*Lehr's damping measure:* Measure  $\xi$  for determining the factor to account for building damping  $\eta$  and the interaction factor for the combination of the modal components according to the CQC method.

$$\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$$

*Building ductility  $k_5$ :* Ratio of the limit deformation and the elastic deformation.

*Factor for regard of the building type:* The factor  $k_4$  should generally be set to 1.

The button  opens a dialog which shows the related spectral acceleration  $k_1 \cdot k_2 \cdot k_4$ .

*Combination method:*

- *Standard:* The program automatically selects one of the two following combination methods (see *Basics > Response spectrum OENORM B 4015*).
- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

## Coefficients as per SIA 261


*Subsoil class:* Define the reaction parameters according to SIA 261, Table 24.

*Reaction parameters  $S, TB, TC, TD$ :* Coefficients for determining the design spectrum.

*Lehr's damping measure:* Measure  $\xi$  for determining the factor that accounts for building damping  $\eta$ . For the design spectrum according to Section 16.2.4.1 5% viscous damping is assumed.

*Importance factor  $\gamma_f$ :* Factor according to Table 25.

*Behavior coefficient (horiz.)  $q$ :* The capacity of a structure to resist the earthquake action in the nonlinear zone with post-limit stiffness can be accounted for with the behavior coefficient  $q$ . For reinforced concrete the value can be taken from Table 14, SIA 262. It is used for the horizontal earthquake action. For vertical earthquake actions  $q$  is internally set to 1.5 by the program.

The button  opens a dialog which shows the related spectral acceleration  $S_d(T) \cdot g / a_{gd}$ .

*Combination method:*

SRSS method: Square root of the sum of the squares.

## Coefficients as per EN 1998-1:2013 (Horizontal Elastic Response Spectrum)

*Ground type:* Define the parameter values according to the basic standard EN 1998-1, Table 3.2 or 3.3.

*Parameters  $S, TB, TC, TD$ :* Coefficients for determining the horizontal elastic response spectrum for the earthquake action.

The parameters are suggested in accordance with the ground type and the type of the response spectrum.

*Lehr's damping measure:* Measure  $\xi$  for determining the factor that accounts for building damping  $\eta$ .

*Importance factor  $\gamma_I$ :* Factor according to Section 4.2.5.

The button  opens a dialog which shows the related spectral acceleration  $S_e(T) / a_g$  (with  $a_g = \gamma_I \cdot a_{gR}$ ).

## Coefficients as per DIN EN 1998-1:2021 (Horizontal Elastic Response Spectrum)

*Subsoil circumstances 475 years:* Define the parameter values according to DIN EN 1998-1, Table NA.1 or NA.2.

*Parameters  $S$ ,  $TB$ ,  $TC$ ,  $TD$ :* Coefficients for determining the horizontal elastic response spectrum for the earthquake action.

The parameters are suggested in accordance with the subsoil circumstances and the spectral acceleration  $S_{aP,R}$ .

*Lehr's damping measure:* Measure  $\xi$  for determining the factor that accounts for building damping  $\eta$ .

*Importance factor  $\gamma_I$ :* Factor according to Table NA.5.

The button  opens a dialog which shows the related spectral acceleration  $S_e(T) / a_g$  (with  $a_g = \gamma_I \cdot a_{gR}$ ).

## Alternative Response Spectrum

*Combination method:*

- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

*Lehr's damping measure:* Damping factor of the structure for calculation of the interaction factor when combining the modal components according to the CQC method.

*Standard:* Function of the standard response spectrum.

*Vertical:* Function of the vertical response spectrum. The standard response spectrum will be used for the vertical direction, if no function is defined.

## Store Static Pseudo Loads

The static pseudo loads of the modal components that are calculated for an earthquake analysis with response spectra are saved and can be subsequently imported into a load case. If necessary, this can be done separately for the accelerations directions.

## Time Step Integration

*Number of time steps:* Number of time steps to be carried out for the integration of the equations of motion.

*Duration of a time step:* Duration of a time step [s].

*Results all  $n$  steps:* Saving every  $n$ -th time step reduces the volume of data. The accuracy of the integration is not affected, but the quality of the result representation is, however.

*Static initial condition (LC):* The internal forces and deformations of the selected load case are applied as static initial condition (condition at time  $t=0$ ) for the direct time step integration. This is important for non linear analyses (e.g. cable structures) to account for the permanent actions previous to the time variable loads. In this case a superposition is not allowed. The loads (e.g. dead load) which are part of the initial condition are also to be taken into account for the corresponding dynamic load case (see below) to ensure the state of equilibrium. They can have a variable progression over the time as well.

*Type of load-time function*

- *Periodic at the overall system:* Congruent cosinusoidal excitation of all degrees of freedom with  $f(t) = \cos(2\pi \cdot f_p \cdot t + \varphi_p)$ .
- *Instationary at the overall system:* Congruent instationary excitation of all degrees of freedom.
- *Node related:* Different load-time functions for the various nodes or degrees of freedom, which are defined in the table 'Node related load-time function'.
- *Load related:* The load-time functions specified in the loads are used.
- *Train crossing:* The train loads are geometrically defined by assigning them to the corresponding track. The track must run completely on or in the structure. If axle loads on the track cannot be assigned to an element, an error message is issued. The time-dependent position of the axle loads is a function of the speed and the start time. If you define other load types in addition to the train loads for a load case, these are considered to be constant across the integration time.

*Instationary load-time function:* Number of the instationary load function.

*Load frequency:* Frequency  $f_p$  of the periodic excitation on the entire system [Hz].

*Phase shift:* Phase shift  $\varphi_p$  of the periodic excitation on the entire system [rad].



## Consider the Following Load Cases

Calculate the load cases from the left list box.

## Consider Defined Node Accelerations

Activate the defined node accelerations.

## Point Masses

This function of the *structure* menu is used to define the point masses for dynamic analysis. They act in addition to the masses from the selected load cases and the mass distribution of the structure, which are automatically determined by the program. The point masses are linked to the element nodes and are represented with a filled circle.

### Off

Remove point masses from the graphical view.

### Enter

Define point masses at the selected nodes by specifying their components.

## Components of a point mass

### Mux, Muy, Muz

Translational masses in direction of the global axes [*t*].

### Mphi.x, Mphi.y, Mphi.z

Rotatory masses about the global axes [*t m<sup>2</sup>*].

## Generate Masses From Loads

It allows you to select load cases from which to create additional nodal masses.

This is done by dividing nodal loads in the global z direction that result from the load cases by the gravitational acceleration (9.81 m/s<sup>2</sup>) and applying them as additional point masses (translational masses) in the x, y and z directions.

The following load types, which do not represent vertical loads in this sense, are ignored by the program.

- Influence surface, Influence line
- Insert
- Dynamic element collapse
- Dynamic train load
- Fire scenarios
- Buckling eigenvalues
- Creep and shrinkage
- Support displacement
- Superposition
- Theory
- Initial strain, Temperature
- Prestressing
- Load group
- Point moment on beams
- Fixed end forces of beams
- Free line moments on beams and area elements
- Line torsional moment on beams

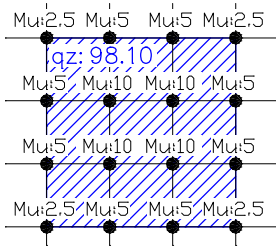
The *Dead load* load type is also not taken into account as the dead load of the system is already automatically included in the mass matrix.

### Load cases

Select load cases to add them to the current action.

### Load factor

The individual load cases are weighted by a factor.



Mass distribution for an element grid of 1 m and a load  $q_z$  of 98.1 kN/m<sup>2</sup>

The point masses of load cases resulting from dynamic analysis can be examined with the database using 'Results, Masses generated from loads'.

## Node Accelerations

The inertial forces that result from the specified node accelerations are added to the loads. Therefore, for pure soil excitation no external loads should be defined.

Node accelerations [ACCE]					
	From Node	To node	ax	ay [m/s <sup>2</sup> ]	az
1	1	1000	1.0000	0.0000	0.0000
2					

### From node, to node

Node group that is accelerated.

### ax, ay, az

Components of the node acceleration in global direction [m/s<sup>2</sup>].

## Modal Damping

This table is used to specify the damping parameters for the natural damping of the system. In the program the following formula is used for modal damping  $\xi_i$ .

$$\xi_i = D + (\alpha + \beta \omega_i^2) / (2 \omega_i)$$

In general it only makes sense to specify the damping either using Lehr's damping measure or the two factors  $\alpha$  and  $\beta$ . Singular damping elements can only be described through a direct integration of the equations of motion.

Modal damping [DAEMPFUNG]				
Eigenmode	D [%]	alpha [1/s]	beta [s]	
1	2.000	0.00000	0.00000	▲
2	2.000	0.00000	0.00000	■
3	2.000	0.00000	0.00000	▼

### Eigenmode

Number of the eigenmode.

### D

Lehr's damping measure of the eigenmode (in % of the critical damping).

### alpha

Mass-proportional damping [1/s].

### beta

Stiffness-proportional damping [s].

## Lehr's Damping Measure

Lehr's damping measure (after Ernst Lehr) is also called damping ratio. The value  $\xi = 1$  corresponds to critical damping.

Lehr's damping measure  $\xi$  for the elastic zone from OENORM B 4015, Table 11.

		$\xi$
Construction type	Steel constructions	
	- screwed	0.005 bis 0.010
	- welded	0.003 bis 0.005
	Reinforced concrete constructions	0.005 bis 0.020
	Prestressed concrete constructions	0.005 bis 0.015
Additions	Wood constructions	0.010 bis 0.030
	Masonry buildings	0.010 bis 0.030
Examples for Buildings (incl. foundation)	Damping installations (e.g. brick lining)	0.005
	Tower-like buildings	
	- made of steel	0.004 bis 0.008
	- made of reinforced concrete	0.006 bis 0.010
	Bridges	
	- made of steel	0.005 bis 0.015
	- made of reinforced or prestressed concrete	0.006 bis 0.020
	Frame constructions	
	- made of steel	0.010 bis 0.020
	- made of reinforced concrete	0.015 bis 0.025

The relationship between Lehr's damping measure  $\xi$  and the logarithmic damping decrement  $\Lambda$  is:

$$\xi = \frac{\Lambda}{\sqrt{(2\pi)^2 + \Lambda^2}}$$

Approximate values for the logarithmic damping decrement are listed e.g. in EN 1991-1-4, Table F.2.

## Node-Related Load-Time Functions

The degrees of freedom of a node group can be assigned to different excitation force functions.

Node related load-time function [DYNLAST]						
	From node	To node	Load freq. [Hz]	Phase shift [rad]	Instationary load time f.	Load component
1	2	11	3.0000	0.0000	0	*
2						

### From node, to node

Node group.

### Load frequency

Excitation frequency  $f_p$  [Hz] for periodic load  $f(t) = \cos(2\pi \cdot f_p \cdot t + \varphi_p)$ .

### Phase shift

Phase shift  $\varphi_p$  for cosinusoidal excitation [rad].

### Instationary load-time function

Number of an instationary load-time function. The load frequency must be set to zero.

### Load component

Decisive load components  $P_x, P_y, P_z, M_x, M_y, M_z$  of a node. Use "\*" to apply all load components.

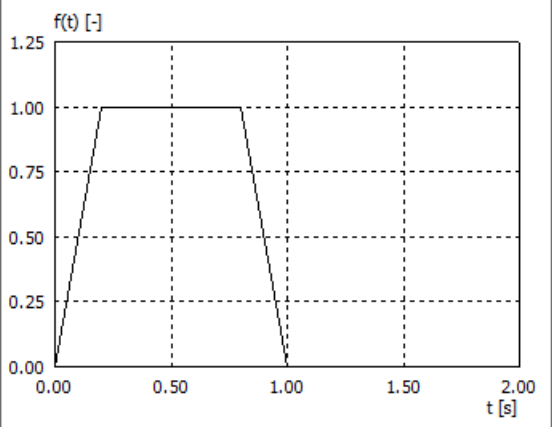
## Instationary Load-Time Function

This dialog is used to define an instationary load-time function that describes the progression  $f(t)$  of the load over time. The function value specifies the load factor at the corresponding time. The last function value is applied until  $t = \infty$ .

**Load-time Function 1** ✕

Table      Periodic     Label:

	t [s]	f(t) [-]
1	0.0000	0.0000
2	0.2000	1.0000
3	0.8000	1.0000
4	1.0000	0.0000
5	2.0000	0.0000
*		



A periodic load-time function is described by the equation  $f(t) = \cos(2\pi \cdot f_p \cdot t + \varphi_p)$  with the load frequency  $f_p$  [Hz] and the phase shift  $\varphi_p$  [rad].

## Rayleigh's Damping

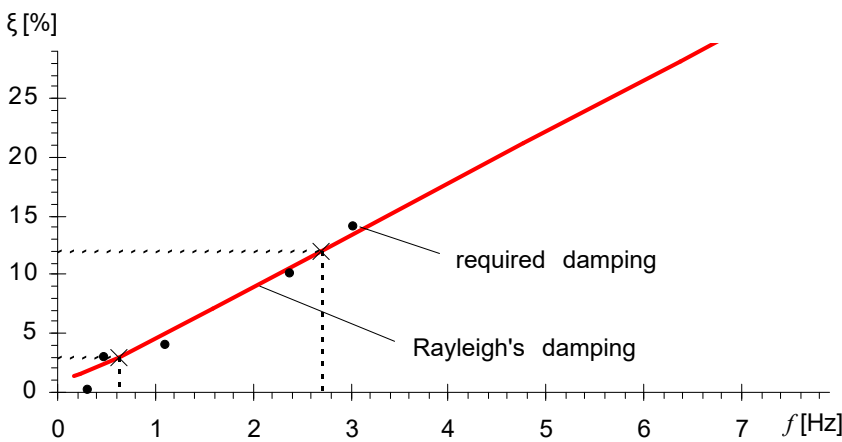
This table is used to specify determining parameters ( $f_i, \xi_i$ ) for mass- and stiffness-proportional damping for direct integration of the equations of motion.

Rayleigh damping [RAYLEIGH]		
	Eigenvalue [Hz]	rel. damp. measure [%]
1. Values	0.637	3.000
2. Values	2.706	12.000

The program applies the damping approach  $C = \alpha \cdot M + \beta \cdot K$ . As the factors  $\alpha$  and  $\beta$  are generally unknown, they are determined internally by the program using the following two equations.

$$\alpha + \beta \cdot \omega_1^2 = 2 \cdot \omega_1 \cdot \xi_1 \quad \alpha + \beta \cdot \omega_2^2 = 2 \cdot \omega_2 \cdot \xi_2 \quad \text{with } \omega_i = 2\pi \cdot f_i$$

The following illustration shows an example of the relationship between resonant frequencies and damping measures. The actual required damping can be specified by measurements, for instance.



## Viscous Damper

Point damper elements are defined by their start and end nodes as well as the damping constant. Point dampers can only be used when directly integrating the equations of motion. Alternatively, the damping constant can also be defined in a spring element.

Viscous damper [DAEMPFER]				
No.	Start-node	End-node	Damping constant [kNs/m]	
1	1	11	12	1.9000
2				

## Excitation Frequency Range Response

The excitation frequencies for the analysis of a stationary response are assigned to a nodal degree of freedom. This is accomplished by entering a frequency range that specifies the frequency steps to be calculated or through the selection of an excitation spectrum using the shown table.

Excitation frequency range of response [HERTZ]							
No.	Node	Degree of freedom	Excitation type	Frequency range		Number of steps	Spectrum
				from [Hz]	to [Hz]		
1	11	ux	Range	0.0000	10.0000	200	
2							

*Nodes*

Nodes for which the response is to be calculated.

*Degree of freedom*

Nodal degree of freedom *ux*, *uy*, *uz*, *phi.x*, *phi.y*, *phi.z* for which a stationary response is to be calculated.

*Excitation type*

Select the excitation frequency range or spectrum.

*Frequency range*

Start and end frequency of the excitation [Hz].

*Number of steps*

Number of calculation steps in the frequency range.

*Spectrum*

Number of the excitation spectrum.

## Excitation Spectrum

This option is used to assign the individual excitation frequencies to load factors. So, for instance, a measured load spectrum (frequency-acceleration value pair) can be applied automatically.

### Excitation frequency

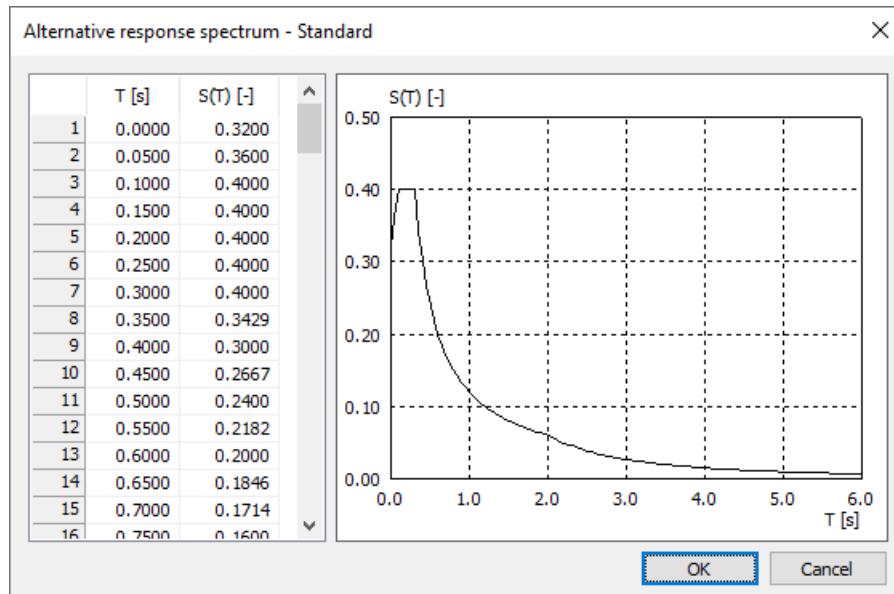
Excitation frequency for which a stationary response is to be calculated [Hz].

### Load factor

Factor for the load at the corresponding excitation frequency.

## Alternative Response Spectrum

This dialog allows you to specify a user-defined standardized response spectrum. The specifications can be made in the analysis settings - dynamics if the corresponding analysis mode is selected.



### Oscillation time T

Vibration period [s].

### Coefficient S(T)

Coefficient of the user-defined standardized response spectrum.

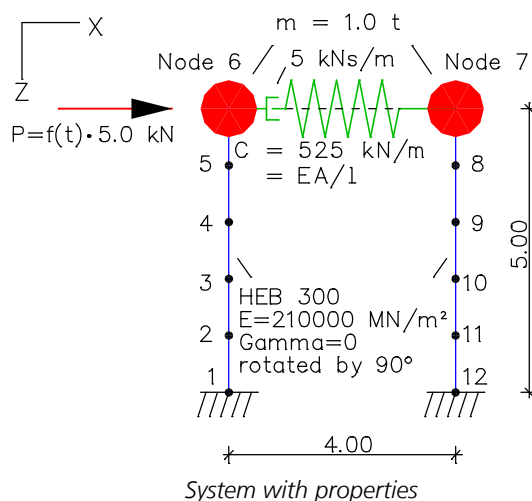
# Examples

## Linked Two-Mass Oscillator

Essential functions of dynamic structure analysis are illustrated based on the example of a two-mass oscillator. A spring element connects the two point masses. The necessary inputs are shown respectively.

The following dynamic analyses are carried out:

- Eigenvalues and eigenmodes.
- Stationary response in an excitation frequency range taking modal damping into account (system response in steady state).
- Periodic load-time function  $f(t) = \sin \omega t$  taking modal damping into account (modal analysis).
- Periodic load-time function  $f(t) = \sin \omega t$  taking a viscous damper and Rayleigh's damping into account (direct analysis).
- Constant load-time function  $f(t) = 1$  taking modal damping into account (modal analysis).
- Instationary load-time function  $f(t) = \text{non-periodic}$  taking modal damping into account (modal analysis).
- Instationary load-time function  $f(t) = \text{increasing linearly}$  taking Rayleigh's damping and breaking of the nonlinear spring into account (direct analysis).
- Response spectrum according to EN 1998-1:2013.



Except for one, all dynamic analyses assume an elastic spring. The viscous damper is only applied in one dynamic analysis.

## Eigenvalues and Eigenmodes

### Setting Dynamics

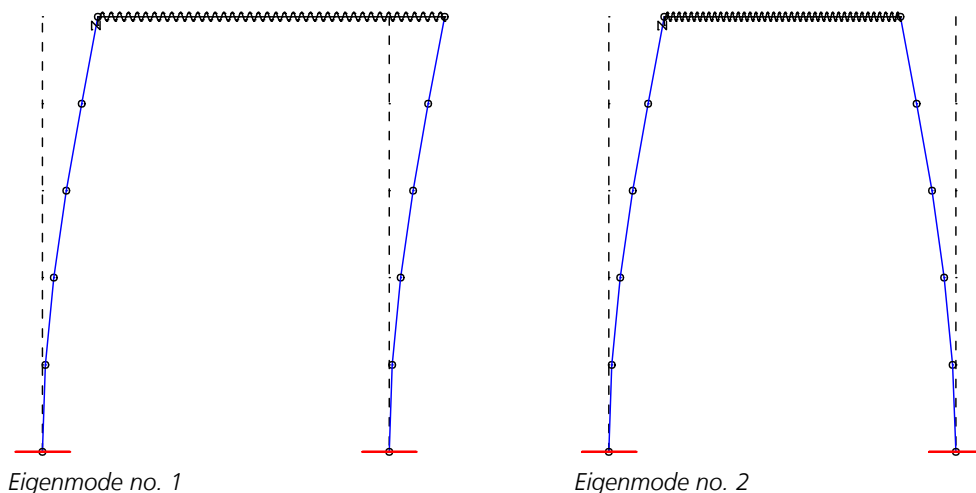
Analysis mode	Eigenvalues
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X

The calculation log includes:

#### Natural frequencies

No.	Eigenvalue [1/s <sup>2</sup> ]	rel. tol. [-]	omega [rad/s]	frequency [Hz]	periode [s]
1	4.3142e+02	7.2896e-27	20.771	3.306	0.302502
2	1.4814e+03	1.1543e-28	38.489	6.126	0.163245

The eigenmodes and resonant frequencies that are determined can be graphically displayed with the *Results* function.



### Stationary Response

The stationary response expresses the system response to a periodically oscillating load in steady state for a defined frequency range (here 0 to 8 Hz). The damping measure is set to 10% for both resonant frequencies. Take note of the reaction peaks in the vicinity of the resonant frequencies.

#### Setting Dynamics

Analysis mode	Stationary response
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X
Actions Regard of defined node accelerations Load Cases	No 1 - Node 6; 5 kN

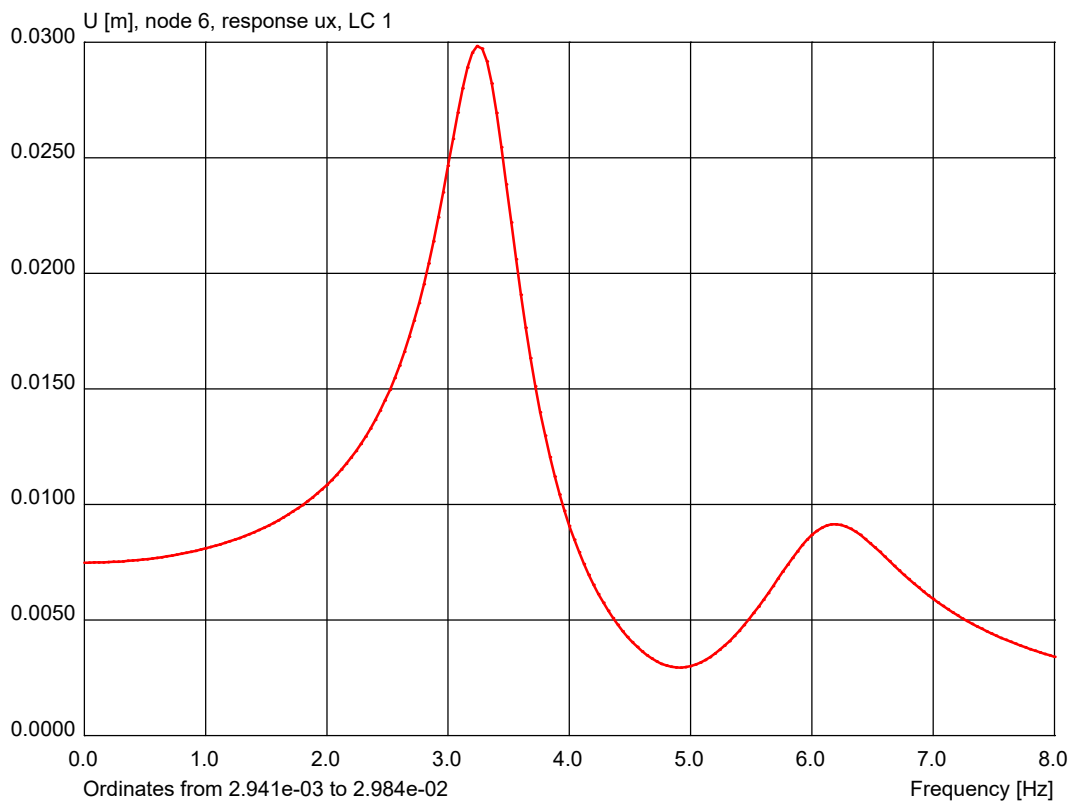
#### Modal damping

Eigenmode	D [%]	alpha [1/s]	β [s]
1	10.000	0.00000	0.00000
2	10.000	0.00000	0.00000

- D Lehr's damping measure
- alpha Mass proportional damping
- β Stiffness proportional damping

#### Excitation frequency range of response

	Node	Degree of freedom	Excitation type	Frequency range		Number of steps	Spectrum
				from[Hz]	to [Hz]		
1	6	ux ...	Range	0.0000	8.0000	200	
2	7	ux ...	Range	0.0000	8.0000	200	





### Periodic Load-Time Function (Modal Analysis)

The system behavior resulting from a sinusoidal load-time function is to be calculated with a system damping of 10%. Because the program assumes a cosinusoidal load-time function, you must define a phase shift of  $-\pi/2$ . The system is excited at its second resonant frequency.

#### Setting Dynamics

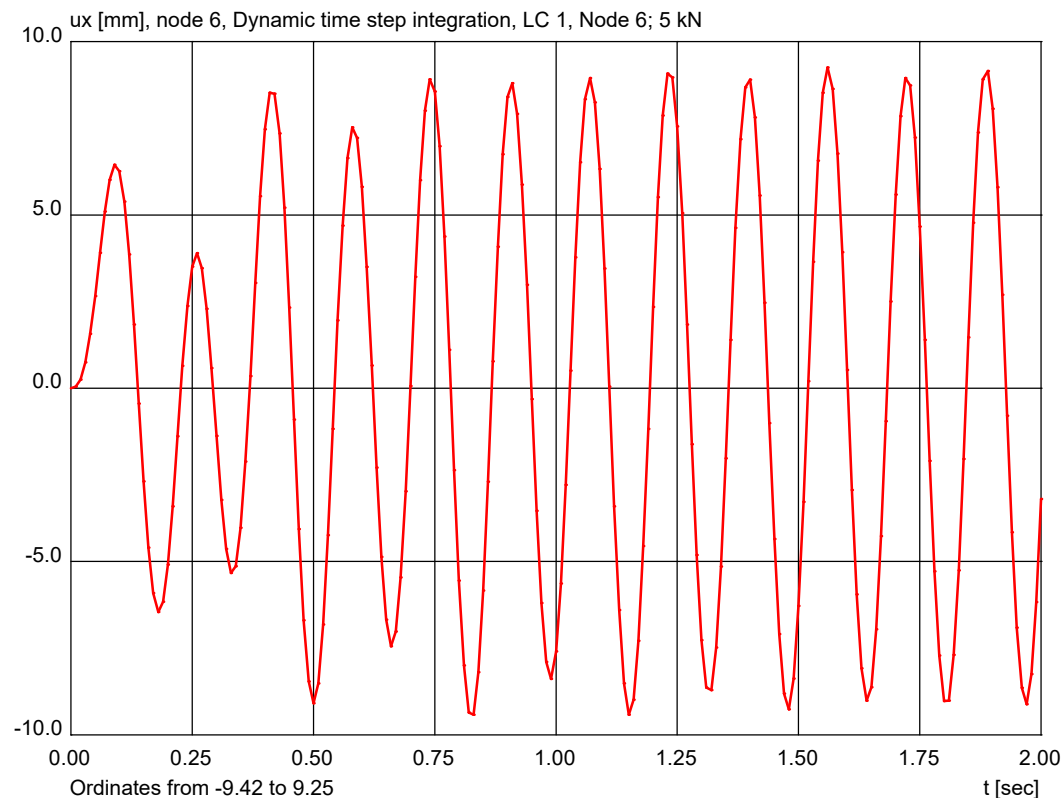
Analysis mode	Modal time step integration
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X
Time step integration Number of time steps	200
Time step period	0.01 [s]
Results every n steps	1
Load-time Function	Periodical for the overall system
Load frequency	6.126 [Hz]
Phase shift	-1.5708 [rad]
Actions Regard of defined node accelerations	No
Load Cases	1 - Node 6; 5 kN

#### Modal damping

Eigenmode	D [%]	alpha [1/s]	β [s]
1	10.000	0.00000	0.00000
2	10.000	0.00000	0.00000

- D Lehr's damping measure
- alpha Mass proportional damping
- β Stiffness proportional damping

After the transient phase the oscillation remains stationary.



### Periodic Load-Time Function (Direct Analysis)

The system behavior resulting from a sinusoidal load-time function is to be calculated taking into account a viscous damper and Rayleigh's damping. Because the program assumes a cosinusoidal load-time function, you must define a phase shift of  $-\pi/2$ . The system is excited at its second resonant frequency.

#### Setting Dynamics

Analysis mode	Direct time step integration
Effect of the masses In direction Around axis	X
Time step integration Number of time steps Time step period Results every n steps Load-time Function Load frequency Phase shift	200 0.01 [s] 1 Periodical for the overall system 6.126 [Hz] -1.5708 [rad]
Actions Regard of defined node accelerations Load Cases	No 1 - Node 6; 5 kN

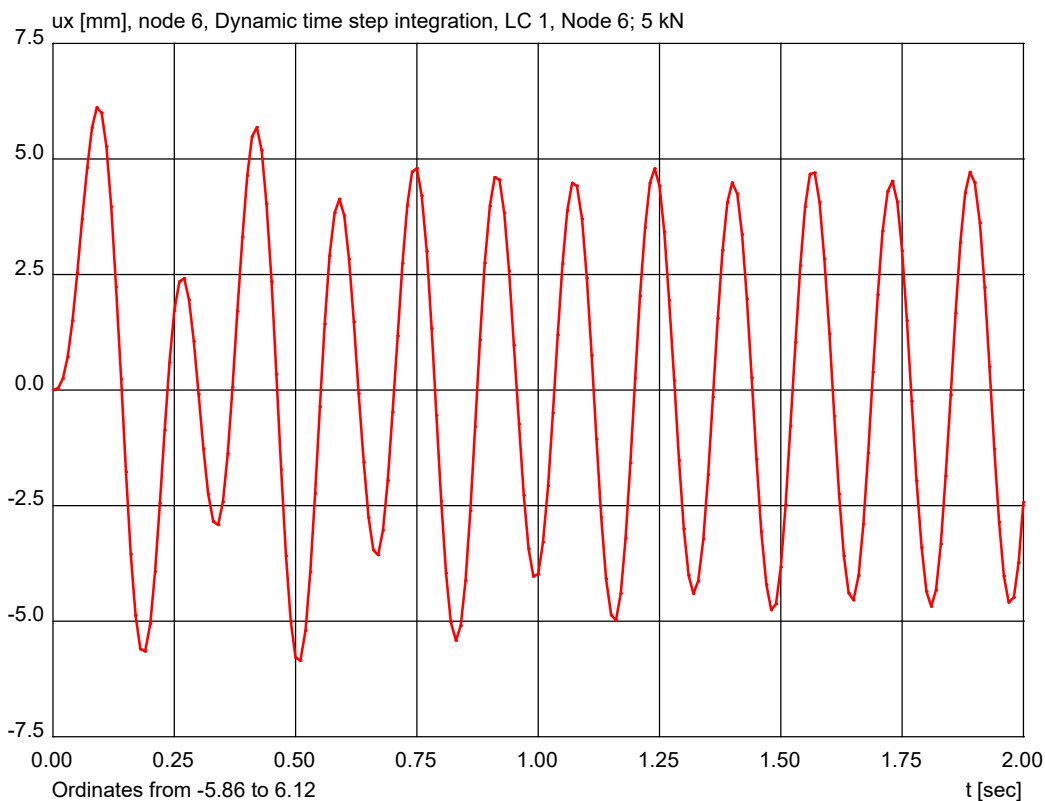
#### Rayleigh damping

	Eigenvalue [Hz]	rel. damp. measure [%]
1. Values	3.307	10.000
2. Values	6.126	10.000

#### Viscous damper

No.	Start- node	End- node	Damping constant [kNs/m]
1	1	6	5.0000

After the transient phase the oscillation remains stationary.



## Constant Load-Time Function

The system behavior resulting from a load that is applied at time  $t = 0$  and continues to apply constantly thereafter is to be analyzed. The analysis assumes a system damping of 10%.

### Setting Dynamics

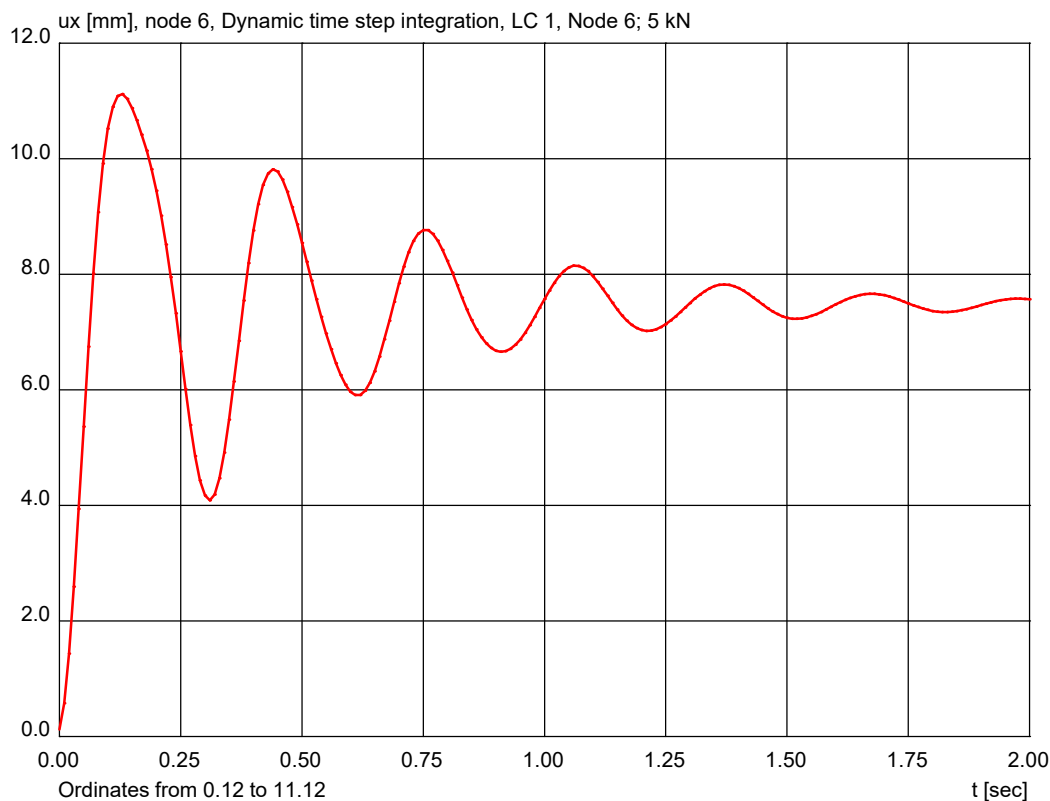
Analysis mode	Modal time step integration
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X
Time step integration Number of time steps	200
Time step period	0.01 [s]
Results every n steps	1
Load-time Function	Periodical for the overall system
Load frequency	0 [Hz]
Phase shift	0 [rad]
Actions Regard of defined node accelerations	No
Load Cases	1 - Node 6; 5 kN

### Modal damping

Eigenmode	D [%]	alpha [1/s]	β [s]
1	10.000	0.00000	0.00000
2	10.000	0.00000	0.00000

D    Lehr's damping measure  
alpha    Mass proportional damping  
β    Stiffness proportional damping

The end value of the damped oscillation is the static displacement  $u_x$ .



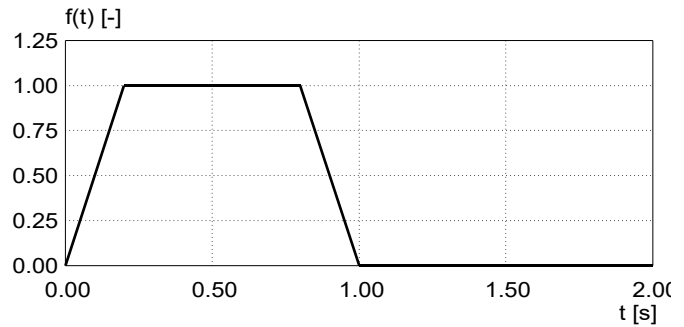
### Instationary Load-Time Function

The system behavior resulting from a non-uniformly oscillating load is to be analyzed. The analysis assumes a system damping of 10%.

#### Setting Dynamics

Analysis mode	Modal time step integration
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X
Time step integration Number of time steps	200
Time step period	0.01 [s]
Results every n steps	1
Load-time Function	1, instationary at the overall system
Actions Regard of defined node accelerations	No
Load Cases	1 - Node 6; 5 kN

#### Load-time Function 1



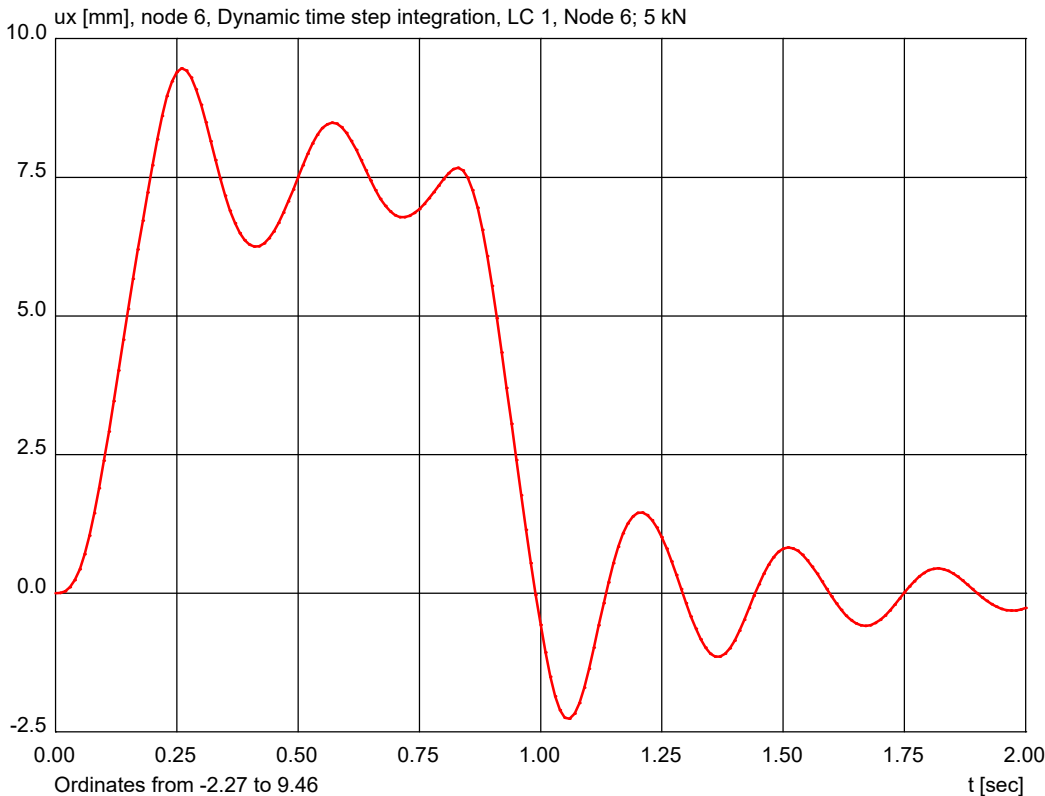
No.	t [s]	f(t) [-]	No.	t [s]	f(t) [-]
1	0.000	0.0000	2	0.200	1.0000
3	0.800	1.0000	4	1.000	0.0000
5	2.000	0.0000			

The function value  $f(t)$  specifies the load factor at each corresponding time.

#### Modal damping

Eigenmode	D [%]	alpha [1/s]	β [s]
1	10.000	0.00000	0.00000
2	10.000	0.00000	0.00000

- D    Lehr's damping measure
- alpha    Mass proportional damping
- β    Stiffness proportional damping



## Linear Load Increase With Breaking Spring

The system behaviour when the nonlinear spring element breaks without viscous damping is to be calculated.

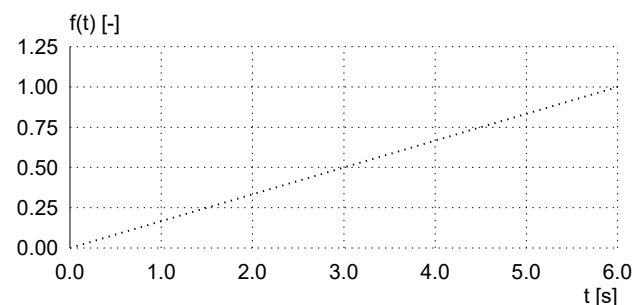
Spring element dialog: Number: 1, Label: Spring breaking at an extension of 2 cm. Characteristics table: ux (Breaking spring, Linear: No, Stiffness: 0), uy (Yes, 0), uz (Yes, 0), phi.x (Yes, 0), phi.y (Yes, 0), phi.z (Yes, 0). Geometry: Node 6 (2, 0, -5), Node 7 (6, 0, -5), x-direction (1, 0, 0). Viscous damping (ux): 0 [kNs/m].

Spring characteristic dialog: Values table: ux [m] vs Fx [MN]. Points: 1 (0, 0), 2 (0.01, 0.00525), 3 (0.02, 0.00525). Label: Breaking spring.

### Setting Dynamics

Analysis mode	Direct time step integration
Effect of the masses In direction Around axis	X
Time step integration Number of time steps Time step period Results every n steps Load-time Function	600 0.01 [s] 1 2, instationary at the overall system
Actions Regard of defined node accelerations Load cases	No 2 - Node 6; -30 kN

Load-time Function 2 - Linear increase of 100 % in 6 s



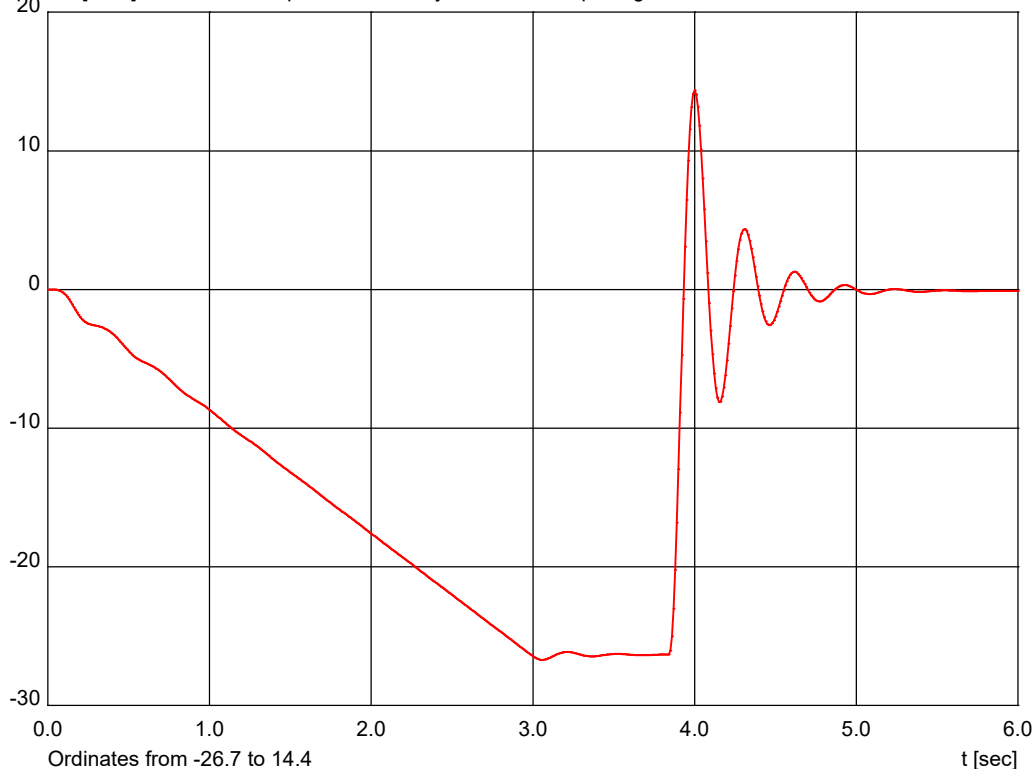
### Rayleigh damping

	Eigenvalue [Hz]	rel. damp. measure [%]
1. Values	3.307	10.000
2. Values	6.126	10.000

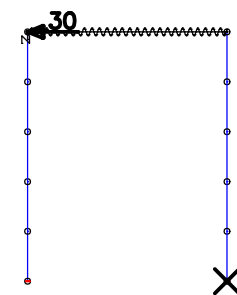
No.	t [s]	f(t) [-]	No.	t [s]	f(t) [-]
1	0.000	0.0000	2	6.000	1.000

The function value  $f(t)$  specifies the load factor at each corresponding time.

20 Mz [kNm], beam 11, field point 2 von 2, Dynamic time step integration, LC 2, Node 6; -30 kN



The plasticization of the spring can be observed after 2.97 s and the breaking after 3.85 s. The right column of the system then swings out to zero.

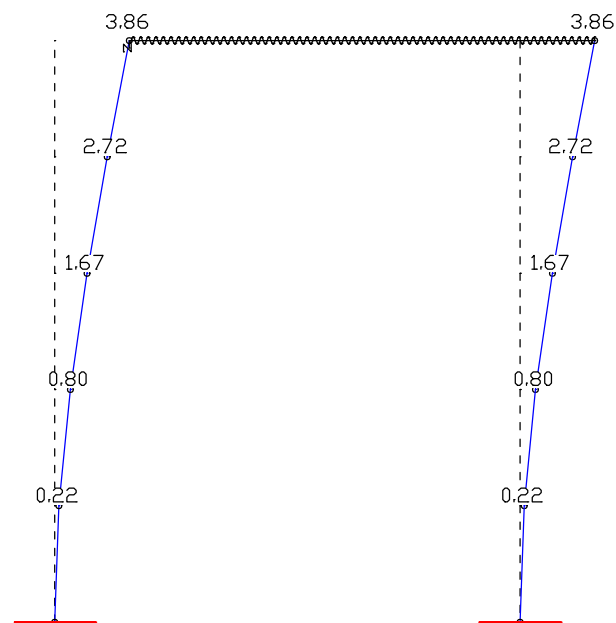


## Response Spectrum EN 1998-1:2013

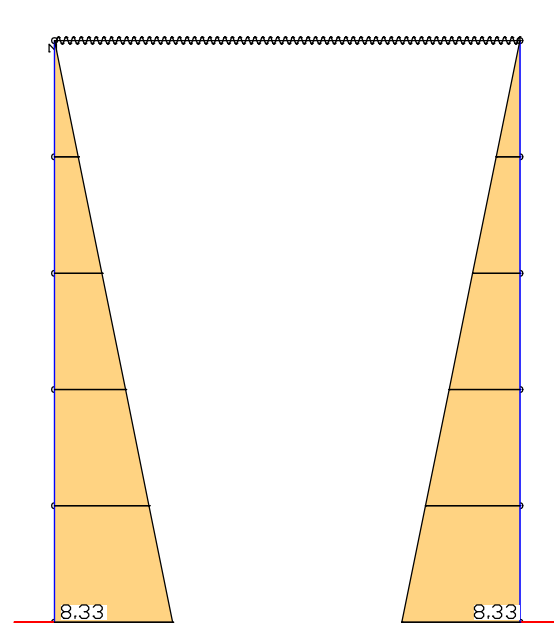
The horizontal static pseudo loads are to be calculated according to the basic standard EN 1998-1 for the eigenmodes under analysis. The results of the evaluation of the response spectrum of the individual eigenmodes are statistically superposed.

### Setting Dynamics

Analysis mode	Response spectrum EN 1998-1:2013
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X
Soil accelerations Apply components separately Referenz system angle alpha	$a_x = 1 \ a_y = 0 \ a_z = 0 \text{ [m/s}^2\text{]}$ No 0 [°]
Coefficients of EN 1998-1:2013 Ground type Response spectrum of type Parameter for response spectrum Lehr's damping measure Importance coeff. gamma I Behavior coefficient q Combination method Store static substitute loads	A - Rock or other rock-like geological formation... 1 S = 1 TB = 0.15 TC = 0.4 TD = 2 0.05 1 1.5 Standard No



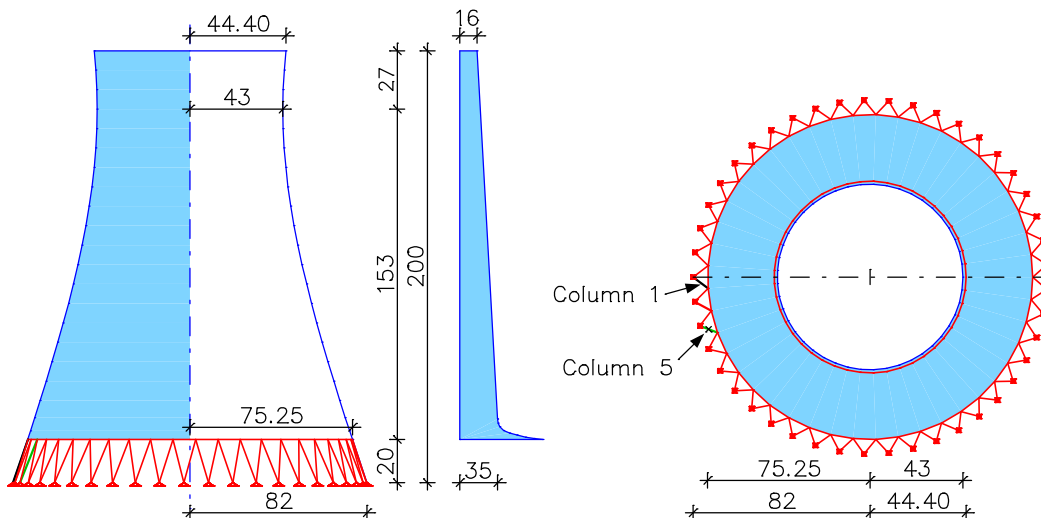
Deformations after statistical superposition [mm]



Moments after statistical superposition [kNm]

# Natural Ventilation Cooling Tower With Eigenmodes and Dynamic Element Collapse

The 200 m high cooling tower shown below serves as an example for the dynamic analysis of a model made of beam and shell elements. Three characteristic eigenmodes are shown for illustration. In addition, the vibration behavior of the structure due to failure (dynamic element collapse) of column 1 was simulated.



### Load data load case 3: Dead load and collapse of column 1

No.	Dynamic element collapse (KOLL)			Collapse time	Factor for eff. stiffness
	from	to el.			
1	1	1		0.5 sec	1e-06

No.	Dead load (EG) referring to material and cross section properties weighting factor in direction		
	X [-]	Y [-]	Z [-]
2	0.0000	0.0000	1.0000

The shell has a continuously changing wall thickness from 16 to 35 cm. At its lower edge, it is connected to a ring beam with a rectangular cross-section (70x55) that is inclined according to the shell and to 90 inclined rectangular columns (55x55). The structure has jointed supports on the 45 lower nodes. The shell consists of C35/45 according to DIN 1045-1. The ring beam and the supports consist of C60/75 acc. to DIN 1045-1.

### Setting Dynamics

Analysis mode	Direct time step integration
Effect of the masses In direction Around axis	X, Y, Z X, Y, Z
Time step integration Number of time steps Time step period Results every n steps Load-time Function Load frequency Phase shift Static initial condition (LC)	240 0.05 [s] 1 Periodical for the overall system 0 [Hz] 0 [rad] 3 - Dead load and collapse of column 1
Actions Regard of defined node accelerations Load cases	No 3 - Dead load and collapse of column 1

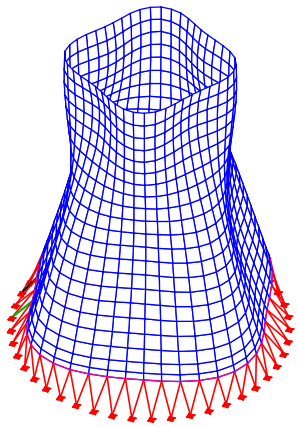
Sudden failure of one or more components (here column 1 after 0.5 sec) causes a dynamic impulse and consequently a considerable additional load on the structure. This can be calculated with the load type *Dynamic element collapse*. For this purpose, the stiffnesses of selected elements are reduced within a direct time step calculation (here with factor 0.000001). The sudden rearrangement caused by the system change causes the impulse.

Rayleigh's damping is defined as follows.

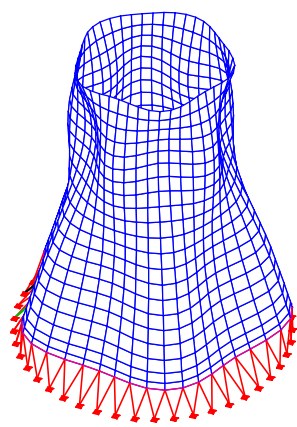
### Rayleigh damping

	Eigenvalue [Hz]	rel. damp. measure [%]
1. Values	0.460	1.000
2. Values	0.590	1.000

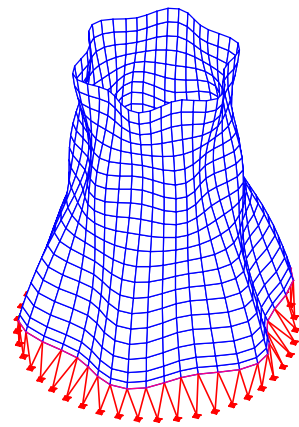
After calculating the direct time step integration, the deformations, internal forces, velocities and accelerations are available for all time steps and all nodes or elements of the system. Following the dynamic analysis, these can be superpositioned (determination of the extreme values) and used for further checks. With direct time step integration there are no eigenmodes or resonant frequencies. To determine this, a further calculation run with corresponding analysis settings (e.g. *Analysis mode - Eigenvalues*) is required.



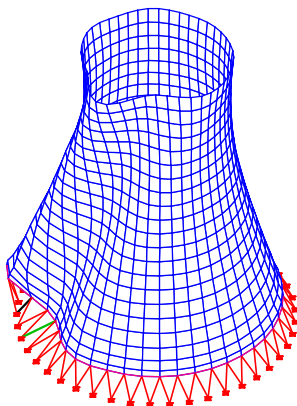
Eigenmode 1 (0.46 Hz)



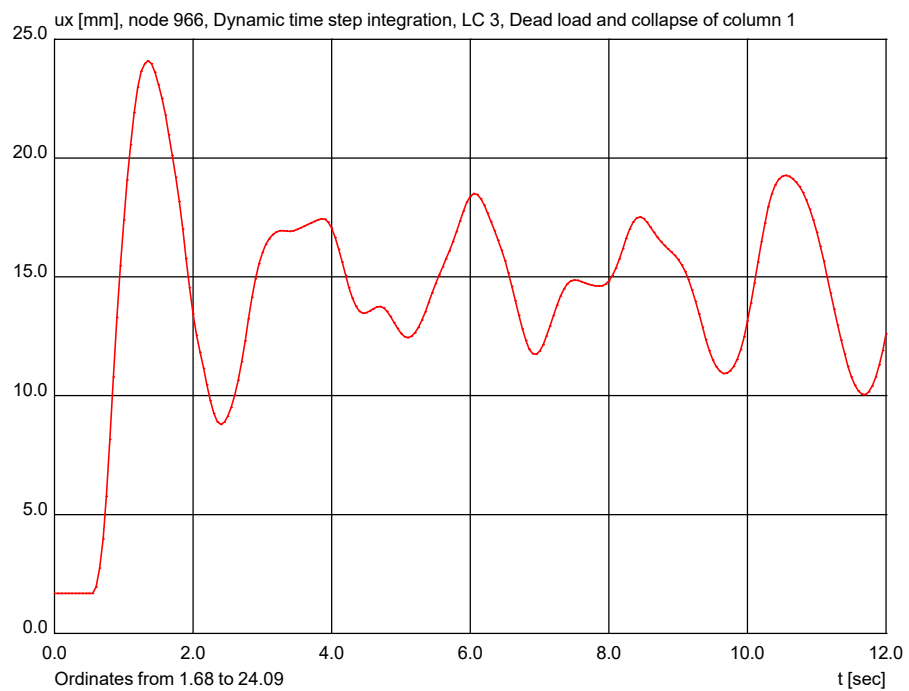
Eigenmode 3 (0.47 Hz)



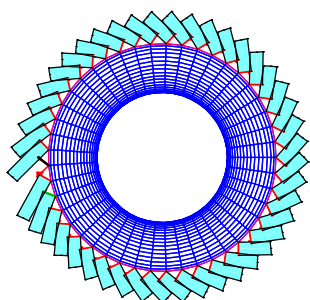
Eigenmode 5 (0.54 Hz)



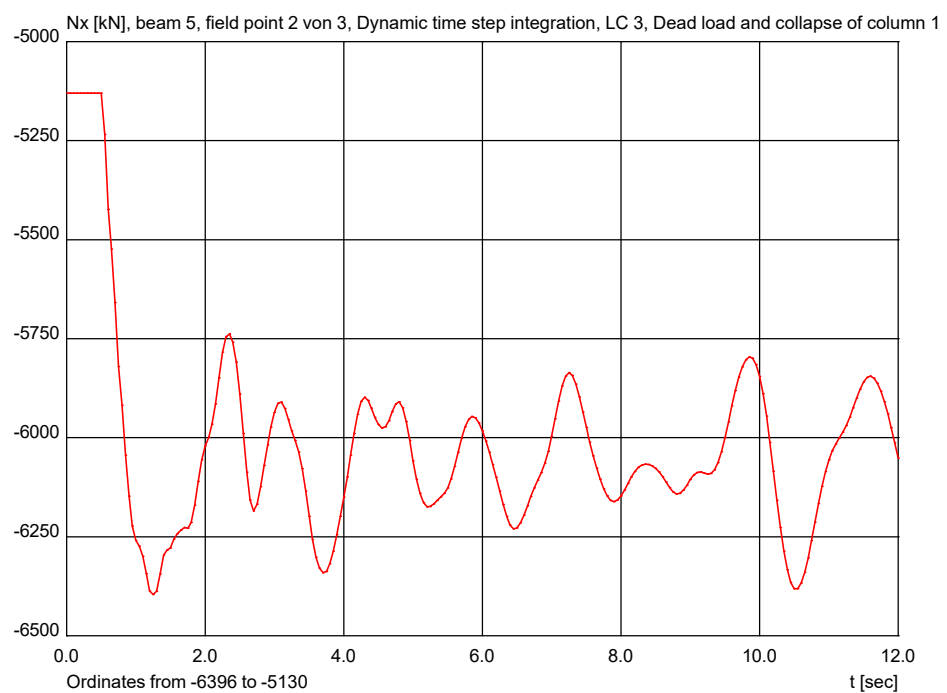
Deformation after 1.35 s  
(superelevated 500 times)



Deformation  $u_x$  of the top node of column 5



Longitudinal force in every  
second column after 1.25 s



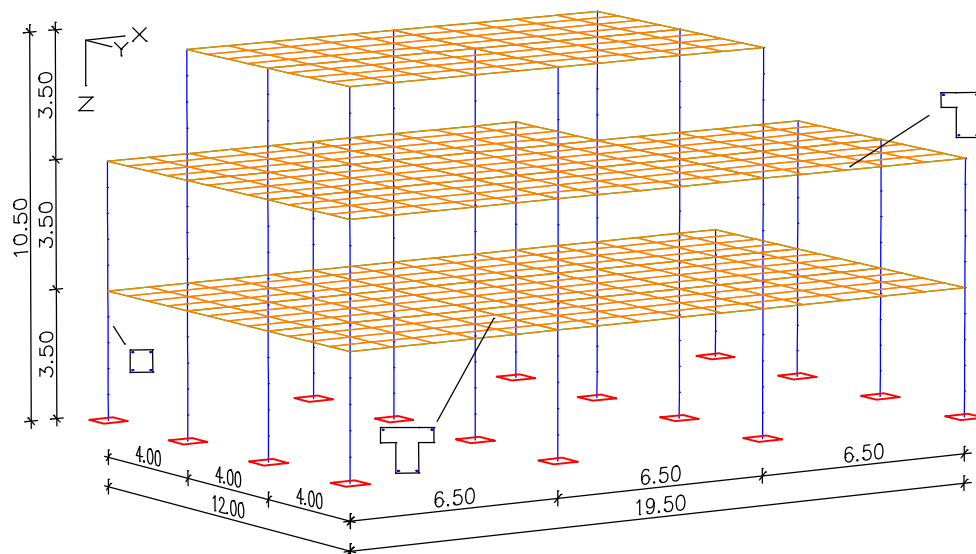
Longitudinal force of column 5



## Earthquake Check According to DIN EN 1998-1:2021

The following example illustrates how to apply the response spectrum method on an irregular 3D structure. The handling of a similar model is also shown in the video (German language)

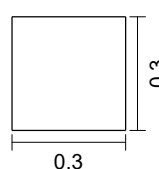
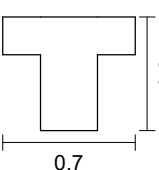
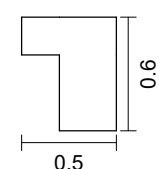

"[https://download.infograph.de/video\\_de/Erdbebennachweis%20nach%20der%20Antwortspektrenmethode.mp4](https://download.infograph.de/video_de/Erdbebennachweis%20nach%20der%20Antwortspektrenmethode.mp4)".



### Material

Concrete C20/25 according to DIN EN 1992-1-1  
Reinforcing steel 500 M / 500 S

### Section properties

1	Polygon 	Columns Centroid [m] $y_s = 0.150$ $z_s = 0.150$ Area [m <sup>2</sup> ] $A = 9.0000e-02$ Moments of inertia [m <sup>4</sup> ] $I_x = 5.0000e-04$ $I_y = 6.7500e-04$ $I_1 = 6.7500e-04$ $I_z = 6.7500e-04$ $I_2 = 6.7500e-04$ Main axis angle [Grad] $\Phi = 0.000$ $I_{yz} = 0.0000e+00$ Ignore Iyz in member stiffnes.
2	Polygon 	T-beams Centroid [m] $y_s = 0.350$ $z_s = 0.238$ Area [m <sup>2</sup> ] $A = 2.6000e-01$ Moments of inertia [m <sup>4</sup> ] $I_x = 3.0000e-03$ $I_y = 7.8821e-03$ $I_1 = 7.8821e-03$ $I_z = 6.6167e-03$ $I_2 = 6.6167e-03$ Main axis angle [Grad] $\Phi = -0.000$ $I_{yz} = 0.0000e+00$ Ignore Iyz in member stiffnes.
3	Polygon 	Edge beams Centroid [m] $y_s = 0.305$ $z_s = 0.264$ Area [m <sup>2</sup> ] $A = 2.2000e-01$ Moments of inertia [m <sup>4</sup> ] $I_x = 3.0000e-03$ $I_y = 6.8424e-03$ $I_1 = 7.5143e-03$ $I_z = 3.5288e-03$ $I_2 = 2.8569e-03$ Main axis angle [Grad] $\Phi = -22.322$ $I_{yz} = 1.6364e-03$ Ignore Iyz in member stiffnes.
4	Area 	Slabs Element thickness [m] $d_z = 0.2000$ torsion-proof Orthotropy $d_{zy}/d_z$ = 1 E-Modulus slab/plain        = 1

### Properties for the earthquake check

$S_{aP,R}$  (475 years, Aachen): 2.557 m/s<sup>2</sup>

Soil class: A

Importance category III:

Behavior coefficient:

Soil acceleration  $a_{gR} = 2.557 / 2.5 = 1.023$  m/s<sup>2</sup>

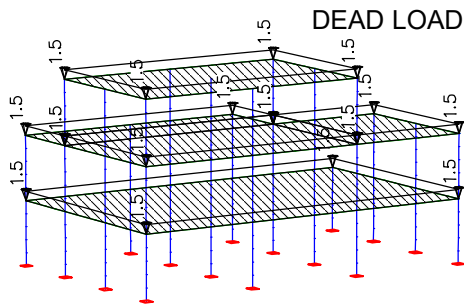
Geological subsoil class: R

$\gamma_1 = 1.2$

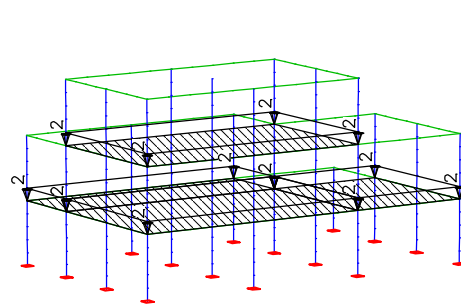
$q = 0.8 \cdot 1.5 = 1.2$

For the displayed framework structure made of reinforced concrete the following calculations are carried out:

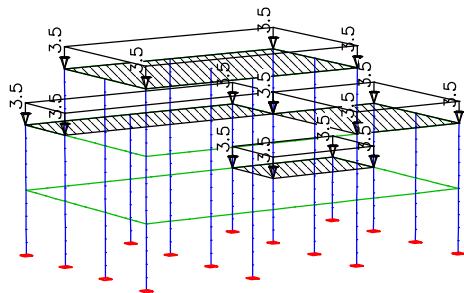
- Determination of the eigenvalues and eigenmodes of the structure model under consideration of the masses according to Section 4.3.3.3.1 of the standard.
- Determination of the horizontal static pseudo loads according to Section 4.3.3.3.1 of the standard.
- Check of the effective modal masses of the structure model.
- Statistical combination of the modal internal forces and displacements according to Section 4.3.3.3.2 of the standard.
- Consideration of random torsion actions according to Section 4.3.3.3.3 of the standard.
- Combination of the stress as a result of the components of the earthquake actions according to Section 4.3.3.5.1 (3) of the standard.
- Combinations of the earthquake action with other actions according to DIN EN 1990.
- Design according to DIN EN 1992-1-1.
- Consideration of influences according to 2nd order theory (P-D effects) according to DIN EN 1998-1, Section 4.4.2.2 (2).



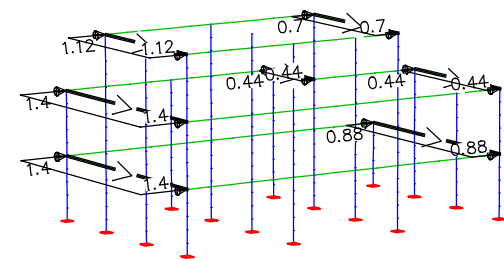
Load case 1: Dead load and expansion



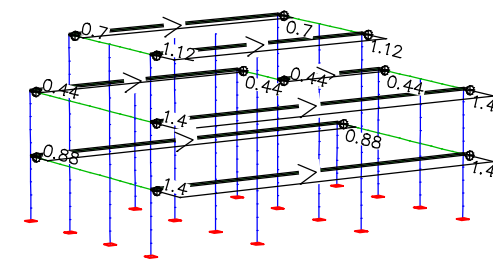
Load case 2: Traffic on inner areas



Load case 3: Traffic on outer areas



Load case 4: Wind in x direction



Load case 5: Wind in y direction

**Calculation settings for dynamic**

Effect of the masses  
in direction X Y Z  
around axis X Y Z

Eigenvalues  
Number of eigenvalues: 5  
Maximum vectorspace : 10  
Iteration accuracy : 1.00 [%]

**Response spectrum DIN EN 1998-1:2021**

Basic values of the soil acceleration  
ax: 1.023 [m/s<sup>2</sup>]  
ay: 1.023 [m/s<sup>2</sup>]  
az: 0.000 [m/s<sup>2</sup>]  
Acceleration components are applied separately  
Angle of the reference system alpha: 0.00 [°]  
Soil behavior : A-R  
Reaction parameter horizontal S: 1.000 TB: 0.100 TC: 0.200 TD: 2.000  
Reaction parameter vertical S: 1.000 TB: 0.050 TC: 0.200 TD: 1.200  
Lehr's damping measure : 0.050  
Importance factor GammaI: 1.200  
Behavior coefficient qh: 1.200 qv: 1.000

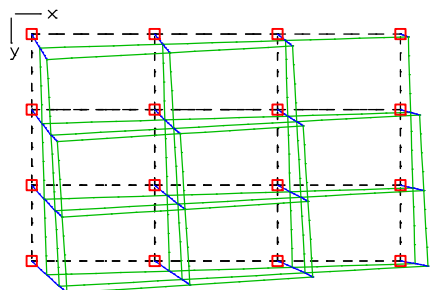
**Generate masses from loads**

Load case	factor
1	1.00
2	0.21
3	0.30

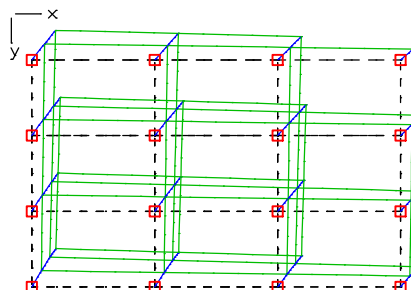
**Sum of the installed masses (global)**

	Mx [t]	My [t]	Mz [t]
Volume masses :	495.034	495.034	495.034
Point masses :	0.000	0.000	0.000
Masses of LC :	121.890	121.890	121.890
Sum :	616.923	616.923	616.923

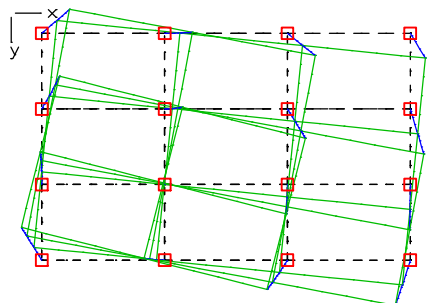
**Resonant frequencies and eigenmodes**



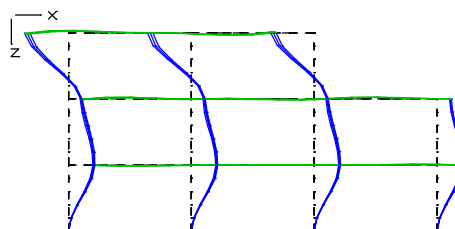
Eigenmode 1:  $f = 1.43$  Hz



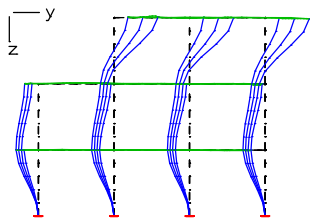
Eigenmode 2:  $f = 1.47$  Hz



Eigenmode 3:  $f = 2.03$  Hz



Eigenmode 4:  $f = 3.54$  Hz



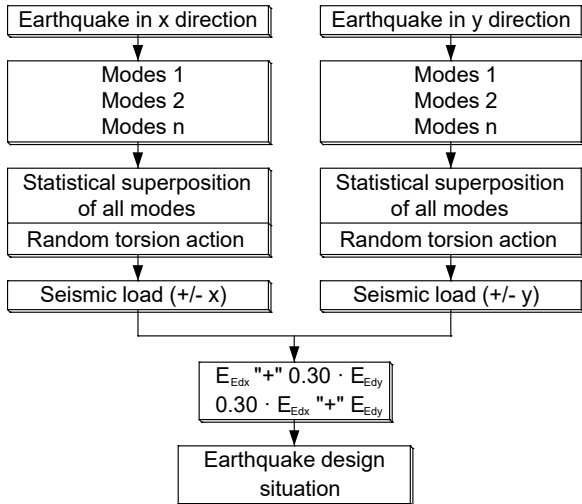
Eigenmode 5:  $f = 3.63$  Hz

**Effective modal masses (global)**

Mode shape	Mx eff [t]	My eff [t]	Mz eff [t]
1	323.206	183.332	0.000
2	209.813	329.520	0.000
3	13.016	37.500	0.000
4	46.327	5.072	0.034
5	3.753	41.475	0.003
Sum:	596.115	596.899	0.037
Part of the total masses:	97 [%]	97 [%]	0 [%]

The effective modal masses that are considered are 97% of the applied masses. This means that according to Section 4.3.3.3.1 (3) of the standard the number of eigenmodes analyzed is sufficient.

**Calculation scheme**



According to Section 4.3.3.5.1 (3), the check for the horizontal earthquake action is to be carried out in two directions that are perpendicular to one another. The calculation scheme shown above depicts the procedure. The statistical superposition of the modal components is carried out by the program. The random torsion actions according to Section 4.3.3.3.3 (1) of the standard are considered in the following manner for this structure.

**Comparative calculation**

Foundation force

$$F_b = \frac{S_d(T)}{a_g} \cdot a_g \cdot m$$

$$= 0.595 \cdot 1.023 \cdot 1.2 \cdot 617$$

$$= 450 \text{ kN for } T = 1/1.43 = 0.7 \text{ s}$$

with

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j}$$

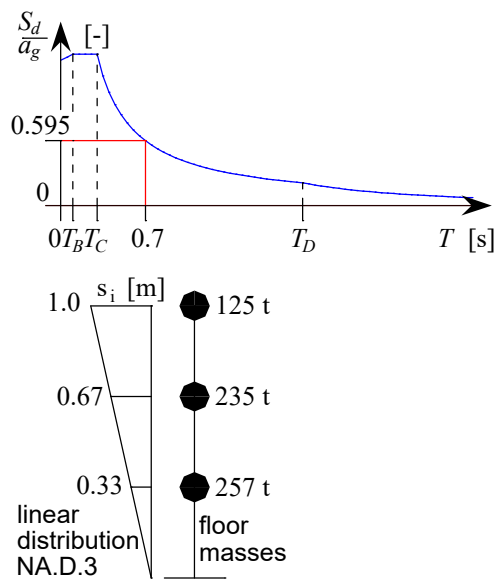
$$M_{1i} = e_{1i} \cdot F_i = h_i \cdot L_i^2 / 6$$

Load ordinate of the trapezoidal load of the story *i*

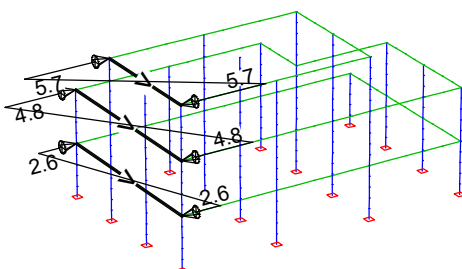
$$h_i = \pm 0.3 \cdot F_i / L_i$$

Eccentricity of  $F_i$

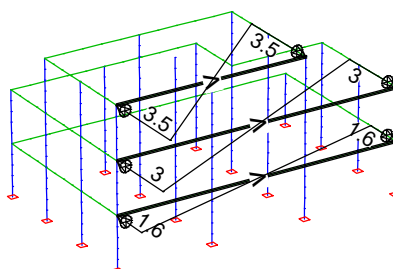
$$e_{ai} = \pm 0.05 \cdot L_i$$



Floor above story	Height $z_i$ [m]	Mass $m_i$ [t]	$s_i \cdot m_i$ [tm]	H-Force $F_i$ [kN]	Width $Lx_i$ [m]	Trap. load $hx_i$ [kN/m]	Width $Ly_i$ [m]	Trap. load $hy_i$ [kN/m]
3	10.5	125	125	153	8	± 5.7	13	± 3.5
2	7	235	157	192	12	± 4.8	19.5	± 3.0
1	3.5	257	86	105	12	± 2.6	19.5	± 1.6
Sum		617	367	450				



Load case 51: Random torsion (x)



Load case 52: Random torsion (y)

## Combination of the stress

Combination of the stress as a result of the components of the earthquake actions according to Section 4.3.3.5.1 (3) of the standard.

### Load case combination 1, Edx "+" 0.3 Edy

1. Variable exclusive action		Factor
	Response spectrum overlaid (x)	-1.000
	Response spectrum overlaid (x)	1.000
2. Variable exclusive action		Factor
	Response spectrum overlaid (y)	-0.300
	Response spectrum overlaid (y)	0.300
3. Variable exclusive action		Factor
51	Random torsion (x)	-1.000
51	Random torsion (x)	1.000
4. Variable exclusive action		Factor
52	Random torsion (y)	-0.300
52	Random torsion (y)	0.300

### Load case combination 2, 0.3 Edx "+" Edy

1. Variable exclusive action		Factor
	Response spectrum overlaid (x)	0.300
	Response spectrum overlaid (x)	-0.300
2. Variable exclusive action		Factor
	Response spectrum overlaid (y)	1.000
	Response spectrum overlaid (y)	-1.000
3. Variable exclusive action		Factor
51	Random torsion (x)	-0.300
51	Random torsion (x)	0.300
4. Variable exclusive action		Factor
52	Random torsion (y)	-1.000
52	Random torsion (y)	1.000

Combinations of the earthquake action with other actions according to DIN EN 1990.

#### DIN EN 1992-1-1 actions

##### Standard design group

##### G - Dead load

$\Gamma_{sup} / \Gamma_{inf} = 1.35 / 1$

##### Load cases

1 G, Dead load

##### QN - Imposed load, traffic load

$\Gamma_{sup} / \Gamma_{inf} = 1.5 / 0$

Combination coefficients for: Superstructures  
Working load - category A: Residential buildings  
 $\Psi_{0.0} / \Psi_{0.1} / \Psi_{0.2} = 0.7 / 0.5 / 0.3$

##### Load cases 1. Variant, inclusive

2 QN, Indoor ceiling  
3 QN, Outdoor ceiling

##### QW - Wind load

$\Gamma_{sup} / \Gamma_{inf} = 1.5 / 0$

Combination coefficients for: Superstructures  
Wind loads  
 $\Psi_{0.0} / \Psi_{0.1} / \Psi_{0.2} = 0.6 / 0.2 / 0$

##### Load cases 1. Variant, exclusive

4 QW, Wind in direction x  
5 QW, Wind in direction y

##### AE - Actions due to earthquake

$\Gamma_{sup} = 1$

##### Load cases 1. Variant, exclusive

K1 Edx "+" 0.3 Edy  
K2 0.3 Edx "+" Edy

**1. Permanent and temporary situation**

Final state

- G Dead load
- QN Imposed load, traffic load
- QW Wind load

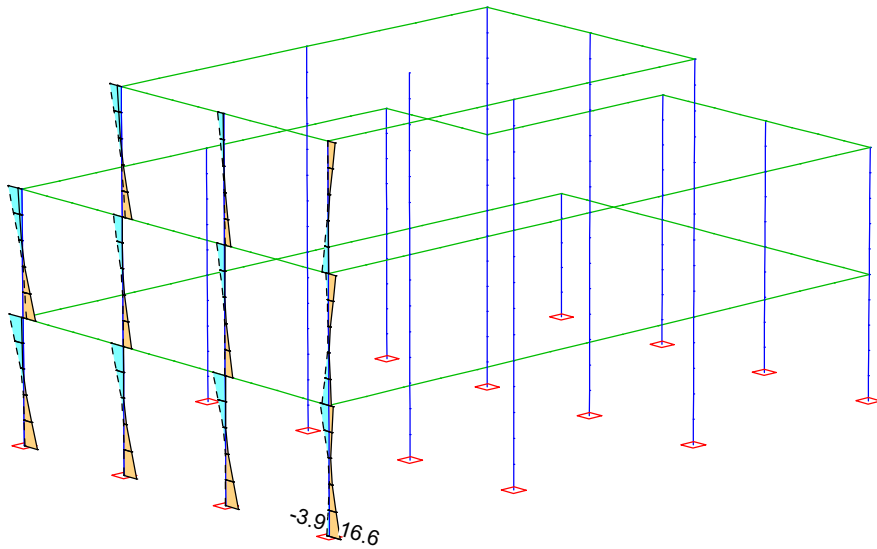
**1. Earthquake situation**

Final state

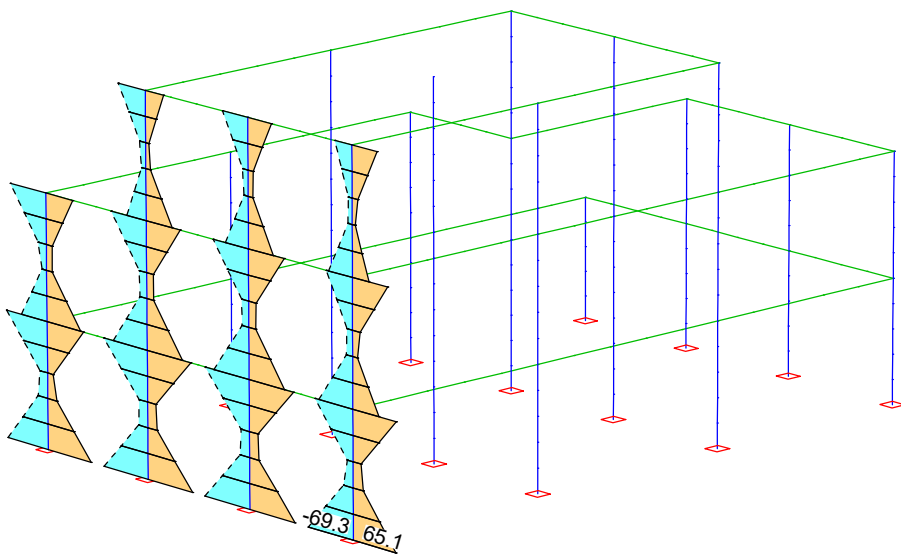
- G Dead load
- QN Imposed load, traffic load
- QW Wind load
- AE Actions due to earthquake

**Results**

The internal forces  $min/max M_z$  displayed below indicate the different influences of wind force (permanent and temporary situation) and earthquake action (earthquake situation) for a selected beam group. It can be seen that the actions from earthquakes are decisive in this case.



Selected internal forces  $min, max M_z$  from the permanent and temporary situation [kNm]



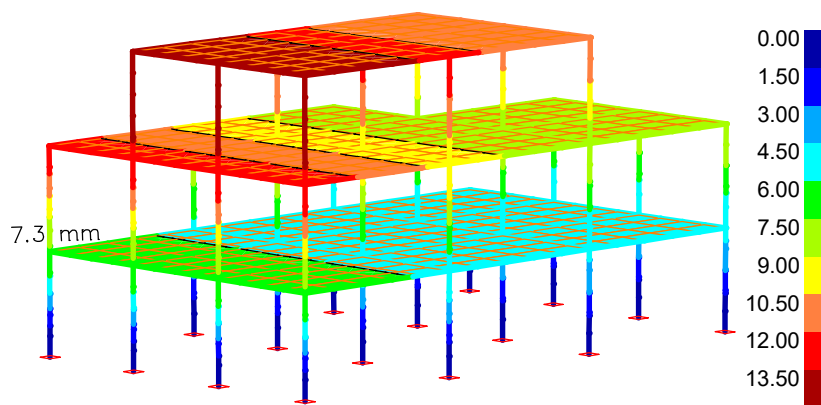
Selected internal forces  $min, max M_z$  from the earthquake situation [kNm]

**Combination information - Internal forces Beam 1 - Loc. 1, Node 1 - Mz min, max  
DIN1992.E.1 : 1. Earthquake situation, DIN EN 1992-1-1**

Load case	Factor	Nx [kN]	My [kNm]	Mz [kNm]	Qy [kN]	Qz [kN]	Mx [kNm]
1 : G, Dead load	1.00	-199.91	4.43	-2.00	1.65	-3.77	0.00
2 : QN, Indoor ceiling	0.30	-6.17	0.25	-0.10	0.09	-0.22	-0.00
3 : QN, Outdoor ceiling	0.30	-5.17	0.05	-0.05	0.04	-0.03	0.00
51 : Random torsion (x)	-0.30	-0.87	-0.63	-1.05	0.58	0.34	0.02
52 : Random torsion (y)	-1.00	-4.77	-3.42	-5.71	3.14	1.85	0.11
Asp.x : Response spectrum overlaid (x)	-0.30	-10.30	-14.52	-3.89	-2.14	-7.88	-0.07
Asp.y : Response spectrum overlaid (y)	-1.00	-65.86	-11.29	-56.47	-31.11	-6.10	-0.37
Mz min		-293.06	-25.13	-69.26	-27.76	-15.81	-0.30
1 : G, Dead load	1.00	-199.91	4.43	-2.00	1.65	-3.77	0.00
51 : Random torsion (x)	0.30	0.87	0.63	1.05	-0.58	-0.34	-0.02
52 : Random torsion (y)	1.00	4.77	3.42	5.71	-3.14	-1.85	-0.11
Asp.x : Response spectrum overlaid (x)	0.30	10.30	14.52	3.89	2.14	7.88	0.07
Asp.y : Response spectrum overlaid (y)	1.00	65.86	11.29	56.47	31.11	6.10	0.37
Mz max		-118.10	34.29	65.12	31.19	8.01	0.31

**Consideration of influences according to 2nd order theory (P-Δ effects)**

The example calculation is carried out for the lowest story according to Section 4.4.2.2 (2) of the standard.



Maximum deformation in y-direction due to the earthquake situation

**Sum of installed loads and support reactions**

LC.	Label	Fx [kN]	Fy [kN]	Fz [kN]
1	G, Dead load	0.000	0.000	5675.250
	Support reactions	-0.000	-0.000	5675.250
2	QN, Indoor ceiling	0.000	0.000	624.000
	Support reactions	-0.000	0.000	624.000
3	QN, Outdoor ceiling	0.000	0.000	819.000
	Support reactions	0.000	0.000	819.000

$$P_{tot} = \Sigma G_k + \Sigma \psi_2 \cdot Q_k = 5675 + 0.3 \cdot (624 + 819) = 6108 \text{ kN}$$

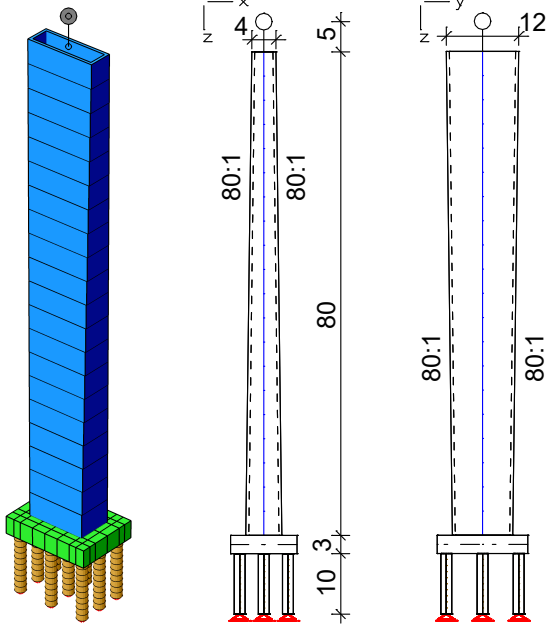
$$d_t = d_s = d_e \cdot q = 7.3 \cdot 1.2 = 8.8 \text{ mm}; \quad h = 3500 \text{ mm}; \quad V_{tot} = F_b = 450 \text{ kN}$$

$$\theta = \frac{P_{tot} \cdot d_t}{V_{tot} \cdot h} = \frac{6108 \cdot 8.8}{450 \cdot 3500} = 0.03 \leq 0.1$$

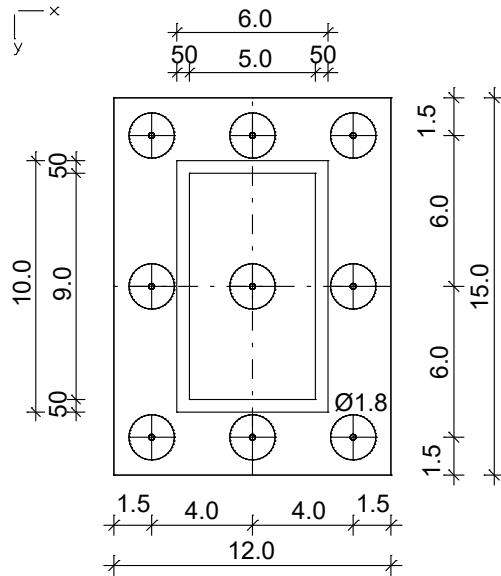
=> **Actions according to the second-order theory do not have to be considered.**

# Earthquake Check of a Bridge Pier According to the ZPA Method

The bridge pier shown below illustrates the response spectrum method using the ZPA method. The structure consists of C35/45 according to DIN EN 1992-1-1. The horizontal bedding moduli for the piles were assumed to be 500 MN/m<sup>3</sup> in this example. In vertical direction the piles are fixed at their lower end. The mass of the superstructure of the bridge assumed for the example was taken into account via a point mass (5000 t) 5 m above the pier. In the context of the example, only the global x-direction is examined and earthquake zone 3 and the subsoil conditions "A-R" according to DIN 4149 are assumed. For better comparability, the statistical combination method SRSS is used in all cases.



Structure and dimensions [m]



Top view with pier cross-section at the clamping point

**Sum of the installed masses (global)**

	Mx [t]	My [t]	Mz [t]
Volume masses :	4888.313	0.000	0.000
Point masses :	5000.000	0.000	0.000
Masses of LC :	0.000	0.000	0.000
<b>Sum :</b>	<b>9888.313</b>	<b>0.000</b>	<b>0.000</b>

**Effective modal masses (global)**

Mode shape	Mx eff [t]	My eff [t]	Mz eff [t]
1	6554.330	0.000	0.000
2	940.564	0.000	0.000
3	521.525	0.000	0.000
4	787.150	0.000	0.000
5	586.590	0.000	0.000
6	0.000	0.000	0.000
7	88.883	0.000	0.000
8	21.056	0.000	0.000
9	342.112	0.000	0.000
<b>Sum:</b>	<b>9842.209</b>	<b>0.000</b>	<b>0.000</b>
<b>Part of the total masses:</b>	<b>100 [%]</b>	<b>0 [%]</b>	<b>0 [%]</b>

As can be seen from the table, with 9 eigenmodes already 100 % of the eigenmasses are excited. This can therefore be regarded as a reference solution. The 1st and 2nd eigenmodes have an effective modal mass of 66.3 % and 9.5 %. The first two eigenmodes thus excite 75.8 % of the masses, which does not meet the standard requirement of 90 % excitation. With the assumed subsoil conditions "A-R", the plateau value of the response spectrum is only reached with the third eigenmode (T = 0.14 sec < TC). The 6th eigenmode is again below the plateau (T = 0.038 sec < TB).

**Calculation settings for dynamic**

```
Effect of the masses
in direction X
around axis

Eigenvalues
Number of eigenvalues: 9
Maximum vectorspace : 40
Iteration accuracy : 1.00 [%]
Response spectrum DIN 4149 (release 04/2005)
Basic values of the soil acceleration
ax: 0.800 [m/s²]
ay: 0.000 [m/s²]
az: 0.000 [m/s²]
Angle of the reference system alpha: 0.00 [°]
Soil behavior : A-R
Reaction parameter S: 1.000 TB: 0.050 TC: 0.200 TD: 2.000
Lehr's damping measure : 0.050
Importance coeff. Gamma1: 1.400
Behavior coefficient q: 1.500
```

**Natural frequencies**

No.	Eigenvalue [1/s²]	rel. tol. [-]	omega [rad/s]	frequency [Hz]	periode [s]
1	1.8975e+00	4.9592e-23	1.377	0.219	4.561308
2	2.2315e+02	2.7791e-25	14.938	2.377	0.420613
3	2.0408e+03	1.6256e-20	45.175	7.190	0.139085
4	7.4138e+03	8.9772e-16	86.103	13.704	0.072973
5	1.5208e+04	1.8038e-13	123.322	19.627	0.050950
6	2.7683e+04	4.2009e-11	166.383	26.481	0.037763
7	3.3484e+04	9.6951e-12	182.987	29.123	0.034337
8	7.4012e+04	3.3768e-10	272.052	43.298	0.023096
9	1.4252e+05	6.1725e-06	377.519	60.084	0.016643

An oscillation time of 0.2 sec was assumed for the ZPA value. Thus the spectral value  $S_{a,ZPA}$  is assumed to be the same size as the plateau value of the response spectrum and thus fits well for oscillation times below 0.2 sec.



**Calculation settings for dynamic**

Effect of the masses  
in direction X  
around axis

**Eigenvalues**

Number of eigenvalues: 2  
Maximum vectorspace : 40  
Iteration accuracy : 1.00 [%]  
Response spectrum DIN 4149 (release 04/2005)

**Basic values of the soil acceleration**

ax: 0.800 [m/s<sup>2</sup>]  
ay: 0.000 [m/s<sup>2</sup>]  
az: 0.000 [m/s<sup>2</sup>]

Angle of the reference system alpha: 0.00 [°]

ZPA-Method (Zero Period Acceleration) activated, oscillation time T\_ZPA: 0.200 [s]

Soil behavior : A-R

Reaction parameter S: 1.000 TB: 0.050 TC: 0.200 TD: 2.000

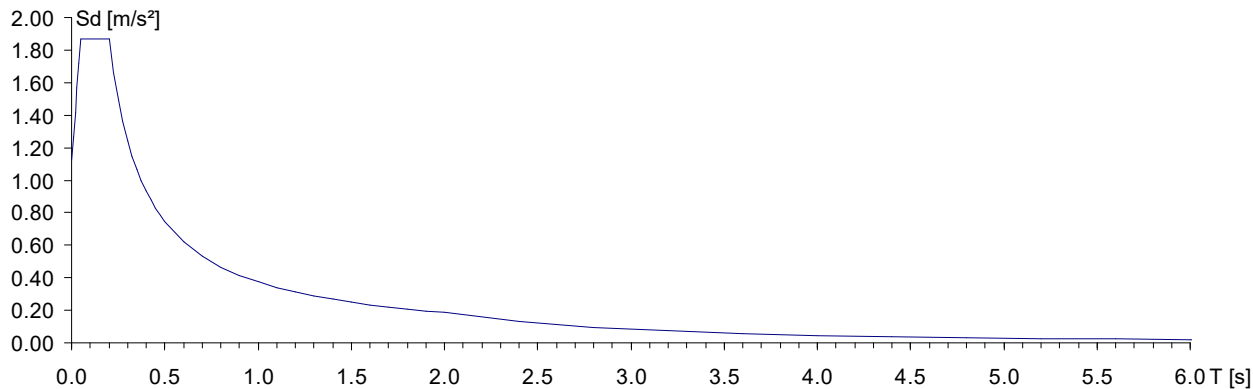
Lehr's damping measure : 0.050

Importance coeff. Gamma: 1.400

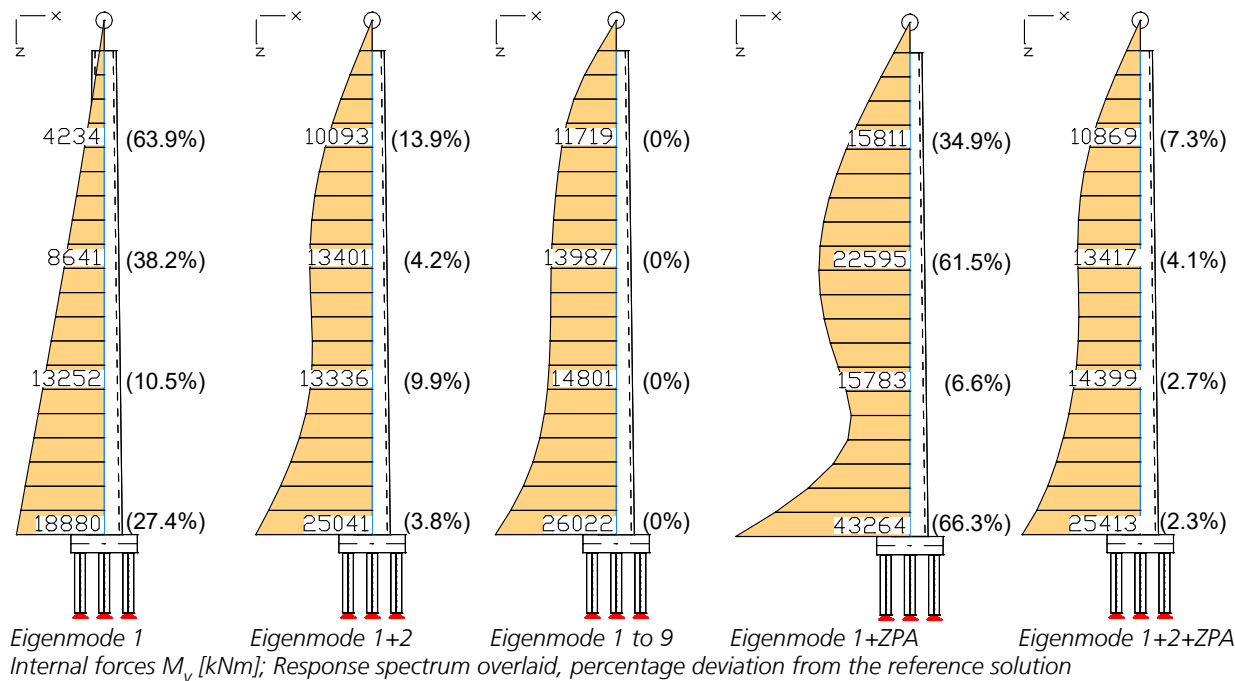
Behavior coefficient q: 1.500

**Effective modal masses (global)**

Mode shape	Mx eff [t]	My eff [t]	Mz eff [t]
1	6554.330	0.000	0.000
2	940.564	0.000	0.000
3	2393.436	0.000	0.000
Sum:	9888.331	0.000	0.000
Part of the total masses:	100 [%]	0 [%]	0 [%]



Response spectrum



As can be seen from the comparison of the internal forces due to the superimposed earthquake loads, the ZPA method with two eigenmodes provides a result similar to the reference solution with 9 eigenmodes. You can see that the ZPA method only approximates high-frequency eigenmodes well. This is one of the reasons for the remaining deviations.

# Pushover Analysis of a Four-storey Reinforced Concrete Frame

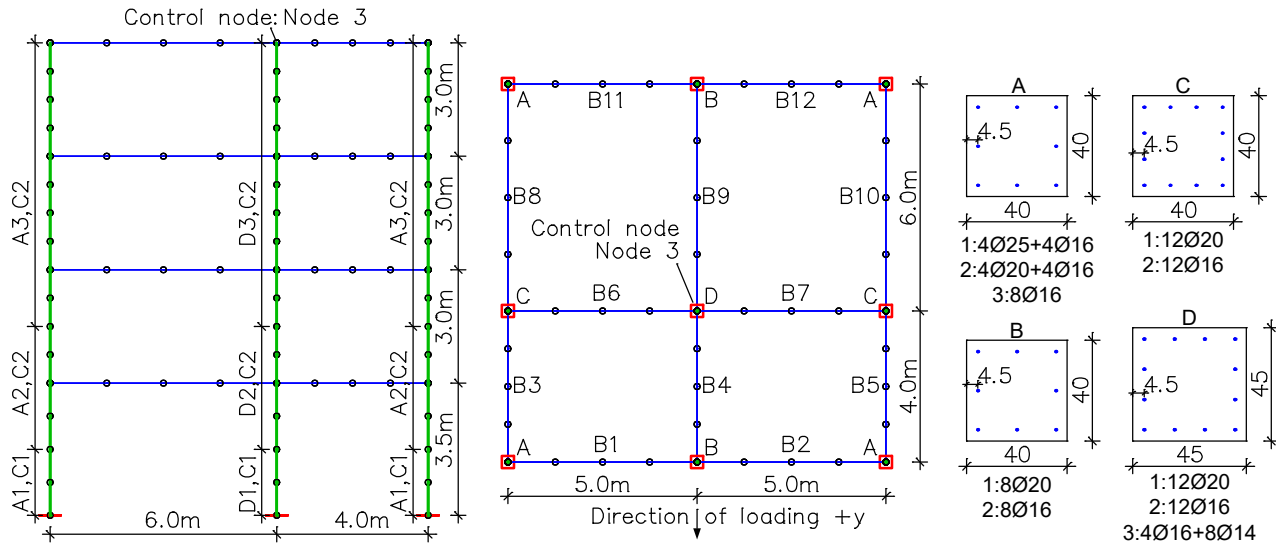
This example shows the procedure for a pushover analysis for a four-storey reinforced concrete frame. The example was selected based on the publication

OS Modeler – Examples of application, Version 1, University of Ljubljana, 2008, Matjaž Dolšek

From the results of the pushover analysis, the target displacement for a defined seismic load is determined. The target displacement is determined by the N2 method.

## Structural properties

The dimensions of the calculation model with the existing reinforcement quantities can be seen in the following figures. The material C25/30 according to EN 1992-1-1 (basic standard) was selected. The yield strength  $f_y$  of all reinforcement is 500 MN/m<sup>2</sup>. The longitudinal reinforcement is strapped in all cross sections.

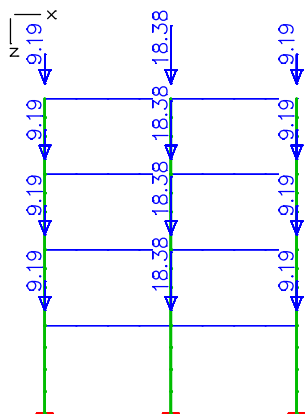


Beam	$b_{eff}$ [cm]	$A_{s,1}$ [cm <sup>2</sup> ]	$A_{s,2}$ [cm <sup>2</sup> ]
B3, B5	70	4.618	4.210
B1, B2, B11, B12	80	4.618	4.210
B8, B10	90	4.618	4.210
B4	110	3.848	3.079
B6, B7	130	3.848	3.079
B9	150	3.848	3.079

On the program side, the implemented method is divided into the calculation steps described under "Pushover calculation according to EN 1998-1:2013".

## Dynamics

In order to reach the storey masses examined in the example (1st floor 87 t, 2nd and 3rd floor 86 t, 4th floor 83 t), the masses involved in the vibration have been supplemented by the missing shares with *Generate masses from loads*.



### Generate masses from loads

Load case	factor
1	1.00

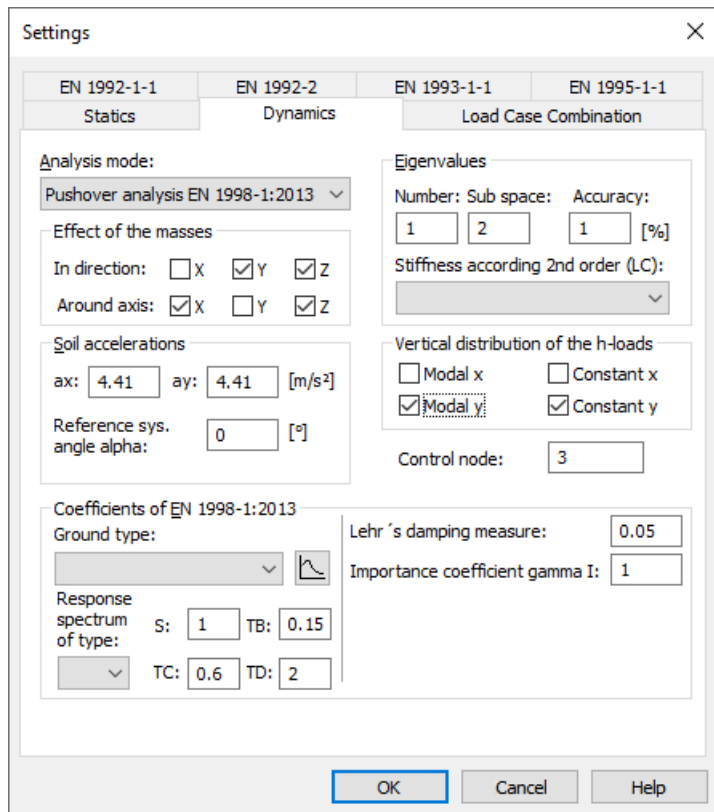
Extract from the listing *Finite elements dynamics* to control the considered masses (87 + 2·86 + 83 = 342 t):

### Sum of the installed masses (global)

	Mx [t]	My [t]	Mz [t]
Volume masses :	0.000	192.180	192.180
Point masses :	0.000	0.000	0.000
Masses of LC :	0.000	149.888	149.888
Sum :	0.000	342.068	342.068

Load case 1: G

For the subsequent pushover analysis i.a. horizontal static pseudo loads are required. These are applied in the appropriate direction to the mass points using the mass assignment and the basic modal form according to EN 1998-1, Annex B.



The dynamics setting dialogue requires beside the analysis mode the following information:

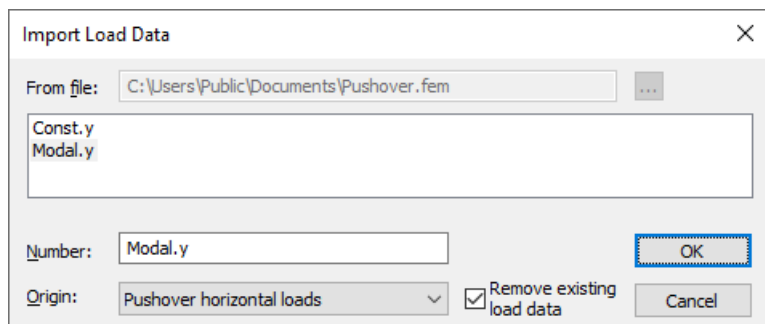
- *Effect of the masses*: In the example the structure should not swing in the x direction or around the y axis.
- *Eigenvalues*: Enough eigenvalues to get the basic model form in the y direction.
- *Soil acceleration*: In the example the masses were accelerated in the y-direction with  $0.45 \cdot g = 4.41 \text{ m/s}^2$ .
- *Coefficients of EN 1998-1:2013*: The elastic response spectrum used in the example is defined by the shown values ( $S, T_B, T_C, T_D$ , *damping measure* and  $\gamma_I$ ).
- *Vertical distribution of the h-loads*: In the example a modal distribution of horizontal loads (*modal y*) is assumed. According to the standard, Section 4.3.3.4.2.2, additionally a *constant* distribution of the h-load (uniform acceleration response) is to investigate.
- *Control node*: By the standard the eigenmode at the control node (usually in the roof level) is to scale to 1. According to Section 4.3.3.4.2.3 and Annex B the control node is also used for calculating the equivalent mass  $m^*$  of an oscillator with a single degree of freedom, the displacement and capacity curves as well as the transformation coefficient  $\Gamma$ .

**Note:**

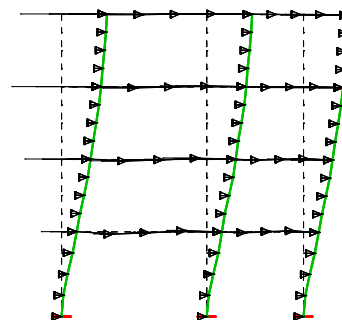
The dynamic calculation can only be performed if on the settings page *Statics* the *Nonlinear analysis* is switched off.

**Nonlinear Structural Analysis**

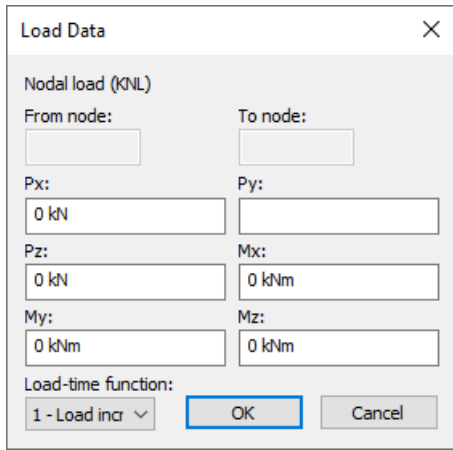
For the nonlinear pushover analysis, load cases are required which include both the static load components (G) and the horizontal load components (earthquakes) to be increased. The horizontal load components to be increased (*Modal.y* or *Const.y*) have already been determined within the dynamic calculation. They have to be assigned to load cases and the degree of load increase has to be defined. In general, the load components are initially defined in separate load cases and then summarized by a load group.



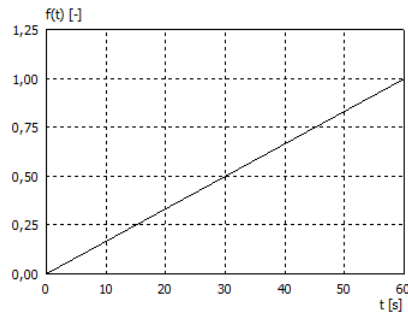
Import of horizontal static pseudo loads in a load case



Eigenmode 1 and imported loads (*Modal.y*)

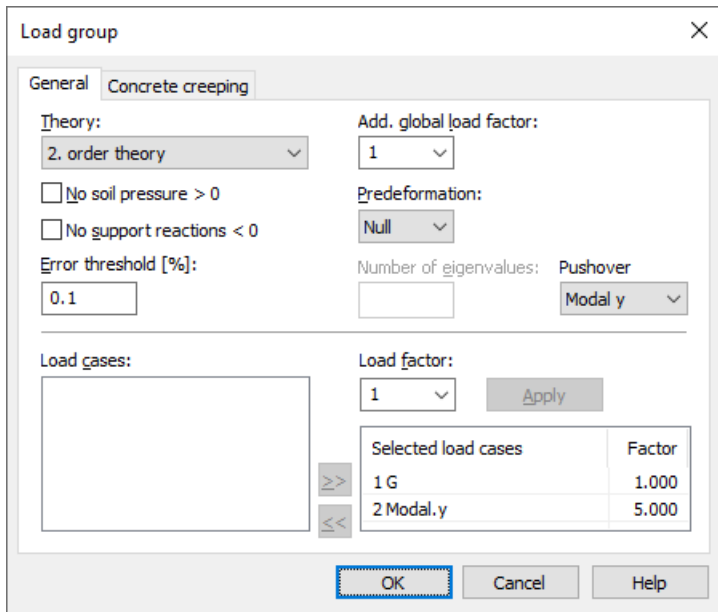


After import into a load case, the node loads still have to be provided with a load-time function, because only the horizontal node loads are to be increased in the context of the nonlinear analysis.



Example of the *load-time function*

The defined increase in loads is not significant for the arc length method. The vertical loads (G) should be constant in all load levels.



For the nonlinear pushover analysis, a load case 3 is defined by a load group in this example.

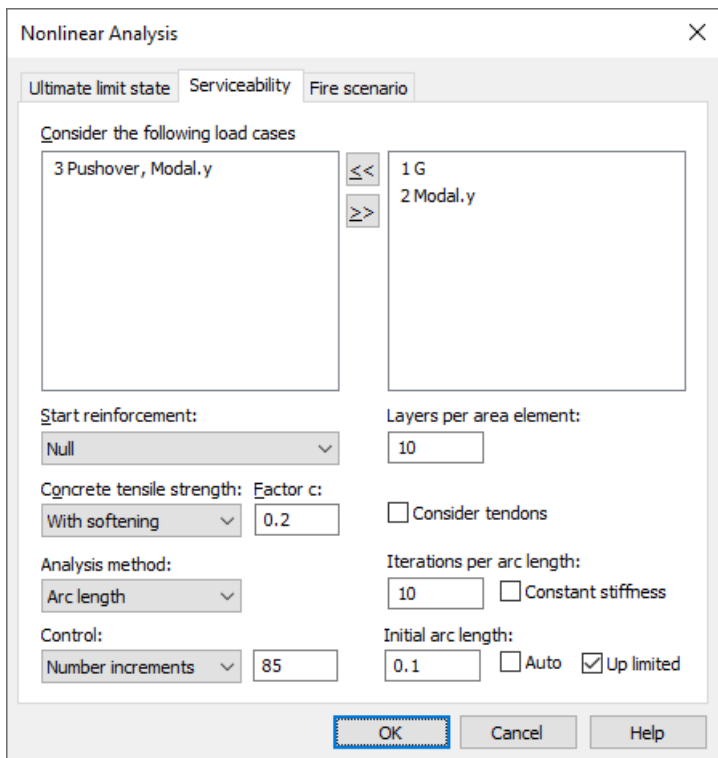
As the structure deforms greatly, the second-order theory is selected. To increase accuracy, the error threshold was set to 0.1 %.

In the earthquake situation, a characteristic G-load is generally assumed (load factor 1.0). The horizontal displacement of the control node should reach approximately 150 % of the target displacement according to Section 4.3.3.4.2.3.

In this example, the arc length method is used. The size of the h-loads reached depends heavily on the parameters of the arc length method in addition to the set factor for load case 2 *Modal.y*. The sign of the factor defines the load direction (here +y).

Furthermore, the *pushover* case matching the h-loads is to be selected (here *Modal.y*).

Load case 3: *Pushover, Modal.y*

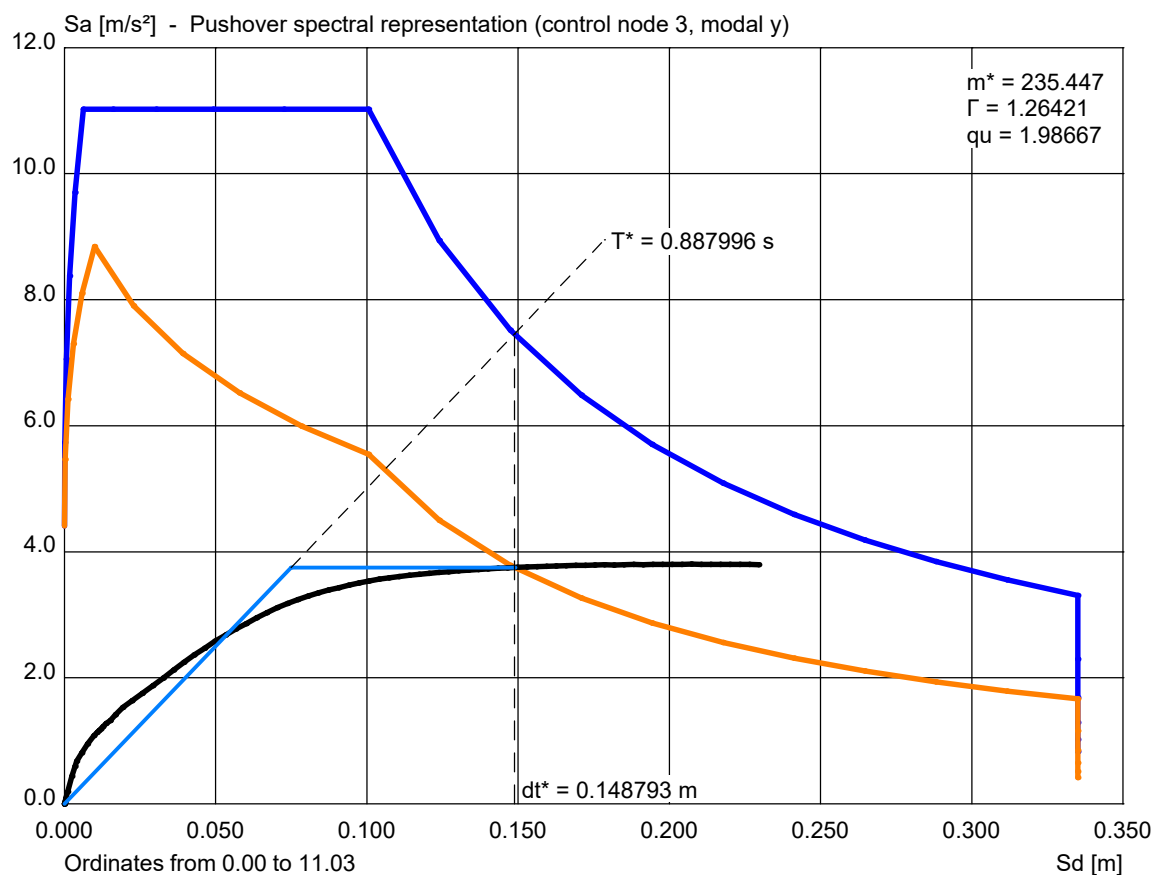
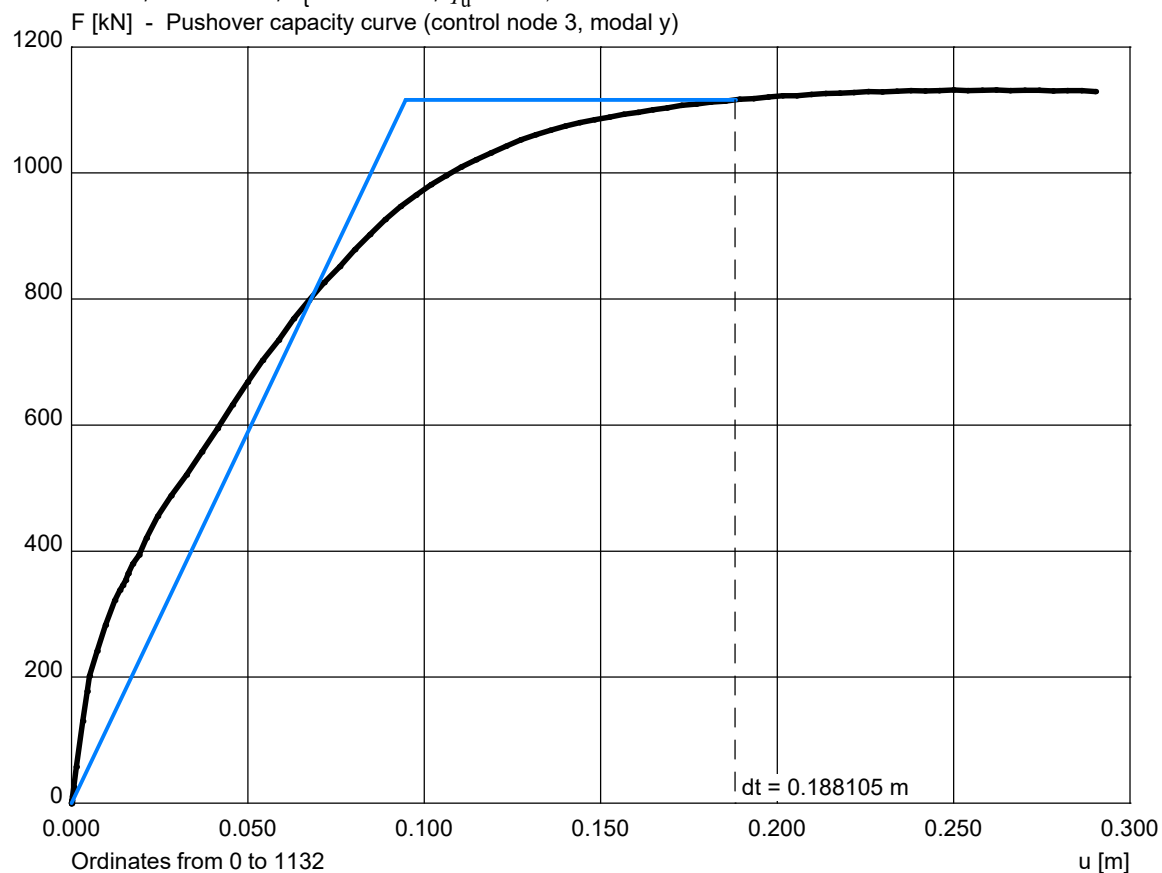


In the settings for *Nonlinear analysis / serviceability* in addition to the selection of the load case and the *arc length* analysis method the following settings have been made:

- *Start reinforcement*: If *Null* is selected, the basic reinforcement of the reinforcing steel layers of the cross sections is used.
- *Concrete tensile strength*: This option defines the behaviour of the concrete in the tensile zone for all concrete cross sections. The comparison calculation assumed a concrete tensile strength to be applied.
- *Control*: The number of increments of 85 selected here must in conjunction with the other parameters of the arc length method be chosen so that a sufficient horizontal displacement of the control node is achieved.
- *Iterations per arc length*: To prevent the arc length becoming very small, the number has been increased to 10.
- *Initial arc length*: This has been limited to the top to get enough intermediate results in the load-displacement curve.

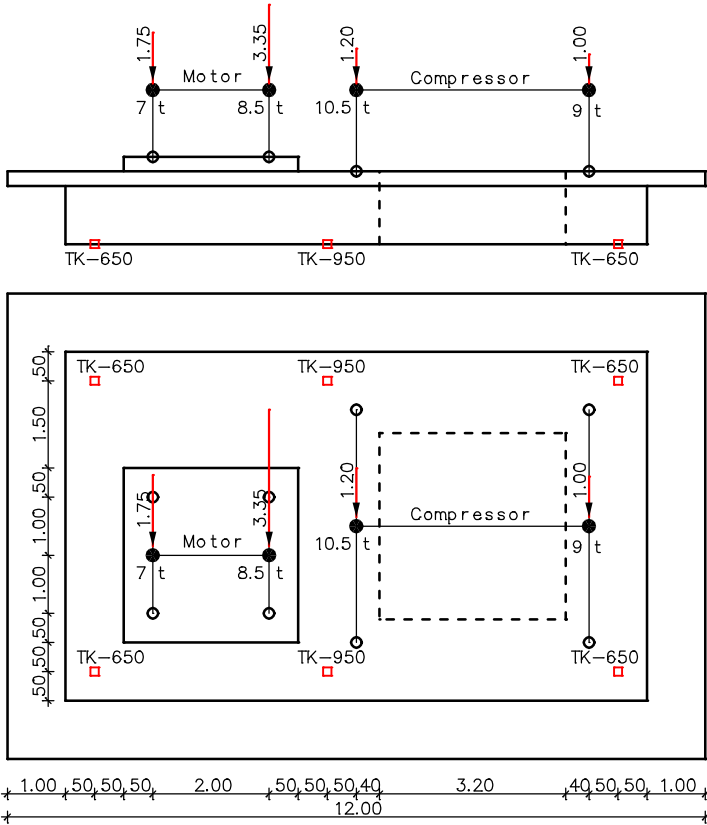
## Results

After the two calculation steps, in addition to the nodal deformations and internal forces per load level, the capacity curve in the multi-mass system and spectral representation is available. Their linearization is iterative according to EN 1998-1, Section B.5. The presented results are in very good agreement with the results of Matjaž Dolšek ( $m^* = 227.9$  t,  $\Gamma = 1.279$ ,  $F = 1102$  kN,  $T^* = 0.87$  s,  $d_t^* = 0.146$  m,  $q_u = 2.01$ ).



## Maschine Foundation With Multiple Exciters

This example treats the modeling and analysis of a machine foundation under dynamic load from two exciters of different frequencies, a motor (operating frequency: 25 Hz) and a compressor (operating frequency: 50 Hz). The foundation is made of concrete C35/45. In order to take into account all eccentricities, a solid model was chosen. The beam structure for the motor and the compressor takes into account the positions of the machine masses (bearing shaft) and the load application points of the imbalance forces. Therewith the deformation and acceleration of the system at these positions can be calculated. The beams have no masses ( $g = 0$ ), because they are included in the point masses of the machines. To avoid restraints, normal force joints are defined at the bearing shafts. The following figure shows the system with the masses and imbalance forces [kN] of the machines.



### Point masses

Nodes	Mass [t]
135	7.0
136	8.5
137	10.5
138	9.0

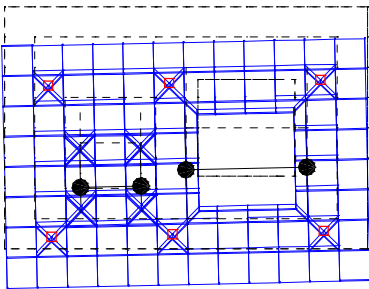
For the beam stiffness the following values are used:

- $A = 0,1 \text{ m}^2$
- $I_x = 10^{-6} \text{ m}^4$
- $I_y = I_z = 0,1 \text{ m}^4$
- E-Modulus = 210000 MN/m<sup>2</sup>
- G-Modulus = 81000 MN/m<sup>2</sup>

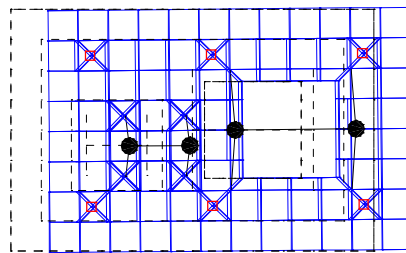
The six supports have two different spring elements specifications (spring rigidities [MN/m]).

GERB	TK-650	TK-950
horizontal	11.9	15.9
vertical	20.6	29.8

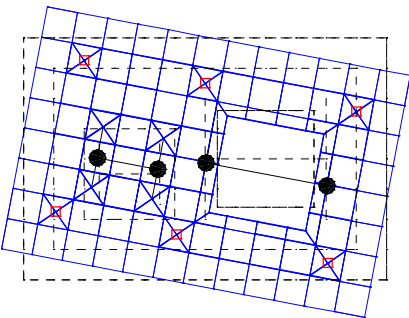
### Eigenmode



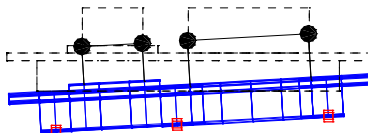
Eigenmode 1:  $\omega = 2.81 \text{ Hz}$



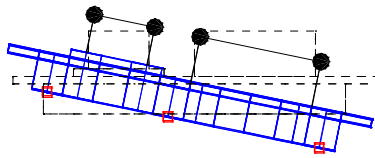
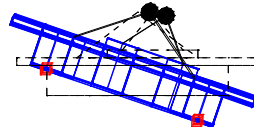
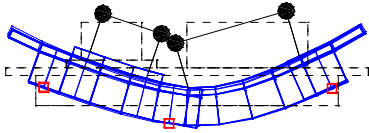
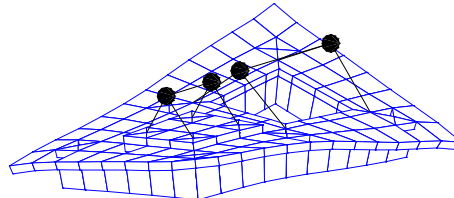
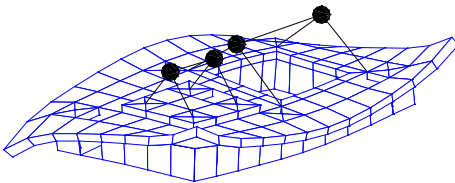
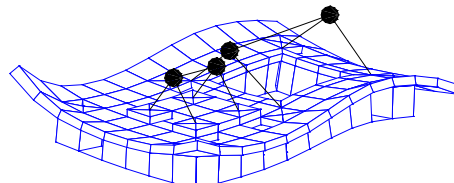
Eigenmode 2:  $\omega = 2.88 \text{ Hz}$



Eigenmode 3:  $\omega = 3.59 \text{ Hz}$



Eigenmode 4:  $\omega = 4.00 \text{ Hz}$

Eigenmode 5:  $\omega = 4.54$  HzEigenmode 6:  $\omega = 5.29$  HzEigenmode 7:  $\omega = 27.56$  HzEigenmode 8:  $\omega = 34.81$  HzEigenmode 9:  $\omega = 66.66$  HzEigenmode 10:  $\omega = 70.82$  Hz

## Evaluation of resonant frequencies

In accordance with the withdrawn DIN 4024, Part 1, Section 5.3.2 the vibration behavior of the machine foundation could be evaluated on the basis of the resonant frequencies. If the following conditions 1 and 2 were fulfilled, further analyses of the vibration behavior were not necessary:

1. Resonant frequency 1. order

$$f_{1(z)} \leq 0.8 \cdot f_m$$

$$f_4 \leq 0.8 \cdot 25 \text{ Hz}$$

$$4.0 \leq 20 \text{ Hz}$$

2. Resonant frequencies of higher order

$$f_n \leq 0.9 \cdot f_m \quad \text{and} \quad f_{n+1} \geq 1.1 \cdot f_m$$

$$f_6 \leq 0.9 \cdot 25 \text{ Hz} \quad f_7 \geq 1.1 \cdot 25 \text{ Hz}$$

$$5,3 \leq 22.5 \text{ Hz} \quad 27,6 \geq 27.5 \text{ Hz}$$

or

$$f_8 \leq 0.9 \cdot 50 \text{ Hz} \quad f_9 \geq 1.1 \cdot 50 \text{ Hz}$$

$$34,8 \leq 45 \text{ Hz} \quad 66,7 \geq 55 \text{ Hz}$$

Terms:

$f_m$  Machine frequency (25 or 50 Hz)

$f_1$  First resonant frequency with the eigenmode in the direction of the exciter (here: z direction)

$f_n$  Highest resonant frequency below the machine frequency

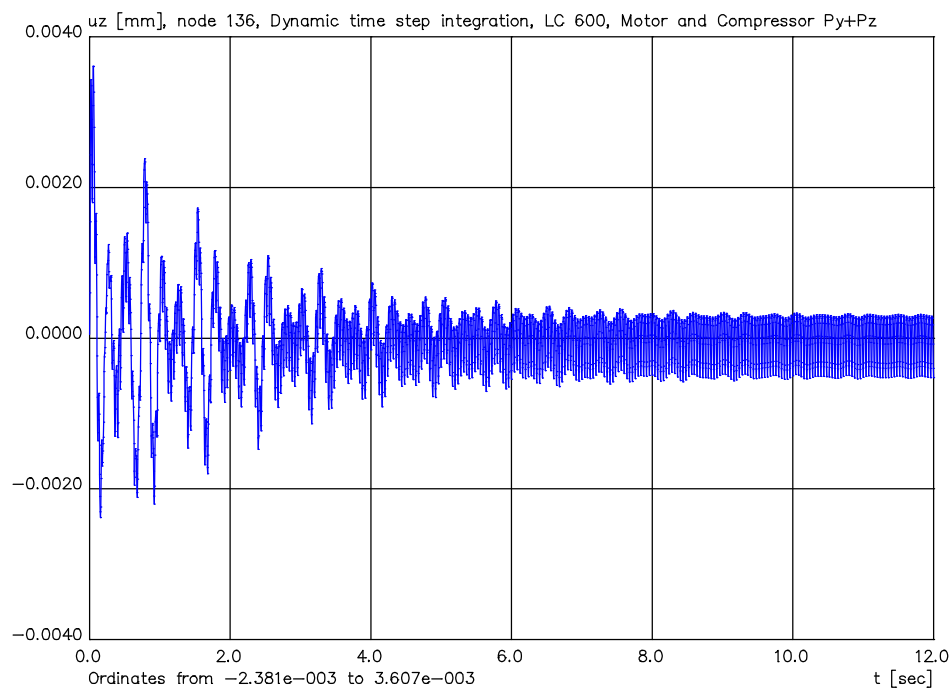
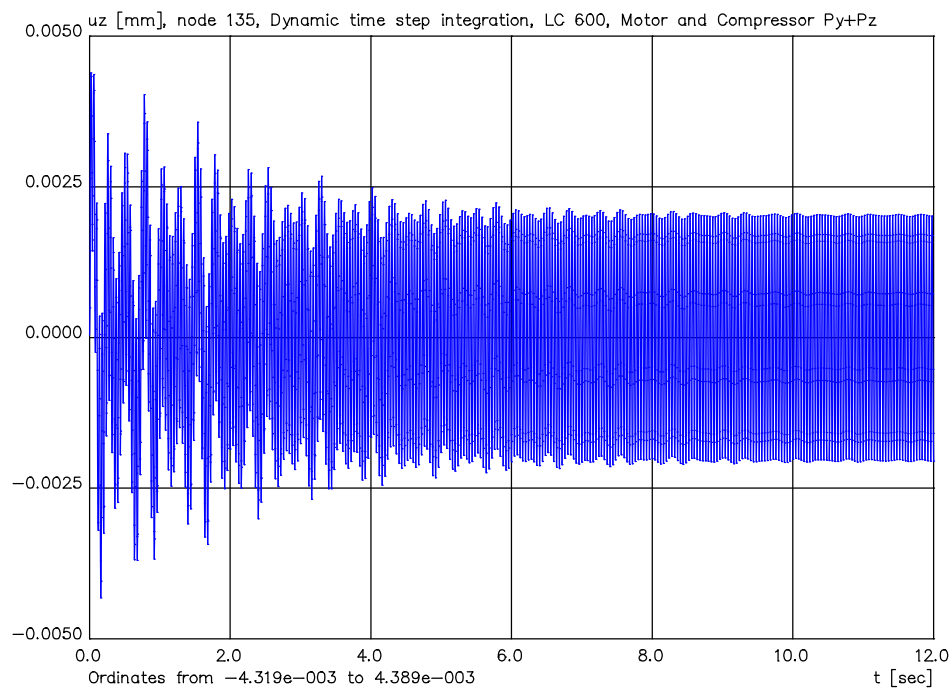
$f_{n+1}$  Lowest resonant frequency above the machine frequency

## Analysis of the forced vibrations

If no positive evaluation of the vibration behavior is possible, a more detailed analysis with the calculation of the forced displacement values can be necessary. The calculation of the stationary response, separately for the different frequent exciters with a Lehr's damping of 2 % for all 10 resonant frequencies, leads to the maximum amplitudes of the displacement  $u_z$  [ $\mu\text{m}$ ] of the bearing shafts which are put together in the following table. It is more unfavorable, if the maximum values of the single deformations of the nodes are added together as done in the table below. In the steady state the sum of the single deformations of the nodes correspond to the results of the time step integration, which is presented in the following section.

Loadcase	Node 135	Node 136	Node 137	Node 138
$P_z$ of motor at 25 Hz	2.007	0.488	0.398	0.916
$P_z$ of compressor at 50 Hz	0.010	0.124	0.169	0.155
$P_y$ of motor at 25 Hz	0.014	0.129	0.007	0.031
$P_y$ of compressor at 50 Hz	0.003	0.008	0.003	0.002
Sum of amplitudes [ $\mu\text{m}$ ]	2.034	0.749	0.577	1.104

A modal time step integration with 6000 time steps and a time increment of 0.002 s under a simultaneous action of all dynamic loads leads to the following deformations of the bearing shaft of the motor.





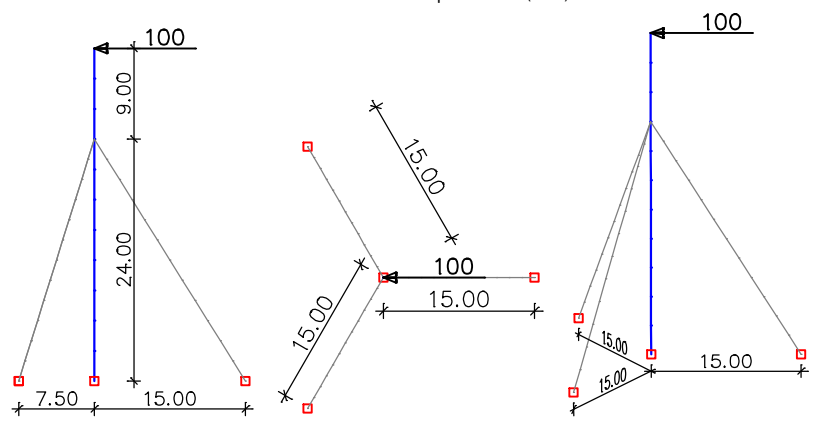
The maximum amplitudes of the displacement  $u_z$  of the nodes 135 to 138 of the bearing shafts after 12 seconds (steady state) are:

Analysis	Node 135	Node 136	Node 137	Node 138
Stationary response: $\Sigma u_z $	2.03 $\mu\text{m}$	0.75 $\mu\text{m}$	0.58 $\mu\text{m}$	1.10 $\mu\text{m}$
Modal analysis	2.05 $\mu\text{m}$	0.53 $\mu\text{m}$	0.58 $\mu\text{m}$	1.10 $\mu\text{m}$

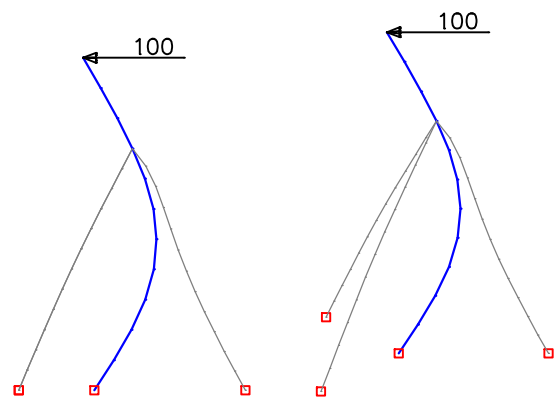
### Braced Mast

This is an example of a mast braced in three directions. The braces are represented by 12 cable elements each. The point load that is applied to the tip of the mast oscillates periodically. The analysis of the structure is carried out with a direct time step integration in 100 time steps over a duration of 2 seconds.

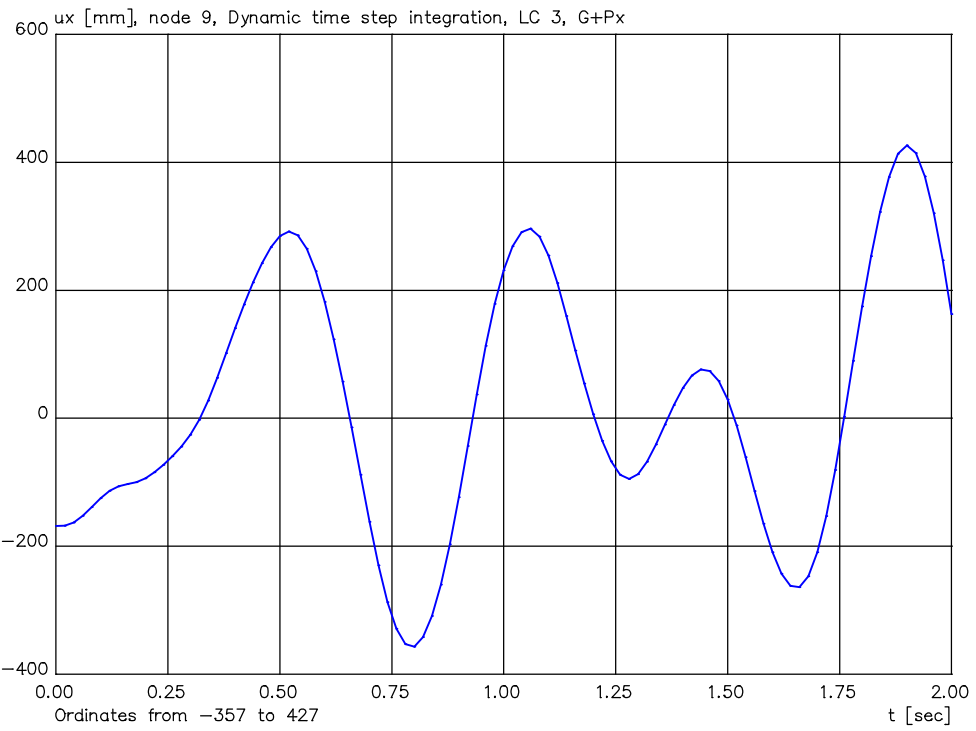
- Mast: Tube (MSH),  $d = 660 \text{ mm}$ ,  $t = 20 \text{ mm}$ , S355
- Cable elements:  $A = 3.57 \text{ cm}^2$ ,  $E = 150000 \text{ MN/m}^2$ , Cable prestress = 200.0 kN
- Load: Dead load and 100 kN periodic (cos) with 2.31 Hz at the mast tip



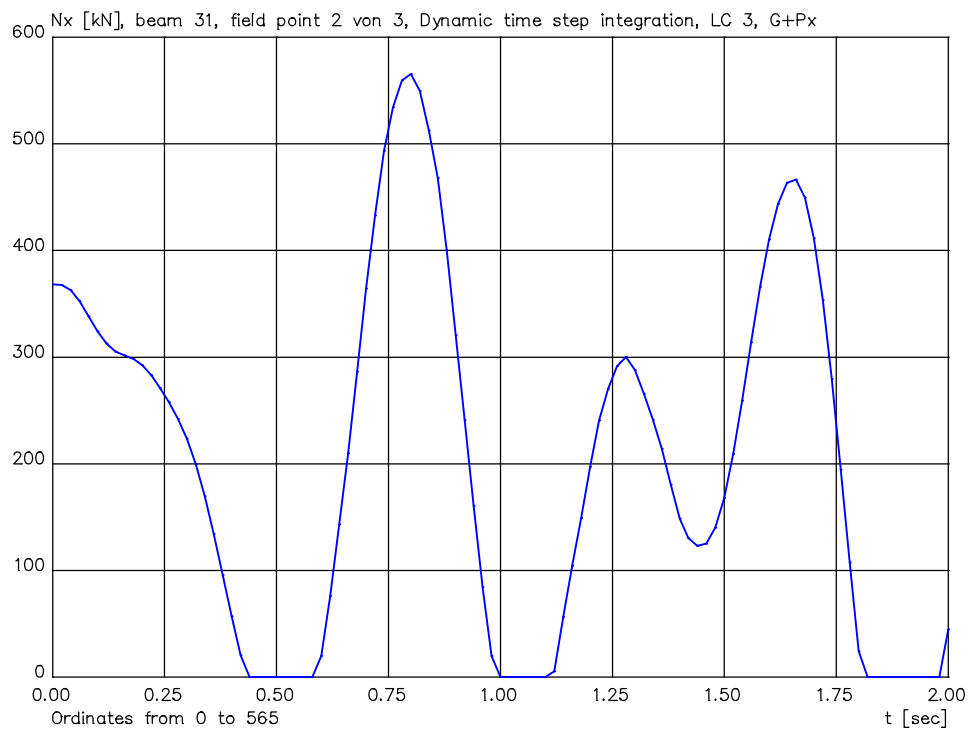
System measurements with load [kN]



XZ view 3D view  
Deformation figure after 1.94 seconds (superelevated 10 times)



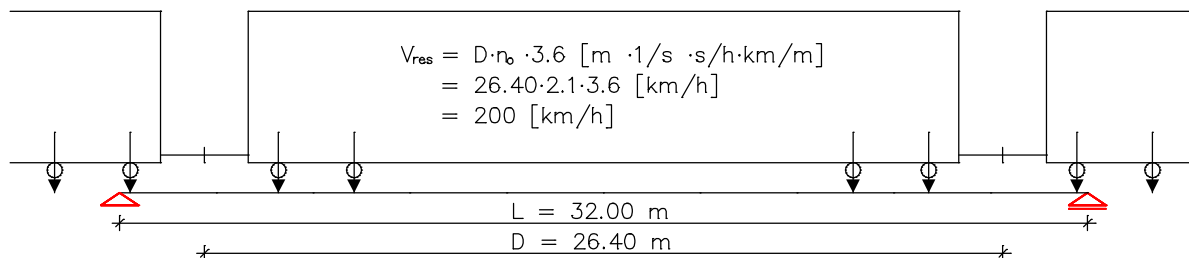
Deformation at the selected node in the x direction



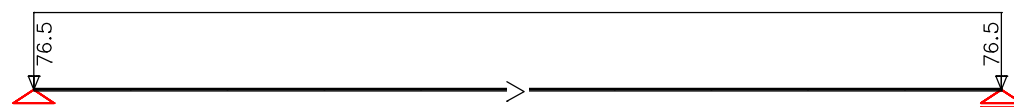
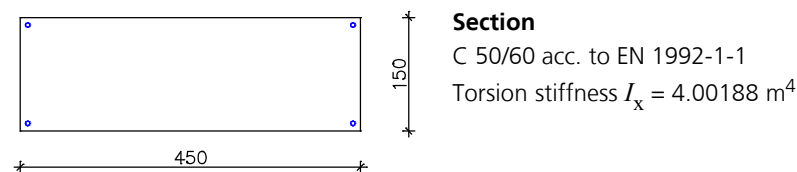
Internal forces  $N_x$  at the selected result location

## Dynamic Train Crossing

With the program the dynamic load by predefined regular trains such as ICE, Thalys etc. or self-defined train loads can be easily examined. In this example, an ICE 1 is supposed to drive over a 32 m single-span bridge made of beam elements.



System with ICE 1, dimensions and resonant speed



In addition to the input of the system and the static load cases the following steps are necessary:

- Definition of the track and specification of the train properties
- Assignment of masses and damping
- Dynamic time step integration
- Representation and process of results

### Track and train definition

The tracks are described by entering a polygon on the structure. It is only necessary that the track lies within the section dimensions of the elements. Eccentricities relative to the beam axes are automatically taken into account.

The load type *Dynamic train load* is to be selected for one load case and the track of the train is to be specified. It can run across area elements and beams with a polygon section. After entering the polygon the dialog for the train definition will be opened:

**Dynamic train load** [X]

Label: ICE 1, 200 km/h    No.: 1

Train: ICE 1    v: [km/h] 200    Start time: 0

Track    Train

	Distance [m]	Axle load [kN]
1	0	195
2	3	195
3	11.46	195
4	14.46	195
5	19.31	130
6	21.81	130
7	38.31	130

OK    Cancel

In this dialog the desired type of train and the speed to be analyzed are to be specified. If multiple trains or tracks are to be analyzed simultaneously (e.g. for a train encounter), the interval can be regulated by varying the departure times. If a user-defined train is selected, the axle distance and axle loads can be edited in the column *Train*. Data from the clipboard can also be inserted.

Therewith the definition of the track is finished.

During the calculation the train is considered as a group of single loads running along the track. The axle loads always take effect in the direction of the global z-axis.

### Assignment of masses and damping

For the dynamic analysis the considered masses are important. In the standard case only the masses from dead weight are taken into account. Additional masses such as from e.g. the ballast bed can be taken into account with *Generate masses from loads*.

#### Sum of the installed masses (global)

	Mx [t]	My [t]	Mz [t]
Volume masses :	550.459	550.459	550.459
Point masses :	0.000	0.000	0.000
Masses of LC :	249.541	249.541	249.541
Sum :	800.000	800.000	800.000

Also a useful damping of the system should be considered.

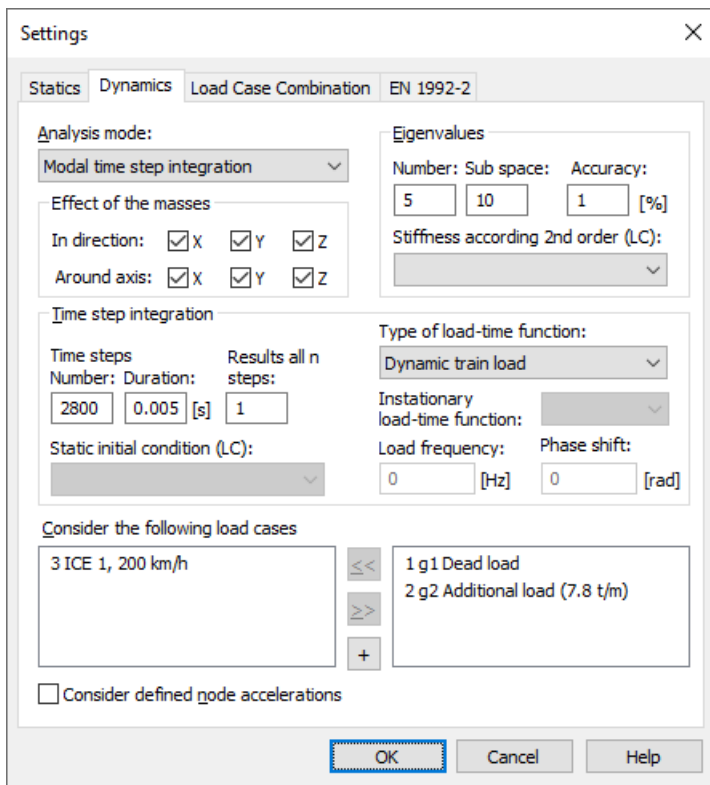
### Modal damping

Eigenmode	D [%]	alpha [1/s]	β [s]
1	1.500	0.00000	0.00000
2	1.500	0.00000	0.00000
3	1.500	0.00000	0.00000
4	1.500	0.00000	0.00000
5	1.500	0.00000	0.00000

- D Lehr's damping measure
- alpha Mass proportional damping
- β Stiffness proportional damping

### Dynamic time-step integration

Dynamic train loads are only taken into account during a dynamic analysis. The specifications are made in the tap *Dynamics* of the analysis settings.



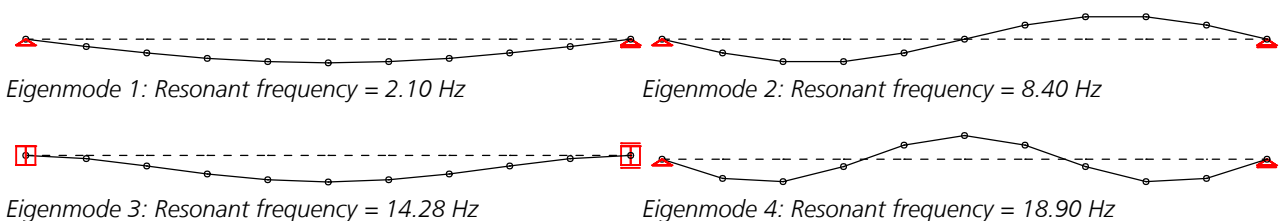
The train crossing can be analyzed using the direct or the modal time-step integration.

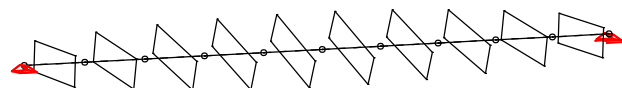
The number of eigen values to be considered depends on the type of structure. The duration and number of the time steps should be chosen in a way to get the desired calculation duration or the route of the train.

The duration of the chosen time steps is substantially specified by the oscillation behavior of the structure. It should be lower than 1/10 of the oscillation time of the decisive resonant frequency.

During the analysis the resonant frequencies, the eigen modes and all time steps are calculated. For each time step the deformations, velocities, accelerations and internal forces are determined and saved.

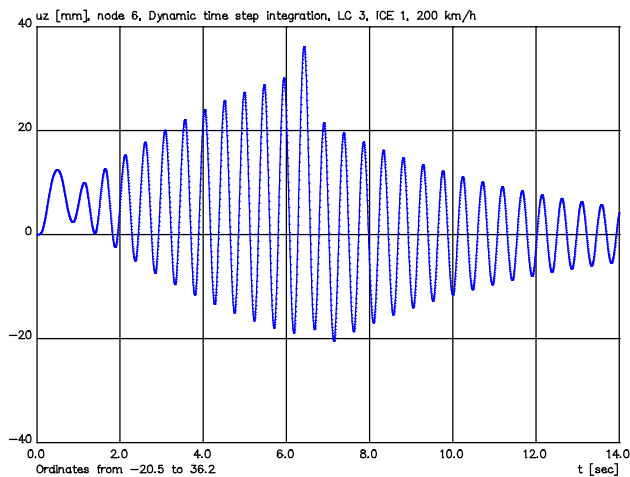
### Represent and process results



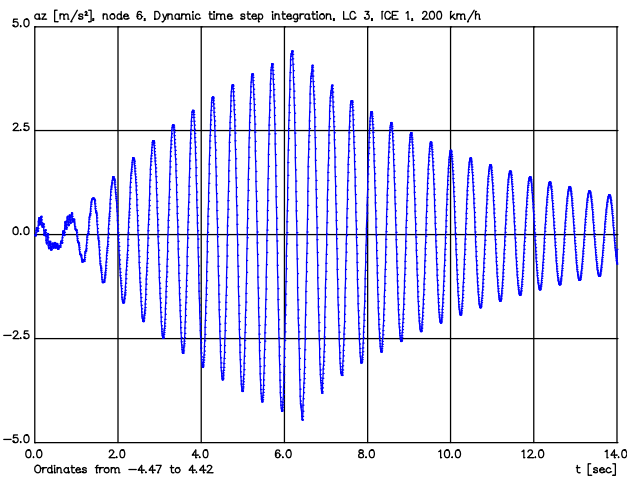


Eigenmode 5: Resonant frequency = 21.69 Hz

Beside the eigen modes with their resonant frequencies the results of the train crossing can be represented for each node over the calculation time. For this the representation area is split into two windows. In the left window the node is to be selected for which the result curve shall be displayed.



Vertical displacement at the mid-span



Vertical acceleration at the mid-span

Some dynamic results like deformations and internal forces can additionally be represented for each time step at the system.

With load case combinations the 'envelope curve' of all time steps for deformations and internal forces can be determined and, where required, can be superposed with static load cases.

**Load case combination 1, ICE 1, 200 km/h**

Exclusive selection		Factor								
3	ICE 1, 200 km/h (Time step integration)	1.000								
	-6.3	-12.0	-16.5	-19.5	-20.5	-19.5	-16.6	-12.1	-6.4	
	11.2	21.3	29.3	34.4	36.2	34.4	29.3	21.3	11.2	
Deformation $u_{z \min, \max}$ from load case combination 1 [mm]										
	-0.11	-0.20	-0.28	-0.33	-0.34	-0.32	-0.28	-0.20	-0.11	
	0.11	0.21	0.28	0.33	0.35	0.33	0.28	0.21	0.11	
Velocities $v_{z \min, \max}$ from load case combination 1 [m/s]										
	-1.51	-2.79	-3.72	-4.31	-4.47	-4.12	-3.45	-2.57	-1.39	
	1.25	2.40	3.34	4.03	4.42	4.40	3.92	2.97	1.61	
Accelerations $a_{z \min, \max}$ from load case combination 1 [m/s²]										

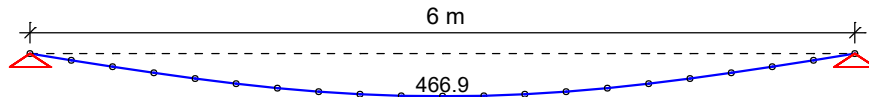
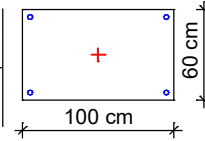
# Design of a Vibration Damper

This example is intended to illustrate the design of a vibration damper, in accordance with the approaches described in *Petersen C. Schwingungsdämpfer im Ingenieurbau*. In this example, the single damper consists of a vertical spring under the beam and a point mass at the lower end of the spring.

## Material properties

	No.	Type	E-Modu. [MN/m <sup>2</sup> ]	G-Modu. [MN/m <sup>2</sup> ]	Poiss. ratio	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]
1	1	C30/37-EN	33000	13800	0.20	1.00e-05	25.000
2	2	C30/37-EN	33000	13800	0.20	1.00e-05	0.000

## Section polygon

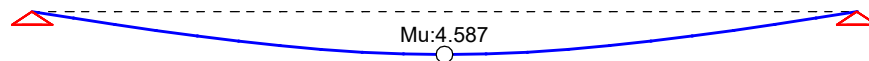


Eigenmode 1: Resonant frequency = 27.20 Hz, Eigenmode u [mm], Factor = 0.7  
Value range (overall system, min/max): 0.00/466.91 [mm]

System A1 mit verteilter Massenbelegung (Material 1) und 1. Eigenform

According to *Petersen*, a vibration damper has the greatest effect at the location and in the direction of the largest amplitude of the eigenmode to be damped. Since in InfoCAD the eigenmodes are normalized according to equation 8, the "kinetic equivalent mass" is  $M_1 = 1/u^2$  of the relevant degree of freedom u of the eigenmode.

Thus,  $M_1 = 1/0.46691^2 = 4,587$  t for the center of the span of the considered beam.



Eigenmode 1: Resonant frequency = 27.00 Hz, Eigenmode u [mm], Factor = 0.7  
Value range (overall system, min/max): 0.00/466.91 [mm]

Point masses in the nodes [t, tm<sup>2</sup>]

Equivalent system B1 with "kinetic equivalent mass" (point mass and weightless material 2) and eigenmode

The figure shows that the equivalent system B1 with only one point mass  $M_1$  has the same amplitude in its eigenmode at the mass point as the original system A1. Due to the additional damper mass  $M_D$  of 0.275 t (approx. 6% of  $M_1$ ), the new equivalent system B2 becomes a two-mass oscillator. According to *Petersen*, the optimum spring stiffness  $k_D$  and viscous damping  $d_D$  of the single damper for a two-mass oscillator are calculated as follows:

$$k_D = (2 \cdot \pi \cdot f_D)^2 \cdot M_D = (2 \cdot \pi \cdot 25.661)^2 \cdot 0,275 \quad \text{with}$$

$$= 7149 \text{ [kN/m]} = 7.149 \text{ [MN/m]} \quad f_D = M_1 / (M_1 + M_D) \cdot f_1$$

$$= 4.587 / (4.587 + 0.275) \cdot 27.2 = 25.661 \text{ [Hz]}$$

and

$$d_D = 2 \cdot M_D \cdot 2 \cdot \pi \cdot f_1 \cdot \zeta_D \quad \zeta_D = \{ 3/8 \cdot M_D \cdot M_1^2 / (M_1 + M_D)^3 \}^{1/2}$$

$$= 2 \cdot 0.275 \cdot 2 \cdot \pi \cdot 27.2 \cdot 0.1374 = 12.915 \text{ [kN s/m]} \quad = \{ 3/8 \cdot 0.275 \cdot 4.587^2 / (4.587 + 0.275)^3 \}^{1/2} = 0.1374 \text{ [-]}$$

## Spring element 1

Geometry	x [m]	y [m]	z [m]	Characteristics [MN/m], [MNm/rad]	
				Comp.	Label
Node 11	3	0	0	ux	Yes
Node 22	3	0	0.5		
x-direction	0	0	1		
z-direction	-1	0	0		

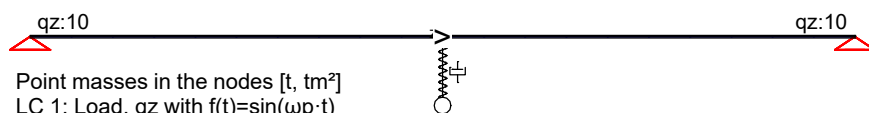
Linear Stiffness: 7.149  
Viscous damping (ux): 12.915 [kNs/m]

In addition, a Lehr's damping factor for all eigenmodes of 1 % is assumed.

## Point masses

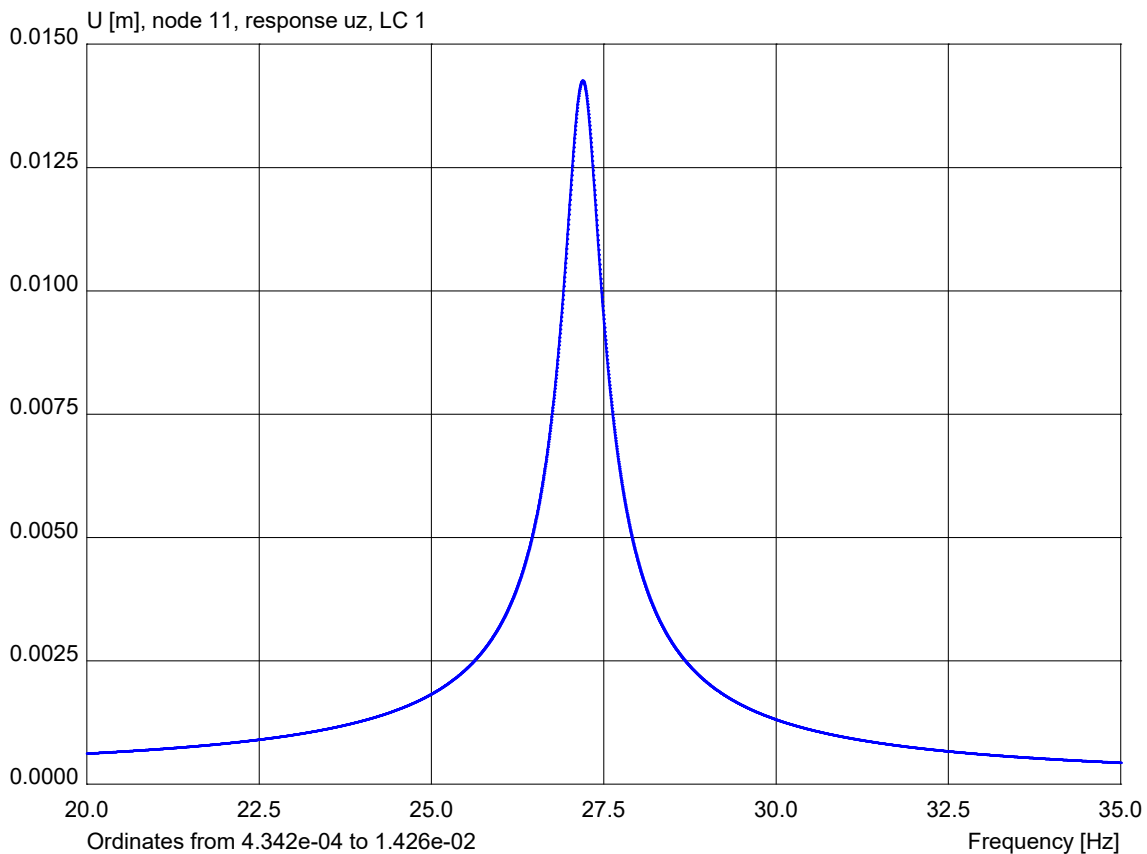
	Node	Mux	Muy	Muz	Mphi.x	Mphi.y	Mphi.z
1	22	0.000e+00	0.000e+00	2.750e-01	0.000e+00	0.000e+00	0.000e+00

The system masses will be activated with the particular weight in material properties.

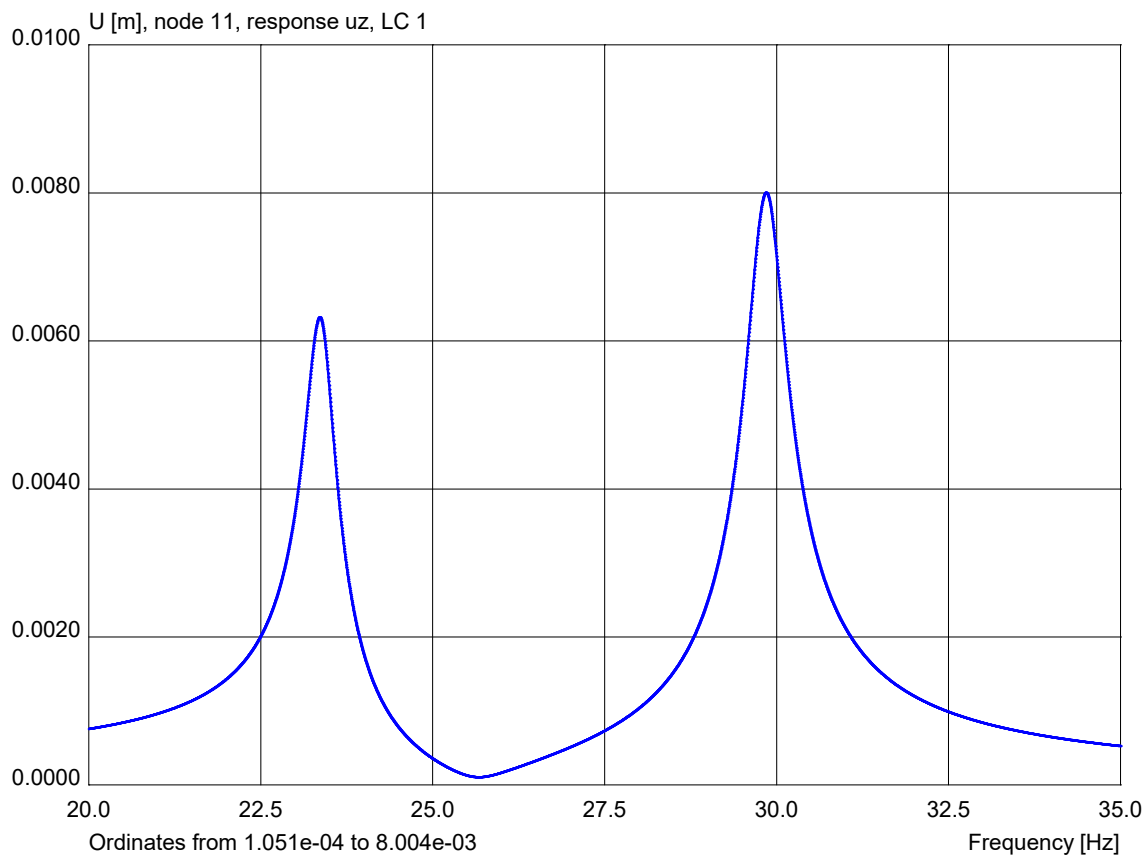


Point masses in the nodes [t, tm<sup>2</sup>]  
LC 1: Load, qz with f(t)=sin(ωp·t)

New system A2 with distributed masses (material 1), damper and the dynamic load

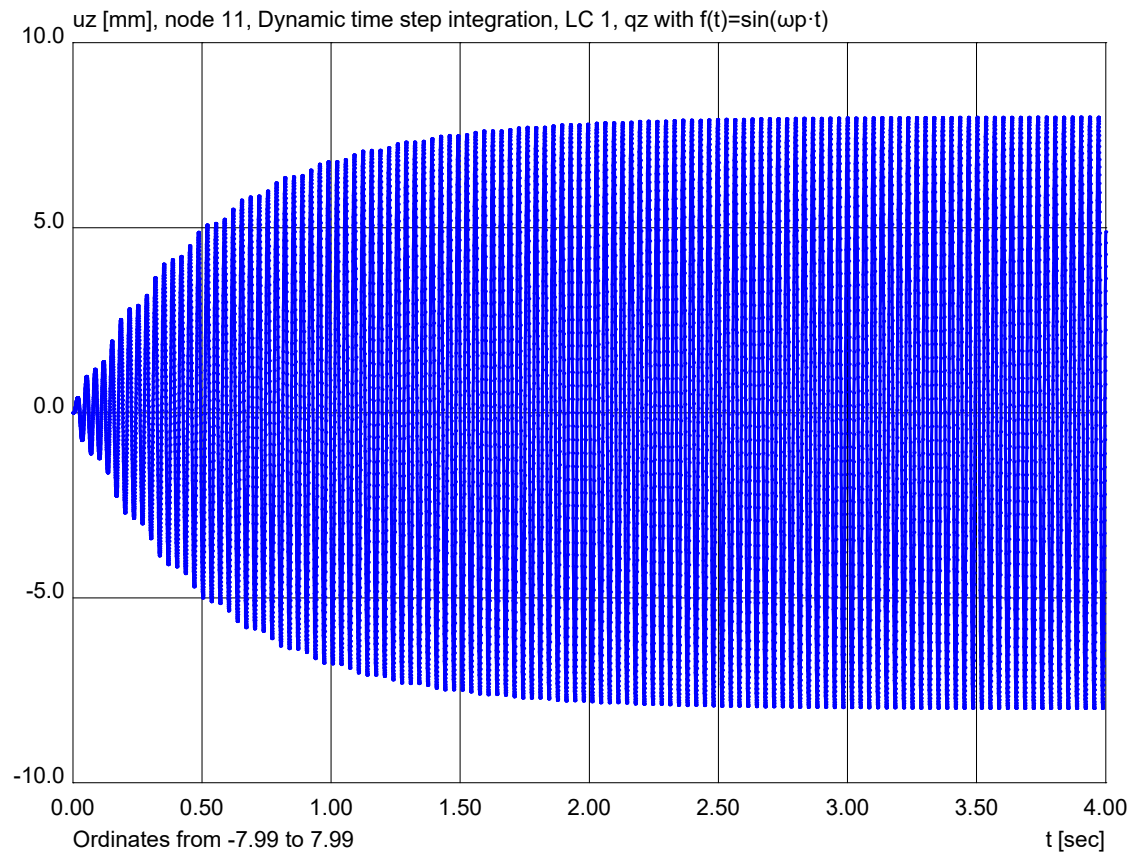


*Response  $u_z$  of the node in the center of the span of system A1 with dead weight but without damper*

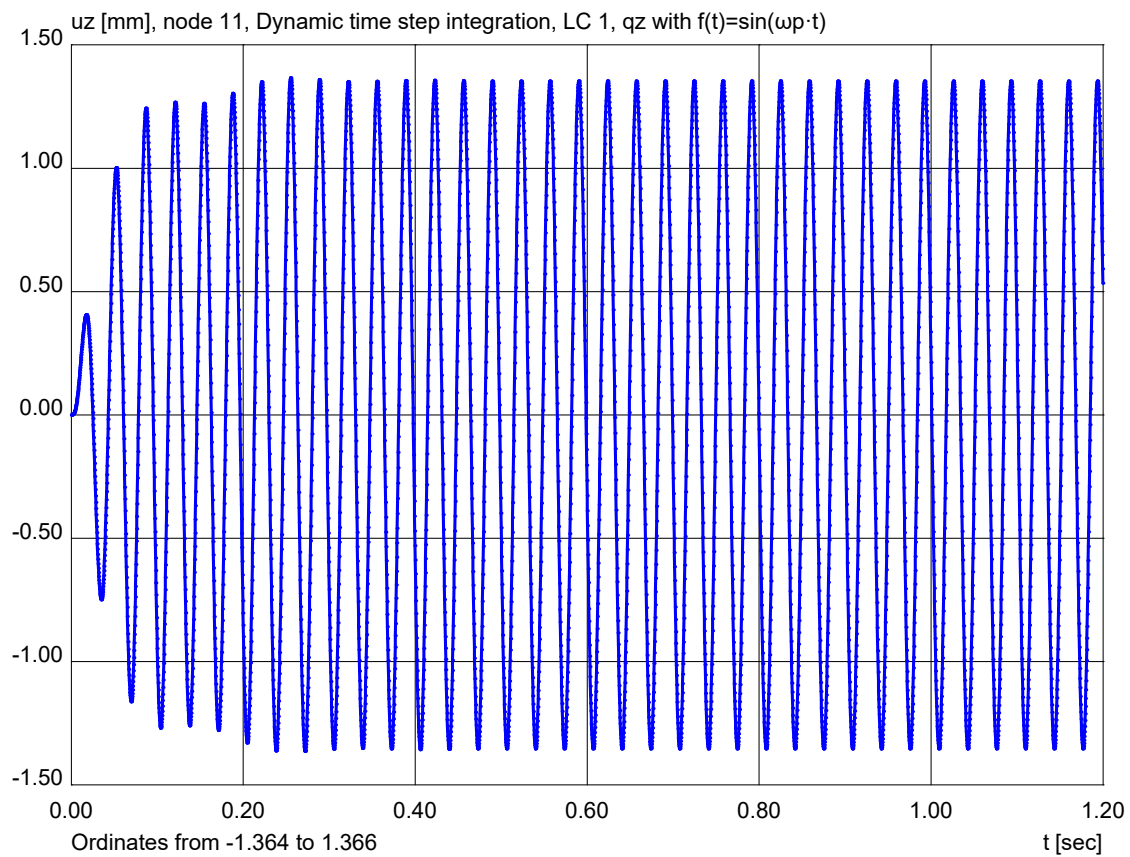


*Response  $u_z$  of the node in the center of the span of system A2 with dead weight and with damper*

The comparison of the results of the response analysis of the two systems A1 and A2 shows well the effect of the damper. The viscous damping cannot be taken into account in the response analysis. For this purpose, a direct time step integration is necessary, which is examined for illustration on system A2 once **without** and once **with** viscous damping in the spring and the sinusoidally oscillating line load with 29.84 Hz (2nd eigenmode) of load case 1.



Displacement  $u_z$  of the node in the center of the span of system A1 without viscous damping



Displacement  $u_z$  of the node in the center of the span of system A2 with viscous damping

The maximum amplitude of the displacement  $u_z$  in the center of the span for the system without viscous damping is 7.99 mm after 4 seconds and thus almost corresponds to the result of the response analysis, which provides the result for the steady state ( $t = \infty$ ,  $u_z = 8$  mm). The displacements  $u_z$  of the system with viscous damping are significantly smaller and much earlier already at their maximum of 1.366 mm.



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Publisher: U.S. Nuclear Regulatory Commission (NRC), NRC Washington D.C. 2012
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Methode der finiten Elemente, Studienausgabe (Methods of Finite Elements, Study Edition).  
Hanser Verlag, München 1984.

# Nonlinear Structural Analysis

## Basics

### Area of Application

The analysis option *Nonlinear analysis* enables the determination of the internal forces and deformations of beam and shell structures made of reinforced concrete and steel as well as stress states of solid elements considering geometric and physical nonlinearities. A distinction can be made between calculations with material characteristics for the ultimate limit state or the serviceability limit state. Due to the considerable numerical complexity, it only makes sense to use this program module for special problems.

Specifically, the following nonlinear effects can be taken into consideration:

- Equilibrium of the deformed system according to the second- or third order theory, if this has been activated for the corresponding load case.
- Beams and area elements made of reinforced concrete according to DIN 1045, DIN 1045-1, OENORM B4700, SIA 262 and EN 1992-1-1.
- Beams and area elements made of steel with bilinear stress-strain curve under consideration of the Huber-von Mises yield criterion and complete interaction with all internal forces.
- Beams and area elements with bilinear stress-strain curve respectively yield criterion according to Raghava or Rankine and individually definable compressive and tensile strength.
- Solids with isotropic damage according to De Vree or Mazars as well as yield criteria according to Huber-Mises, Raghava, Rankine, Mohr-Coulomb, Drucker-Prager or Lubliner with linear hardening.
- Solids with combined plasticity and damage.
- Compression-flexible beams.
- Beam and area element bedding with bilinear bedding curve perpendicular to and alongside the beam.
- Solid elements with bilinear bedding curve in the element coordinate system.
- Pressed steel layers in bond with concrete in beams can be considered with a bilinear stress-strain curve.
- Tendon groups in bond with concrete can be taken into account with a bilinear stress-strain curve.
- Prestressing without bond (only load) can also be simulated.
- Beams and area elements made of steel and reinforced concrete in case of fire according to EN 1992-1-2 and EN 1993-1-2 or beams made of timber in case of fire according to EN 1995-1-2 (see chapter '*Structural analysis for fire scenarios*').

### Analysis Method

The analysis model used is based on the finite element method. To carry out a nonlinear system analysis, the standard beam elements are replaced internally with shear-flexible nonlinear beams with an increased displacement function. For the treatment of area and volume structures nonlinear layer elements and solid elements are used. In contrast to the linear calculation and in order to account for the nonlinear or even discontinuous properties of the structure, a finer subdivision is generally necessary. The program assumes that the actions defined in the load cases are weighted with the appropriate partial safety factors.

#### Requirements

- The beams are assumed to be straight.
- The area elements are flat.
- Area and beam sections are constant for each element.
- The dimensions of the section are small compared with the other system dimensions.
- Shear deformations of beams are accounted for with a shear distortion that is constant across the section, meaning the section remains plane after the deformation but is no longer perpendicular to the beam axis.
- The mathematical curvature is linearized (second-order theory).
- The load is slowly increased to its final value and does not undergo any deviation in direction as a result of the system deformation.

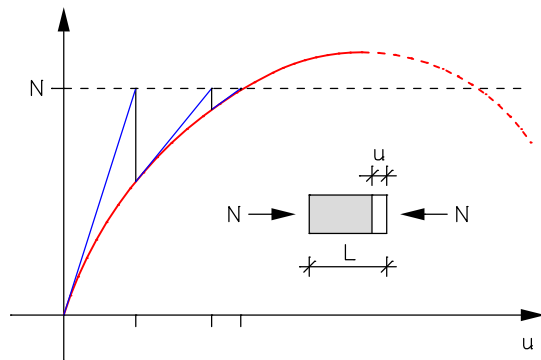
#### Equilibrium iteration

To solve the nonlinear system of equations, both the incremental Newton-Raphson method and the arc length method can be used. In both methods, the tangential stiffness matrix is computed in each iteration depending on the current internal force and deformation state. Alternatively, a constant stiffness matrix (*linear stiffness method*) can be used. Here, the stiffness matrix is computed only once at the beginning of the numerical simulation and in the following assumed to be constant.

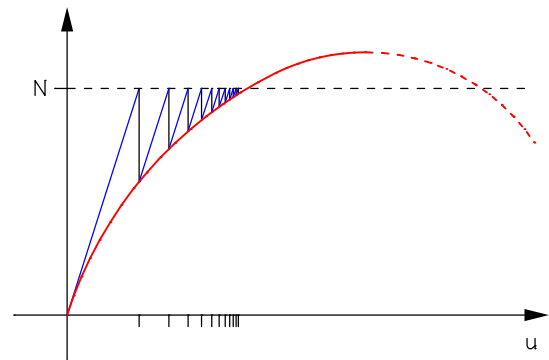
The following calculation methods are available:

- *Newton method:*  
The iteration is load-controlled until the residual (length of the error vector) falls below a required error limit. If the limit is not reached within the maximum allowable iteration steps per load level, the iteration is aborted and the load factor ( $\leq 1.0$ ) is determined by interval nesting.
- *Newton time steps method (NTS):*  
The solution method corresponds to the Newton method, except that for each time step the load present at that time may be different according to the defined load-time function. Based on the solution of the last time step, a solution for the next time step is determined. If it is not possible to fall below the error limit within the maximum iteration steps per load level, the solution for an intermediate point in time is determined by interval nesting.
- *Arc length method (ARC):*  
In this method, in each iteration cycle, starting from the last converged state, the solution of an extended nonlinear system of equations along a circular arc with a given radius (arc length) is sought. As a result of the additional constraint that has to be considered (equation of a generalized sphere), there is another unknown besides the deformation vector - the load factor. Due to the combined load-displacement-controlled method, the behavior of structures in the supercritical range (including *snap through* and *snap back*) can be analyzed. If, within the maximum allowable iteration steps per arc length, the error is larger than the given tolerance, a solution for a smaller arc length is determined. The end of the calculation is controlled either by the number of increments (arc lengths, iteration cycles), the 1st bifurcation point, or a load factor to be reached. It is possible to increase the load within a load case only for selected loads. For this purpose, an arbitrary load-time history must be assigned to these loads. All other loads are constant. The shape of the load-time function is irrelevant.

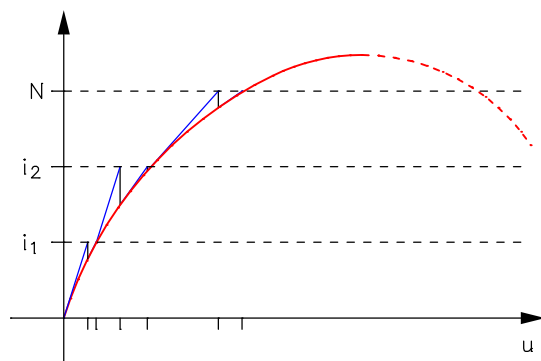
The following figures illustrate the different iteration progressions using the example of a beam with one degree of freedom.



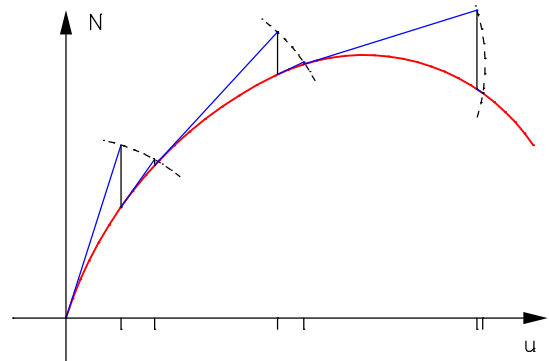
Newton method with tangential stiffness



Newton method with constant stiffness



Newton method with time steps and tangential stiffness



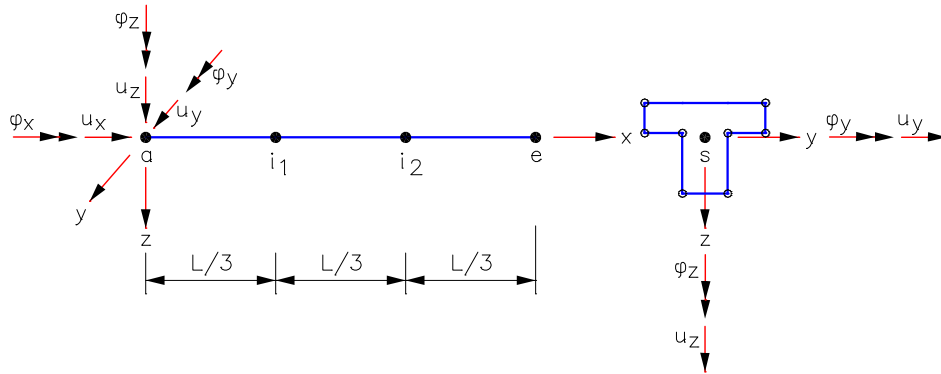
Arc length method with tangential stiffness

When performing a check on the entire system according to OENORM B4700 (Section 3.4.3.3), you need to keep in mind that the decisive action combination may only amount to 80% of the reachable system limit load. To account for this, you can, for example, introduce an additional load factor of 1.25.

For reinforced concrete components the existing reinforcement forms the basis of the calculation. If desired, an automatic reinforcement increase can be carried out for the load bearing capacity check to achieve the required load level.

## Finite Beam Elements

A shear-flexible beam element with 4 nodes is used. Both inner nodes are hidden to the user. Each node has the degrees of freedom  $u_x, u_y, u_z, \varphi_x, \varphi_y$  and  $\varphi_z$ .



Beam element with node degrees of freedom

Because independent deformation functions are used for the displacement and rotation of shear-flexible elements, all deformations of the given beam element are approximated by third-order polynomials.

This allows an equivalent representation of the strains and curvatures, which is especially advantageous for determining the physical nonlinearities. All internal forces are described by second-order polynomials (quadratic parabolas).

In connection to shell elements equivalent shear fixed beam elements are used to provide compatibility.

For linear calculation and constant element load in the longitudinal or lateral direction the results are precise. Other loads, such as point loads in the element or trapezoidal loads in the element, can only be approximated. The quality of the results depends on the chosen structure discretization.

### Kinematics

$$u_x(x, y, z) = u_x(x) - y \cdot \varphi_z(x) + z \cdot \varphi_y(x)$$

$$u_y(x, y, z) = u_y(x) - z \cdot \varphi_x(x)$$

$$u_z(x, y, z) = u_z(x) - y \cdot \varphi_x(x)$$

### Distortion-displacement relationships

$$\epsilon_x = \frac{\partial u_x}{\partial x} + \frac{1}{2} \left[ \left( \frac{\partial u_y}{\partial x} \right)^2 + \left( \frac{\partial u_z}{\partial x} \right)^2 \right]$$

$$\gamma_{xy} = \frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x}$$

$$\gamma_{xz} = \frac{\partial u_z}{\partial x} + \frac{\partial u_x}{\partial z}$$

$$\vartheta' = \frac{\partial \varphi_x}{\partial x}$$

$$\epsilon_y = \epsilon_z = \gamma_{yz} = 0$$

### Virtual internal work

$$\Psi(a) = \int_V \delta \epsilon \cdot \sigma \, dV + \int_V \delta \gamma \cdot \tau \, dV + \int_L \delta \vartheta' \cdot M_x \, dx$$

Normal force    Shear    Torsion  
Bending

$a$     Nodal degrees of freedom

**Tangential stiffness matrix**

$$\frac{d\Psi}{da} = K_T = \int_V \left( dB^T \cdot \sigma + B^T \cdot \frac{d\sigma}{d\varepsilon} \right) dV + K_{\text{shear}} + K_{\text{torsion}}$$

$$= K_0 + K_G + K_L + K_{\text{shear}} + K_{\text{torsion}}$$

with  $d\varepsilon = B \cdot da$  and  $B = B_0 + B_L$

If necessary, terms from the bedding of the elements can be added.

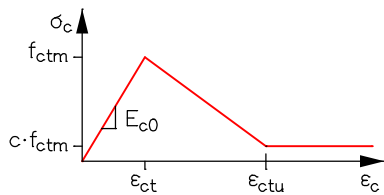
**Section Analysis**

In general, a nonlinear analysis can only be performed for polygon sections, database sections and steel sections. For all other section types and for the material types *Beton* and *Timber* an elastic material behavior is assumed. In order to determine the nonlinear stiffnesses and internal forces, you need to perform a numeric integration of the stresses and their derivatives across the section area. The procedure differs depending on the material.

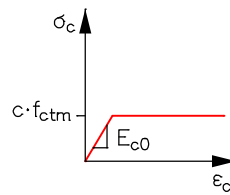
**Reinforced Concrete Beams**

Based on the stress-strain curves shown further below, the resulting internal forces  $N_x$ ,  $M_y$  and  $M_z$  at the section can be determined for a known strain state by integrating the normal stresses. For determining the strain and stress states at the section the following assumptions are made:

- The sections remain flat at every point in time during the deformation, even when the section is cracked as a result of exceeding the concrete tensile strength.
- The concrete and reinforcement are perfectly bonded.
- The tensile strength of the concrete is usually ignored, but it can be taken into account if desired.
- The tensile section of the stress-strain curves can be described using one of the two following curves:



With softening



Bilinear

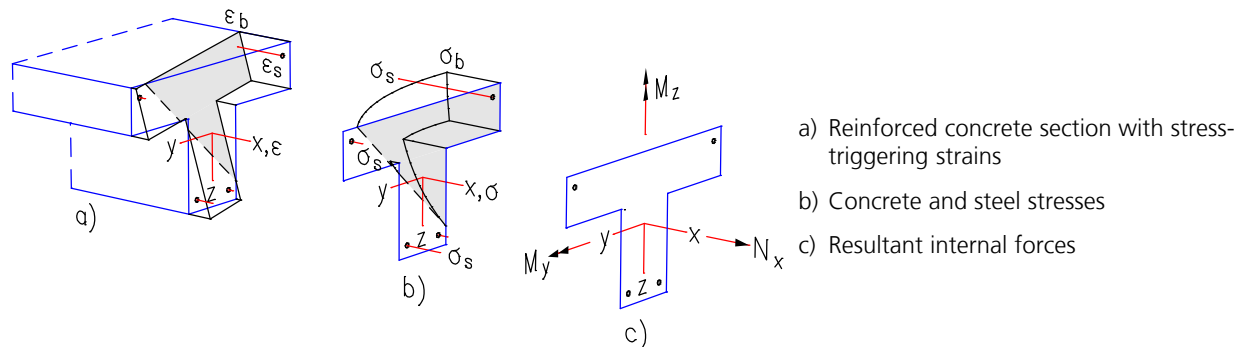
$\epsilon_{ct}$  Strain when reaching the concrete tensile strength

$\epsilon_{ctu}$  Strain after exceeding the specific fracture energy  $G_f$

$$G_f = G_{f0} \cdot \left( \frac{f_{cm}}{f_{cmo}} \right)^{0.7} \quad G_{f0} \approx 3.0 \cdot 10^{-5} \frac{\text{MNm}}{\text{m}^2} \quad f_{cmo} = 10 \frac{\text{MN}}{\text{m}^2}$$

$f_{cm}$  Average concrete compressive strength

- The tension stiffening effect of the concrete is accounted for by the residual tensile stress  $\sigma = c \cdot f_{ctm}$  or  $c \cdot \beta_{bz}$ . The constant  $c$  can be set by the user (default: 0.1).



- a) Reinforced concrete section with stress-triggering strains
- b) Concrete and steel stresses
- c) Resultant internal forces

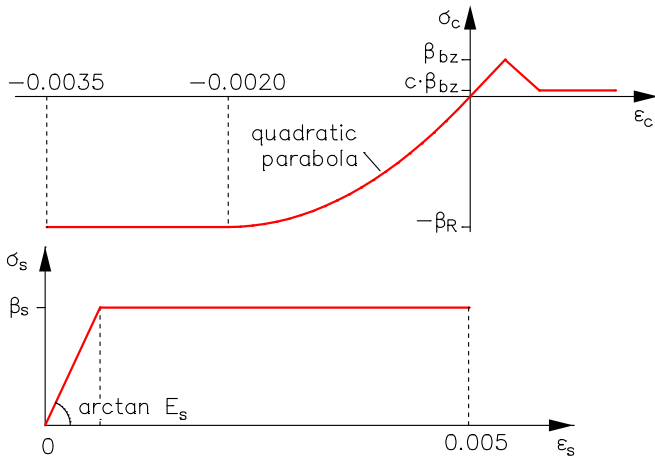
Depending on the chosen material type, the following stress-strain curves are applied. The tensile section of the concrete is only activated as an option.

### Stress-Strain-Curves for the Ultimate Limit State Check

The following material partial safety factors are implemented. For Concretes according to EN 1992-1-1 or OENORM B 4700 the user-defined material safety factors for the permanent and temporary design situation or fundamental combination are used.

The stress-strain curves of prestressing steel have a rising branch with  $E_p$  and a horizontal upper branch at  $f_{pd}$ . Depending on the concrete, the following strengths are assumed for  $f_{pd}$ :

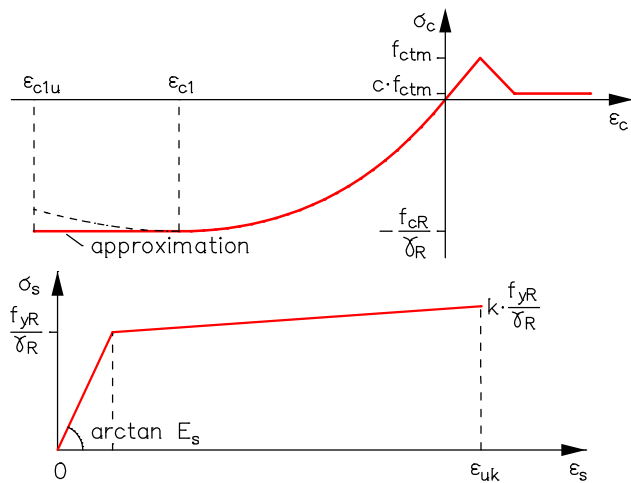
- Concrete DIN 1045:1988 :  $f_{pd} = \beta_s$
- Concrete DIN EN 1992-1-1, DIN 1045-1 :  $f_{pd} = 1,1 \cdot f_{p0,1k} / \gamma_R$  mit  $\gamma_R = 1.3$
- Concrete EN 1992-1-1, ÖNORM B 4700, SIA 262 :  $f_{pd} = f_{p0,1k} / \gamma_s$  mit  $\gamma_s = 1.15$



Concrete acc. to DIN 1045:1988, Figure 11  
 $\beta_R$  acc. to Table 12  
 $\sigma_c = -\beta_R \cdot (\epsilon_b - 1/4 \cdot \epsilon_b^2)$  for  $0 \geq \epsilon_b \geq -2 \%$

Reinforcing steel acc. to DIN 1045:1988, Figure 12  
 $E_s = 210000 \text{ MN/m}^2$  acc. to Figure 12

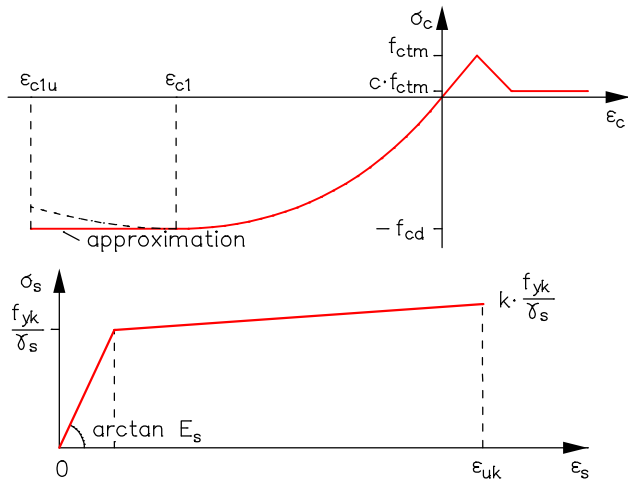
Stress-strain curves according to DIN 1045



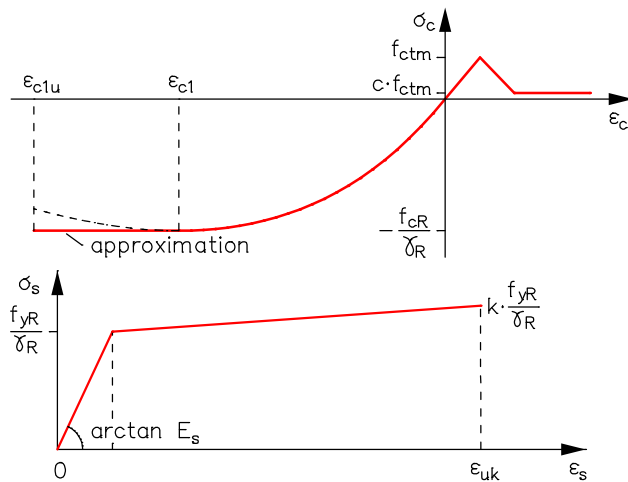
Concrete acc. to DIN 1045-1, 9.1.5  
 $\sigma_c$  acc. to Equation (62) or Figure 22 for  $0 \geq \epsilon_c \geq \epsilon_{c1}$   
 with  $f_{cR} / \gamma_R$  instead of  $f_c$ ;  $\gamma_R = 1.3$   
 $E_{c0m}$  or  $E_{lc0m}$  instead of  $E_{c0}$   
 $f_{cR}$  acc. to Eq. (23) or Eq. (24)  
 Normal concrete:  $\alpha = 0.85$   
 Lightweight concrete:  $f_{lck}$  instead of  $f_{ck}$  and  $\alpha = 0.75$

Reinforcing steel acc. to DIN 1045-1, 9.2.3 Figure 26  
 $f_{yR} = 1,1 \cdot f_{yk}$ ;  $\gamma_R = 1.3$  acc. to 8.5.1 (4)  
 $k = 1.05$ ;  $\epsilon_{uk} = 2.5\%$  acc. to Table 11, Column 1 or 2  
 $E_s = 200000 \text{ MN/m}^2$  acc. to 9.2.4 (4)

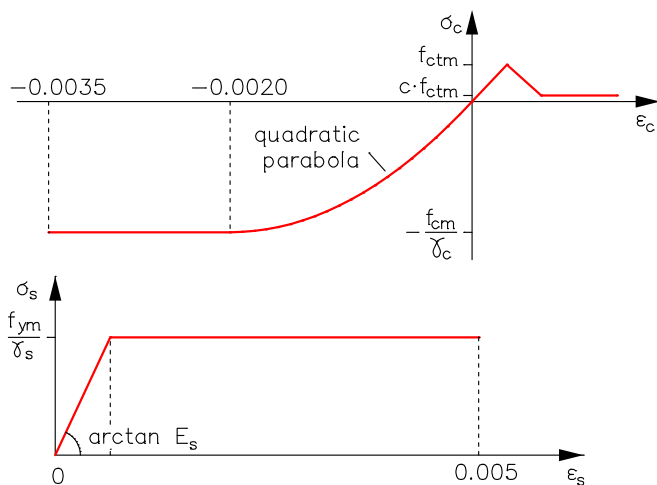
Stress-strain curves according to DIN 1045-1



Stress-strain curves according to EN 1992-1-1



Stress-strain curves according to DIN EN 1992-1-1



Stress-strain curves according to OENORM B 4700

Concrete acc. to EN 1992-1-1, 3.1.5  
 $\sigma_c$  acc. to Equation (3.14) for  $0 \geq \epsilon_c \geq \epsilon_{c1}$  or  $\epsilon_{lc1}$   
 with  $f_{cd}$  or  $f_{lcd}$  instead of  $f_{cm}$  and acc. Eq. (5.20)  
 $E_{cd} = E_{cm}/\gamma_c$  instead of  $E_{cm}$   
 with  $\gamma_{cE} = 1.2$  and  $E_{cm}$  as specified  
 Normal concrete:  $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c$  (Eq. 3.15) with  $\alpha_{cc} = 1.0$   
 Lightweight concrete:  $f_{lcd} = \alpha_{lcc} \cdot f_{lck}/\gamma_c$  (Eq. 11.3.15) with  $\alpha_{lcc} = 0.85$   
 $\gamma_c$  as specified (Table 2.1N, Row 1,  $\gamma_c = 1.5$ )

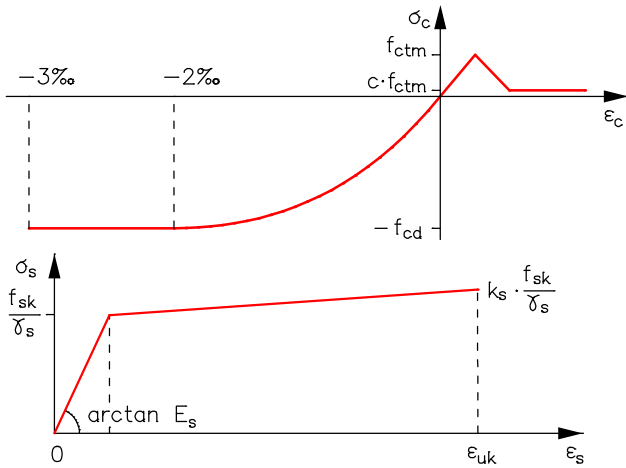
Reinforcing steel acc. to EN 1992-1-1, 3.2.7 Figure 3.8  
 $k = 1.05$ ;  $\epsilon_{uk} = 2.5\%$  acc. to Table C.1, Column A  
 $E_s = 200000 \text{ MN/m}^2$  acc. to 3.2.7 (4)  
 $\gamma_s = 1.15$  acc. to Table 2.1N, Row 1

Concrete acc. to DIN EN 1992-1-1, 3.1.5  
 $\sigma_c$  acc. to Equation (3.14) for  $0 \geq \epsilon_c \geq \epsilon_{c1}$  or  $\epsilon_{lc1}$   
 with  $f_{cR}/\gamma_R$  instead of  $f_{cm}$  and  $E_{cm}/\gamma_R$  instead of  $E_{cm}$   
 with  $\gamma_R = 1.3$  and  $E_{cm}$  as specified  
 $f_{cR}$  acc. to Eq. (NA.5.12.7)  
 Normal concrete:  $f_{cR} = 0.85 \cdot \alpha_{cc} \cdot f_{ck}$  with  $\alpha_{cc} = 0.85$   
 Lightweight concrete:  $f_{cR} = 0.85 \cdot \alpha_{lcc} \cdot f_{lck}$  with  $\alpha_{lcc} = 0.75$

Reinforcing steel acc. to DIN EN 1992-1-1, 3.2.7 Figure 3.8  
 $f_{yR} = 1.1 \cdot f_{yk}$  acc. to Eq. (NA.5.12.2);  $\gamma_R = 1.3$   
 $k = 1.05$ ;  $\epsilon_{uk} = 2.5\%$  acc. to Eq. (NA.5.12.4)  
 $E_s = 200000 \text{ MN/m}^2$  acc. to 3.2.7 (4)

Concrete acc. to OENORM B 4700, 3.4.1.1 Figure 7  
 with  $f_{cm} = f_{ck} + 7.5 \text{ (N/mm}^2\text{)}$  acc. to Equation (10c) and  
 $f_{ck} = 0.75 \cdot f_{cwk}$  acc. to 3.4.1.1 (2)  
 $\gamma_c$  as specified (Table 1, Row 1,  $\gamma_c = 1.5$ )

Reinforcing steel acc. to OENORM B 4700, 3.4.1.2 Figure 9  
 with  $f_{ym} = f_{yk} + 10 \text{ (N/mm}^2\text{)}$  acc. to Equation (10d)  
 $E_s = 200000 \text{ MN/m}^2$  acc. to Figure 9  
 $\gamma_s = 1.15$  acc. to Table 1, Row 1



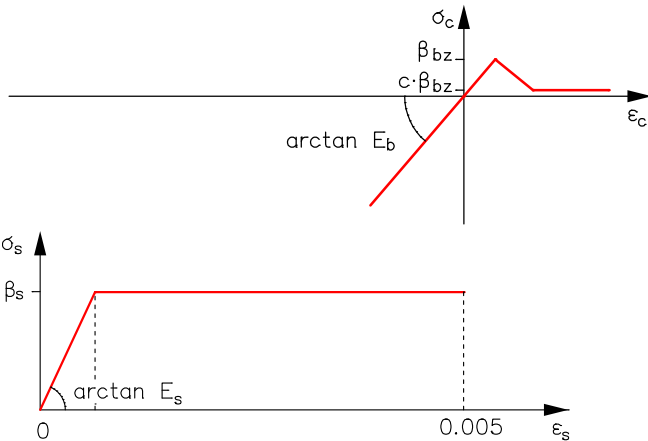
Stress-strain curves according to SIA 262

Concrete acc. to SIA 262, 4.2.1.6 and Figure 12  
 $\sigma_c$  acc. to Equation (28) for  $0 \geq \epsilon_c \geq -2 \text{ ‰}$  with  
 $f_{cd}$  acc. to Eq. (2) and Eq. (26);  $\gamma_c = 1.5$  acc. to 2.3.2.6  
 $E_{cd} = E_{cm} / \gamma_{cE}$  acc. to Eq. (33)  
 $E_{cm}$  acc. to Eq. (10) and Eq. (11)  
 $\gamma_{cE} = 1.2$  acc. to 4.2.1.17

Reinforcing steel acc. to SIA 262, 4.2.2.2, Figure 16  
 with  $f_{sd} = f_{sk} / \gamma_s$  acc. to Eq. (4);  $\gamma_s = 1.15$  acc. to 2.3.2.6  
 $k_s = 1.05$  and  $\epsilon_{uk} = 2 \text{ ‰}$  acc. to Table 9, Column A  
 $E_s = 205000 \text{ MN/m}^2$  acc. to Figure 16

### Stress-Strain-Curves for the Serviceability Check

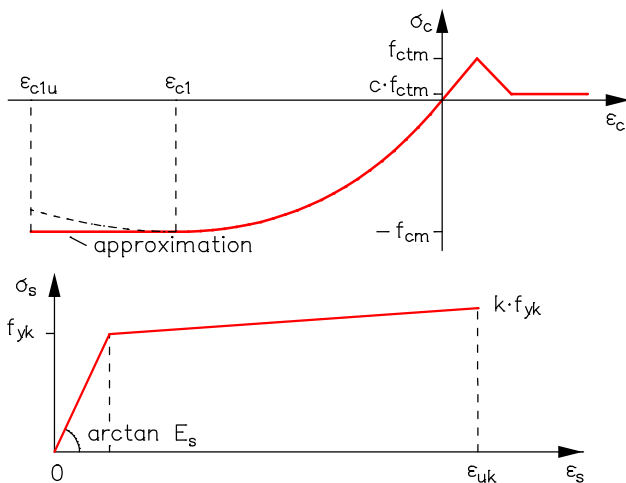
The serviceability check is based on the average strengths of the materials. The partial safety factors are assumed to be 1.0. The stress-strain curves of prestressing steel have a rising branch with  $E_p$  and a horizontal upper branch at  $f_{p0,1k}$  or  $\beta_s$ .



Stress-strain curves according to DIN 1045

Concrete with linear stiffness in the compression zone without failure limit

Reinforcing steel acc. to DIN 1045:1988, Figure 12  
 $E_s = 210000 \text{ MN/m}^2$  acc. to 6.2.1

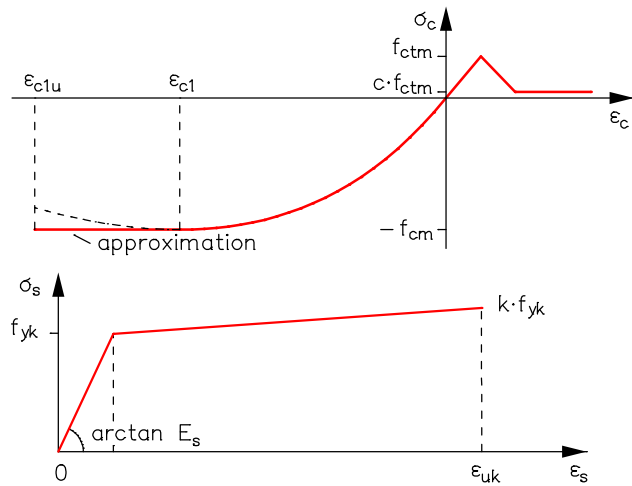


Stress-strain curves according to DIN 1045-1

Concrete acc. to DIN 1045-1, 9.1.5  
 $\sigma_c$  acc. to Equation (62) or Figure 22 for  $0 \geq \epsilon_c \geq \epsilon_{c1}$   
 with  $f_{cm} = f_{ck} + 8 \text{ (N/mm}^2\text{)}$  or  $f_{lcm} = f_{lck} + 8 \text{ (N/mm}^2\text{)}$   
 instead of  $f_c$  and  $E_{c0m}$  or  $E_{lc0m}$  instead of  $E_{c0}$

Reinforcing steel acc. to DIN 1045-1, 9.2.3 Figure 26  
 $k = 1.05$ ;  $\epsilon_{uk} = 2.5 \text{ ‰}$  acc. to Table 11, Column 1 or 2  
 $E_s = 200.000 \text{ MN/m}^2$  acc. to 9.2.4 (4)

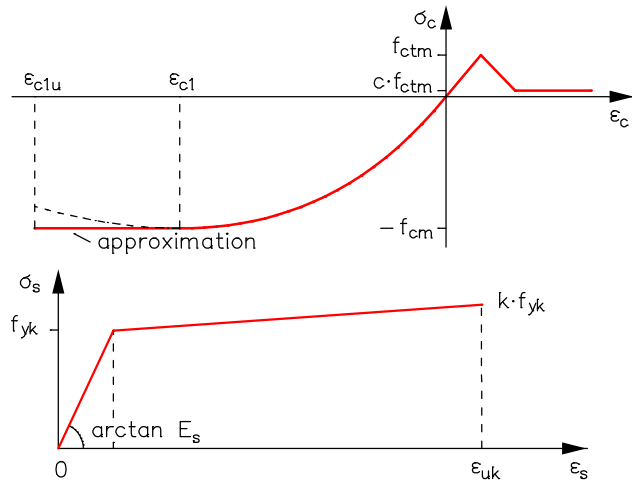




Stress-strain curves according to EN 1992-1-1

Concrete acc. to EN 1992-1-1, 3.1.5  
 $\sigma_c$  acc. to Equation (3.14) for  $0 \geq \epsilon_c \geq \epsilon_{c1}$  or  $\epsilon_{lc1}$   
 with  $f_{cm} = f_{ck} + 8$  (N/mm<sup>2</sup>) acc. to Table 3.1  
 or  $f_{lcm}$  acc. to Table 11.3.1  
 and  $E_{cm}$  or  $E_{lcm}$  as specified

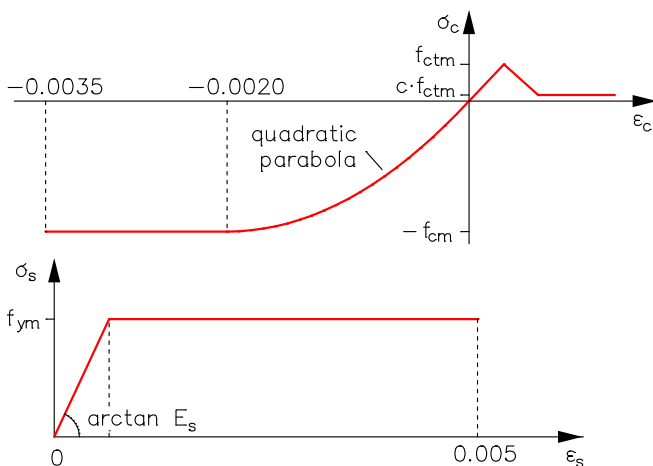
Reinforcing steel acc. to EN 1992-1-1, 3.2.7 Figure 3.8  
 $k = 1.05$  ;  $\epsilon_{uk} = 2.5\%$  acc. to Table C.1, Column A  
 $E_s = 200000$  MN/m<sup>2</sup> acc. to 3.2.7 (4)



Stress-strain curves according to DIN EN 1992-1-1

Concrete acc. to DIN EN 1992-1-1, 3.1.5  
 $\sigma_c$  acc. to Equation (3.14) for  $0 \geq \epsilon_c \geq \epsilon_{c1}$  or  $\epsilon_{lc1}$   
 with  $f_{cm} = f_{ck} + 8$  (N/mm<sup>2</sup>) acc. to Table 3.1  
 or  $f_{lcm}$  acc. to Table 11.3.1  
 and  $E_{cm}$  or  $E_{lcm}$  as specified

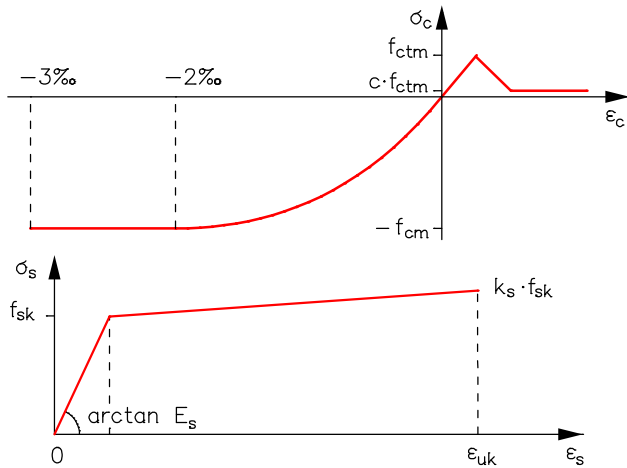
Reinforcing steel acc. to DIN EN 1992-1-1, 3.2.7 Figure 3.8  
 $k = 1.05$  ;  $\epsilon_{uk} = 2.5\%$   
 $E_s = 200000$  MN/m<sup>2</sup>



Stress-strain curves according to OENORM B 4700

Concrete acc. to OENORM B 4700, 3.4.1.1 Figure 7  
 with  $f_{cm} = f_{ck} + 7.5$  (N/mm<sup>2</sup>) acc. to Equation (10c) and  
 $f_{ck} = 0.75 \cdot f_{cwk}$  acc. to 3.4.1.1 (2)

Reinforcing steel acc. to OENORM B 4700, 3.4.1.2 Figure 9  
 with  $f_{ym} = f_{yk} + 10$  (N/mm<sup>2</sup>) acc. to Equation (10d)  
 $E_s = 200000$  MN/m<sup>2</sup> acc. to Figure 9



Stress-strain curves according to SIA 262

Concrete acc. to SIA 262, 4.2.1.6 and Figure 12  
 $\sigma_c$  acc. to Equation (28) for  $0 \geq \epsilon_c \geq -2 \text{ ‰}$  with  
 $f_{cm} = f_{ck} + 8 \text{ (N/mm}^2\text{)}$  acc. to Eq. (6) instead of  $f_{cd}$   
 $E_{cm}$  acc. to Eq. (10) and Eq. (11) instead of  $E_{cd}$

Reinforcing steel acc. to SIA 262, 4.2.2.2, Figure 16  
 with  $f_{sk}$  instead of  $f_{sd}$   
 $k_s = 1.05$  and  $\epsilon_{uk} = 2 \text{ ‰}$  acc. to Table 9, Column A  
 $E_s = 205000 \text{ MN/m}^2$  acc. to Figure 16

### Torsional Stiffness

When calculating the torsional stiffness of the section, it is assumed that it decreases at the same rate as the bending stiffness.

$$GI_{x,II} = \frac{1}{2} \cdot GI_{x,I} \cdot \left( \frac{EI_{y,II}}{EI_{y,I}} + \frac{EI_{z,II}}{EI_{z,I}} \right)$$

The opposite case, meaning a reduction of the stiffnesses as a result of torsional load, cannot be analyzed. The physical nonlinear analysis of a purely torsional load is also not possible for reinforced concrete.

### Check of the Limit Strains (Ultimat Limit State Check)

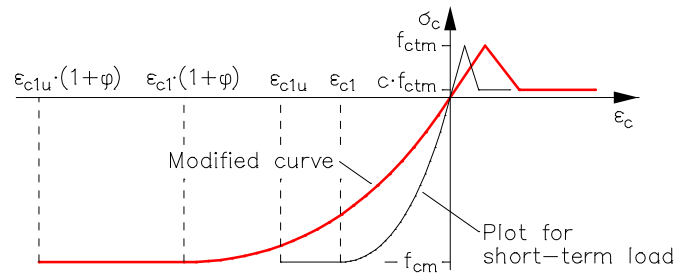
After completion of the equilibrium iteration the permissible limit strains for concrete and reinforcing steel are checked in the case of structures consisting only of beam elements, if the *Newton* analysis method is used. If the permissible values are exceeded the reinforcement will be increased or the load-bearing capacity will be reduced.

### Automatic Reinforcement Increase (Ultimate Limit State Check)

If requested an automatic increase of reinforcement can be made for structures consisting only of beam elements with the *Newton* analysis method to reach the full load-bearing capacity. Initially based on the desired start reinforcement the reachable load level is determined. If the reached load-bearing capacity considering the limit strains is lower then 100%, the reinforcement will be increased and the iteration starts again. If no load-bearing capacity exists because of insufficient start reinforcement a base reinforcement can be specified within the reinforcing steel definition and the design can be performed again.

## Concrete Creep

The consideration of concrete creep in nonlinear analysis is realized by modifying the underlying stress-strain curves. These are scaled in strain-direction with the factor  $(1+\phi)$ . Also the corresponding limit strains are multiplied by  $(1+\phi)$ . The following figure shows the qualitative approach.



The described procedure postulates that the creep-generating stresses remain constant during the entire creep-period. Due to stress redistributions, for example to the reinforcement, this can not be guaranteed. Therefore an insignificant overestimation of the creep-deformations can occur.

Stress relaxation during a constant strain state can also be modelled by the described procedure. It leads, however, to an overestimation of the remaining stresses due to the non-linear stress-strain curves.

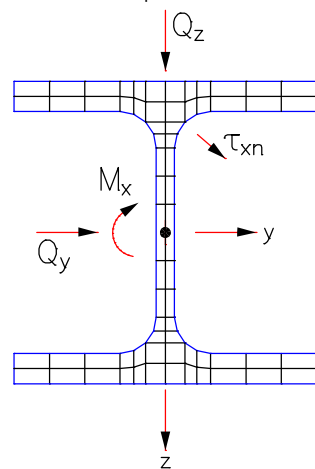
The nonlinear concrete creep can be activated in the 'Load group' dialog.

This analysis approach applies analogous for area elements.

The plausibility of the achievable calculation results as well as the mode of action of different approaches for the concrete tensile strength and the *tension stiffening* are demonstrated by some examples in the 'Examples' section.

## Steel Beams

The section geometry is determined by the polygon boundary. In order to perform the section analysis, the program internally generates a mesh. The implemented algorithm delivers all section properties, stresses and internal forces on the basis of the quasi-harmonic differential equation and others.



Steel cross-section with internal mesh

The shear load is described by the following boundary value problem:

$$\frac{\partial^2 \varphi}{\partial y^2} + \frac{\partial^2 \varphi}{\partial z^2} = 0 \quad \text{(Differential equation)}$$

$$\tau_{xn} = 0 \quad \text{(Boundary condition)}$$

$$\sqrt{\sigma_x^2 + 3\tau_{xy}^2 + 3\tau_{xz}^2} - \sigma_{R,d} = 0 \quad \text{(Yield criterion)}$$

The differential equation holds equally for  $Q_y$ ,  $Q_z$  and  $M_x$ . Depending on the load, the boundary condition delivers different boundary values of the solution function  $\varphi$ . The solution is arrived at by integral representation of the boundary value problem and discretization through finite differential equation elements. The internal forces are determined by numerical integration of the stresses across the section under consideration of the Huber-von Mises yield criterion. The interaction of all internal forces can be considered.

At the ultimate limit state, the material safety factor  $\gamma_M$  is taken into account as follows:

- Construction steel according to DIN 18800 and the general material type *Stahl*:  
The factor  $\gamma_M$  of the fundamental combination according to DIN 18800 is decisive.
- Construction steel according to EN 10025-2:  
In accordance with EN 1993-1-1, Chapter 6.1 (1),  $\gamma_M$  is assumed to be  $\gamma_{M0} = 1.0$ .

The serviceability check is generally calculated with a material safety factor of  $\gamma_M = 1.0$ .

On the basis of DIN 18800, the interaction between normal and shear stresses is considered only when one of the following three conditions is met:

$$Q_y \geq 0.25 \cdot Q_{pl,y,d}$$

$$Q_z \geq 0.33 \cdot Q_{pl,z,d}$$

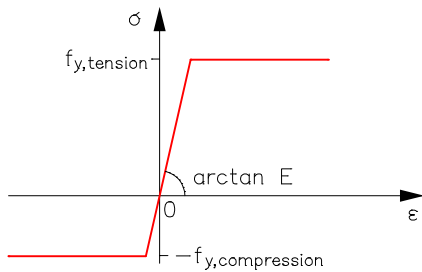
$$M_x \geq 0.20 \cdot M_{pl,x,d}$$

Limiting the bending moments  $M_{pl,y,d}$  and  $M_{pl,z,d}$  according to DIN 18800 to 1.25 times the corresponding elastic limit moment does not occur.

Stability failure due to cross-section buckling or lateral torsional buckling is not taken into account.

### Beams of Free Material

For polygon sections of the general material type *Frei*, the following bilinear stress-strain curve with user-defined tensile strength ( $f_{y,tension}$ ) and compressive strength ( $f_{y,compression}$ ) is used.



Stress-strain curve for the material type *Frei*

Due to the differing strengths in the compressive and tensile section, the Huber-von Mises yield criterion is not used here. The interaction of normal and shear stresses is also not accounted for.

### Area Elements

So-called layer elements are used to enable the integration of nonlinear stresses across the area section. This is done by determining the stresses in each layer according to the plain stress theory, two-dimensional stress state, under consideration of the physical nonlinearities. The integration of the stresses to internal forces is carried out with the help of the trapezoidal rule.

$$n_x = t \cdot \left( \sigma_{x0} + \sigma_{xn} + 2 \cdot \sum_{i=1}^{n-1} \sigma_{xi} \right)$$

$$n_y = t \cdot \left( \sigma_{y0} + \sigma_{yn} + 2 \cdot \sum_{i=1}^{n-1} \sigma_{yi} \right)$$

$$n_{xy} = t \cdot \left( \tau_{xy0} + \tau_{xyn} + 2 \cdot \sum_{i=1}^{n-1} \tau_{xyi} \right)$$

$$m_x = t \cdot \left( \sigma_{x0} \cdot z_0 + \sigma_{xn} \cdot z_n + 2 \cdot \sum_{i=1}^{n-1} \sigma_{xi} \cdot z_i \right)$$

$$m_y = t \cdot \left( \sigma_{y0} \cdot z_0 + \sigma_{yn} \cdot z_n + 2 \cdot \sum_{i=1}^{n-1} \sigma_{yi} \cdot z_i \right)$$

$$m_{xy} = t \cdot \left( \tau_{xy0} \cdot z_0 + \tau_{xyn} \cdot z_n + 2 \cdot \sum_{i=1}^{n-1} \tau_{xyi} \cdot z_i \right)$$

## Reinforced Concrete Area Elements

The biaxial concrete behavior is realized with the help of the concept of equivalent one-axial strains (Finite Elemente im Stahlbetonbau (Finite Elements in Reinforced Concrete Construction), Stempniewski and Eibl, Betonkalender 1993). The existing strain state is first transformed into the principal direction in each corresponding layer and then reformed to the equivalent relationship of the form

$$\begin{aligned} \begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} = 0 \end{Bmatrix} &= \frac{1}{1-\nu^2} \begin{Bmatrix} E_1 & \nu \cdot \sqrt{E_1 \cdot E_2} & 0 \\ \nu \cdot \sqrt{E_1 \cdot E_2} & E_2 & 0 \\ 0 & 0 & (1-\nu^2) \cdot G \end{Bmatrix} \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{12} = 0 \end{Bmatrix} & \text{with} \\ & \varepsilon_{c,1} = \varepsilon_{11} + \nu \cdot \sqrt{\frac{E_2}{E_1}} \cdot \varepsilon_{22} \\ & \varepsilon_{c,2} = \nu \cdot \sqrt{\frac{E_1}{E_2}} \cdot \varepsilon_{11} + \varepsilon_{22} \\ & G = \frac{1}{4 \cdot (1-\nu^2)} \left[ E_1 + E_2 - 2 \cdot \nu \cdot \sqrt{E_1 \cdot E_2} \right] \end{aligned}$$

$$= \frac{1}{1-\nu^2} \begin{Bmatrix} E_1 & 0 & 0 \\ 0 & E_2 & 0 \\ 0 & 0 & (1-\nu^2) \cdot G \end{Bmatrix} \begin{Bmatrix} \varepsilon_{c,1} \\ \varepsilon_{c,2} \\ \varepsilon_{12} = 0 \end{Bmatrix}$$

The shear modulus  $G$  is considered to be direction independent in this approach. The poisson's ratio  $\nu$  is assumed to be constant.

The required tangential stiffnesses  $E_1$  and  $E_2$  and equivalent biaxial concrete compressive stresses  $\sigma_{c,i}$  are iterated using the Saenz curve

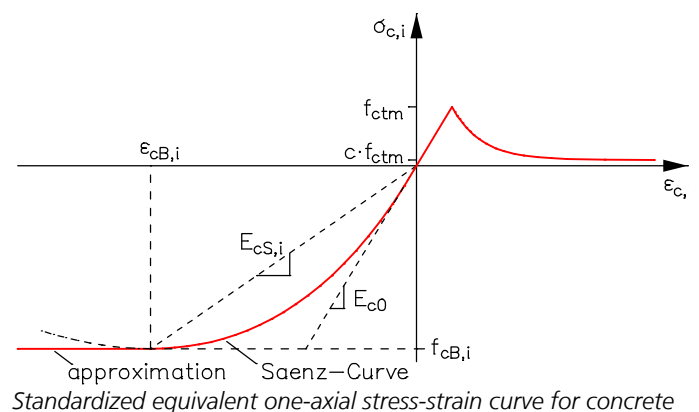
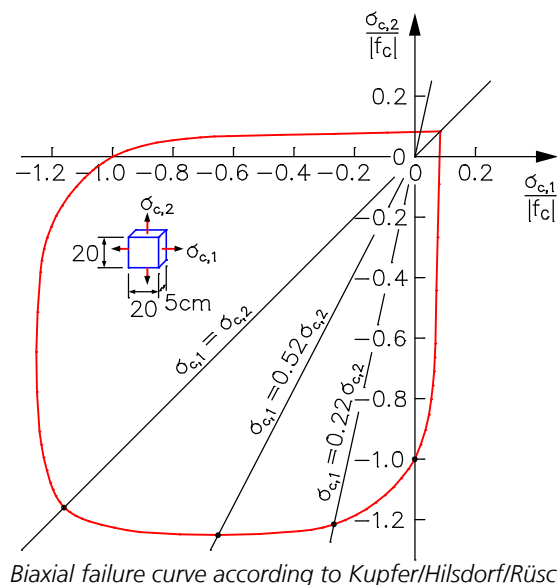
$$\sigma_{c,i} = \frac{E_{c0} \cdot \varepsilon_{c,i}}{1 + \left( \frac{E_{c0}}{E_{cS,i}} - 2 \right) \frac{\varepsilon_{c,i}}{\varepsilon_{cB,i}} + \left( \frac{\varepsilon_{c,i}}{\varepsilon_{cB,i}} \right)^2} \quad \text{with } i = 1, 2 \text{ (principal directions)}$$

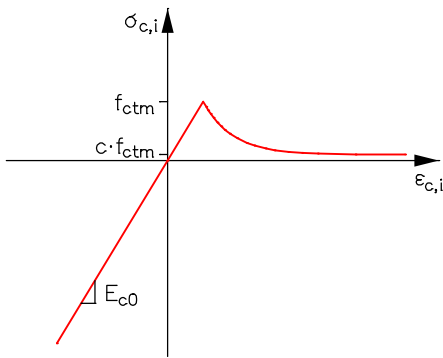
under consideration of the biaxial stiffnesses according to Kupfer/Hilsdorf/Rüsch.

Only when checking the serviceability according to DIN 1045 it is assumed that the compressive area exhibits a linear elastic material behavior.

Parameters for the stress-strain relationship used are:

- Initial stiffness  $E_{c0}$  of the one-axial stress-strain curve of the selected concrete.
- Equivalent biaxial concrete strains  $\varepsilon_{c,i}$
- Factor of the biaxial failure curve  $k_i$  according to Kupfer/Hilsdorf/Rüsch depending on  $\sigma_{c,1} / \sigma_{c,2}$ .
- Secant stiffness  $E_{cS,i} = f_{cB,i} / \varepsilon_{cB,i}$  when the biaxial concrete compressive strength  $f_{cB,i}$  is reached.
- Biaxial concrete compressive strength  $f_{cB,i} = k_i \cdot f_c$
- Concrete compressive strength  $f_c$  of the one-axial stress-strain curve of the selected concrete. The values described in the section *Reinforced concrete beams* are used.
- Compression  $\varepsilon_{cB,i}$  when  $f_{cB,i}$  is reached. The value  $\varepsilon_{cB,i}$  is derived from the function parameters  $k_i$  and  $\varepsilon_{c1}$  according to Darwin and Pecknold.
- Compression  $\varepsilon_{c1}$  when the strength  $f_c$  of the one-axial stress-strain curve of the selected concrete is reached.





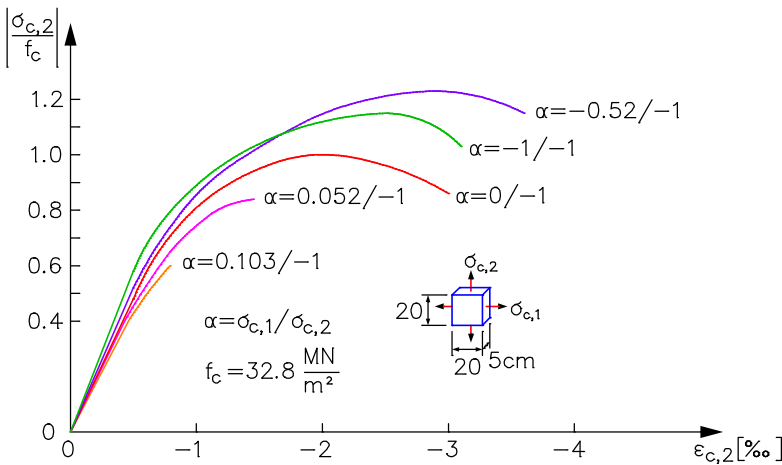
Stress-strain curve for the serviceability check according to DIN 1045

The consideration of the concrete creep occurs analog to the procedure for beams through modification of the underlying stress-strain curves and can be activated in the 'Load group' dialog.

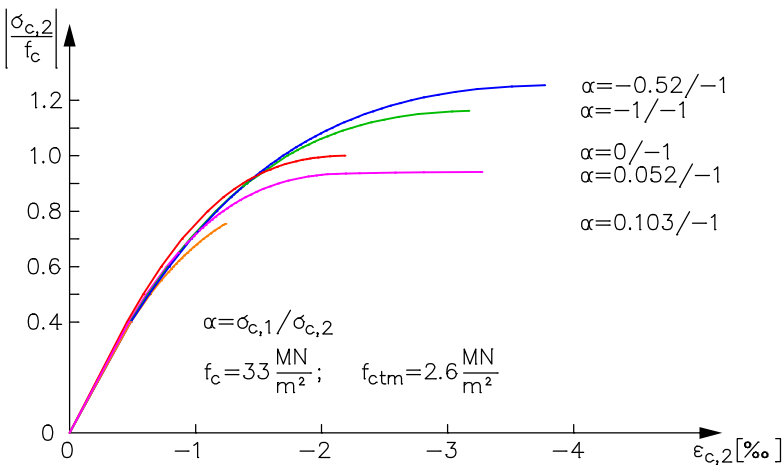
**Interaction with the shear stresses from the lateral forces is not considered here.** During the specification of the crack direction the so-called 'rotating crack model' is assumed, meaning that the crack direction can change during the nonlinear iteration as a function of the strain state. Conversely, a fixed crack direction after the initial crack formation can lead to an overestimation of the load-bearing capacity.

**The Finite Element module currently does not check the limit strain nor does it automatically increase reinforcement to improve the load-bearing capacity due to the significant numerical complexity.**

For clarification of the implemented stress-strain relationships for concrete, the curves measured in the biaxial test according to Kupfer/Hilsdorf/Rüsch are compared with those calculated using the approach described above.

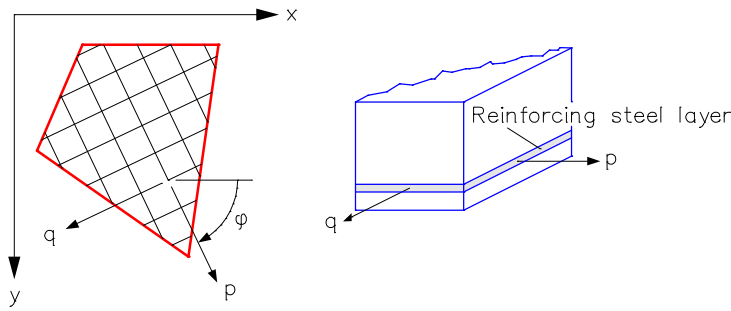


Biaxial test according to Kupfer/Hilsdorf/Rüsch



Calculation results for C25/30;  $f_{cm} = 25 + 8 = 33 \text{ MN/m}^2$

The existing **reinforcing steel** is modeled as a 'blurred' orthogonal reinforcement mesh. The corresponding strain state is transformed into the reinforcement directions (p, q) of the respective reinforcing steel layer. The stress-strain curve of the selected reinforced concrete standard is used to determine the stress.



### Area Elements of Steel and Free Material

For area elements made of steel the Huber v.Mises yield criterion is used. The material safety factor specified in the section *Steel beams* apply accordingly. For area elements of the material type *Frei*, the *Raghava* yield criterion, as described below, or the *Rankine* yield criterion can be chosen.

$$F = J_2 - 1/3 \cdot f_c \cdot f_t + (f_c - f_t) \cdot \sigma_m = 0$$

with

$$\sigma_m = 1/3 (\sigma_x + \sigma_y + \sigma_z)$$

$$J_2 = 1/2 (\sigma_x^2 + \sigma_y^2 + \sigma_z^2) + \tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2$$

$$\sigma_x = \sigma_x - \sigma_m$$

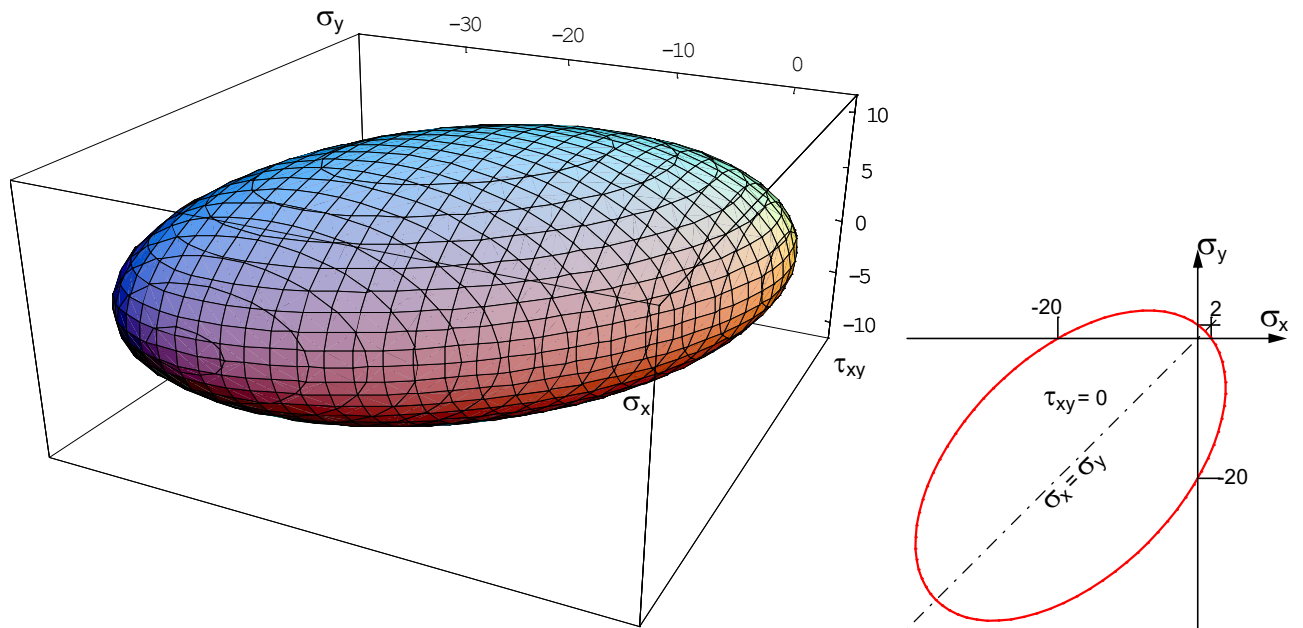
$$\sigma_y = \sigma_y - \sigma_m$$

$$\sigma_z = \sigma_z - \sigma_m$$

Starting from the strain state, a layer is iterated on the yield surface with the help of a 'backward Euler return' that, in conjunction with the Newton-Raphson method, ensures quadratic convergence. (Non-linear Finite Element Analysis of Solids and Structures, M.A. Crisfield, Publisher John Wiley & Sons).

The yield surface mentioned above is illustrated below for the 2D stress state, as used for layer elements, for the principal stresses  $\sigma_1$  and  $\sigma_2$  as well as for the components  $\sigma_x$ ,  $\sigma_y$  and  $\tau_{xy}$ .

In this example  $f_c = 20 \text{ MN/m}^2$  and  $f_t = 2 \text{ MN/m}^2$ .



Yield surface for the 2D stress state according to Raghava

## Solid Elements

A distinction is made between solid elements made of steel, a standardized concrete type and solid elements with the material type *Frei*.

For solid elements made of the first two material types, the yield criterion according to *Raghava* (ideal plastic) is used. For the standardized concrete types, the compressive strength  $f_c$  is set to  $f_{ck}$  in the serviceability and to  $f_{cd}$  in the ultimate limit state. Softening of the material in tension is not currently supported for these material types.

If, on the other hand, a structure is discretized with solid elements of the material type *Frei*, three different model approaches are available to represent the nonlinear material behavior (i.a. with hardening and softening in tension and compression). Depending on the problem to be analyzed, the numerical calculation can be carried out either by using pure damage and plasticity models or a combination of plasticity and damage. Damage surfaces, yield criteria and all parameters associated with the respective model are to be selected or adapted in the dialog for the material properties. An isotropic material behavior is assumed for all models.

In the following, the model approaches (damage, plasticity and combined model) are briefly described. Detailed derivations etc. can be found in a large number of publications

[e.g. *Kachanov* (1958, 1986), *Chaboche* (1981), *Lemaitre & Chaboche* (1990), *Chen & Han* (1988)].

## Damage

The constitutive relations of a isotropic damage model are as follows:

$$\sigma = (1 - D) \cdot C : \varepsilon = (1 - D) \cdot \bar{\sigma}, \quad (C: \text{elasticity tensor, } \varepsilon: \text{strain tensor}) \quad (V1)$$

where  $\bar{\sigma}$  are denoted as effective and  $\sigma$  as nominal stresses. The degradation of the material is determined using the scalar damage parameter  $D(\eta)$ , which is calculated here as a function of the equivalent strain  $\eta$ . In *InfoCAD* only exponential functions are used to represent the loss of stiffness. All functions have in common that the damage parameter takes the value zero ( $D = 0 \rightarrow \sigma = C : \varepsilon$ ) for undamaged material and the value one ( $D = 1 \rightarrow \sigma = 0$ ) for completely damaged material.

*InfoCAD* provides two damage models, the *De Vree et al.* (1995) model and the *Mazars et al.* (2015) model. The models differ both in the shape of the damage surface and in the equation used to calculate the damage parameter. As a result, materials with both identical (*De Vree*) and different (*Mazars*) tension-compression behavior can be modeled. A method for minimizing mesh dependency is not currently considered in either model.

## De Vree

In the *De Vree* model, elastic material behavior is assumed until the maximum tensile ( $f_t$ ) and compressive ( $f_c$ ) stresses are reached. The stress-strain curve after reaching the respective yield stress (softening) is also qualitatively identical. In *InfoCAD*, a combination of equivalent strain according to *De Vree et al.* (1995)

$$\eta = K_1 \cdot I_1 + \sqrt{(K_1 \cdot I_1)^2 + K_2 \cdot J_2}, \quad \text{with: } \begin{aligned} K &= \frac{f_c}{f_t}, & K_1 &= \frac{K-1}{2K \cdot (1-2\nu)}, & K_2 &= \frac{3}{K \cdot (1+\nu)^2} \\ I_1 &= \varepsilon_1 + \varepsilon_2 + \varepsilon_3 & & \text{(1. invariant of the strain tensor)} \\ J_2 &= [(\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2]/6 & & \text{(2. deviatoric invariant)} \end{aligned} \quad (V2)$$

and exponential damage function according to *Peerlings* (1999)

$$D(\eta) = 1 - (1 - A) \cdot \frac{\eta_0}{\eta} - A \cdot \frac{\eta_0}{\eta} \cdot \exp(B \cdot (\eta_0 - \eta)), \quad \text{with: } \eta_0 = \frac{f_t}{E} \quad (V3)$$

is used, where  $D(\eta) = 0$  if  $\eta \leq \eta_0$ . In the latter formulation, the residual tensile or compressive strength is controlled by parameter  $A$  ( $\sigma^{res} = (1 - A) \cdot \sigma_{c,t}$ ) and the damage evolution (slow or fast increase of damage) by parameter  $B$ , see Figure 1.



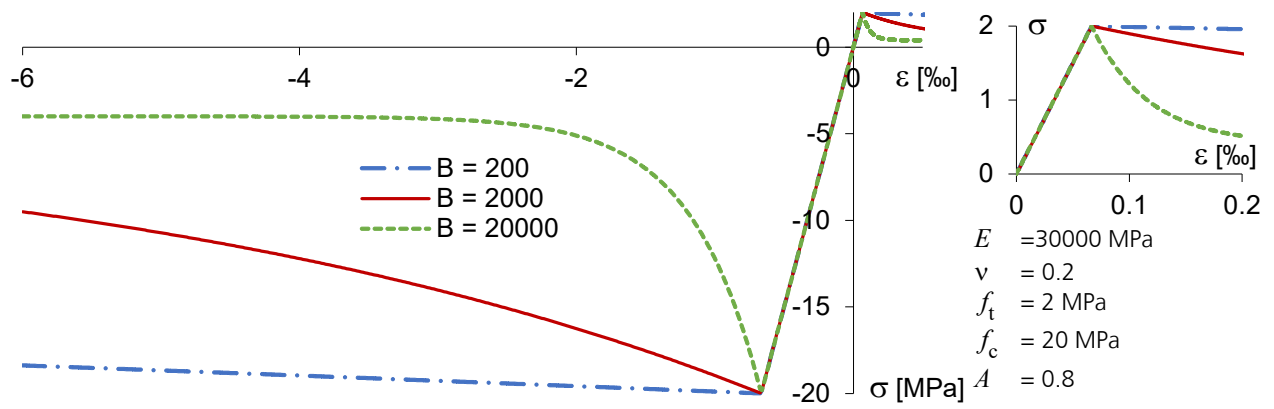


Figure 1: Uniaxial stress-strain diagram (de Vree / Peerlings)

With increasing stiffness degradation, the damage asymptotically approaches the value 1. In addition to the three-dimensional view, Figure 2 also shows the damage surface for the plane stress state ( $\sigma_3=0$ ). Based on this (cf. with Kupfer et al. (1969)) and the uniaxial stress-strain relationship, it becomes clear that the compressive behavior of concrete cannot be adequately represented with this damage model.

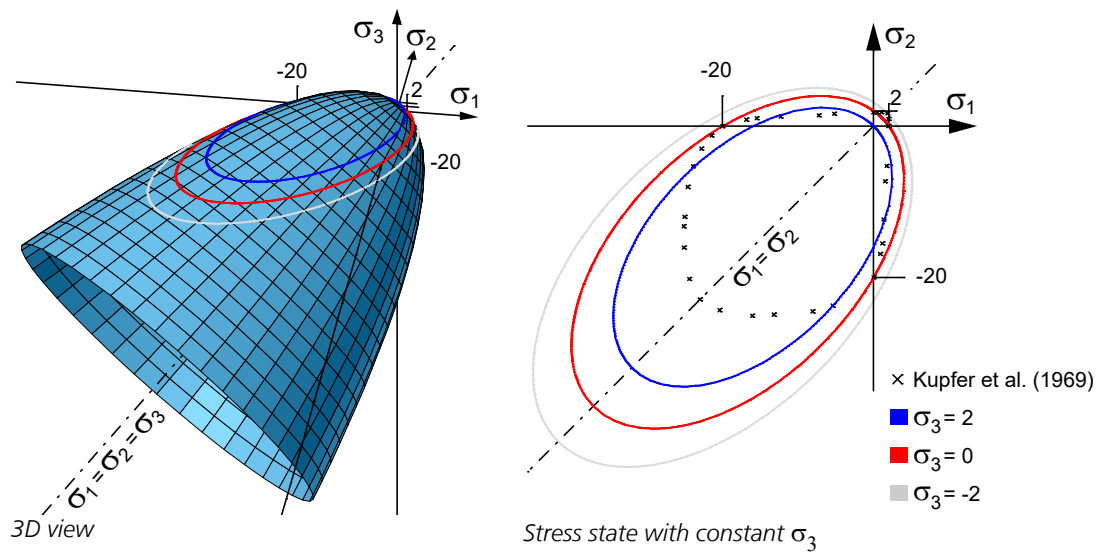


Figure 2: Damage (de Vree) in the principal stress space,  $f_c = 20$  MPa,  $f_t = 2$  MPa

On the one hand, the nonlinear relationship between stresses and strains is not reproduced realistically, this applies both before and after reaching the compressive strength  $f_c$ , and on the other hand, the compressive-compressive failure is significantly overestimated. However, problems with dominant tensile failure can be predicted with sufficient accuracy using this model. The counterpart to De Vree's damage surface is the yield surface according to Raghava (1973) (or Mises-Schleicher (1926)) in the case of plastic material behavior, see Figure 9. In analogy to the yield surface (Raghava  $\rightarrow$  Mises), in the plane stress state ( $\sigma_3=0$ ) the damage surface degenerates to an ellipse symmetrical to the stress origin for identical values of tensile and compressive strength ( $f_t = f_c \rightarrow K = 1$ , see Eq. V2).

### Mazars' $\mu$ -Model

The second isotropic damage model implemented in InfoCAD is based on the publications of Mazars et al. (2015, 2017). The equivalent strains are determined separately for tension ( $\eta_t$ ) and compression ( $\eta_c$ ):

$$\eta_t = \frac{I_1}{2 \cdot (1-2\nu)} + \frac{\sqrt{J_e}}{2 \cdot (1+\nu)} \quad \text{and} \quad \eta_c = \frac{I_1}{5 \cdot (1-2\nu)} + \frac{6 \cdot \sqrt{J_e}}{5 \cdot (1+\nu)}, \quad \text{with } J_e = [(\epsilon_1 - \epsilon_2)^2 + (\epsilon_2 - \epsilon_3)^2 + (\epsilon_3 - \epsilon_1)^2] / 2 \quad (V4)$$

The triaxial factor  $r$  (Lubliner et al. 1989) is subsequently used to calculate the equivalent total strain corresponding to the respective stress state:

$$\eta = r \cdot \eta_t + (1-r) \cdot \eta_c, \quad \text{with: } r = \frac{\sum_{i=1}^3 \langle \sigma_i \rangle}{\sum_{i=1}^3 |\sigma_i|}. \quad (V5)$$

Depending on the triaxial factor  $r$  and the respective parameters for generating the damage function in tension ( $A_t, B_t$ ) and compression ( $A_c, B_c$ ), a damage related to the three-dimensional stress state is formulated. With the two parameters  $A$  and  $B$

$$A = (2 \cdot r^2 (1 - 2 \cdot k) - r \cdot (1 - 4 \cdot k)) \cdot A_t + (2 \cdot r^2 - 3 \cdot r + 1) \cdot A_c \tag{V6}$$

$$B = r^{r^2 - 2 \cdot r + 2} \cdot B_t + (1 - r^{r^2 - 2 \cdot r + 2}) \cdot B_c \tag{V7}$$

the damage parameter finally yields to:

$$D(\eta) = 1 - (1 - A) \cdot \frac{\eta_0}{\eta} - A \cdot \exp(B \cdot (\eta_0 - \eta)), \quad \text{with: } \eta_0 = r \cdot \eta_{T0} + (1 - r) \cdot \eta_{C0} \tag{V8}$$

Also here applies:  $D(\eta) = 0$  if  $\eta \leq \eta_0$ . The shear parameter  $k$  is set to 0.7 according to the recommendation in Mazars *et al.* (2017). For the two limiting cases  $r = 1$  (pure tension) and  $r = 0$  (pure compression), the result is:  $A = A_t$  and  $B = B_t$  or  $A = A_c$  and  $B = B_c$ , see Eqs. V6 and V7. In contrast to the tensile range, where damage to the material only starts when the maximum tensile stress  $f_t$  is reached, in the compressive range a loss of stiffness already occurs before the compressive strength  $f_c$  is reached.

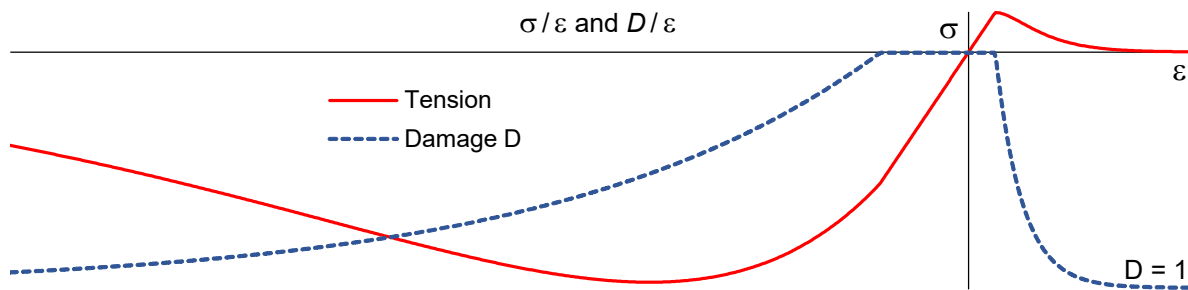


Figure 3: Uniaxial stress-strain and damage-strain diagram (Mazars'  $\mu$ -Model)

This is evident from both the damage surfaces (Figure 4) and the nonlinear stress-strain relationships even before the maximum value is reached (see Figure 3). Thus, in contrast to the *De Vree* model, the compressive behavior of concrete can also be represented satisfactorily. The factor  $\eta_0 / \eta$ , which was not considered by Mazars *et al.* (2015) in the third term of the damage function, is decisive for this, see Eqs. V3 and Gl. V8.

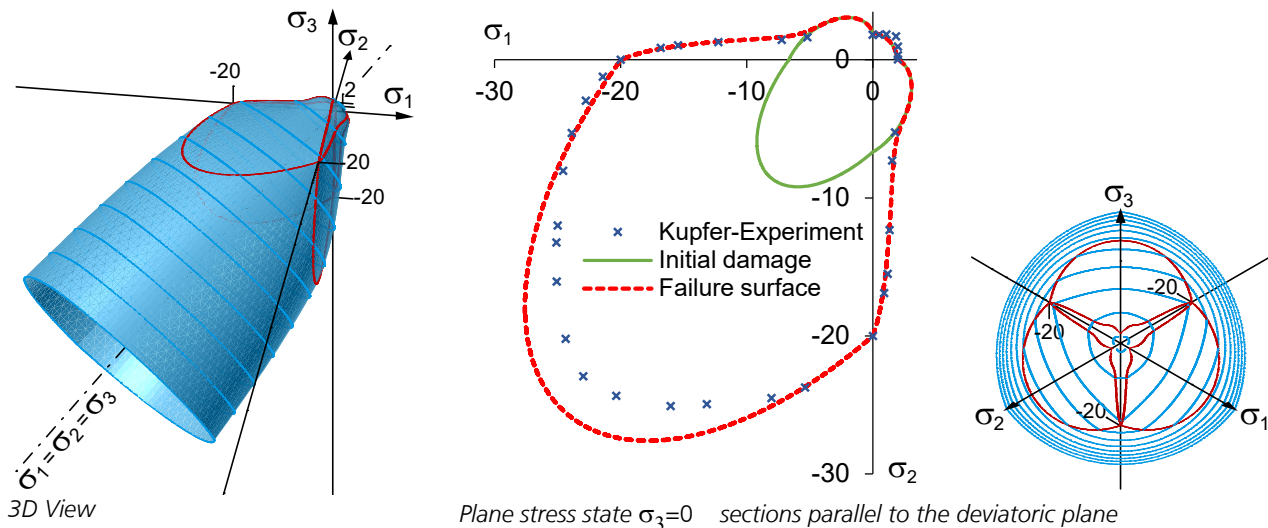


Figure 4: Damage surface (Mazars'  $\mu$ -Model) in the principal stress space,  $f_c = 20$  MPa,  $f_t = 2$  MPa

The respective threshold values  $\eta_{T0}$  (uniaxial tension) and  $\eta_{C0}$  (uniaxial compression) are defined as follows:

$$\eta_{T0} = \frac{f_t}{E} \quad \text{and} \quad \eta_{C0} = \frac{10}{3} \cdot \eta_{T0} \tag{V9}$$

The compressive threshold value is 10/3 times the tensile threshold value (Gafoor & Dinkler 2020). Analogous to the exponential damage function according to Peerlings (Gl. V3), the residual strength is controlled with the parameters  $A_t$  (tension) and  $A_c$  (compression) and the stiffness degradation with the parameters  $B_t$  (tension) and  $B_c$  (compression). However, due to the changed function compared to Peerlings as well as the definition of the compression threshold  $\eta_{C0}$ ,

there are some dependencies that have to be considered when using the  $\mu$ -model. First, since the compression range of the stress-strain relation is determined by the tensile strength  $f_t$ , the residual tensile strength is also defined by it:

$$\sigma_t^{\text{res}} = (1 - A_t) \cdot f_t \quad \text{or} \quad \sigma_c^{\text{res}} = (1 - A_c) \cdot \frac{10}{3} \cdot f_t \quad (\text{V10})$$

Secondly, the parameters  $A$  ( $A_t, A_c$ ) and  $B$  ( $B_t, B_c$ ) not only influence the residual load-carrying capacity and induce a fast or slow increase of the damage (see Figure 1), but also (with the exception of  $A_t$ ) directly influence the maximum value of the compressive or tensile stress ( $f_c$  or  $f_t$ ). This can be seen from the uniaxial relationships shown in Figure 5 ( $A_t$  and  $A_c$  constant) and Figure 6 ( $B_t$  and  $B_c$  constant).

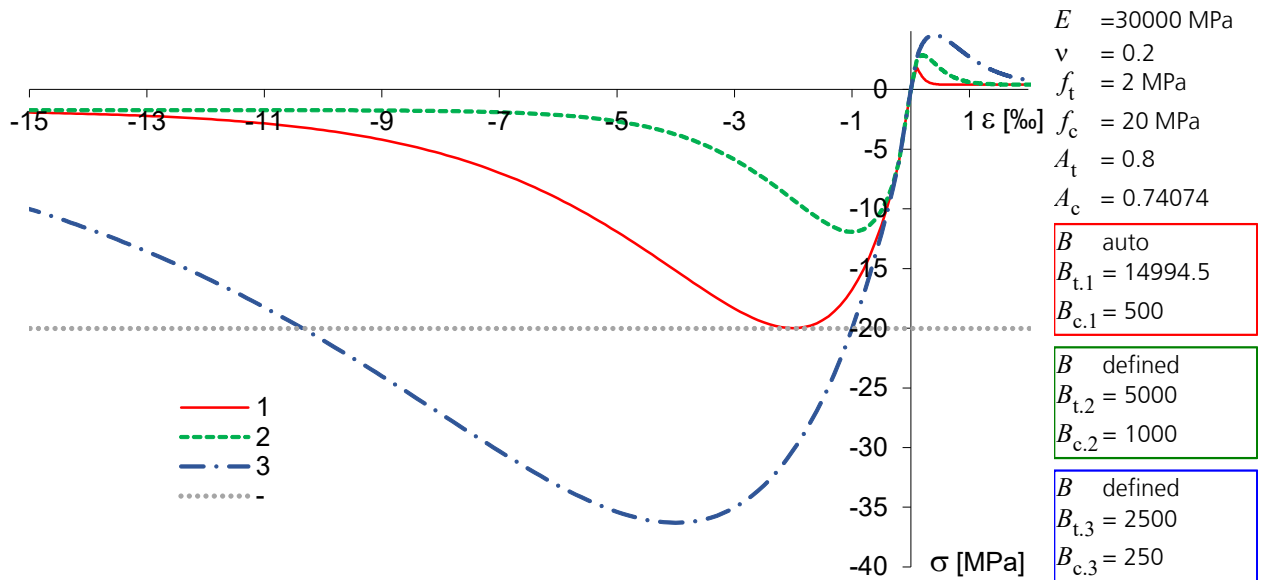


Figure 5: Uniaxial stress-strain diagram (Mazars'  $\mu$ -Model). Influence of the parameter  $B$ .

In order to make both the input of the stress-strain relationship as flexible as possible and to guarantee compliance with the tensile and compressive strength selected in the dialog (without parameter study), two alternatives are available in InfoCAD. One with and one without entering the parameters  $B_t$  and  $B_c$ . With the input of both parameters, the selected tensile and compressive strength does not correspond to the maximum values of the stress-strain relationship (see 2 and 3 in Figures 5 and 6). Without input, on the other hand, i.e. if the check mark is removed from 'Enter  $B$ ', the strengths previously defined in the dialog are automatically obtained (see 1 in Figures 5 and 6).

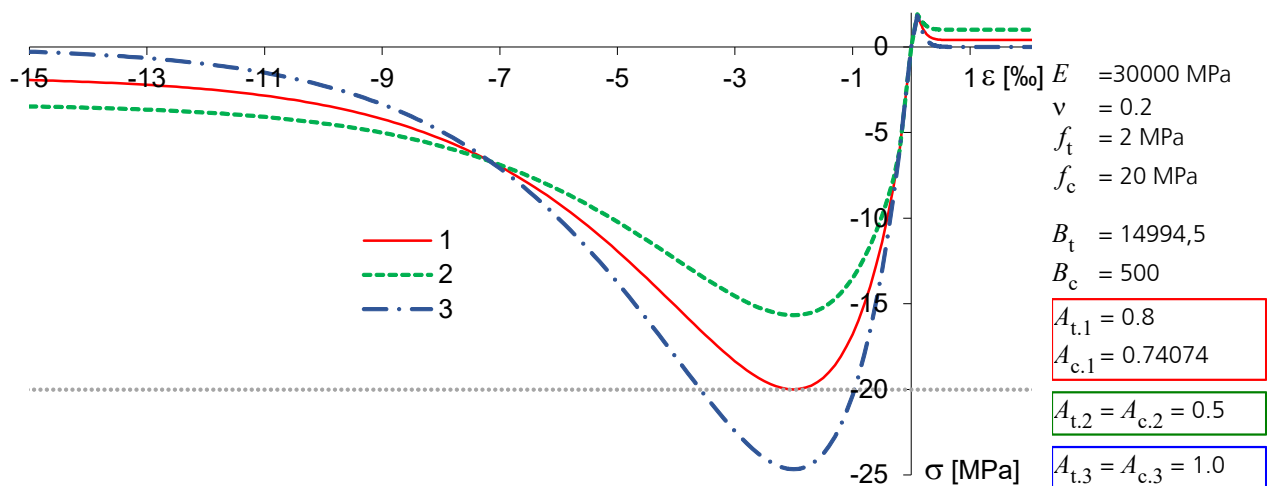


Figure 6: Uniaxial stress-strain diagram (Mazars'  $\mu$ -Model). Influence of the parameter  $A$ .

In the standard case, the limit strain  $\epsilon_1$  results from the parameters selected in the dialog for the material properties. However, if the compressive strength  $f_c$  is to correspond to a certain limiting strain  $\epsilon_1$ , it is necessary to adjust the compressive parameters  $A_c$  and  $B_c$ . For this purpose,  $B_c$  ( $B_c = 1 / \epsilon_1$ ) can first be determined and then the parameter  $A_c$  according to Equation V11:

$$A_c = (\eta_{C0} - f_c / E) / (\eta_{C0} - \exp(B_c \cdot \eta_{C0} - 1) / B_c) \quad (\text{V11})$$

**Examples**

To validate the  $\mu$ -model implemented in InfoCAD and verify the numerically predicted stiffness degradation due to uniaxial and biaxial loading, a comparison is made with the results of *Mazars et al.* (2015) as well as a comparison with experimental results (*Kupfer et al.* 1969). Figure 8 shows the stress-strain relationship resulting from uniaxial load. *InfoCAD* gives the same results as published by *Mazars et al.* For completeness, it is noted that the damage thresholds are determined internally here (Eq. V9) and are not explicitly given as model parameters ( $\eta_{T0}$  and  $\eta_{C0}$ ).

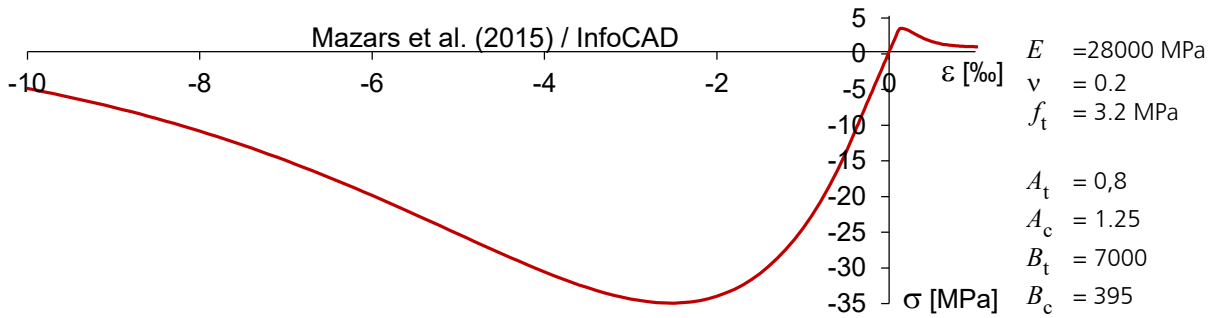


Figure 7: Uniaxial stress-strain relation (numerical)

The damage surface visualized in Figure 4 already shows that the biaxial failure curve of concrete (*Kupfer et al.*) can also be reproduced well with the  $\mu$ -model. In order to additionally evaluate the quality of the numerically predicted stiffness degradation for a biaxial loading (plane stress state  $\sigma_3=0$ ), corresponding finite element analyses were carried out and their results plotted together with the experimental results of *Kupfer et al.* in Figure 8. While all compressive components of the principal strains agree sufficiently accurately with the measured values, the tensile components in the region of the maximum values deviate from the recorded values. This is due to the fact that with an isotropic damage formulation, the poisson's ratio  $\nu$  remains constant (*Mazars et al.*). In addition, this model slightly overestimates the biaxial compressive strength. Regarding the partial discrepancy between numerical and experimental results, it should be mentioned here again that the stress-strain relationship, as described in the previous sections, can be adapted to the existing material behavior at any time by changing the four damage parameters ( $A_t, B_t, A_c$  and  $B_c$ ) as well as the tensile strength.

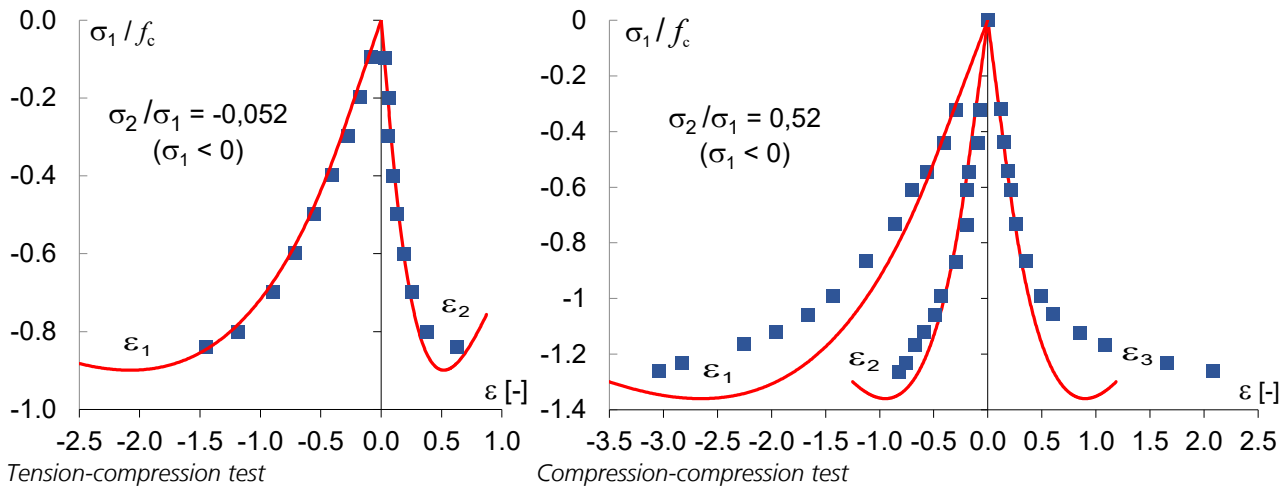


Figure 8: Two-axial stress-strain relationships. Comparison of numerical (— *InfoCAD*: *Mazars'*  $\mu$ -model) and experimental (■ *Kupfer et al.*) results.  
Parameters:  $f_t = 3.2$  MPa,  $E = 30$  GPa,  $\nu = 0.2$ ,  $A_t = 1$ ,  $A_c = 1.25$ ,  $B_t = 10000$ ,  $B_c = 459$

## Plasticity

The yield criteria available in InfoCAD are explained below:

- *Raghava*
- *Huber-v.Mises*
- *Rankine*
- *Mohr-Coulomb*
- *Drucker-Prager*
- *Lubliner*

The *Rankine*, *Mohr-Coulomb*, *Drucker-Prager* and *Lubliner* yield criteria are only available for solid elements with the material type *Frei*. The desired plasticity model can be selected in the material properties dialog.

It should be noted that the following equations do not take into account isotropic hardening. Initially, an ideal plastic material behavior is assumed. Explanations of the linear hardening considered in the FE program are given at the end of this section.

Due to the split of the strain tensor ( $\epsilon$ ) into an elastic ( $\epsilon^{el}$ ) and a plastic ( $\epsilon^{pl}$ ) part, the constitutive relations are defined as follows:

$$\sigma = C : \epsilon^{el} = C : (\epsilon - \epsilon^{pl}), \quad (C: \text{elasticity tensor}) \tag{V12}$$

Where the stresses here are also named effective stresses.

### Raghava

For solid elements made of steel and standardized concrete, only the yield criterion according to *Raghava et al. (1973)*, *Schleicher (1926)* is used:

$$F(\sigma) = J_2 + \alpha \cdot I_1 - \beta = 0, \quad \text{with } \alpha = (f_c - f_t)/3, \beta = (f_c \cdot f_t)/3, \tag{V13}$$

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3 \text{ and } J_2 = [(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]/6$$

Figure 9 shows the yield surface in the principal stress space. To illustrate the difference to the biaxial failure curve according to *Kupfer et al. (1969)*, the yield surface in the plane stress state ( $\sigma_3=0$ ) is shown in addition to the three-dimensional surface.

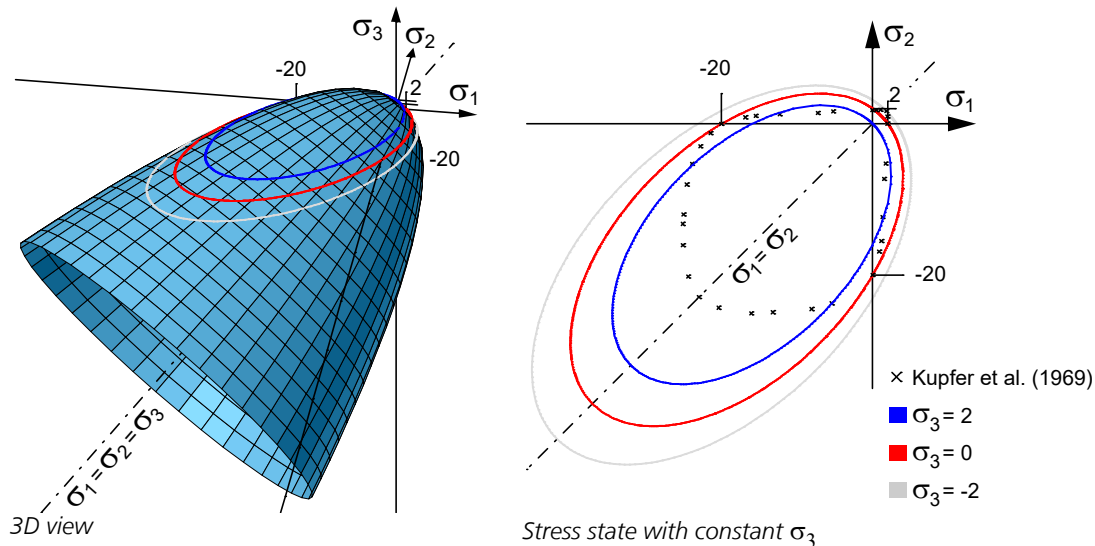


Figure 9: Yield surface according to *Raghava* in the principal stress space,  $f_{ck} = 20 \text{ MPa}$ ,  $f_{ctm} = 2 \text{ MPa}$

Analogous to the *De Vree* damage model (Figure 2), the compression-compression zone for concrete is significantly overestimated.

### Huber-v.Mises

If tensile and compressive strength are identical ( $f_c = f_t = \sigma_y$ ), the yield criterion of *Raghava et al.* (1973) is equivalent to the *Huber-v. Mises* yield criterion:

$$F(\sigma) = J_2 - \sigma_y^2 / 3 = 0 \tag{V14}$$

The representation of the yield surface in the principal stress space results in the typical *Huber-v.Mises* cylinder, whose center line coincides with the hydrostatic axis, see Figure 10. Due to the fact that here hydrostatic stresses (in contrast to the modified approach of *Raghava*, Eq. V13) have no influence on the yield behavior, this plasticity model is only suitable for a few materials, such as steel. This is also evident from the symmetrical yield surface in the plane stress state ( $\sigma_3=0$ ).

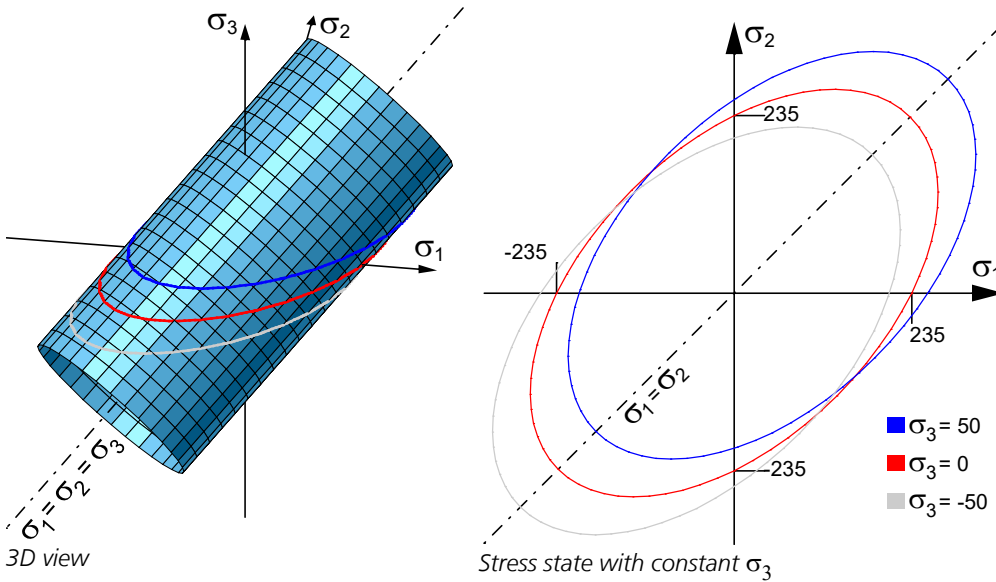


Figure 10: Yield surface according to Huber-v.Mises in the principal stress space,  $f_c = 235 \text{ MPa}$ ,  $f_t = 235 \text{ MPa}$

### Rankine

A plasticity criterion that can be used in addition to the two previously mentioned for solid elements of the material type *Frei* is based on the work of *Rankine* (1858). It is mainly suitable for brittle materials, since only one of the three principal stresses must exceed the elastic limit ( $\sigma_{1,2,3} > f_{c,t}$ ) for plastic yield to occur. The remaining two principal stresses have no effect on material failure. The yield surface composed of two parts is shown in Figure 11.

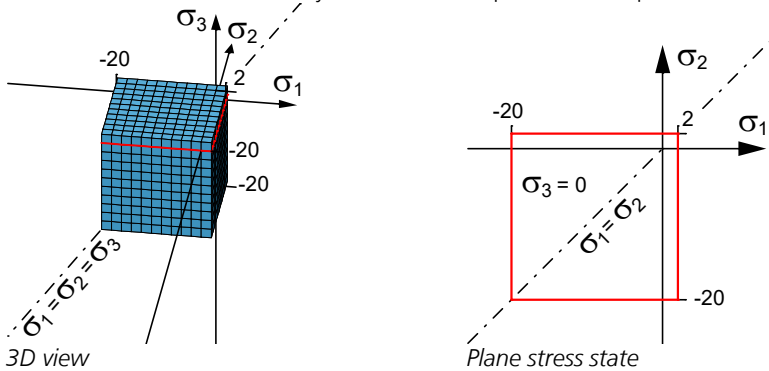


Figure 11: Yield surface according to Rankine in the principal stress space,  $f_c = 20 \text{ MPa}$ ,  $f_t = 2 \text{ MPa}$

With the first invariant of the stress tensor  $I_1$ , the second deviatoric invariant  $J_2$ , the Lode angle  $\Theta$  and the respective strengths (t: tension, c: compression), the following form of the yield criterion is obtained:

$$F_{t,c}(\sigma) = 2 \cdot \sqrt{3} \cdot J_2 \cdot \cos\Theta + I_1 - 3 \cdot f_{t,c} = 0 \tag{V15}$$

### Mohr-Coulomb

A frequently used plasticity model for estimating the load-bearing behavior of soils (i.a. bearing capacity failure of foundation and embankment failure) was developed by *Coulomb* in the 18th century and generalized by *Mohr* towards the end of the 19th century: the *Mohr-Coulomb* model. Here, a linear relationship is assumed between the shear strength of a material and the normal stresses occurring in the shear joint, with the resistance that occurs during sliding within the shear joint being defined by the angle of friction

$\varphi$ . With additional consideration of the cohesion  $c$ , i.e. the adhesive strength of individual soil particles, the following form of the yield condition is obtained:

$$F(\sigma) = \tau + \sigma \cdot \tan(\varphi) - c = 0,$$

With the use of *Mohr's* stress circle, the above equation can also be formulated as a function of the maximum and minimum principal stress:

$$F(\sigma) = (\sigma_{\max} - \sigma_{\min}) + (\sigma_{\max} + \sigma_{\min}) \cdot \sin(\varphi) - 2 \cdot c \cdot \cos(\varphi) = 0 \tag{V16}$$

The *Mohr-Coulomb* yield surface is described by a total of six planar surfaces, which form an irregular six-sided pyramid in the three-dimensional stress space, see Figure 12. Its shape can also be clearly understood by the showb sections perpendicular to the hydrostatic axis (deviatoric plane). In addition, the influence of material parameters such as friction angle  $\varphi$  and cohesion  $c$  becomes obvious when comparing yield surfaces generated with both concrete (Figure 12, left) and soil (right) parameters.

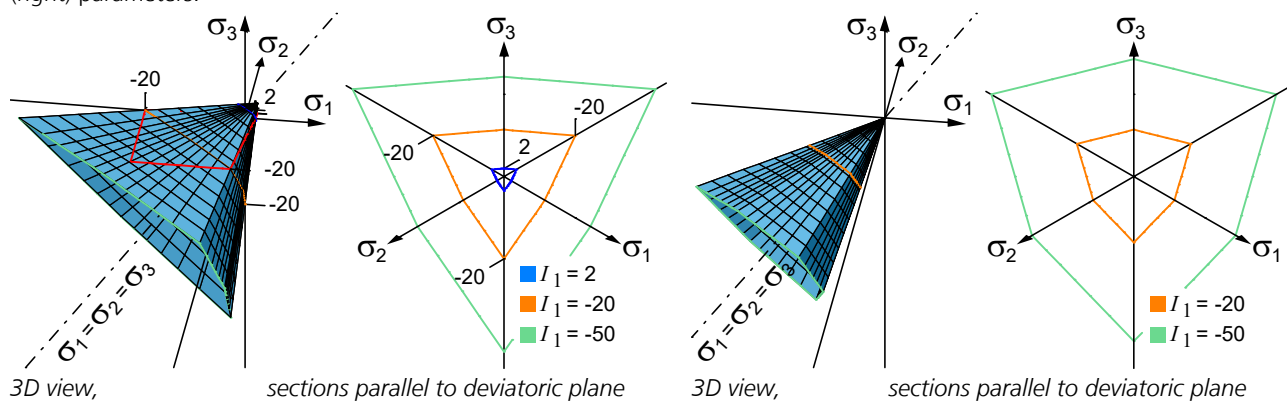


Figure 12: Yield surface according to Mohr-Coulomb in the principal stress space

In order to prevent an overestimation of the volume increase during plastic yielding (dilatancy), the general case of the non-associative yielding rule has been implemented in *InfoCAD*. Accordingly, in addition to the two previously known input values ( $\varphi$  and  $c$ ), a further parameter must be specified in the dialog for the material properties, namely the dilatancy angle ( $\psi \leq \varphi$ ). For the special case of an associated yield rule, the friction angle and dilatancy angle are identical ( $\varphi = \psi$ ). Currently, a maximum friction angle of  $60^\circ$  can be entered in *InfoCAD*. A friction angle of zero results in the yield function according to *Tresca*.

### Drucker-Prager

In contrast to *Rankine* and *Mohr-Coulomb*, the yield surface of the *Drucker-Prager* plasticity model (here: with adjustment to uniaxial strengths) has no edges. The smooth surface of the *Drucker-Prager* cone (see Figure 13) is an approximation of the *Mohr-Coulomb* criterion and is determined in the same way as for *Raghava* by the two invariants and the respective strengths for tension and compression:

$$F(\sigma) = \sqrt{J_2} + \alpha \cdot I_1 - \beta \cdot c = 0, \text{ with: } \alpha = \frac{1}{\sqrt{3}} \cdot \frac{f_c - f_t}{f_c + f_t}, \quad \beta = \frac{2}{\sqrt{3}} \text{ and } c = \frac{f_c \cdot f_t}{f_c + f_t} \tag{V17}$$

If the tensile and compressive strength are identical, the hydrostatic dependency is omitted here as well and this results to the *Huber-v.Mises* yield criterion again.

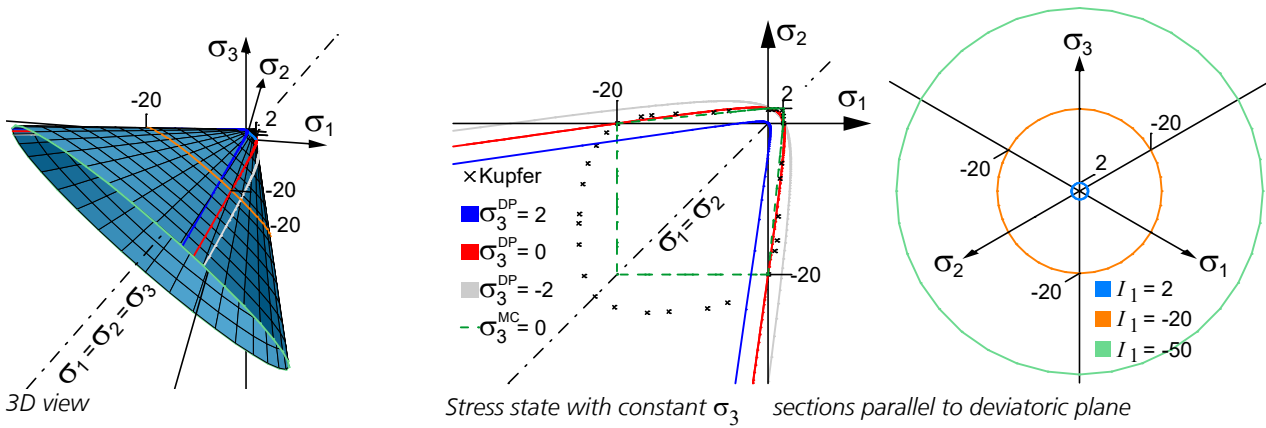


Figure 13: Yield surface according to Drucker-Prager in the principal stress space,  $f_c = 20 \text{ MPa}$ ,  $f_t = 2 \text{ MPa}$

In addition, it is clear from Figure 13 that for the strengths used here, contrary to Mohr-Coulomb, no failure occurs in the compression-compression zone (plane stress state  $\sigma_3=0$ ). For other values, however, this is quite possible. The following relationship must be considered: the smaller the difference between compressive and tensile strength, the smaller the angle between the yield surface and the hydrostatic axis.

### Lubliner

While all previously described plasticity models can only inadequately represent the fracture criteria of concrete or its biaxial material behavior, the yield surface according to Lubliner et al. (1989) in the plane stress state ( $\sigma_3=0$ ) agrees almost exactly with the failure surface experimentally determined by Kupfer et al. (1969), see Figure 14 (center).

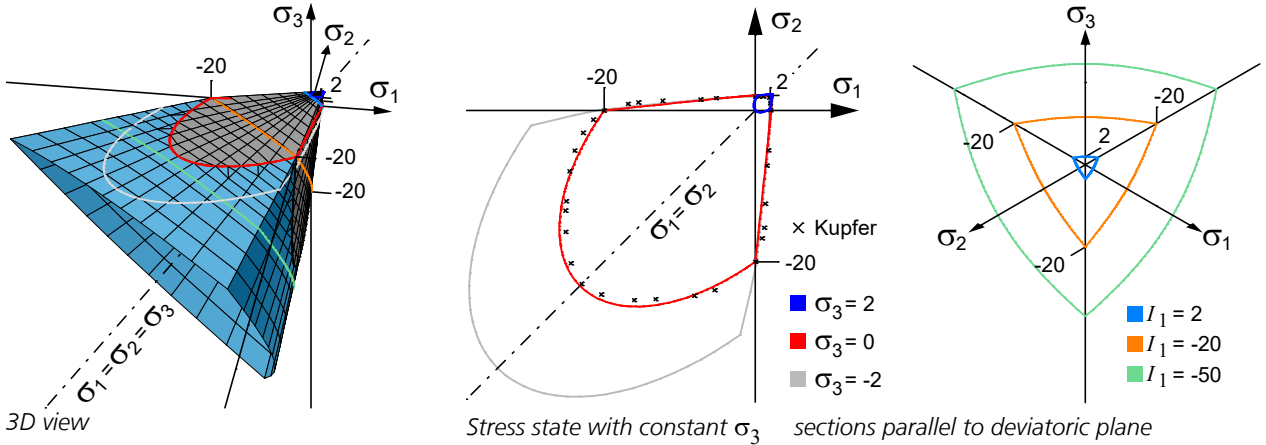


Figure 14: Yield surface according to Lubliner in the principal stress space,  $f_c = 20 \text{ MPa}$ ,  $f_t = 2 \text{ MPa}$ ,  $f_{cb} = 1,16 \cdot f_c$

The Lubliner surface is composed of three components. This is clearly shown by the yield criterion, which contains maximum values of the positive  $\langle \sigma_{\max} \rangle$  and negative  $\langle -\sigma_{\max} \rangle$  stresses put in Macaulay brackets

$$F = \underbrace{\sqrt{3 \cdot J_2}}_1 + \underbrace{\alpha \cdot I_1}_2 + \underbrace{\beta \cdot \langle \sigma_{\max} \rangle}_3 - \underbrace{\gamma \cdot \langle -\sigma_{\max} \rangle}_4 - \underbrace{(1-\alpha) \cdot f_c}_5 = 0, \tag{V18}$$

where

$$\alpha = \frac{(f_{cb}/f_c) - 1}{2 \cdot (f_{cb}/f_c) - 1}, \quad \beta = (1-\alpha) \cdot \frac{f_c}{f_t} - (1+\alpha), \quad \gamma = \frac{3 \cdot (1-\varphi_c)}{2 \cdot \varphi_c - 1} \quad \text{and} \quad \varphi_c = \frac{2}{3}. \tag{V19}$$

Depending on the stress state, a reduced form of equation V18 results. As already mentioned, three cases are possible:

- 1) Tensile stresses present -> fourth term of the right-hand side is omitted.
- 2) Biaxial compression -> third and fourth term omitted.
- 3) Triaxial compression -> third term omitted.

Due to the congruent areas at biaxial and triaxial compression, the governing yield surface can finally be described by two terms. In Figure 14 (left), these are shown in color for differentiation: gray area -> tensile stresses present, blue area -> triaxial compression. In addition to the tensile and compressive strength, the equibiaxial compressive strength factor (factor  $= f_{cb}/f_c$ ) can also be adjusted in the material properties dialog for the Lubliner model. The default value (for concrete) is



suggested to be 1.16 times the uniaxial compressive strength ( $f_c$ ):

$$f_{cb} = 1.16 \cdot f_c \tag{V20}$$

The shape of the yield surface is also influenced by the ratio of tensile and compressive meridians at triaxial compression, which is described by the parameter  $\varphi_c$  (see Eq. V19). Values between 0.5 and 1 are theoretically possible ( $0.5 < \varphi_c \leq 1$ ). Displayed in deviator plane, these two limits result in a triangle ( $\varphi_c = 0.5$  Rankine) or a circle ( $\varphi_c = 1$  Drucker-Prager). Here the value was set to  $\varphi_c = 2/3$  according to the recommendation in Lubliner et al. (1989) and Richart et al. (1928).

In the same way as with Mohr-Coulomb, a non-associative yield rule rather than an associative yield rule is used to ensure sufficient dilatancy, i.e. volume increase during plastic yielding. Plastic potential and yield condition are now no longer identical. The plastic strain increments are determined by deriving a potential function

$$\Delta \varepsilon^{pl} = \Delta \lambda \frac{\partial G(\sigma)}{\partial \sigma} \tag{V21}$$

and, as a result, are no longer perpendicular to the yield surface. Following the Lubliner yield condition (V18), which is composed of different Drucker-Prager surfaces, a Drucker-Prager type function is also used as the plastic potential. The potential function is as follows:

$$G(\sigma) = \sqrt{2 \cdot J_2 + (\tan(\psi) \cdot f_t \cdot \xi)^2} + \frac{\tan(\psi)}{3} \cdot I_1 \tag{V22}$$

In addition to the two stress invariants  $I_1$  and  $J_2$ , the input variables are the uniaxial tensile strength  $f_t$ , the dimensionless parameter  $\xi$ , and the dilatancy angle  $\psi$ . Both the parameter  $\xi$  and the dilatancy angle  $\psi$  significantly determine the shape of the potential function, see Fig. 15 ( $\xi = 0$ : linear,  $\xi > 0$ : hyperbolic).

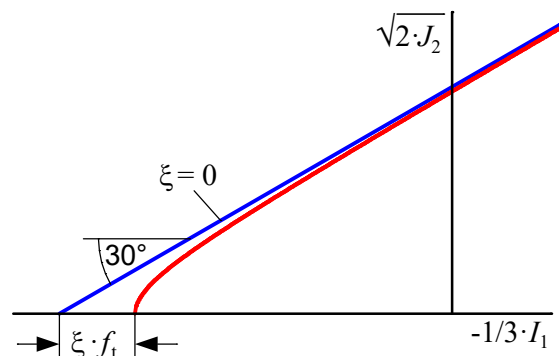


Figure 15: Linear and hyperbolic potential function

While in almost all scientific publications (thus also in InfoCAD) the parameter  $\xi$  is consistently set to 0.1, very different values for  $\psi$  are proposed for describing the dilatancy behavior of concrete. Good results have been obtained with dilatancy angles from  $\psi = 11.3^\circ$  to  $\psi = 56^\circ$ , see Poliotti & Bairan (2019).

Considering this range of values, a dilatancy angle of  $\psi = 30^\circ$  is chosen in InfoCAD.

Note: From a non-associative yield rule follows an asymmetric elasto-plastic tangent and thus an asymmetric stiffness matrix. For an efficient solution of the equation system, the elasto-plastic tangent is symmetrized in InfoCAD, see Pande et al. (1990). However, to ensure numerical stability, it is recommended to perform the calculation with a constant stiffness.

## Linear Hardening

In contrast to the material type *Stahl* or a standardized concrete type, the material type *Frei* offers the possibility to consider a linear hardening. Its degree is defined by the hardening modulus  $H$ , which is to be specified in the dialog for the material properties. When entering the modulus, the plastic material behavior is also directly defined. While a linear hardening can be realized by a positive hardening modulus, the modulus is set to zero for modeling an ideal plastic behavior.

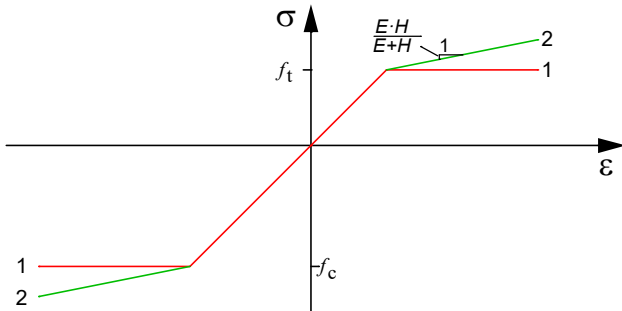


Figure 16: Uniaxial stress-strain diagram. Material behavior:  
1) ideally plastic, 2) with linear hardening for Rankine

Using the *Rankine* yield condition as an example, it is shown below how the hardening is to be included in the respective equations. An isotropic hardening is characterized by an expanding yield surface in the stress space; in contrast to kinematic hardening, the orientation does not change. To realize this expansion, a term  $q(\kappa)$  dependent on the internal (hardening) variable  $\kappa$  is added to the initial yield stress  $f_{t,c}$  (cf. Eq. V15):

$$F_{t,c}(\sigma, \kappa) = 2 \cdot \sqrt{3} \cdot J_2 \cdot \cos\Theta + I_1 - 3 \cdot (f_{t,c} + q(\kappa)) = 0 \quad (V23)$$

In *InfoCAD*, this term is described by the following linear function:  $q(\kappa) = H \cdot \kappa$ . A detailed derivation and explanation of the numerical implementation is omitted here. Instead, we refer to the publications by *Chen & Han* (1988) and *Simo & Hughes* (1998).

## Combined Model (Plasticity-Damage)

The constitutive relations of the elasto-plastic damage model are obtained by combining Eq. V1 and Eq. V12 as follows:

$$\sigma = (1 - D) C : (\varepsilon - \varepsilon^{pl}) \quad (V24)$$

A coupling of stresses  $\sigma$  and effective stresses  $\bar{\sigma}$  is carried out here by means of the scalar damage variable  $D$ , which reduces the stiffness isotropically in analogy to the procedure for the pure damage models:

$$\sigma = (1 - D) \cdot \bar{\sigma} \quad (V25)$$

To determine the plastic strains  $\varepsilon^{pl}$ , the yield condition according to *Lubliner et al.* (V18) is first evaluated taking into account the effective stresses:

$$F(\bar{\sigma}, \tilde{\varepsilon}^{pl}) = \sqrt{3} \cdot J_2 + \alpha \cdot I_1 + \beta(\tilde{\varepsilon}^{pl}) \cdot \langle \bar{\sigma}_{max} \rangle - \gamma \cdot \langle -\bar{\sigma}_{max} \rangle - (1 - \alpha) \cdot \bar{\sigma}_c(\varepsilon_c^{pl}) \leq 0 \quad (V26)$$

Input variables are, in addition to the effective stresses, two internal variables (*Lee & Fenves*, 1998), the tensile ( $\varepsilon_t^{pl}$ ) and compression ( $\varepsilon_c^{pl}$ ) plastic strains:

$$\tilde{\varepsilon}^{pl} = [\varepsilon_t^{pl} \quad \varepsilon_c^{pl}]^T \quad (V27)$$

Their evolutions, which describe the hardening and softening of the material, can be calculated as a function of the eigenvalues of the plastic strain tensor and the triaxial factor  $r(\bar{\sigma})$  (V5) as follows:

$$\dot{\varepsilon}^{pl} = \begin{bmatrix} \dot{\varepsilon}_t^{pl} \\ \dot{\varepsilon}_c^{pl} \end{bmatrix} = \begin{bmatrix} r(\bar{\sigma}) & 0 & 0 \\ 0 & 0 & 1 - r(\bar{\sigma}) \end{bmatrix} \begin{bmatrix} \dot{\varepsilon}_1^{pl} \\ \dot{\varepsilon}_2^{pl} \\ \dot{\varepsilon}_3^{pl} \end{bmatrix}, \text{ mit: } \dot{\varepsilon}_1^{pl} \geq \dot{\varepsilon}_2^{pl} \geq \dot{\varepsilon}_3^{pl} \quad (V28)$$

The factor  $\beta(\tilde{\varepsilon}^{pl})$  in Eq. V26 is here, in the case of the combined model (plasticity and damage), not constant (cf. Eq. V19), but dependent on the two internal variables:

$$\beta(\tilde{\varepsilon}^{pl}) = (1 - \alpha) \cdot \frac{\sigma_c(\varepsilon_c^{pl})}{\sigma_t(\varepsilon_t^{pl})} - (1 + \alpha) \quad (V29)$$

After iterative calculation of the plastic multiplier  $\Delta\lambda$  with the return mapping method, the scalar damage parameter  $D$  is determined as a function of the internal variable  $\tilde{\varepsilon}^{pl}$ . Due to the separation of tensile and compressive plastic strains, the same procedure is used for the damage parameter. With the parameter for tension

$$d_t = 1 - \frac{\sigma_t \cdot E_0^{-1}}{\varepsilon_t^{pl} \cdot (1/b_t - 1) + \sigma_t \cdot E_0^{-1}} \quad (V30)$$

and compression

$$d_c = 1 - \frac{\sigma_c \cdot E_0^{-1}}{\varepsilon_c^{pl} \cdot (1/b_c - 1) + \sigma_c \cdot E_0^{-1}} \quad (V31)$$

the isotropic damage for the three-dimensional stress state is finally determined (V23):

$$D = 1 - (1 - s_c \cdot d_t) \cdot (1 - s_t \cdot d_c) \quad (V32)$$

Since it is currently not possible to simulate load changes (hysteresis) in *InfoCAD*, opening or closing of cracks is not taken into account when calculating the damage parameter ( $s_c = s_t = 1$ ).

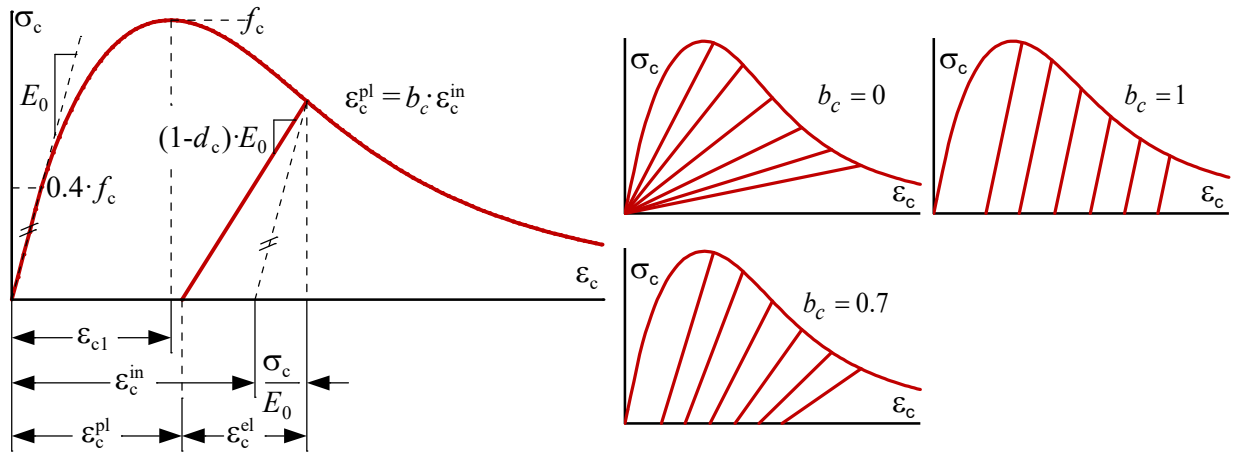


Figure 17: Uniaxial compressive stresses

For a realistic description of the material behavior, uniaxial stress-strain relationships are used, which are based on a decomposition of the strains into elastic, plastic or inelastic components. Separated for compression and tension (elongation), the relationships are as follows (Figs. 17 and 18):

$$\varepsilon_c = \varepsilon_c^{el} + \varepsilon_c^{pl} = \varepsilon_c^{in} + \sigma_c / E_0, \text{ with: } \varepsilon_c^{pl} = b_c \cdot \varepsilon_c^{in} \quad (V33)$$

$$\varepsilon_t = \varepsilon_t^{el} + \varepsilon_t^{pl} = \varepsilon_t^{in} + \sigma_t / E_0, \text{ with: } \varepsilon_t^{pl} = b_t \cdot \varepsilon_t^{in} \quad (V34)$$

Here, the factor  $b_c$  or  $b_t$  determines the ratio of plastic and inelastic strains and thus also the inclination of any unloading paths. While no damage of the material occurs with a factor of  $b_{c,t} = 1$  ( $D = 0$ ) and loading and unloading paths have the same inclination, no plastic strains remain after unloading with a factor of  $b_{c,t} = 0$ . The influence of the parameter can be clearly illustrated by means of both limiting cases (see also Figs. 17 and 18). However, these two cases do not represent a realistic material behavior of concrete during loading and unloading processes. Experimental results often show that the ratio of plastic to inelastic strains takes the value  $b_c = 0.7$  (compression) and  $b_t = 0.1$  (tension). Consequently, these two values are also used in *InfoCAD*.

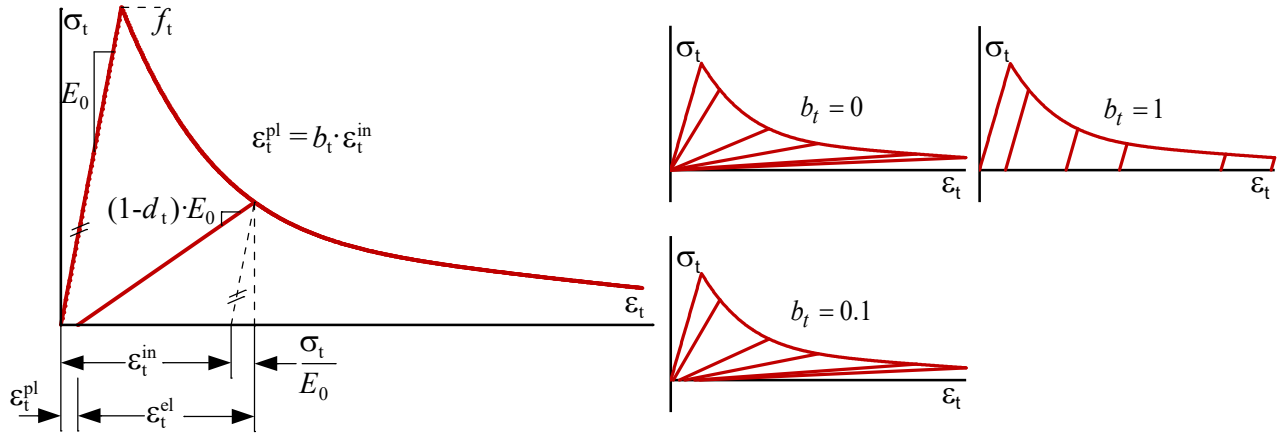


Figure 18: Uniaxial tensile stresses

The uniaxial stress-strain relations implemented here are defined separately for certain areas. First, the functions for a pure compressive stress state are explained. The first of a total of three sections describes the linear elastic material behavior at the start of loading:

$$\sigma_c(\varepsilon_c) = E_0 \cdot \varepsilon_c \quad \text{for} \quad \sigma_c(\varepsilon_c) \leq 0.4 \cdot f_c \quad (\text{V35})$$

Following the curves according to the standard, 0.4 times the compressive strength is taken as the initial yield stress here. After reaching this limit value, the tangential stiffness (initial:  $E_0$ ) is successively reduced until a horizontal curve is obtained at  $\sigma_c(\varepsilon_c) = f_c$ . For this second section, the following analytical function from *CEB-FIP Model Code 1990* is used due to its good agreement with experimental results:

$$\sigma_c(\varepsilon_c) = \frac{E_{ci} \cdot \frac{\varepsilon_c}{f_c} - \left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^2}{1 + \left(E_{ci} \cdot \frac{\varepsilon_{c1}}{f_c} - 2\right) \cdot \frac{\varepsilon_c}{\varepsilon_{c1}}} \cdot f_c \quad \text{for} \quad 0.4 \cdot f_c < \sigma_c(\varepsilon_c) \leq f_c \quad (\text{V36})$$

To ensure a continuous transition between the first two sections of the stress-strain relation, the following secant modulus is used to evaluate Eq. V36 instead of the elastic modulus of the undamaged material ( $E_{ci} = E_0$ ):

$$E_{ci} = \frac{2}{3 \cdot E_c} \cdot \left(\frac{f_c}{\varepsilon_{c1}}\right)^2 - \frac{4 \cdot f_c}{3 \cdot \varepsilon_{c1}} + \frac{5}{3} \cdot E_0 \quad (\text{V37})$$

The limit compression  $\varepsilon_{c1}$  is to be specified a-priori in the same way as the compressive strength in the dialog of the material properties. For a realistic representation of the post-cracking behavior of concrete and minimization of the mesh dependency, the approach of *Krätzig & Pölling* (2004) is used here. This third and final section of the stress-strain relationship under monotonic compressive loading is described by the following function:

$$\sigma_c(\varepsilon_c) = \left( \frac{2 + \gamma_c \cdot f_c \cdot \varepsilon_{c1}}{2 \cdot f_c} - \gamma_c \cdot \varepsilon_c + \frac{\gamma_c \cdot (\varepsilon_c)^2}{2 \cdot \varepsilon_{c1}} \right)^{-1} \quad \text{for} \quad \sigma_c(\varepsilon_c) > f_c \quad (\text{V38})$$

with

$$\gamma_c = \frac{\pi^2 \cdot f_c \cdot \varepsilon_{c1}}{2 \cdot \left( g_c - \frac{f_c}{2} \cdot \left( \varepsilon_{c1} \cdot (1 - b_c) + \frac{b_c \cdot f_c}{E_0} \right) \right)^2} > 0 \quad (\text{V39})$$

and the element and volume based compressive fracture energy:  $g_c = G_c / l_c$ . In order to nearly eliminate the mesh dependence during softening, the compressive fracture energy  $G_c$  is coupled to the element size (volume  $V_e$ ). Simplified for solid elements, the characteristic length  $l_c$  is:  $l_c = V_e^{1/3}$  (*Oliver*, 1989). Experimental tests on specimens have also shown that for normal concrete the amount of compressive fracture energy  $G_c$  is in the range between 10 and 25 kN/m (*Vonk*, 1992). In *InfoCAD*, however, values between 5 and 500 kN/m are permitted.

The influence of the energy on the free parameter  $\gamma_c$ , which defines the area under the stress-strain curve, is evident from Eq. V39. Due to the condition  $\gamma_c > 0$ , a negative denominator must also be excluded. Consequently, to avoid a snap-back behavior, the characteristic length should not exceed the following value (*Krätzig & Pölling*, 2004):

$$l_c \leq G_c / (f_c \cdot (\varepsilon_{c1} \cdot (1 - b_c) + b_c \cdot f_c / E_c)) \quad (V40)$$

For a monotonic compressive loading, the stress-strain diagram consisting of three parts is shown in Figure 17 (left) and Figure 19. Figure 19 also clearly shows the good agreement between numerical (*InfoCAD*) and experimental (*Sinha et al., 1964*) results.

Except for the compressive fracture energy (here:  $G_c = 26 \text{ kN/m}$ ), all parameters for the comparative calculation were taken from the publication by *Krätzig & Pölling (2004)*.

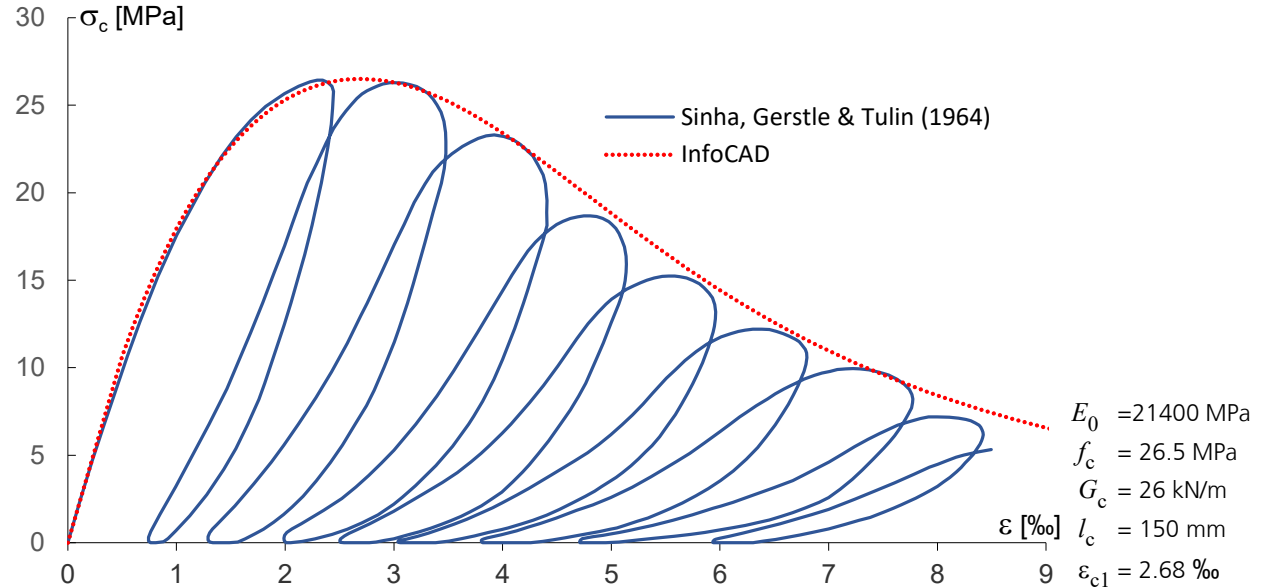


Figure 19: Uniaxial stress-strain diagram (compression): Comparison of experimental (*Sinha, Gerstle & Tulin, 1964*) and numerical (*InfoCAD*) results

After describing the stress-strain relationship for compressive loading implemented in *InfoCAD*, the functions for a tensile load are now specified below. Only two sections need to be defined for this purpose. The first section describes the linear elastic material behavior up to the tensile stress  $f_t$

$$\sigma_t(\varepsilon_t) = E_0 \cdot \varepsilon_t \quad \text{for } \sigma_t(\varepsilon_t) \leq f_t \quad (V41)$$

and the second section describes the softening material behavior (decreasing branch). The function, which realistically represents the softening of concrete under monotonic tensile loading, is based on the scientific work of *Cornelissen et al. (1986)* and *Hordjijk (1992)*. The following stress-crack opening relationship was empirically determined during experiments:

$$\sigma_t(w) = f_t \cdot \left[ \left[ 1 + \left( \frac{c_1 \cdot w}{w_c} \right)^3 \right] \exp\left(-c_2 \frac{w}{w_c}\right) - \frac{w}{w_c} \cdot (1 + c_1)^3 \cdot \exp(-c_2) \right], \quad \text{with: } c_1 = 3 \text{ and } c_2 = 6.93 \quad (V42)$$

Using the inelastic strains  $\varepsilon_t^{\text{in}}$  (V34), the stress-crack opening relation is finally transformed into a stress-strain relation:

$$\sigma_t(\varepsilon_t) = \sigma_t(w = \varepsilon_t^{\text{in}} \cdot l_c) \quad \text{for } \sigma_t(\varepsilon_t) > f_t \quad (V43)$$

The volume-specific fracture energy  $g_f = G_f / l_c$  and thus the dependency of the decreasing branch on the element size (*Bažant & Oh, 1983*) is already taken into account in Eq. V43:

$$G_f = \int_0^{w_c} \sigma_t(w) dw = l_c \cdot \int_0^{w_c/l_c} \sigma_t(\varepsilon_t) d\varepsilon_t \quad (V44)$$

After solving the integral over the critical crack width  $w_c$ , it follows:

$$G_f = 0.195 \cdot w_c \cdot f_t \quad (V45)$$

In addition to the tensile strength  $f_t$ , the fracture energy  $G_f$  or the critical crack width  $w_c$  must also be specified as a material parameter. In *InfoCAD*, the fracture energy was selected as the input variable (see *Material Properties* dialog). However, the above equation can be used to directly determine the fracture energy if the limit crack width is known.

Figure 20 shows the stress-strain relationship for monotonic tensile loading. Here it becomes clear that, using the function generated by *Cornelissen et al. (1986)* and *Hordjijk (1992)*, negative stresses are to be expected as soon as the crack width  $w$  corresponding to the strain exceeds the critical crack width  $w_c$ :  $w = \varepsilon_t^{\text{in}} \cdot l_c > w_c$  (see V42). Since negative stresses are not permissible, an adjustment must be made for this strain range in the numerical implementation.

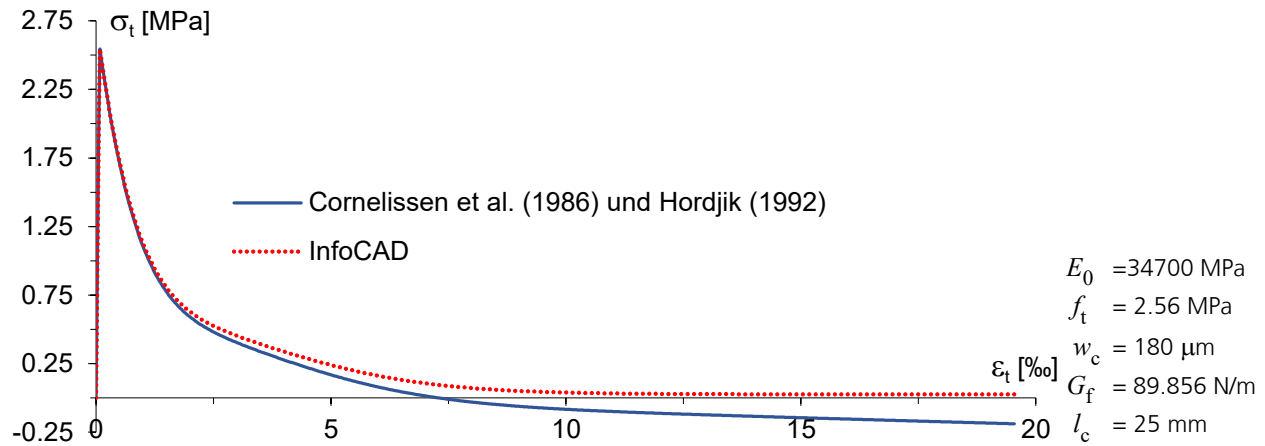


Figure 20: Uniaxial stress-strain diagram

For example, in order to exclude a kink in the stress-strain relationship, which occurs when the critical crack width is exceeded if the stress in this area were set to zero  $\langle \sigma_t(\epsilon_t) \rangle$ , the stress-crack opening relationship (V42) is slightly modified for the calculation with *InfoCAD*. On the one hand, the third term is neglected, thus excluding negative stresses, and on the other hand, a factor for the residual tensile strength  $r$  is introduced so that stresses never become zero even at infinitely large strains, but asymptotically approach the value  $\sigma_t(\epsilon_t) = r \cdot f_t$  (here  $r = 0.02$ ). The adjusted stress-crack opening relation is then:

$$\sigma_t(w) = f_t \cdot \left( 1 - (1-r) + \left[ 1 + \left( \frac{c_1 \cdot w}{w_c} \right)^3 \right] \cdot (1-r) \cdot \exp\left( -c_2 \frac{w}{w_c} \right) \right) \quad (V46)$$

In addition to the original form, the modified stress-strain relationship for monotonic tensile stresses is also shown in Figure 20. In addition to the different functional curves after reaching the critical crack width, it becomes clear that the deviation of both tensile stresses before reaching the critical crack width is negligible.

In the same way as before for compressive stress, the characteristic length  $l_c$  should not exceed a certain value to avoid *snap-back* behavior. The limit is defined as follows [ Irwin (1958), Hillerborg et al. (1976), Bažant & Oh (1983)]:

$$l_c < G_f \cdot E_c / f_t^2$$

To demonstrate the capability of the stress-strain relationship implemented in *InfoCAD*, numerical and experimental results are also compared here. Figure 21 shows the good agreement.

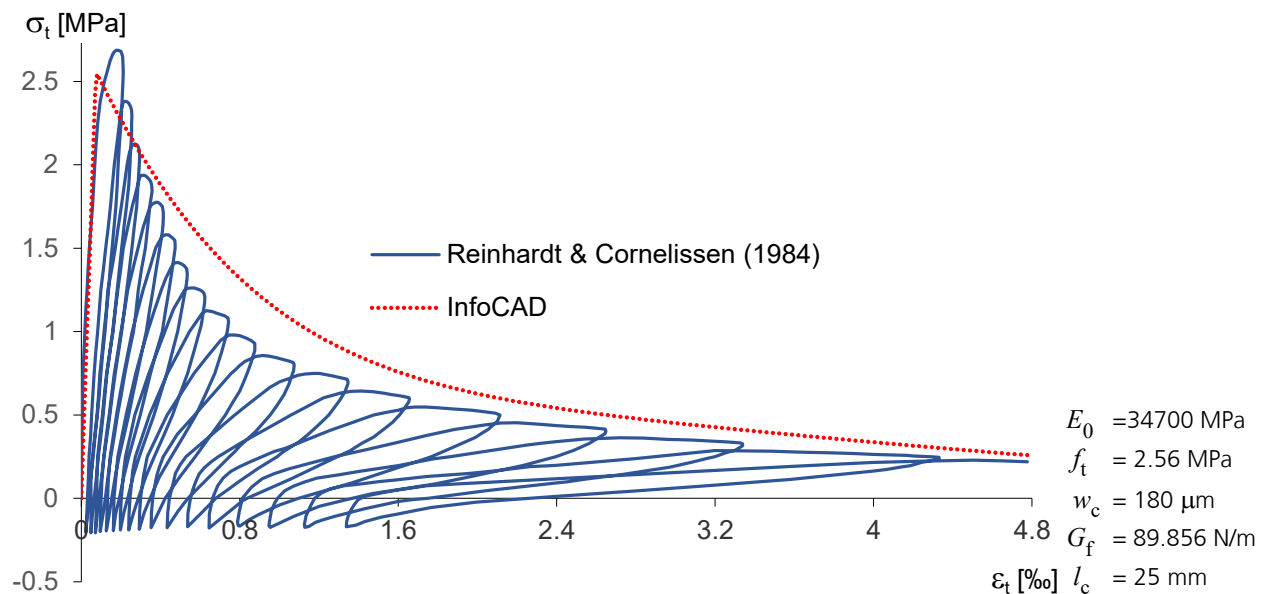


Figure 21: Uniaxial stress-strain diagram (tension): Comparison of experimental (Reinhardt & Cornelissen, 1984) and numerical (*InfoCAD*) results

## Prestressed Structures

Prestressing is often considered only as an external load. Therefore, no stress redistributions between concrete and prestressing steel can be taken into account. But the consideration of this redistributions is also possible. For this purpose, the tendons are included in the element stiffness matrices during the calculation. This method is implemented for all element types. The internal forces (normal forces, bending moments, lateral forces) given by the program always correspond to the concrete section with its reinforcing steel layers. When analyzing composite elements, these alone are not in equilibrium with the external forces since the tendon group forces must be applied while taking into account their spatial orientation. Because the program assumes that a tendon runs straight between the entry and exit points of an element, an adequate FE mesh is especially important for beam elements. Area and solid models, on the other hand, generally exhibit a sufficiently fine discretization.

The stress-strain curves of prestressing steel in the serviceability limit state have a rising branch with  $E_p$  and a horizontal upper branch at  $f_{p0,1k}$  or  $\beta_s$ . In the ultimate limit state the upper branch is at  $f_{pd}$ . Depending on the concrete, the following strengths are assumed for  $f_{pd}$ .

- Concrete *DIN 1045:1988*  $f_{pd} = \beta_s$
- Concrete *DIN EN 1992-1-1, DIN 1045-1*  $f_{pd} = 1,1 \cdot f_{p0,1k} / \gamma_R$  mit  $\gamma_R = 1.3$
- Concrete *EN 1992-1-1, ÖNORM B 4700, SIA 262*  $f_{pd} = f_{p0,1k} / \gamma_s$  mit  $\gamma_s = 1.15$

## Notes on Convergence Behavior

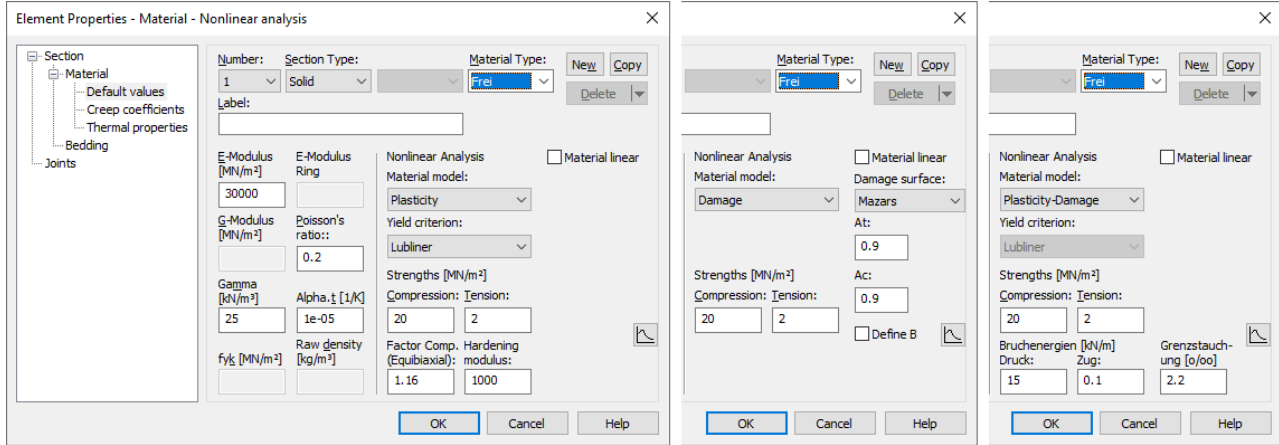
The implemented analysis methods (Newton method, arc length method) with tangential stiffness matrix result in a stable convergence behavior given a consistent relationship between the stress-strain relation and its derivative. As previously mentioned, this is especially true of bilinear materials. Reinforced concrete, however, typically displays a poor convergence due to its more complex material properties. This is caused by crack formation, not continuously differentiable stress-strain relationships, two-component materials, etc.

Especially for checks of the serviceability and determination of the tensile stiffness with softening, markedly worse convergence can result due to the negative tangential stiffness in the softening area. If this results in a singular global stiffness matrix, it is possible to assume a bilinear function in the tensile section or to perform a calculation without tensile strength.

# Material properties

In principle, only area, solid, polygon, database cross-sections and structural steel sections can be subjected to a nonlinear analysis. For all other cross-section types and for the material types *Beton* and *Timber*, linear material behavior is always used as a basis.

In the dialog described below for solid cross-sections with the material type *Frei*, non-linear material properties are specified. The material properties not described here, e.g. modulus of elasticity, are described in the general part of the manual.



## Material model

*Plasticity*

*Damage*

*Plasticity-Damage*

## Yield criterion

- *Drucker-Prager*
- *Huber- von Mises*
- *Lubliner*
- *Mohr-Coulomb*
- *Raghava*
- *Rankine*

## Damage surface

- *De Vree*: Damage model according to *De Vree* with the parameters *A* and *B*.
- *Mazars*: Damage model according to *Mazars* with the parameters  $A_t$ ,  $B_t$ ,  $A_c$  and  $B_c$ .

## Strengths [MN/m<sup>2</sup>]

*Compression*: Yield strength under compressive stresses  $f_c$  (positive value)

*Tension*: Yield strength under tensile stresses  $f_t$  (positive value)

## Material parameters for Mohr-Coulomb

*Friction angle [°]*: Resistance that occurs when sliding in a shear joint

*Dilatancy angle [°]*: Affects the change in volume during plastic deformation

*Cohesion [MPa]*: Adhesive strength or internal cohesion of individual particles (e.g. in soil or concrete).


## More material parameters

*Factor comp. (equibiaxial)*: The standard case for concrete  $f_{cb} / f_c = 1.16$  is suggested, which is used for the yield condition according to *Lubliner*. (positive value).

*Hardening modulus*: Value describing hardening at strains above tensile and compressive strength.

*Fracture energy compression / tension [kJ/m]*: Specific fracture energies for the material model *plasticity-damage*.

*Ultimate compressive strain [%]*: Strain  $\epsilon_{c1}$  under the maximum stress  $f_c$  for the material model *plasticity-damage*.

The uniaxial stress-strain relation can be visualized via the button  and adjusted if necessary. For the combined model (plasticity-damage), only the characteristic lengths has to be specified for the visualization. In the calculation, this is determined internally in dependence of the element volume ( $l_c$  or  $g_c$  in Eq. V38 and  $g_t$  in Eq. V42).

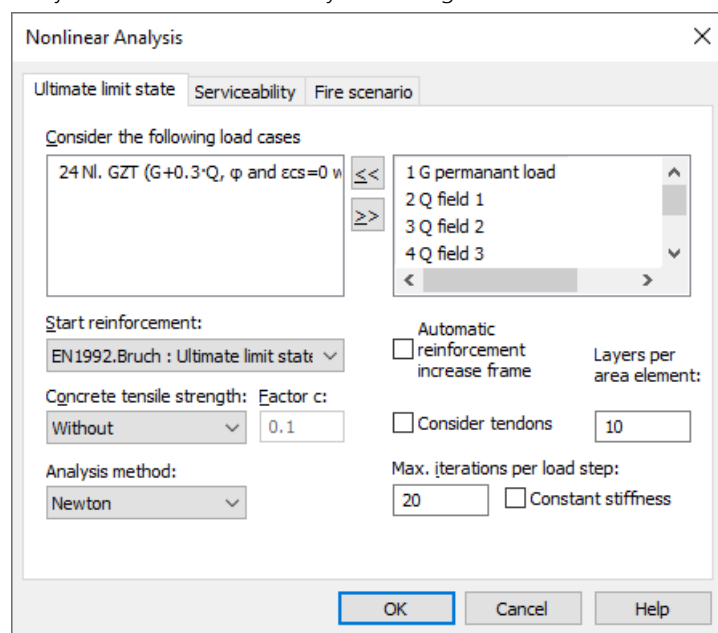
## Material linear

This switch causes a linear-elastic material behavior for this cross-section in the nonlinear system analysis.



## Analysis Settings

The following settings are made on the *Ultimate Limit state* and the *Serviceability* tabs in the *Settings* for the nonlinear analysis of the menu item *Analysis - Settings*.



With the nonlinear system analysis, load cases are calculated under consideration of physical and geometrical nonlinearities, whereby the latter only becomes active if the second- or third order theory is activated in the load case. The load bearing capacity and serviceability check as well as the stability check for fire differ according to the load case that is to be checked, the material safety, the different stress-strain-curves and the consideration of the concrete tensile strength.

### Consider the following load cases

The load cases from the left list box are calculated.

### Start reinforcement

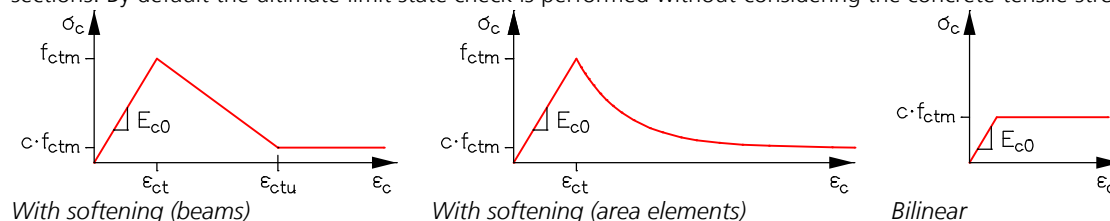
The nonlinear system analysis is carried out on reinforced concrete sections based on the reinforcement selected here. This results from a reinforced concrete design carried out in advance. The starting reinforcement *Null* corresponds to the base reinforcement of the reinforcing steel layers. When performing a check for fire scenarios, special conditions apply as explained in the *Structural Analysis for Fire Scenarios* chapter.

### Automatic reinforcement increase frame

For the ultimate limit state check of pure frameworks a reinforcement increase is carried out for reinforced concrete sections to achieve the required load-bearing safety.

### Concrete tensile strength; Factor c

This option defines the behavior in the tensile zone for the nonlinear internal forces calculation for all reinforced concrete sections. By default the ultimate limit state check is performed without considering the concrete tensile strength.



### Layers per area element

Number of integration levels of an area element. Members subject to bending should be calculated with 10 layers. Structure mostly subject to normal forces can be adequately analyzed with 2 layers.

### Consider tendons

The tendons are considered in the calculation of the element stiffness matrices.

**Constant stiffness**

The iteration is done with a constant stiffness matrix. If the switch is not set, then a tangential stiffness matrix is used.

**Phi-c Reduction**

Method for determining the stability of embankments (serviceability tab). Successive reduction of soil parameters (friction angle  $\phi$  and cohesion  $c$ ) until the failure condition (embankment failure) occurs. The *Phi-c Reduction* according to *Fellenius* is only activated in conjunction with the *Mohr-Coulomb* plasticity model. If the switch is set but no *Mohr-Coulomb* material is defined, the method is not used.

**Analysis method**

The following methods are available:

- **Newton, Newton time steps**

Analysis method:		Max. iterations per load step:	
<input type="text" value="Newton time steps"/>	<input type="checkbox"/> Phi-C-Reduction	<input type="text" value="20"/>	<input type="checkbox"/> Constant stiffness
Unit of time:	T. increment:	Duration:	
<input type="text" value="Seconds"/>	<input type="text" value="1"/>	<input type="text" value="60"/>	

*Max. iterations per load step:*

Maximum number of iteration steps, in order to reach convergence within one load step. If the error threshold is exceeded, the iteration is interrupted and an expectable reduced load level is determined on the basis of load. If it is not possible to fall below the error limit within the iteration steps per load step, the iteration is aborted and load-controlled the loadable load factor ( $\leq 1.0$ ) or the load at an intermediate time is determined by interval nesting.

*Time increment and Duration:*

The load-time function is defined in the database in the folder *Structure description > Instationary load-time function* and then assigned to loads. The time increment and the time duration must be adapted to the load time function. With regard to the special procedure for activating the *Phi-c Reduction*, please refer to the section 'Bearing capacity failure of foundation and embankment failure'.

- **Arc length**

Analysis method:		Iterations per arc length:	
<input type="text" value="Arc length"/>	<input type="checkbox"/> Phi-C-Reduction	<input type="text" value="3"/>	<input type="checkbox"/> Constant stiffness
Control:	Initial arc length:		
<input type="text" value="Number increments"/>	<input type="text" value="30"/>	<input type="text" value="0.1"/>	<input type="checkbox"/> Auto <input type="checkbox"/> Up limited

*Initial arc length:*

The initial arc length can either be entered manually (between 0.001 and 100) or calculated automatically depending on the system stiffness and load. While the arc length does not exceed the maximum value of 100 in the numerical calculation, significantly smaller arc lengths than 0.001 can occur. If the arc length is reduced to less than  $1e-12$  ( $1e-5$  for Solids with Material type *Frei*), the calculation will be aborted. The input value is at the same time the maximum value, if a checkmark is set at "Up limited".

*Iterations per arc length:*

Due to the influence of the iteration number on the arc length during the calculation, when the selected value is reached due to insufficient accuracy of the numerical solution, the number is doubled once. If divergent behavior occurs during the subsequent iterations, or if the error tolerance is not reached even after the increase, the arc length is halved.

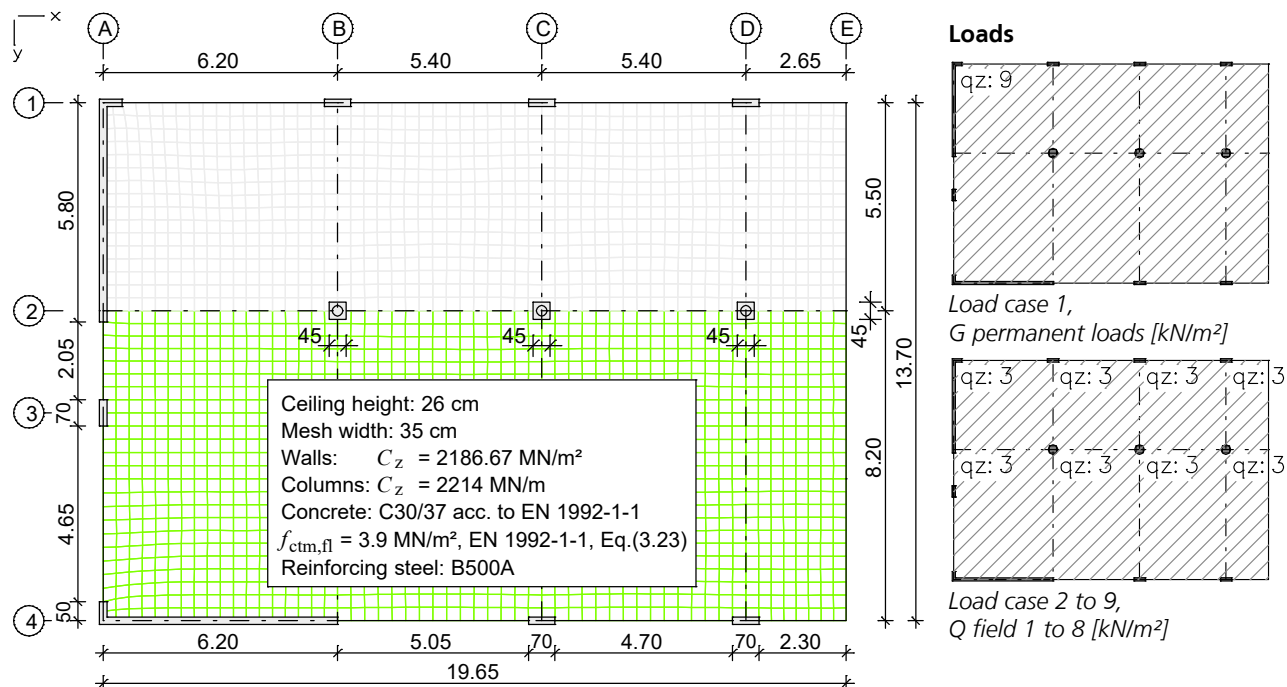
*Control:*

There are three ways to control the arc length method. This can be done via the number of increments to be calculated as well as via the first bifurcation point (inactive at constant stiffness) or, if a certain load level is to be achieved, via the load factor (correct to four decimal places).

# Examples

## Flat Ceiling With Cantilever in State II

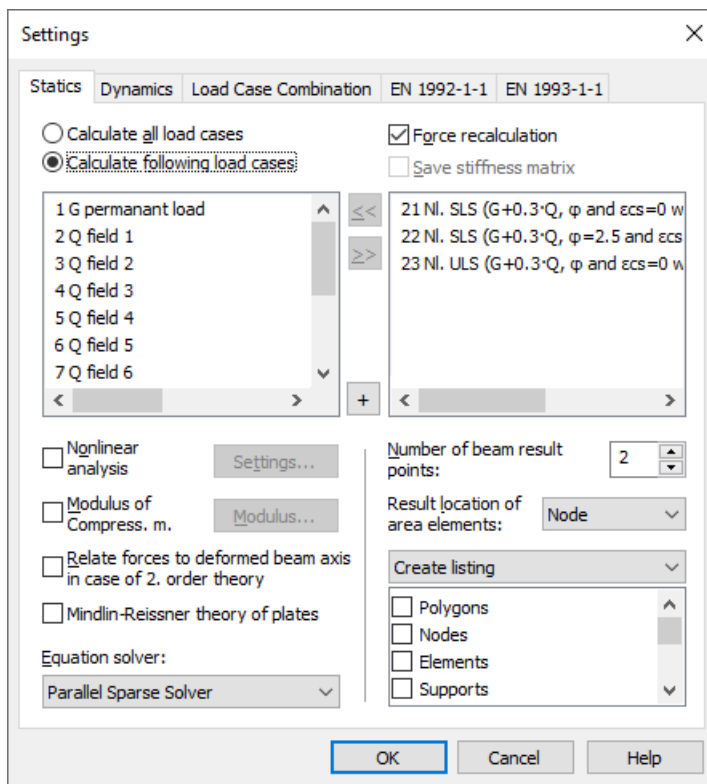
This example shows a ceiling slab that has already been analyzed in the *EN 1992-1-1 design manual*. For this slab, deformations in state II in the serviceability and the ultimate limit states are to be determined for comparison with the elastic calculation. serviceability limit state and ultimate limit state.



Element system and loads from "Beispiele zur Bemessung nach Eurocode 2 – Band 2: Ingenieurbau"

Due to the normal forces to be expected in the slab, shell elements are to be used for the nonlinear structural analysis and horizontal support supports should preferably be free of restraint. Because the longitudinal reinforcement has an essential influence on the deformations within a nonlinear structural analysis, a realistic reinforcement arrangement is to be ensured. For this reason, a linear-elastic static calculation of load cases 1 to 9 with subsequent design according to EN 1992-1-1 using the basic reinforcement given below is carried out. For the subsequent nonlinear system analysis, load cases 21 to 23 have already been entered with load groups.

In load cases 22, the nonlinear creeping of the concrete is additionally taken into account with the creep coefficient  $\phi = 2.5$  and the shrinkage of the concrete is assumed to be  $\epsilon_{CS} = -40 \cdot 10^{-5}$ .



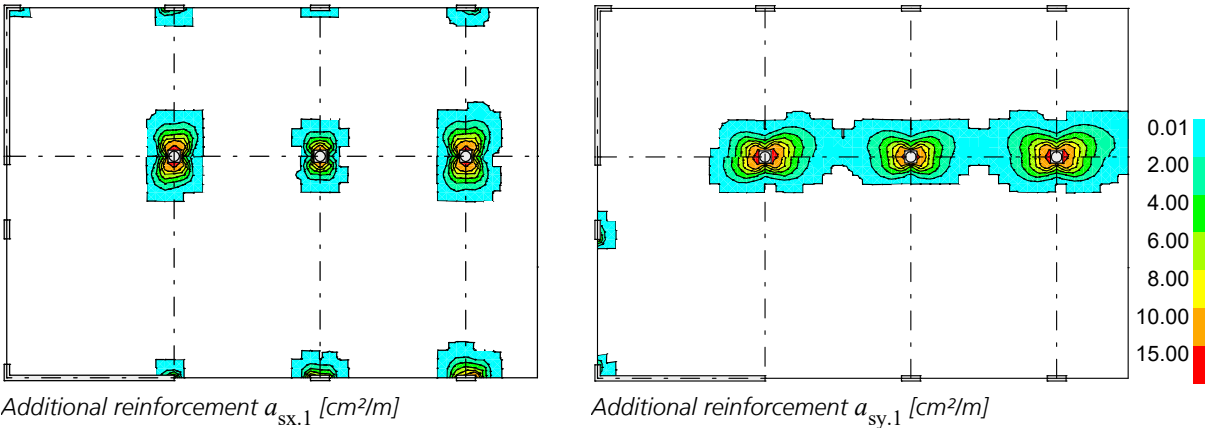
**Reinforcement for area elements**

No.	Lay.	Qual.	d1x [m]	d2x [m]	asx [cm <sup>2</sup> /m]	d1y [m]	d2y [m]	asy [cm <sup>2</sup> /m]	as fix	Rolling
1	1	500M	0.045	0.035	5.670	0.035	0.025	5.650		Warm
	2	500M			7.850			5.030		Warm
2	1	500M	0.045	0.035	5.670	0.035	0.025	5.650		Warm
	2	500M			7.850			11.340		Warm

as Base reinforcement  
 d1 Distance from the upper edge  
 d2 Distance from the lower edge  
 The z axis of the element system points to the lower edge

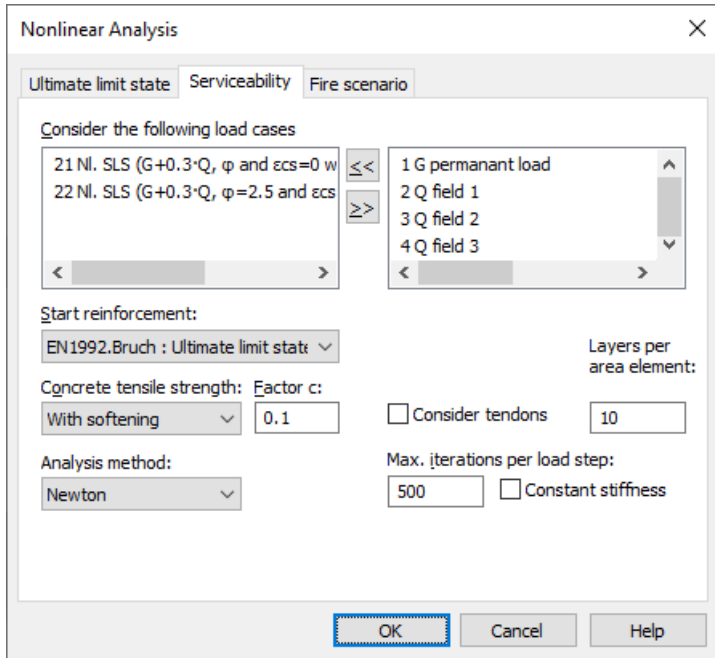
**Bending reinforcement from design of the permanent and temporary situation**

Only in the area around the columns and the corners does a reinforcement increase result in the upper reinforcement layers, which approximately corresponds to the additional reinforcement provided in the literature example.



**Analysis settings**

After the elastic static calculation of load cases 1 to 9 and the subsequent design according to EN 1992-1-1, the nonlinear system analysis is selected in the static analysis settings and the following settings are made.



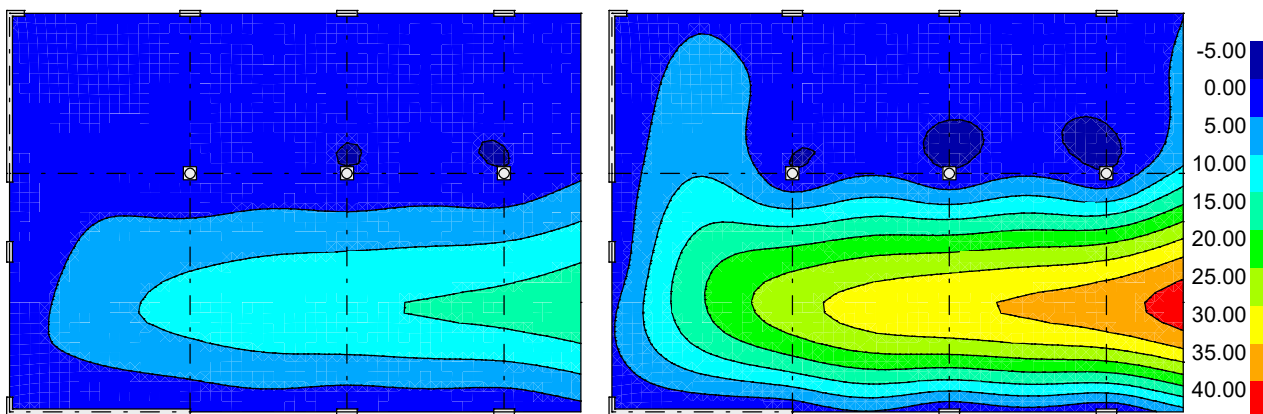
For the load cases (21 and 22) for serviceability, softening and a residual tensile strength of  $c \cdot f_{ctm,fl} = 0.1 \cdot 3.9 = 0.39 \text{ MN/m}^2$  are calculated in the tensile zone of the concrete.

Load case 23 for the ultimate limit state is calculated without considering the concrete tensile strength.

In order to increase the accuracy of the calculation, the error threshold is set to 0.1 % in the load groups. Therefore, the number of maximum iterations per load level is increased to 500.

**Deformations**

The different deformation results are compared below. For better comparison all non-linearly calculated load cases, including the one in the ULS, are calculated with the load  $G+0.3\cdot Q$ . In all cases a load factor of 1.0 can be achieved.



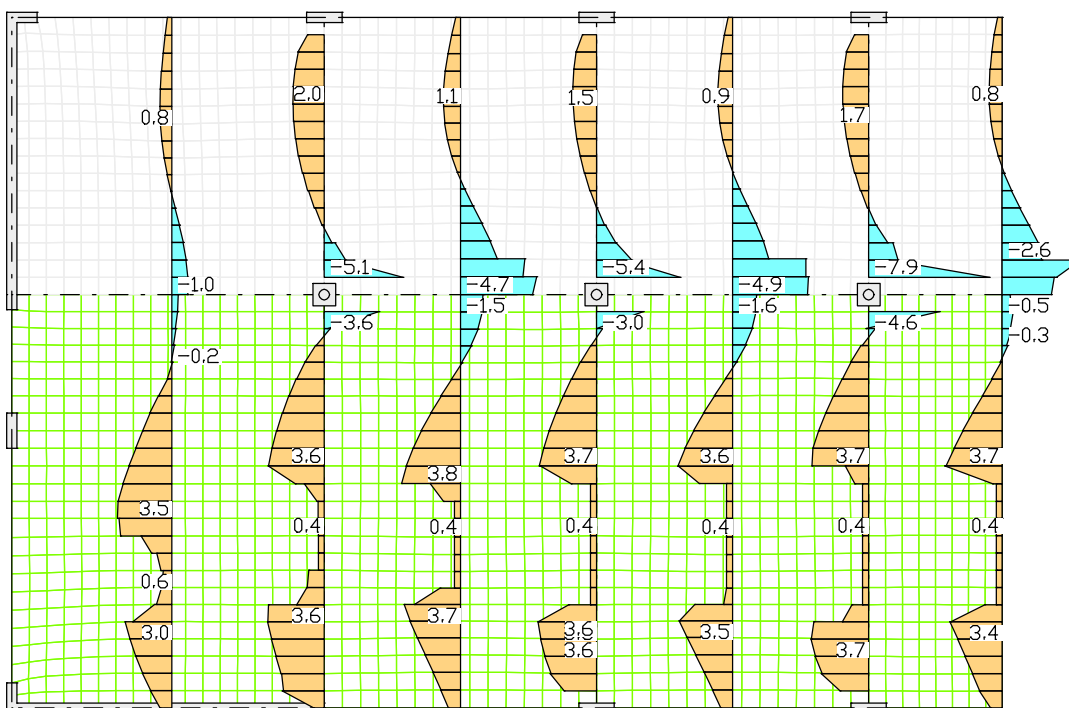
Load cases 21: Deformations  $u_z$  [mm]

Load cases 22: Deformations  $u_z$  [mm]

In the following, the maximum calculated deformations in state II are compared with the results determined in state I (see example for EN 1992-1-1):

Load case	max $u_z$ [mm]	
	Calculation	Literature
(11) $G+0.3\cdot Q$ with elastic material behavior (state I)	10	10
(12) Creeping ( $\varphi=2.5$ by LC 11) with elastic material behavior (state I)	23	-
(13) Total (load case 11 + load case 12) (state I)	33	-
21 NI. SLS ( $G+0.3\cdot Q$ , $\varphi$ and $\epsilon_{cs}=0$ with softening)	18	20
22 NI. SLS ( $G+0.3\cdot Q$ , $\varphi=2.5$ and $\epsilon_{cs}=-40\cdot 10^{-5}$ with softening)	43	40
23 NI. ULS ( $G+0.3\cdot Q$ , $\varphi$ and $\epsilon_{cs}=0$ without concrete tensile strength)	56	-

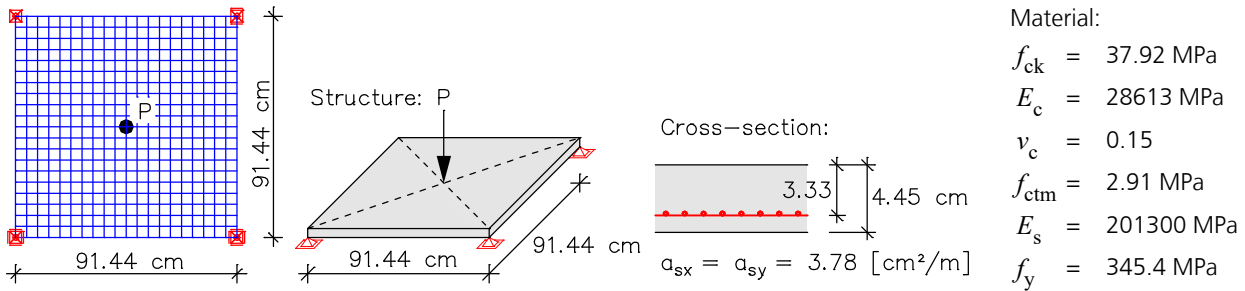
**Concrete stresses**



Load case 22: Concrete stresses [MN/m<sup>2</sup>] in y-direction at the bottom

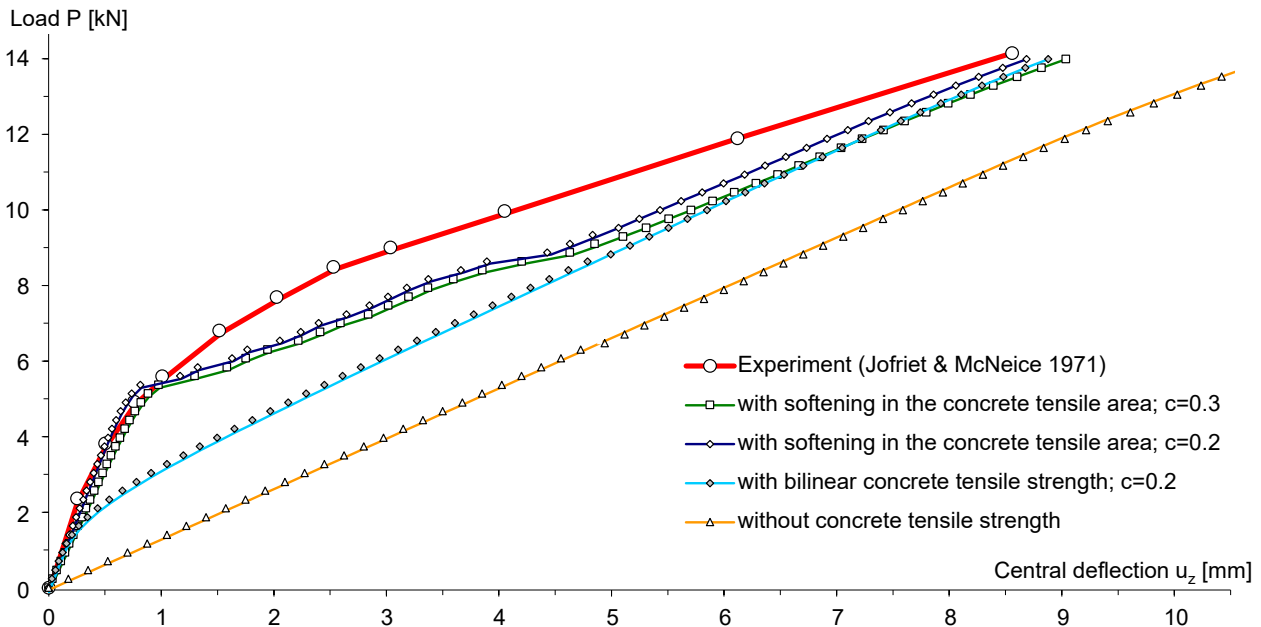
## Experiment of Fracture of a Reinforced Concrete Slab

The system described below was analyzed experimentally by Jofriet & McNeice in 1971 (taken from Krätzig & Meschke 2001 ). The goal is to demonstrate the plausibility of the achieved calculation results. For the crosscheck a system with 20x20 shell elements was used as illustrated below.



Element system with supports, load and the specified material parameters.

The load-displacement curve determined in the test for the slab middle is contrasted with the results of the static calculation in the following diagram. For the SLS the calculation method "Newton time steps" was chosen with 60 time increments and an error threshold of 0.1%. The material parameters were set according to the specifications of the authors. The material type CX-EN (EN 1992-1-1) was used for this.



Load-displacement curves from the crosscheck in SLS and experiment (Jofriet/McNeice)

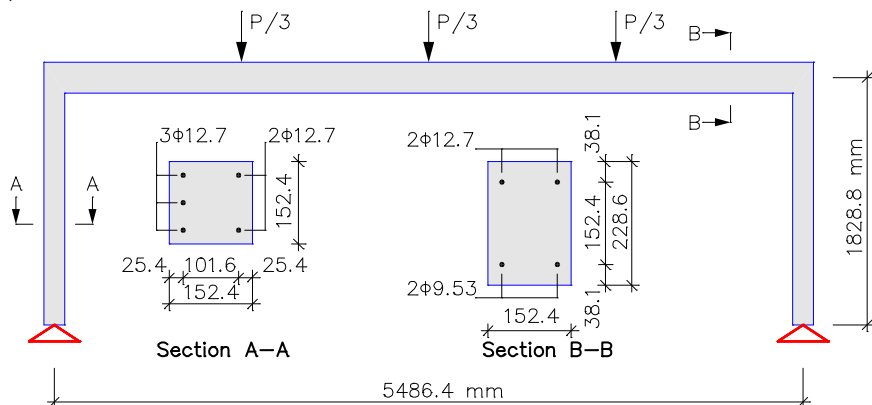
In order to demonstrate the mode of action of the methods implemented in the program for the concrete tensile stresses, four variants were calculated. The curve accounting for the concrete tensile strength with softening ( $c=0.3$ ) exhibits the closest agreement with the test results. The behavior at the beginning of crack formation as well as close to load-bearing capacity display a large level of agreement.

The curve for the bilinear behavior in the concrete tensile area was calculated with the value ( $c=0.2$ ). The stiffness of the slab is thus, as expected, underestimated at the beginning of crack formation. The load-bearing safety is, however, hardly influenced by this. This means that the calculation is on the safe side.

The curve for the 'naked state II' is for the most part determined by the characteristic curve of the reinforcing steel and thus has exhibits nearly linear behavior in the area under examination.

## Experiment of a Reinforced Concrete Frame

The following reinforced concrete frame was analyzed by Ernst, Smith, Riveland & Pierce in 1973 (taken from Krätzig & Meschke 2001). The goal is to demonstrate the plausibility of the achieved calculation results as well as the mode of action of different approaches for determining concrete tensile strength and tension stiffening. The static system with material parameters is shown below.



System with sections, load and dimensions [mm]

### Material parameters of the experiment:

Concrete:  $f_{cm} = 40.82 \text{ MPa}$ ,  $\epsilon_{c1} = -2.36$   
 Reinforcing steel:  $\varnothing 9.53: f_y = 472.3 \text{ MPa}$ ,  $E_s = 209 \text{ GPa}$  (cold-drawn)  
 $\varnothing 12.7: f_y = 455.0 \text{ MPa}$ ,  $E_s = 200 \text{ GPa}$  (hot-rolled)

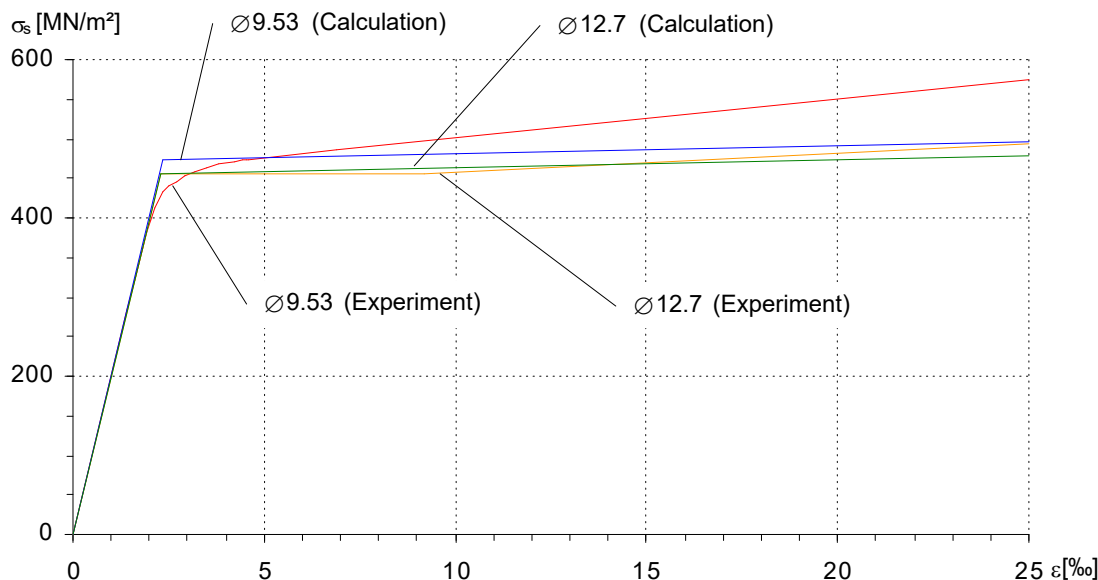
### Material parameters of the crosscheck:

Concrete: CX-EN (EN 1992-1-1),  $E_c = 33600 \text{ MPa}$ ,  $\phi_c = 0.07$   
 $f_{ck} = f_{cm} - 8 = 32.82 \text{ MPa}$ ,  $f_{ctm} = 2.2 \text{ MPa}$   
 Reinforcing steel:  $\varnothing 9.53: f_{yk} = 472.3 \text{ MPa}$ ,  $\varnothing 12.7: f_{yk} = 455.0 \text{ MPa}$   
 $E_s = 200 \text{ GPa}$ ,  $(f_t/f_y) \cdot k = 1.05$  (bilinear)

### Analysis settings:

SLS with softening ( $c = 0.2$ )  
 or without concrete tensile strength  
 Analysis method: Arc length  
 Control: Number of increments = 64 / 52  
 Iterations per arc length: 20  
 Stiffness: tangential  
 Initial arc length:  $\leq 0.03$

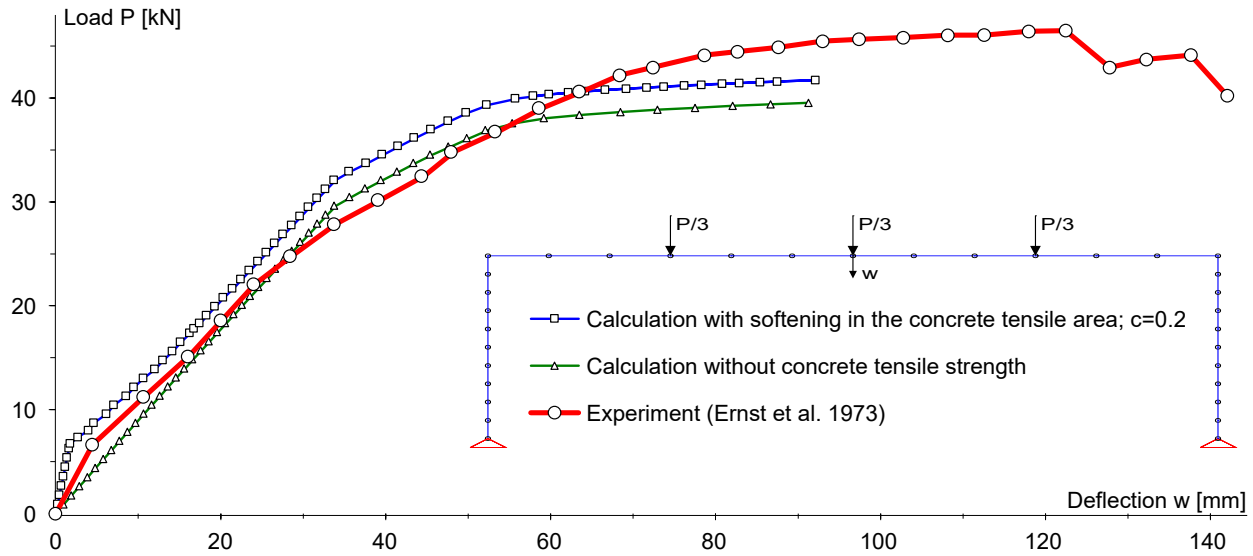
The following figure shows stress-strain-curves of the reinforcing steel used for the experiment and the crosscheck within the strain area up to 25 ‰.



Stress-strain-curves of reinforcement steel from experiment and calculation

The crosscheck was performed according to the arc length method and the second-order theory. Softening in the concrete tensile area was considered for the first calculation variant whereas no concrete tensile strength was assumed for the second variant. According to the experiment, cracking occurs at a limit load of  $P = 5 \text{ kN}$  corresponding to a concrete tensile strength of about 2.2 MPa. To take into account the stress-strain curves of the concrete according to the experiment, a concrete tensile strength  $f_{ctm}$  of 2.2 MPa and a creep coefficient  $\phi_c$  of 0.07 were assumed. The attaining of the system limit load is indicated by stability failure. The failure is caused by reinforcement flow at a load of approx. 35-40 kN and formation of a plastic hinge in the frame center. The slightly lower failure load is due to different stress-strain-curves for reinforcing

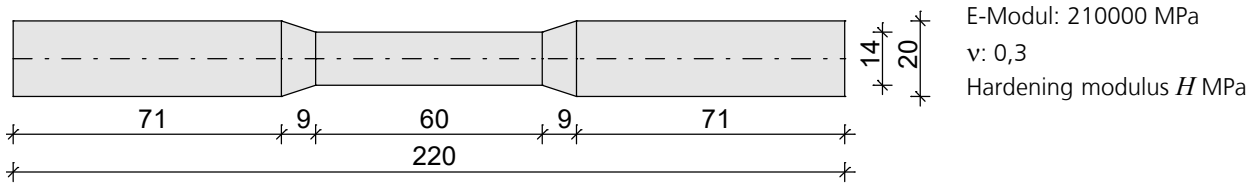
steel decisive for the experiment and the crosscheck. As expected, the calculation without tensile strength results in a lower bound for the load-displacement curve of the frame center.



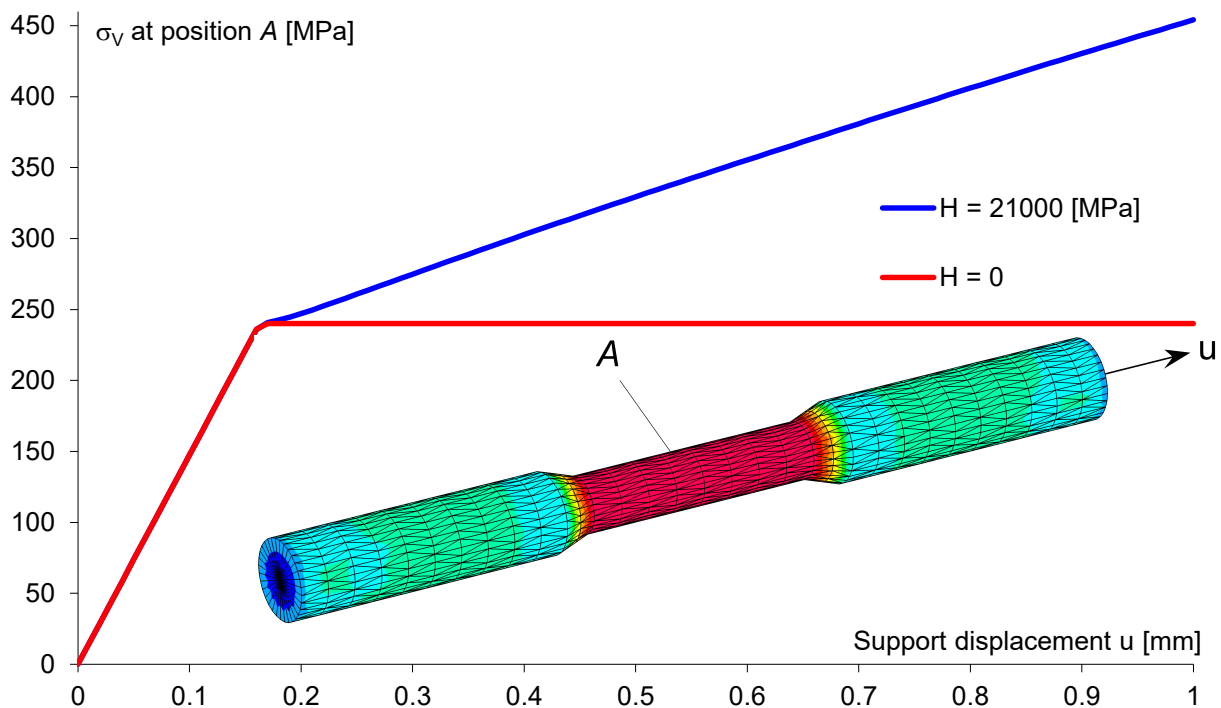
Load-displacement curves from the crosscheck and experiment (Ernst et al. 1973)

### Pull Test With Hardening

The recalculation of the pull test shows the behavior of steel with or without strain hardening. A solid model with tetrahedron elements VT10 is used. To simulate the hardening of the steel the material type *Frei* with the yield criterion *Huber-v.Mises* is used. The 220 mm long specimen, shown below, reacts with plastic behavior when the yield strength ( $f_{yk} = 240$  MPa) is exceeded, as can be easily seen from the stress-displacement curve. The calculation was performed deformation-controlled using the arc length method.



System with dimensions [mm]

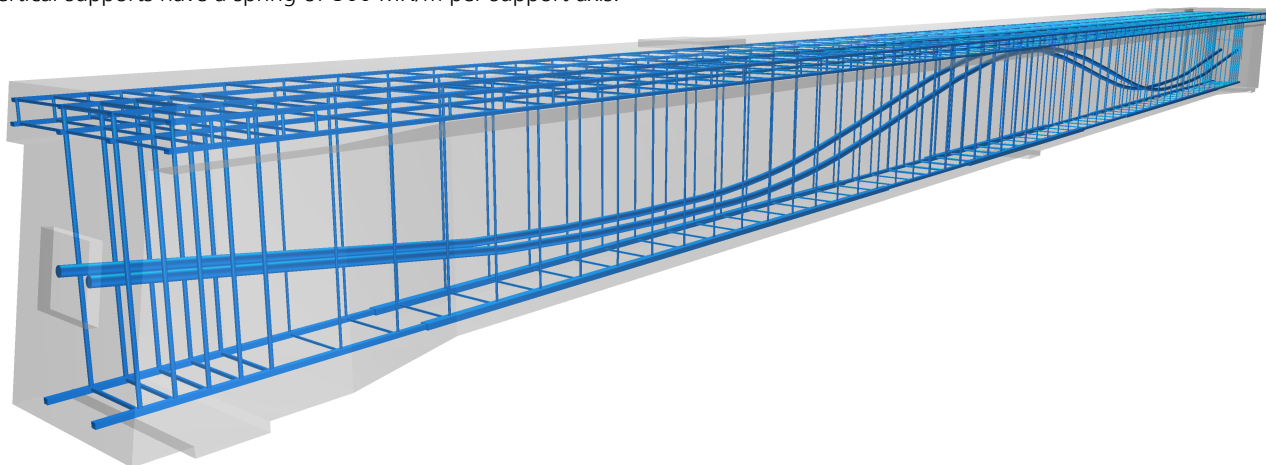


Evolution of the equivalent stress  $\sigma_v$  with increasing length change of the specimen

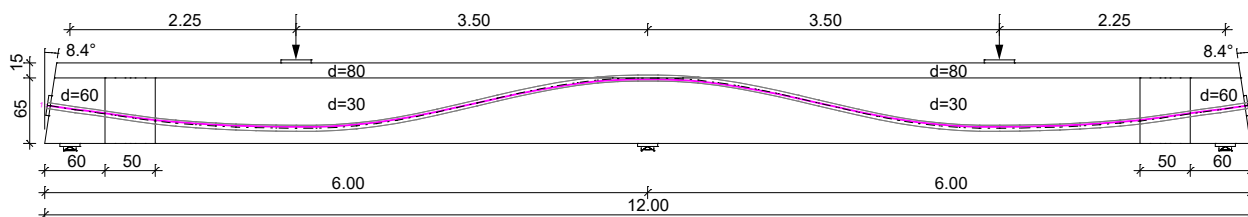


## Prestressed Concrete Structure

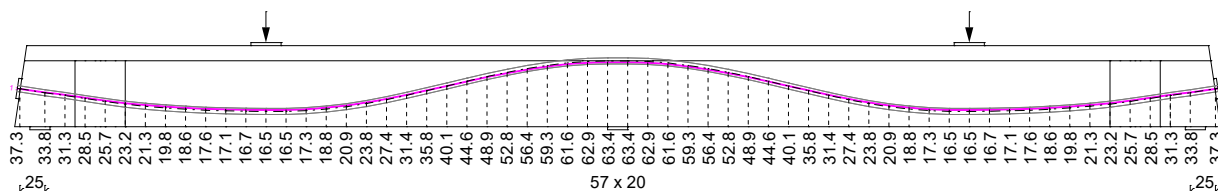
In this example an experiment of a prestressed two-span beam DLT 2.1 with subsequent bond has been recalculated, which has been carried out at the TU Dortmund as part of a research project of the BAST (Maurer et al., 2015). The dimensions of the test beam are shown below. The structure has a T-beam cross section with a constant height of 80 cm. Also the flange dimensions are constant about the length of the beam with  $b/h = 80/15$  cm. The web thickness within the fields and at the inner support is 30 cm. In the area of the end supports the web thickness increase to 60 cm. A mesh width of 5 cm was chosen for the calculation model. The material type *Frei* was selected for the solid model made of cuboid elements VQ83. For the reinforcing steel layers, steel beams were used for the longitudinal and stirrup reinforcement. The applied material properties according to *Hegger et al., BAST, Book B 150 (2020)* are given below. The tendon group is automatically implemented in the element stiffness matrices of the solid elements by the program during the load case calculation. The vertical supports have a spring of 500 MN/m per support axis.



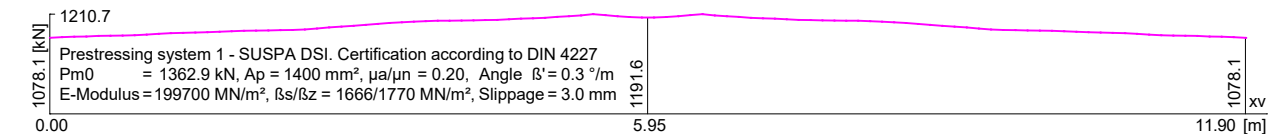
3D view of the calculation model with solid elements in the system viewer



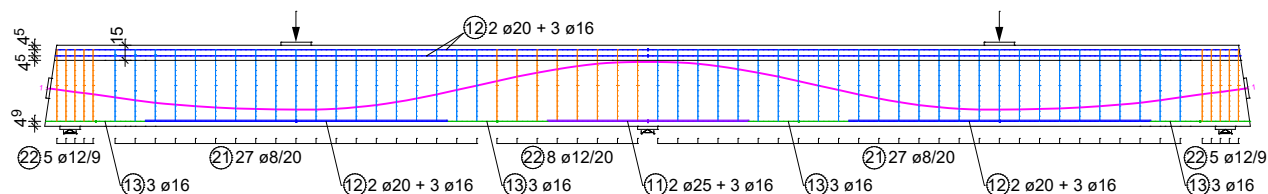
View with system dimensions and depiction of the tendon group



Distance between center axis of the tendon group and the bottom edge of the beam



Prestressing force of the tendon group applied for the calculation



View of the reinforcement (beams) of the calculation model

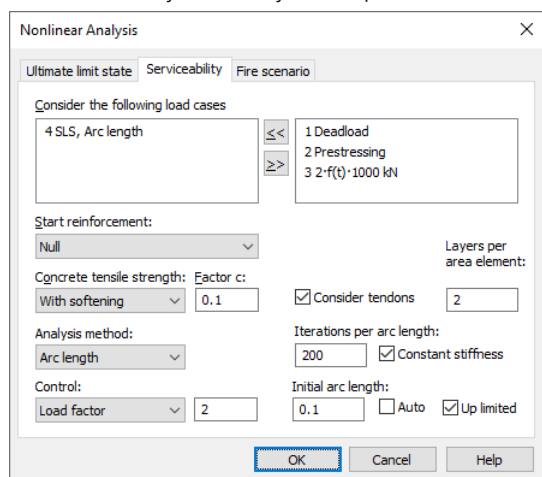
**Reinforcement**

Se-No.	Description	$A_s$ [mm <sup>2</sup> ]	E-Modulus [MN/m <sup>2</sup> ]	$f_{yk}$ [MN/m <sup>2</sup> ]
11	Longitudinal reinf. $\varnothing 25 + 1.5 \cdot \varnothing 16$ (bottom)	792	199501	555
12	Longitudinal reinf. $\varnothing 20 + 1.5 \cdot \varnothing 16$ (bottom)	616	200506	567
13	Longitudinal reinf. $1.5 \cdot \varnothing 16$ (bottom)	302	205200	554
14	Longitudinal reinf. $2 \cdot \varnothing 12$ (top)	226	200750	532
21	Stirrups $\varnothing 8$	50	184000	475
22	Stirrups $\varnothing 12$	113	200750	532

**Material properties of solid elements made of concrete**

Material type	Frei
Material model	Plasticity-damage
E-Modul [MN/m <sup>2</sup> ]	32860
Gamma [kN/m <sup>3</sup> ]	23
Poisson's ratio [-]	0.2
Compr. strength [MN/m <sup>2</sup> ]	$f_{cm} = f_{ck} + 8 = 41.15$
Tensile strength [MN/m <sup>2</sup> ]	$f_{ct} = 2.688$
Ultimate compressive strain [%]	2.363
Compr. fracture energy [kJ/m]	15
Tensile fracture energy [kJ/m]	0.09

The steel plates at the prestressing anchors, the supports and the presses were assumed to be linear-elastic. The nonlinear system analysis was performed with the following settings:

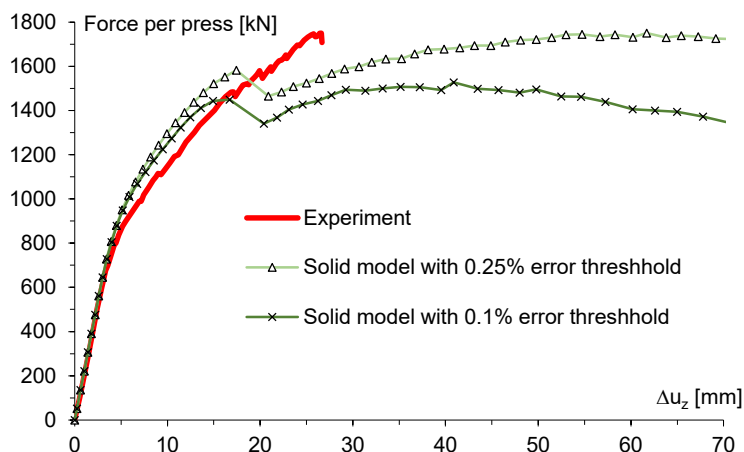


**Load data load case 4: SLS, Arc length**

Load group (GRL)  
 Theory: 1. order theory  
 No soil pressure > 0: No; No support reac. < 0: No; Error threshold [%]: 0.10  
 Additional global load factor: 1.00; Predeformation: 0  
 Consider concrete creeping in the nonlinear analysis: No

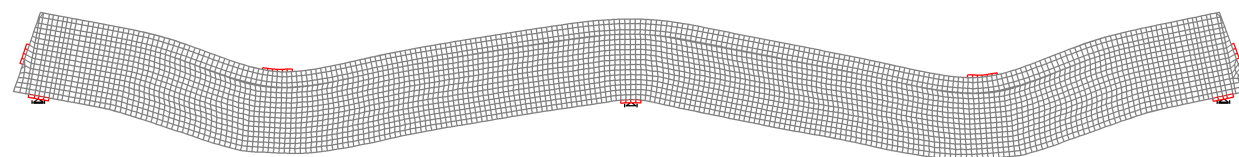
**Selected load cases**

No.	Label	Factor
1	Deadload	1
2	Prestressing	1
3	2·f(t)·1000 kN	1

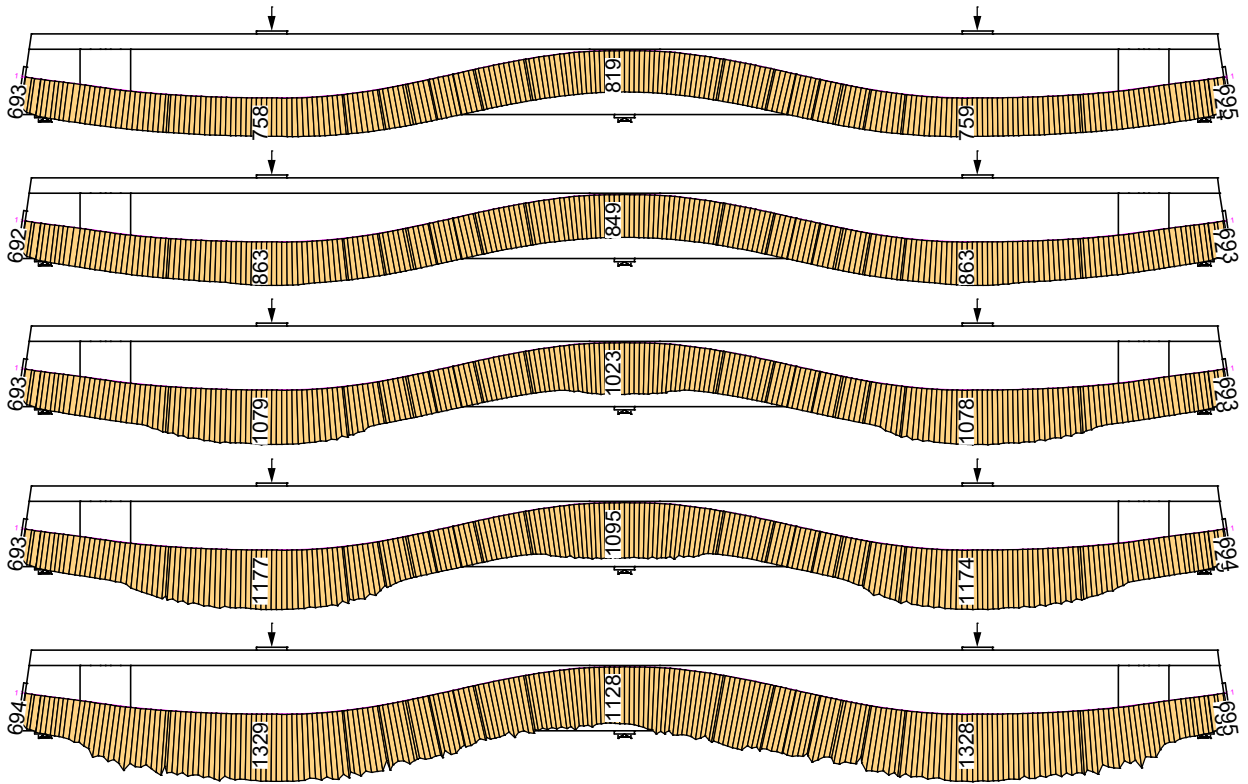


At about 700 kN, all load-displacement curves show a kink in their curve due to cracking. In the experiment, reinforcements were installed at a press load of 1743 kN because of the imminent failure of the structure. As can be seen in the figure on the left, the structure is very sensitive to the set error threshold. The following figures were calculated with the error threshold of 0.1%.

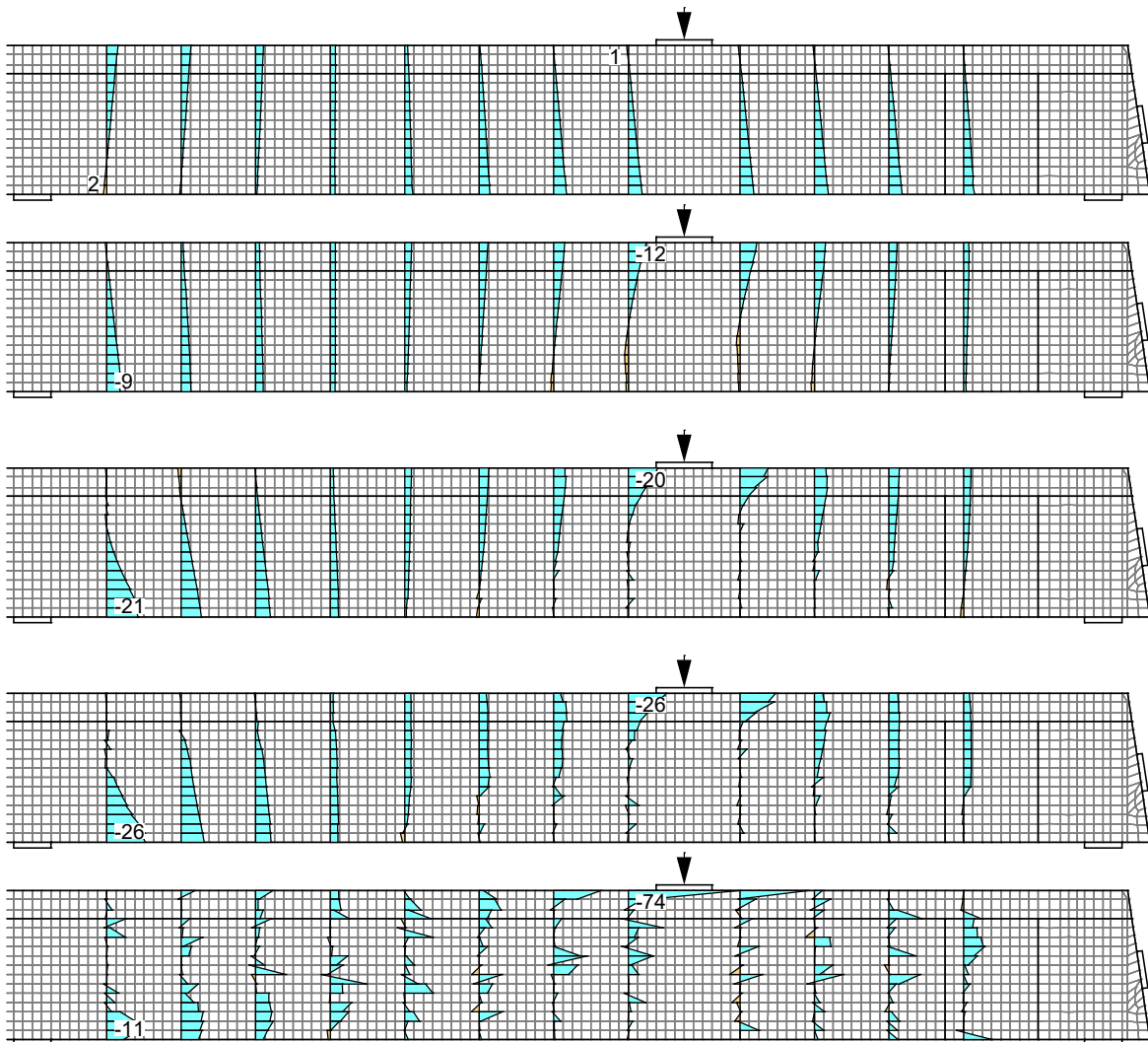
Load-deformation curves at the load location in field 1 from the comparative calculation and the experiment (Maurer et al., 2015)



Deflection for press forces of 1500 kN (20 fold superelevated)



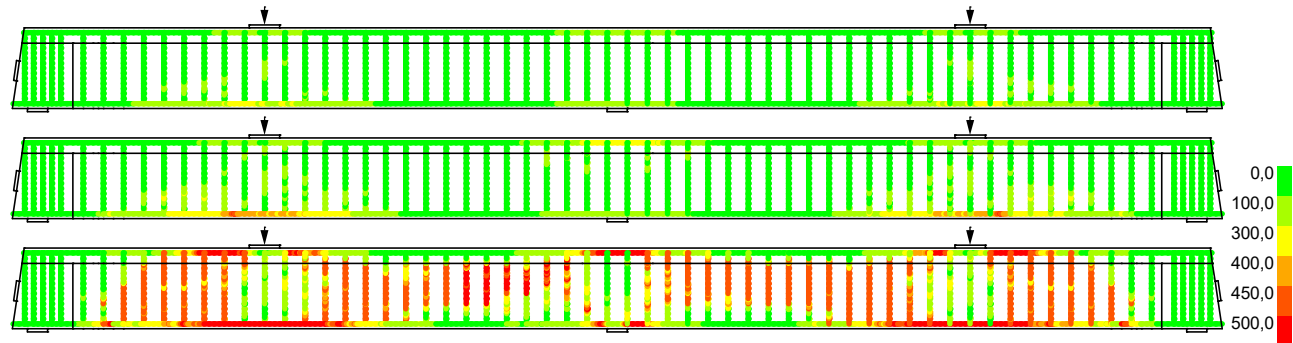
Prestressing steel stresses  $\sigma_p$  [MN/m<sup>2</sup>] for press forces of 0 / 800 / 1200 / 1400 / 1540 kN



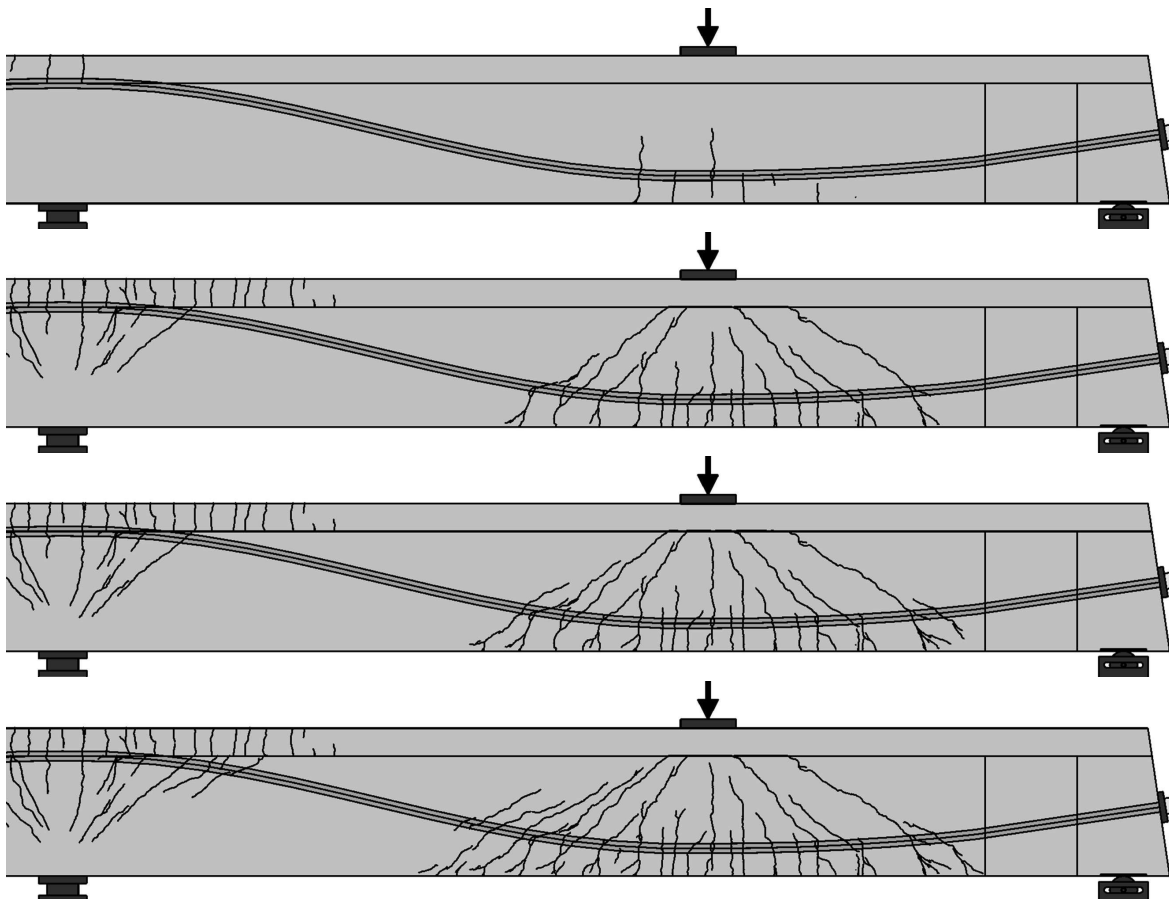
Horizontal stresses  $\sigma_x$  [MN/m<sup>2</sup>] between the stirrups at press forces of 0 / 800 / 1200 / 1400 / 1540 kN



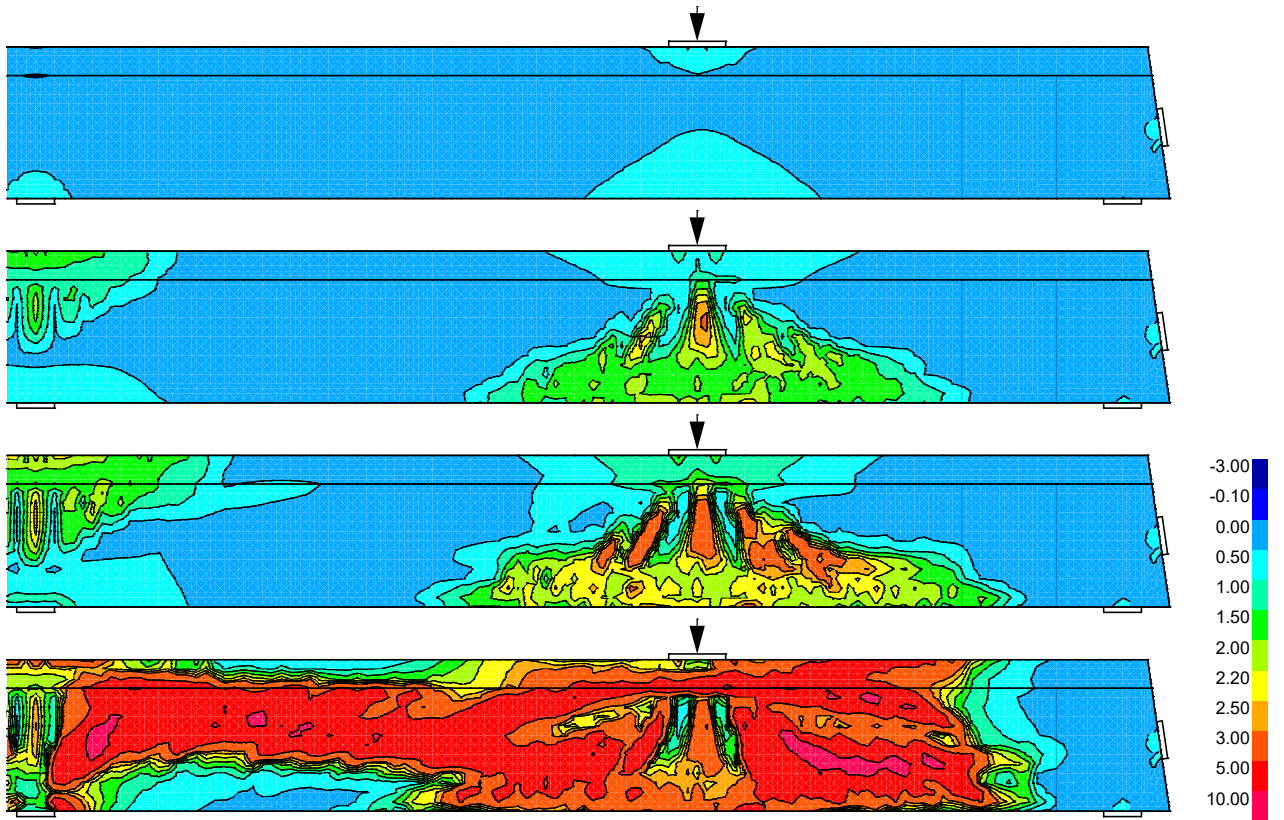
Measured stirrup stress  $\sigma$  [MN/m<sup>2</sup>] at press forces of 1200 / 1400 / 1540 kN BAST Book B 150, A.2.2.1 (c), (d), (e)



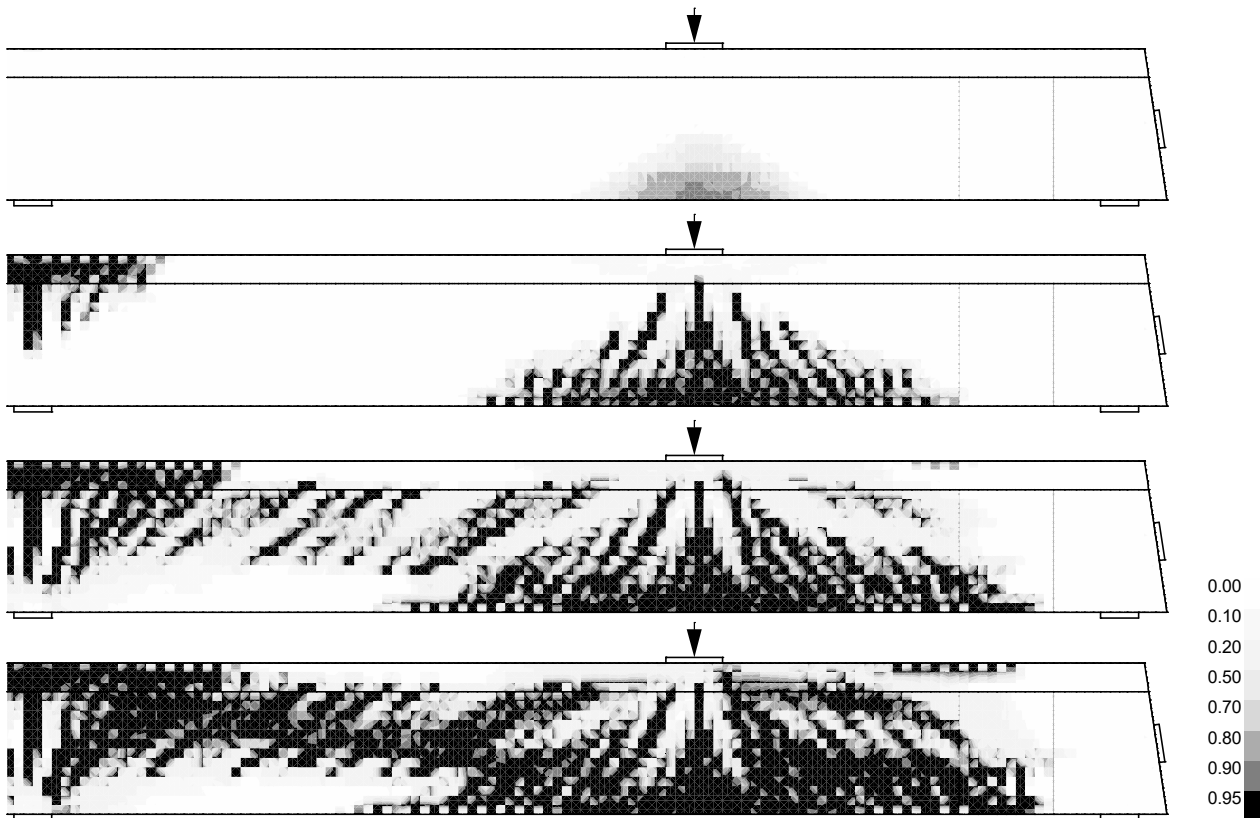
Stirrup stresses  $\sigma_v$  [MN/m<sup>2</sup>] at press forces of 1200 / 1400 / 1540 kN



Crack formation of the experiment for press forces of 800 / 1200 / 1400 / 1540 kN (BAST Book B 150, A.2.1.1 (b), (c), (e))



Principle strain  $\epsilon_1$  for press forces of 800 / 1200 / 1400 / 1540 kN



Damage for press forces of 800 / 1200 / 1400 / 1540 kN

## Reinforced Concrete Corbel

Based on the experimental results of *Mehmel and Freitag*, which were published in 1967 in the journal "Der Bauingenieur", the damage models implemented in *InfoCAD* are analyzed below. The main focus is on the numerically predicted load-bearing capacity and damage prediction ('crack propagation'). Differently reinforced rectangular and trapezoidal corbels were investigated in the test series. The load was applied either directly or indirectly via additionally laterally arranged auxiliary corbels. For the comparative study, a reinforced rectangular corbel with direct load application is used, since numerical results with different approaches for the definition of the material behavior are already available for this test (I.2, reinforcement according to *Mehmel*). Figure 1 shows the corbel created with *InfoCAD*, including the geometry, boundary conditions and reinforcement. A three-dimensional calculation is essential here, since a simplified finite element simulation with plain or shell elements would underestimate the load-bearing capacity by about 40% (*Krejci and Mang*, 1986).

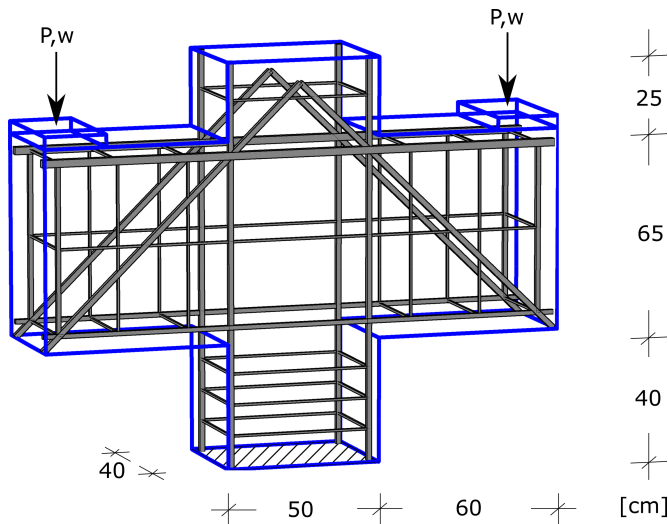


Figure 1: Geometry, boundary conditions and reinforcement

Two simplifications were made when modeling the reinforcement. The first simplification results from the facts that the distance of the stirrups in the lower part of the column was not specified by *Mehmel and Freitag* and that a larger number has no decisive influence on the load-bearing behavior. Instead of seven stirrups, only three equidistant arranged stirrups were considered. The second simplification concerns the longitudinal reinforcement in the lower and upper part of the crossbeam as well as the diagonal reinforcement. Reinforcement bars that are directly adjacent to each other were combined into one cross-section:

Crossbeam top:  $2 \times [2 \times \text{Ø}16 + 1 \times \text{Ø}14]$ , diagonal bars:  $4 \times [2 \times \text{Ø}14]$  (cf. *Mehmel and Freitag*, 1967). The reinforcement in the thickness direction was assumed to be  $\text{Ø}8$  everywhere. A detailed list of bar diameters used, quantities installed and strengths is given in Table 1.

Quantity	Type	Ø	Location	E-Modulus [MPa]	$f_{yk}$ [MPa]
9	Stirrups	8	Crossbeam: 5, Column: 4	206000	430
4	Bars	8	Crossbeam: 2x1 left, 2x1 right	206000	430
10	Bars	16	Crossbeam: 2x2 top, 2x1 bottom Crossbeam: 2x1 left, 2x1 right	206000	430
10	Bars	14	Crossbeam: 2x1 top, Diagonal: 4x2	206000	430
4	Bars	18	Column: 4x1	206000	430

Table 1: Quantity and diameters of reinforcement bars and material properties

In addition to determining the steel bar strengths ( $\sim 4400 \text{ kg/cm}^2$  corresponds to  $\sim 430 \text{ MPa}$ ), the concrete compressive strength was also determined during the tests. The result was a cube compressive strength of  $f_c = 226 \text{ kg/cm}^2$ . The modulus of elasticity and the tensile strength are not mentioned in the publication by *Mehmel and Freitag*. In order to compare the *InfoCAD* results with the other numerical results, the material parameters for concrete were chosen as follows:

$$E = 21870 \text{ MPa}, \nu = 0.2, f_c = 22.6 \text{ MPa}, f_t = 2.3 \text{ MPa}.$$

The numerical analysis is performed using both isotropic damage models available in *InfoCAD* (*De Vree* and *Mazars*). Table 2 shows the parameters for defining the corresponding damage functions or stress-strain relations.

N	Damage model	A	B	$A_t$	$B_t$	$A_c$	$B_c$
1	De Vree	0.45	400	-	-	-	-
2	Mazars	-	-	0.7	auto	0	auto

Table 2: Damage parameters

Taking advantage of the double symmetry, the calculation is performed with only one quarter of the reinforced concrete corbel. In addition, the influence of the element size on the numerical result is shown on the basis of two different discretizations (see Figure 2; Mesh A: 4097 tetrahedral elements and 324 beam elements, Mesh B: 94968 tetrahedral elements and 939 beam elements).

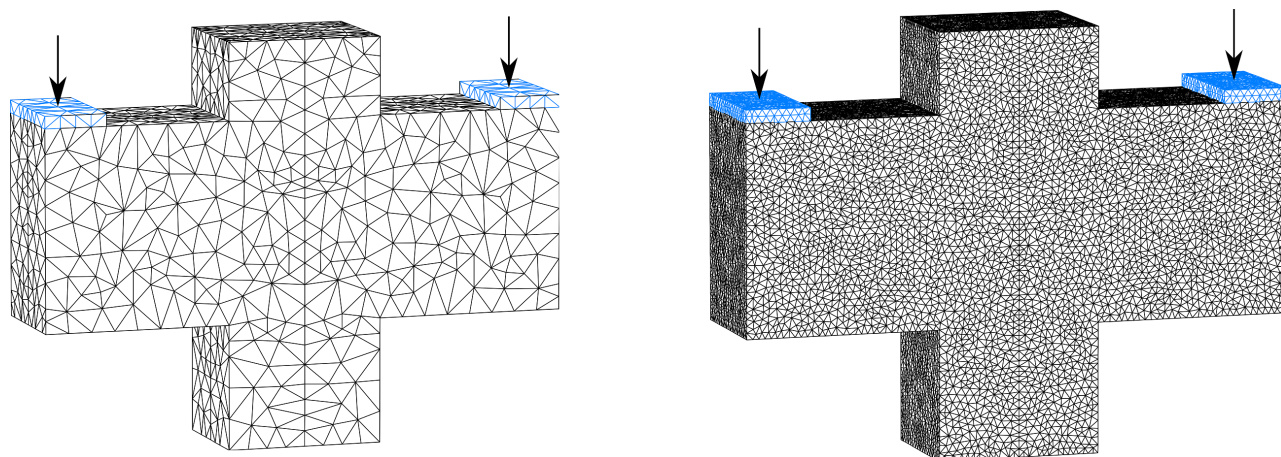


Figure 2: Discretization of the reinforced concrete corbel

The arc length method was used as the calculation method in all cases: Error threshold 1%; Iterations per arc length 20; Initial arc length 0.1; Up limited; Constant stiffness. A displacement-controlled load was applied. Steel plates are placed at the load application points.

Figure 3 illustrates the crack propagation (traced here) recorded photographically during test execution. The condition of the concrete within the white areas before reaching the failure load is not known, see also *Mehmel and Freitag (1967)*.

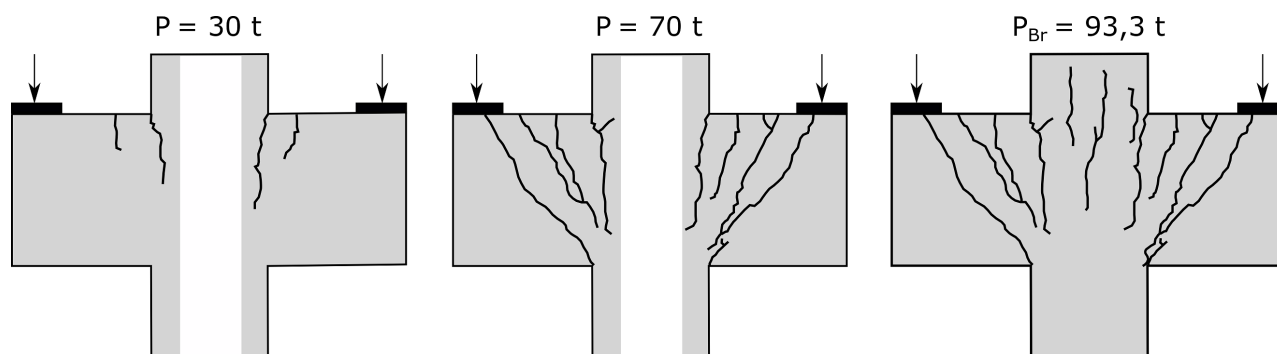


Figure 3: Crack propagation during test execution

The numerically determined damage distributions shown in Figure 4 are comparable to the crack distributions observed during the experiment. Due to the continuum-based damage formulation used in *InfoCAD*, the cracks are not defined discretely but smeared. Therefore, there is a direct correlation between element size and 'crack width'. A comparison of the damage distribution resulting from different discretization (see Figure 2) shows this very clearly, cf. Figure 4 top (Mesh A) and bottom (Mesh B).

The crack propagation associated with the load increase causes a significant increase of the stresses in the reinforcing bars in the damage-relevant areas of the structure, see Figure 5. While in the test the diagonal bars started yielding before all other bars, in the numerical simulation the internal stirrups of the connected crossbeams reach the yield stress first. This is due to the fact that numerically, contrary to the observations during the experiment, up to a load of 70 t, no gaping crack occurs from the point of load application to the indented corner at the other side of the load (cf. Figure 3 and 4, center). After verification of damage distributions and steel stresses, the two damage models are finally analyzed with regard to their predictions of the load-bearing capacity.

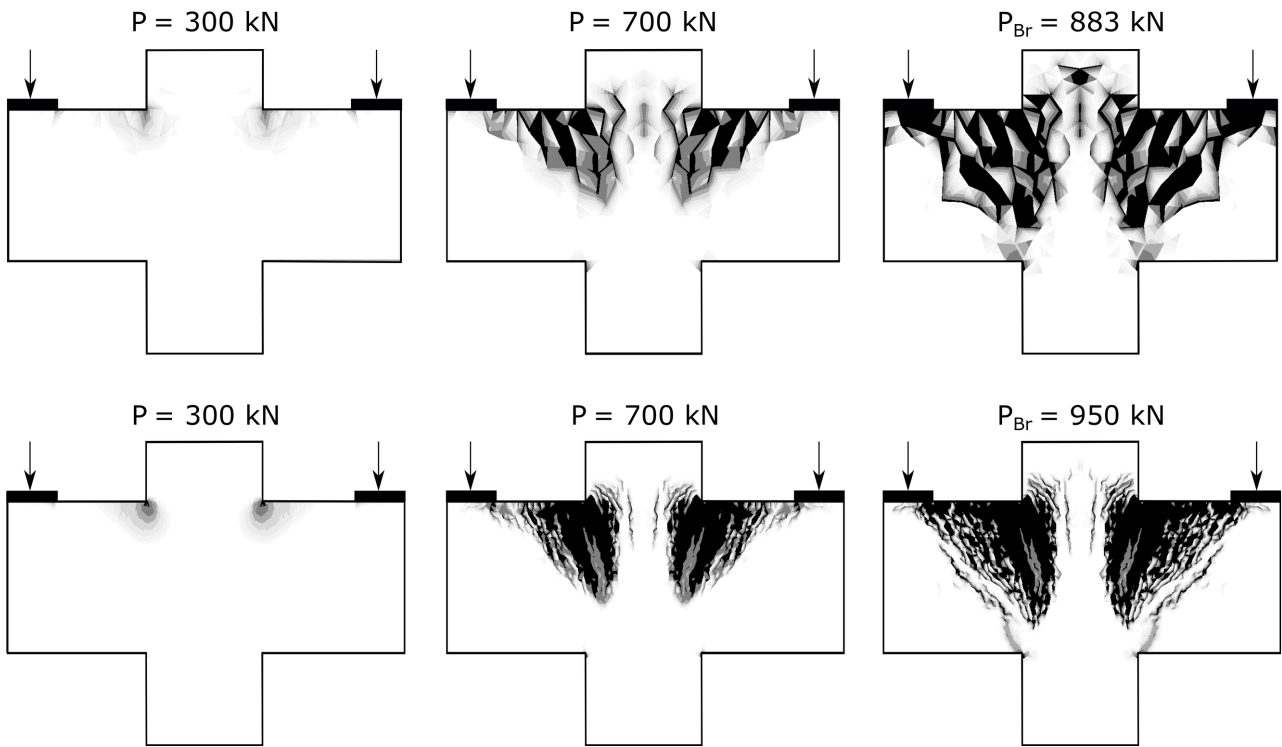


Figure 4: Damage distribution – De Vree ( $D = 0$ : white,  $D > 0,9$ : black)

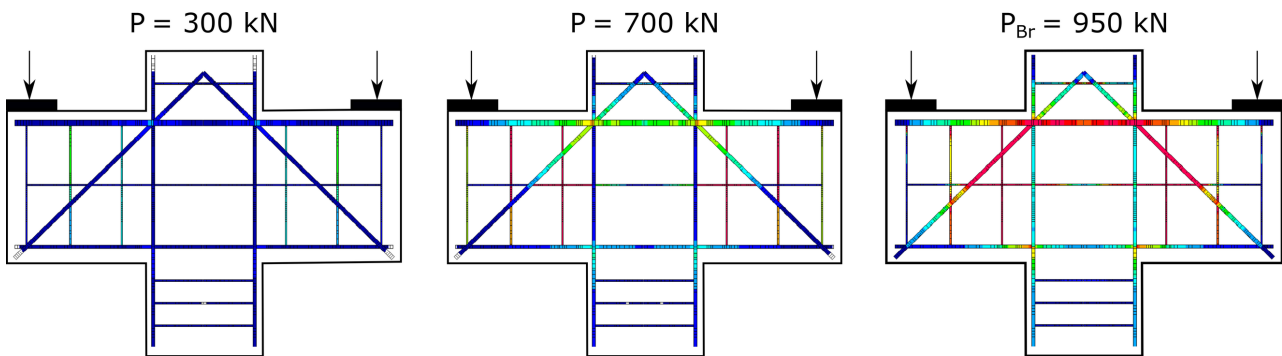


Figure 5: Steel stresses (Mesh B) – De Vree ( $\sigma_v = 0$ : white,  $\sigma_v = 430\text{MPa}$ : red)

A very good agreement is achieved here for the ultimate load. While an ultimate load of  $P_{Br} = 93.3$  tons was determined in the test, values between 883 kN and 962 kN are predicted numerically (depending on the damage model and discretization).

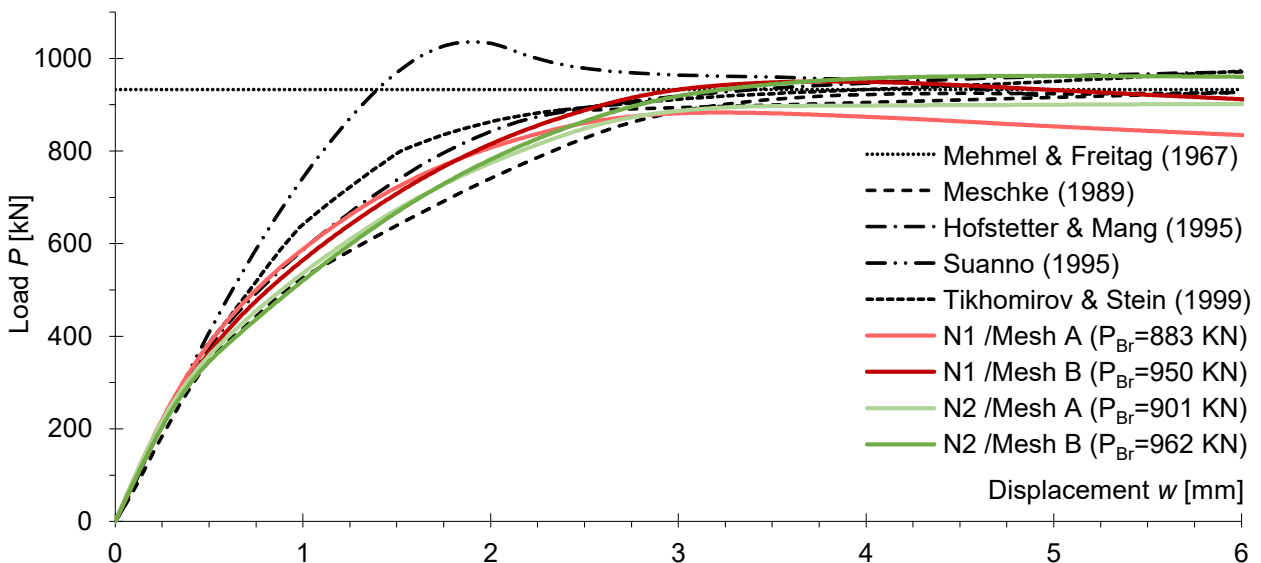


Figure 6: Comparison of experimentally determined ultimate load (93.3 t) and numerically determined load-displacement curves (chosen parameters, see Table 2)



Unfortunately, no comparison of load-deformation curves can be made because the load history was not recorded during the tests. Therefore, simulation results from a total of four different publications (Meschke 1989, Hofstetter & Mang 1995, Suanno 1995, Tikhomirov & Stein 1999) serve as reference solutions, see Figure 6. Taking the *InfoCAD* solutions into account, only minor differences between the individual load-displacement diagrams can be identified. First of all, all curves are almost congruent up to a load of approx. 350 kN. While the model of Meschke (1989) predicts an early degradation of the corbel stiffness with an almost ideal plastic yielding after reaching the ultimate load, the model used by Suanno (1995) results in only a minimal stiffness degradation before a relatively strong decrease after exceeding the maximum load occur. With the model of Tikhomirov & Stein (1999), no definite ultimate load can be determined. Here, an increase in load results in a linear increase of the load-bearing capacity. In the same way as in Meschke (1989), Hofstetter & Mang (1995) also show an ideal-plastic material behavior after reaching the ultimate load. The *InfoCAD* results are in between the results from the latter two publications. While the Mazars model also predicts nearly ideal-plastic yielding after reaching the maximum load, the parameters chosen when using the *De Vree* model imply a slight degradation of the corbel stiffness. To achieve ideal-plastic material behavior, the parameter  $A$  (*De Vree*) or  $A_{t,c}$  (*Mazars*) must be set to zero.

## Unreinforced Concrete Structures

The validation of the implemented elasto-plastic damage model (plasticity-damage), which is available in *InfoCAD* for solid elements of the material type *Frei*, is done, among others, with the help of the experimental results of Petersson (1981) and Winkler (2001). Their experimental results serve as a reference and are often used in the development of nonlinear material models to verify numerically determined solutions, such as load-displacement curves. Both structures discussed below are unreinforced. First, Petersson's 3-point bending beam (with a notch in the center of the span) is analyzed. Due to the geometry and boundary conditions, the 'crack' that occurs with increasing load is in *Mode I* only (opening of the crack edges perpendicular to the crack plane). If additional sliding of the 'crack edges' (*Mode II*) is observed, the 'crack' is in the so-called *mixed mode*. This type of crack loading occurs with Winkler's L-shaped slab.

### 3-point bending test according to Petersson

The dimensions of the single-span beam with a notch in the center investigated by Petersson (1981) in his dissertation, as well as its boundary conditions and discretization with solid elements (VQ83) are shown in Figure 1. The width of the notch (2 cm) corresponds to the smallest mesh width below the load application (Rots et al., 1985).

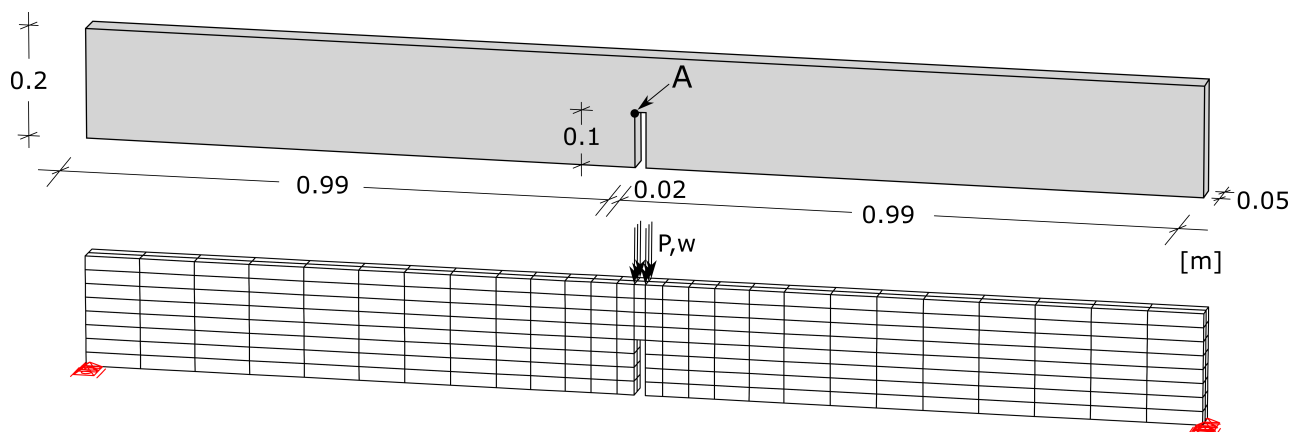


Figure 1: Geometry, boundary conditions and discretization (424 elements)

The point A drawn in the figure below is used to evaluate the inelastic and plastic strains and the damage over the entire load path. The load on the statically determinate supported 3-point bending beam was applied in a displacement-controlled manner.

Parameters	Material		Arc length method	
		E-Modulus [MPa]	30000	Error threshold [%]
	Poisson's ratio [-]	0.2	Number of iterations	200
	Compressive strength $f_c$ [MPa]	43	Initial arc length	0.01
	Tensile strength $f_t$ [MPa]	3.33	Upward limited	yes

Table 1: Parameters (material and arc length method)

The arc length method was used as the calculation method. Together with the material parameters used, the selected error tolerance, number of iterations and initial arc length are shown in Table 1. In order to show differences between the implemented nonlinear solid models (here: combined model according to Lubliner and Lee & Fenves as well as pure damage according to *de Vree*), the influence of the *de Vree* model parameters on the numerically predicted ultimate load and softening is analyzed in addition to the validation of the elasto-plastic damage model (in *InfoCAD*: plasticity-damage). A total of four different simulations are performed for this purpose. All model parameters used are listed in Table 2. While in

the case of the combined model (N1), the fracture energies measured by means of concrete specimens during the test (concrete age: 28 days / average values:  $G_t = 124 \text{ N/m}$ ,  $G_c = 23 \text{ kN/m}$ ) are directly included in the numerical calculation and determine both the ultimate load and the softening process, in the case of the *De Vree* model (currently no consideration of fracture energy regularization), an adjustment of the stress-strain relationship must be made via parameters A and B.

N	Material model	A	B	$G_t$	$G_c$	$\epsilon_{c1}$
1	Plasticity-damage	x	x	124 N/m	23 kN/m	2.2
2	Damage / De Vree	0.98	10	x	x	x
3	Damage / De Vree	0.98	100	x	x	x
4	Damage / De Vree	0.98	1000	x	x	x
5	Damage / De Vree	0.98	230	x	x	x

Table 2: Parameters material models

The difference between the two models or in their handling can be seen in the load-displacement curves (*InfoCAD*) illustrated in Figures 2 and 3.

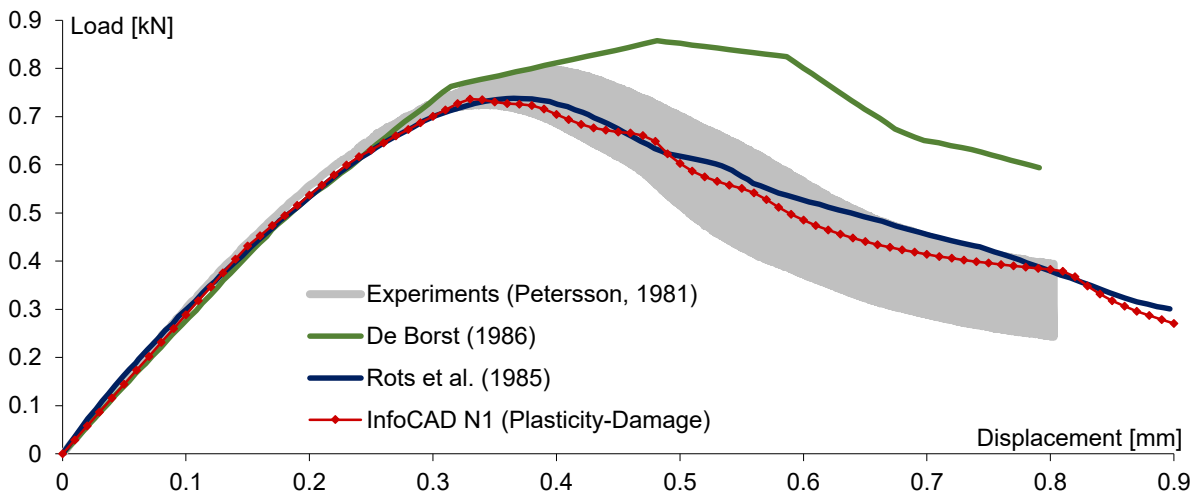


Figure 2: Comparison of experimentally and numerically determined load-displacement curves. (*InfoCAD*: plasticity-damage)

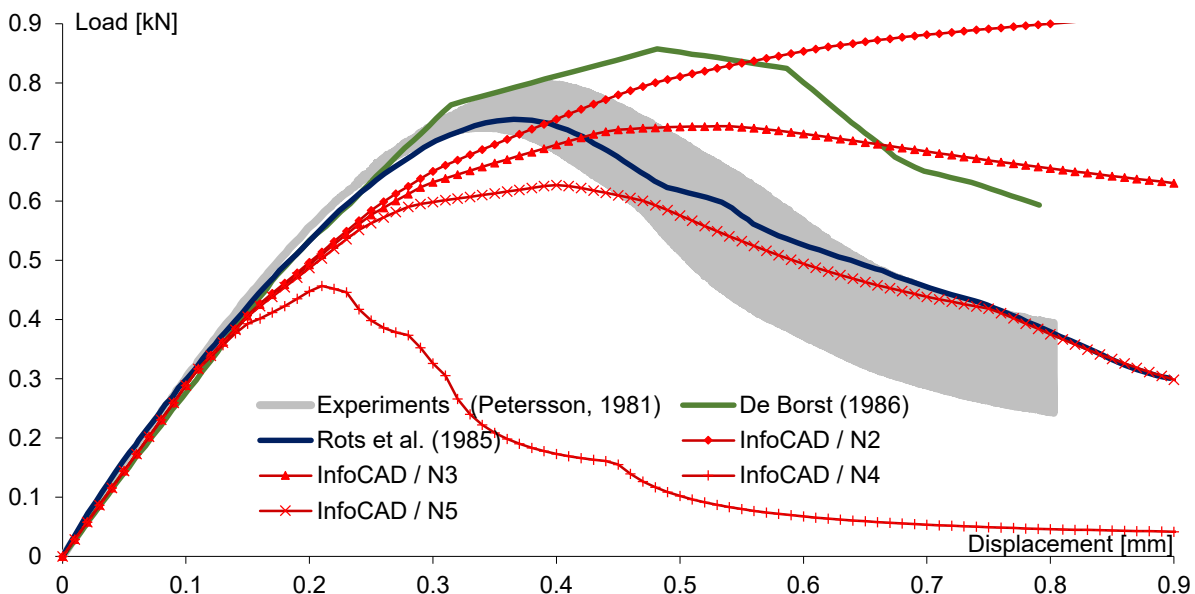


Figure 3: Comparison of experimentally and numerically determined load-displacement curves. (*InfoCAD*: *De Vree*)

Using the elasto-plastic damage model (plasticity-damage) and the fracture energies ( $G_t$ ,  $G_c$ ) known from experiments, very good agreement is obtained with the load-displacement curves recorded during the experiment. An adjustment of the model parameters is not necessary. Both, the ultimate load and the softening material behavior are numerically predicted with sufficient accuracy. This is also shown by a comparison with other numerical results (*De Borst*, 1986; *Rots et al.* 1985). The situation is different with the results shown in Figure 3. It is obvious that the *De Vree* model requires a parameter study for a realistic representation of a particular material behavior. The ultimate load and the degree of stiffness reduction are

strongly coupled. Modeling a large increase in damage results in a reduced ultimate load, while a slow increase in damage results in an increased ultimate load (controlled by parameter B).

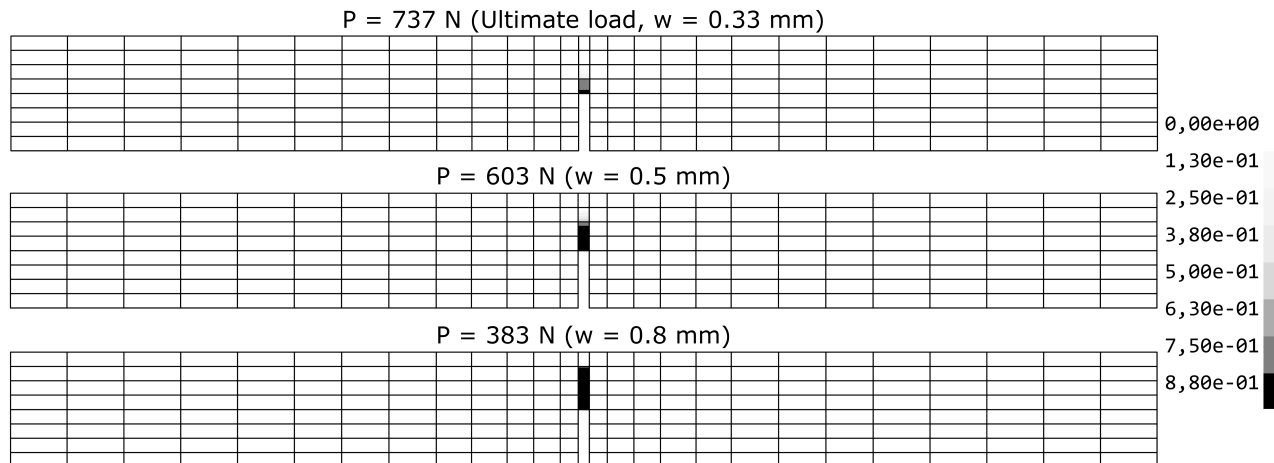


Figure 4: Propagation of damage with increasing load (plasticity-damage)

For further analysis, but also to show possible result representations in *InfoCAD*, further numerical results of the combined model are evaluated here in addition to the load-displacement curve (see in Figure 2). These include the damage occurring with increasing load (Figure 4) as well as the inelastic ( $\epsilon^{in}$ ) and plastic ( $\epsilon^{pl}$ ) strains (Figure 5).

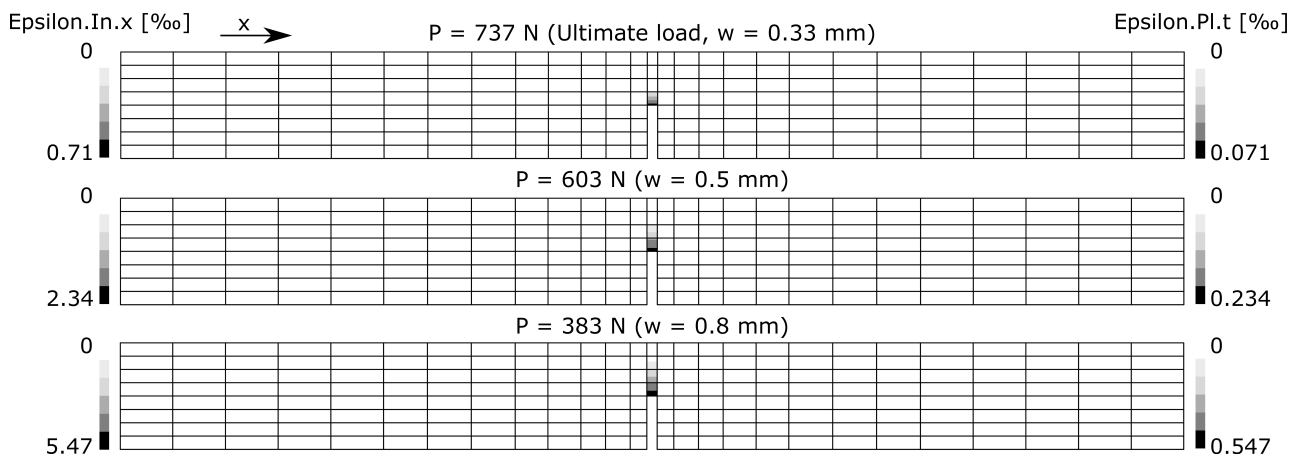


Figure 5: Propagation of inelastic (*In.x*) and plastic (*Pl.t*) strains with increasing load (plasticity-damage).

In addition, the relationship between the latter quantities (for uniaxial tension:  $\epsilon_t^{pl} / \epsilon_t^{in} = 0,1$ , see combined model, Figure 17), can be directly derived from the results of the *Peterson* experiment.

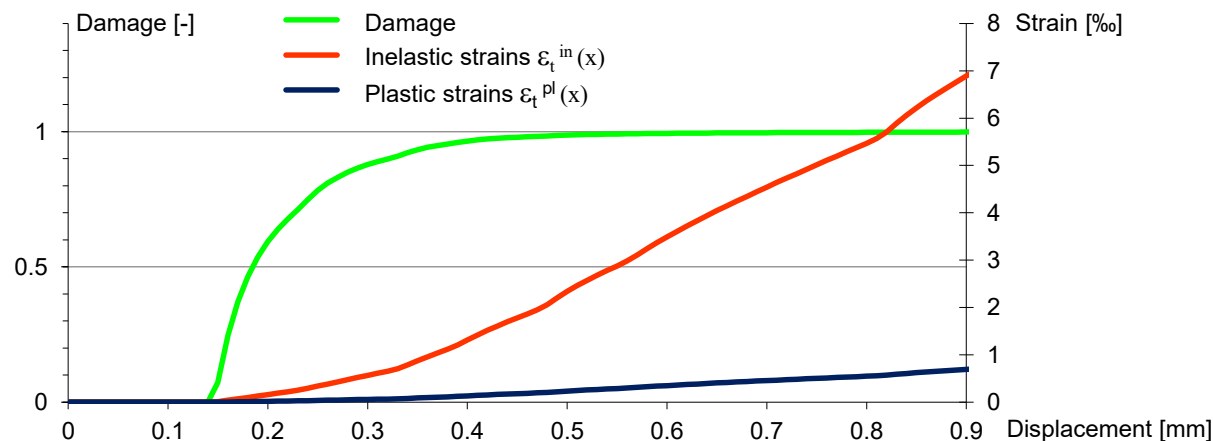


Figure 6: Propagation of damage and inelastic and plastic strains at point A. (see figure 1, plasticity-damage)

Either by looking at the contour plots, which also vividly illustrate the different degradation state within the entire structure, including when the ultimate load is reached ('crack initiation and crack propagation' above the notch), or by looking at Figure 6, which illustrates the propagation of both the damage and the inelastic and plastic strains over the entire loading path (the evaluation is done at point A, see Figure 1).

**L-slab according to Winkler**

In order to confirm the very good reproduction capability of the elasto-plastic damage model (plasticity-damage) with respect to stiffness degradation and damage propagation, as established in the analysis of the single-span beam (Pettersson, 1981), the numerical results are verified on the basis of a second series of tests. This is an unreinforced L-shaped slab investigated by Winkler (2001) in his dissertation.

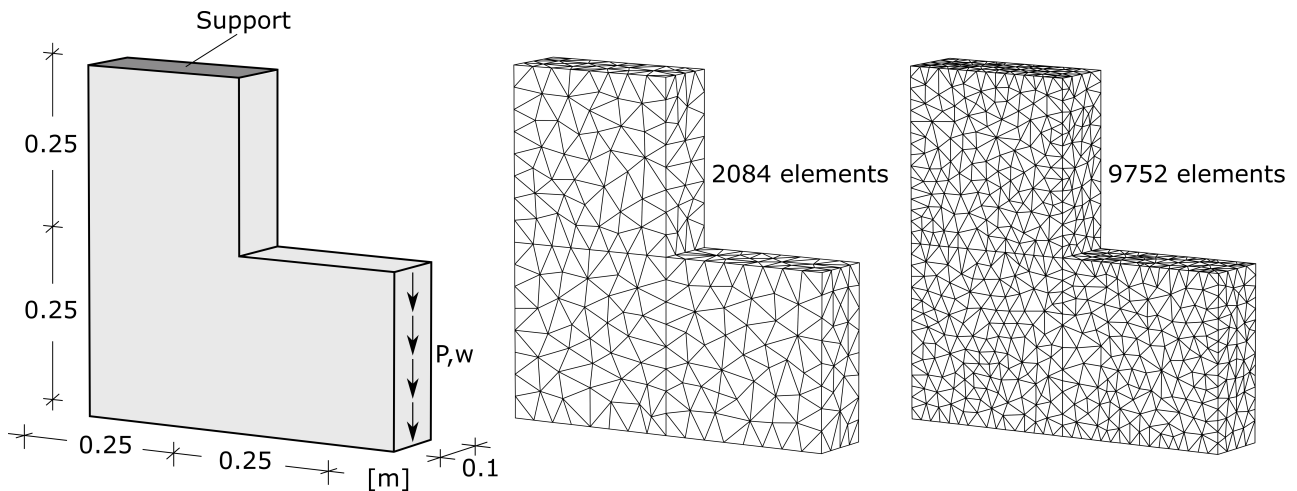


Figure 7: Geometry, boundary conditions and discretizations

The geometric dimensions and boundary conditions are shown in Figure 7. Furthermore, two discretizations with different mesh widths are shown. Numerical analyses should provide information on the influence of the element size on the softening behavior. In order to minimize the stresses and thus any damage in the support area, the L-slab is supported in a statically determinate manner. In addition, in analogy to the 3-point bending beam, the load is also applied here in a displacement-controlled manner. As a result of the arc length method, which is again used as a calculation method, the representation of load-displacement curves as well as the increasing damage at selected nodes of the discretized structure over the course of the load is directly possible. The same applies to the inelastic and plastic strains.

Parameters	Material		Arc length method	
	E-Modulus [MPa]	25850	Error threshold [%]	0.01
Poisson's ratio [-]	0.18	Number of iterations	50	
Compressive strength $f_c$ [MPa]	33	Initial arc length	0.02	
Tensile strength $f_t$ [MPa]	2.7	Upward limited	yes	

Table 3: Parameters (material and arc length method)

However, in contrast to the evaluation of the results of the single-span beam, in the following only load-displacement curves are verified or compared with the curves recorded during the experiment. However, two contour plots are included, which clearly show the relationship between deformation and damage ('crack progression'). Table 3 contains relevant material parameters and all necessary parameters for controlling the calculation with the arc length method. On the other hand, two further material parameters, which represent input parameters of the combined model, namely the two fracture energies (tension:  $G_t$ , compression:  $G_c$ ), are shown in Table 4. The default values defined in *InfoCAD* ( $G_t = 100 \text{ N/m}$ ,  $G_c = 15 \text{ kN/m}$ ,  $\epsilon_{c1} = 2.2 \text{ p.m.}$ ) were not changed for the FE simulations.

There is also a comparison of combined model (plasticity-damage) and pure damage model (*De Vree*) in this validation example. The chosen parameters (*A* and *B*) for generating the uniaxial stress-strain relation according to *Peerlings* (1999) can also be taken from Table 4.

N	Material model	<i>A</i>	<i>B</i>	$G_t$	$G_c$	$\epsilon_{c1}$
1	<i>Damage / De Vree</i>	0.98	140	x	x	x
2	<i>Plasticity - damage</i>	x	x	100 N/m	15 kN/m	2.2

Table 4: Parameters material models

Again, using the elasto-plastic damage model shows a very good correlation between numerically and experimentally determined load-displacement curves, see Figure 8. The ultimate load is predicted with sufficient accuracy for both discretizations. However, due to the smaller characteristic length in the case of the fine mesh (9752 elements) and the associated increased element-specific fracture energy, a minimally lower stiffness reduction in the post-failure region can be observed.

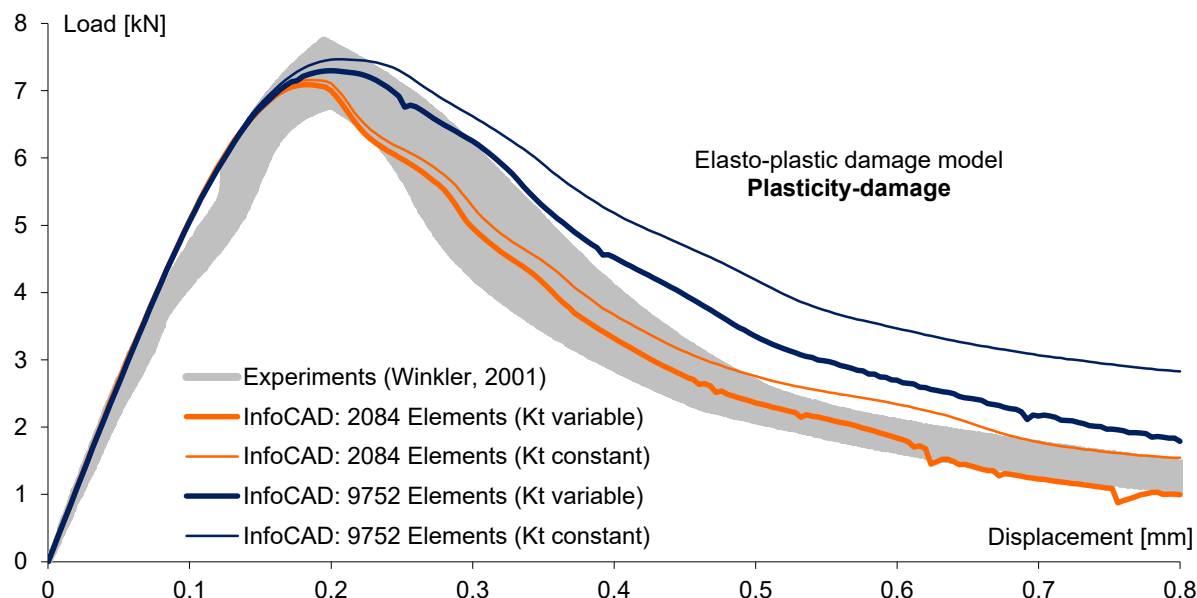


Figure 8: Comparison of experimentally (Winkler, 2001) and numerically (InfoCAD) determined load-displacement curves

For both FE meshes, the analysis was also performed with constant stiffness (linear stiffness method). As expected, the curves are above those with variable stiffness. However, especially for complex structural analyses and stress conditions, constant stiffness is recommended for the calculation. Figure 9 shows the vertical displacement (left) together with the damage distribution (right) for the last load step ( $w = 0,8$  mm).

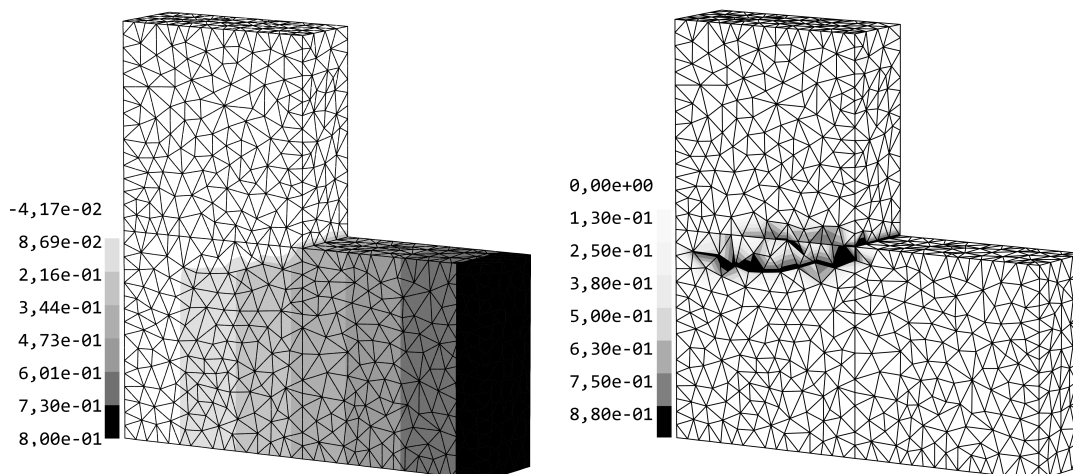


Figure 9: Vertical displacement (left) and damage propagation (right)

The resulting, almost stress-free, 'crack' propagates in the same way as in the experiment (Winkler, 2001) in a slight arc from the inward corner to the opposite side of the slab. Both contour plots are from the simulation with fine discretization and variable stiffness.

While the combined model already achieves very good results with the standard model parameters (Table 4), the use of the De Vree model requires an adjustment of the parameters (possibly several times) in order to reproduce the real material behavior (see previous section). A parameter study carried out in advance, but not described in detail here, showed an acceptable stress-strain relationship for the parameters shown in Table 4. The ultimate load is slightly underestimated with the fine FE mesh, but the post-cracking behavior is predicted with sufficient accuracy. Above this load-displacement curve lies, as expected, the curve determined with broad discretization. In this case, the calculation model delivers a too low degradation of the material after exceeding the ultimate load. The post-cracking behavior thus deviates from the softening curve observed in the experiment.

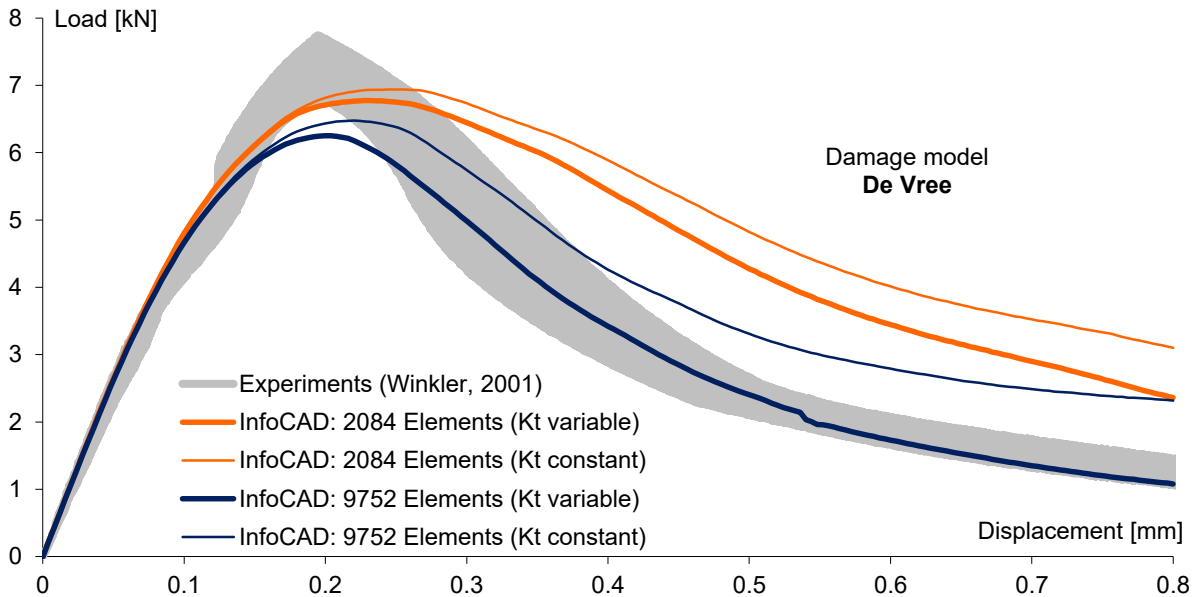


Figure 10: Comparison of experimentally and numerically (InfoCAD) determined load-displacement curves

The interrelationships between ultimate load and softening process as well as any parameter studies to be carried out to generate an adequate uniaxial stress-strain relationship have already been discussed in the case of the single-span beam with notch (Pettersson, 1981). Repeated explanations are therefore not provided here and instead reference is made to the previous section.

## Bearing Capacity Failure of Foundation and Embankment Failure

The *Mohr-Coulomb* plasticity model is available in *InfoCAD* for determining the load-bearing capacity of soils and predicting possible failure scenarios such as the formation of slip surfaces. The model can be used both for foundation failure analyses and for estimating the stability of embankments (*Phi-c Reduction*).

### Bearing Capacity Failure of Foundation

The verification of the numerical results is done here on the basis of the problem considered by *Zienkiewicz et al. (1975)*. In the same way as in the cited publication, the numerical analysis is performed using both associative and non-associative flow rule. In *InfoCAD*, the approach used for the calculation is determined by the appropriate choice of the dilatancy angle  $\psi$ . In Figure 1, all material parameters (dialog of material properties) are shown together with the dimensions and boundary conditions of the soil model (half system).

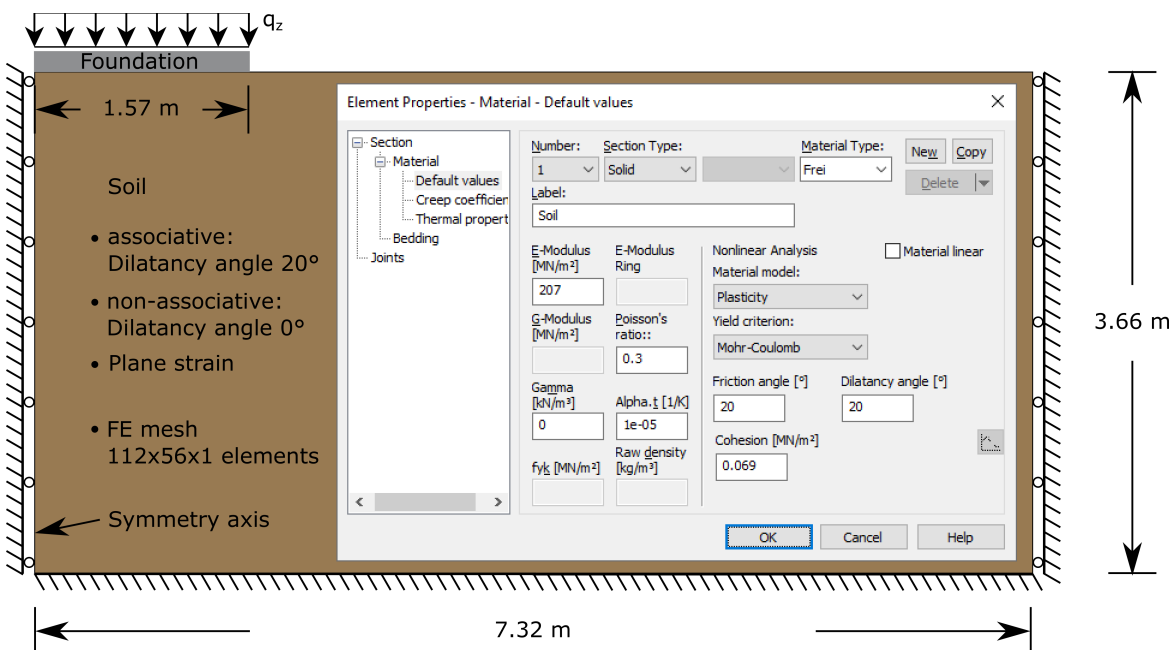


Figure 1: Dimensions (thickness  $d = 1$  m), boundary conditions and material parameters

In order to show the influence of the foundation stiffness on the deformation behavior and the bearing capacity of the soil,

two cases are investigated, a rigid and a flexible foundation. Numerically, this is implemented by a displacement-controlled (rigid) or load-controlled (flexible) calculation, where the superimposed load is applied directly to the weightless soil. The settlement underneath the foundation as well as the lateral upward displacement of the soil (bulging) that occurs when the load is increased can be illustrated using Figure 2.

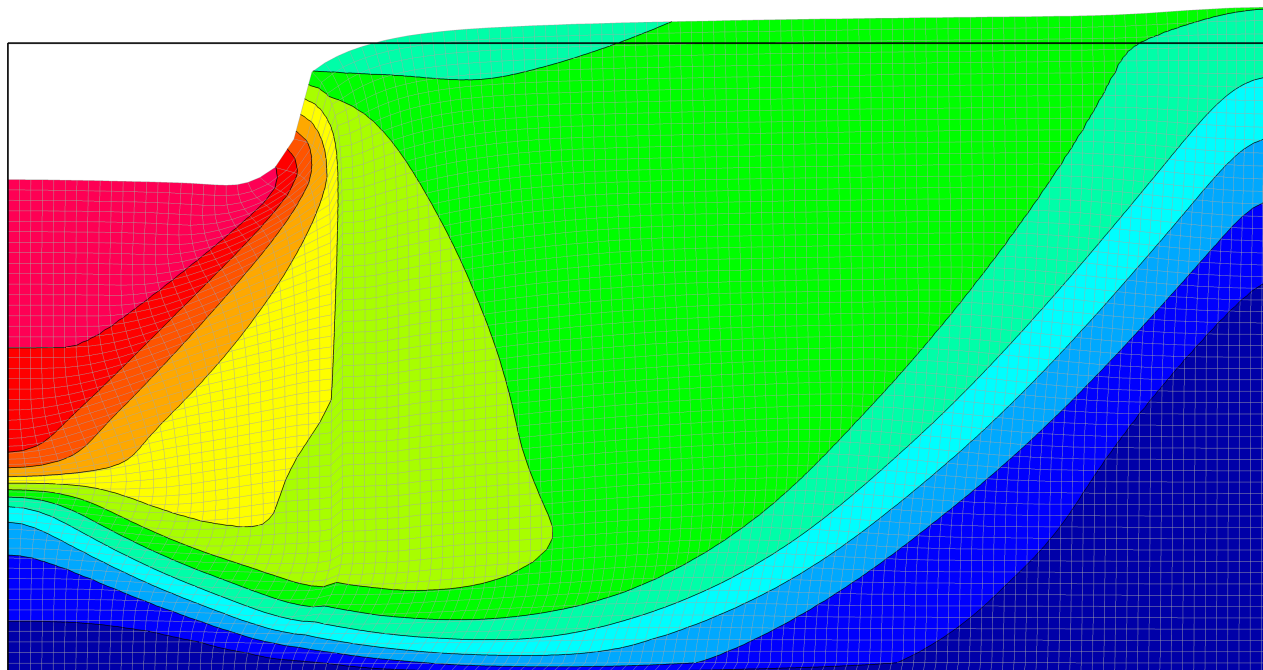


Figure 2: Deformation of the soil layer - flexible foundation

The mechanism of the bearing capacity failure of foundation (active wedge, radial shear zone, passive wedge) becomes even clearer using Figure 3 than when considering the deformations. When displaying the equivalent plastic strains, the shape of the slip surface becomes obvious. In addition to fracture figures, the *Mohr-Coulomb* plasticity model available in *InfoCAD* can also be used to numerically predict the ultimate load of the soil (maximum soil pressure).

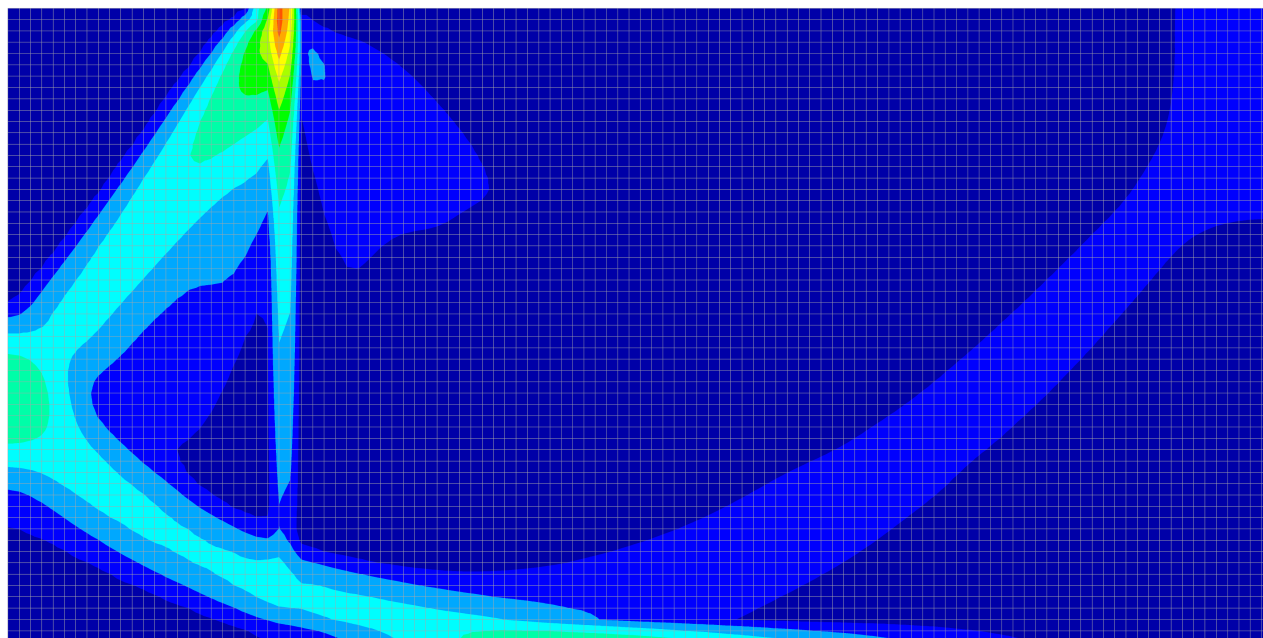


Figure 3: Equivalent plastic strains  $\epsilon_v$  - flexible foundation (maximum values form sliding surface)

The analyses performed with the arc length method (number of iterations: 20, initial arc length: 1 (rigid) bzw. 0.1 (flexible), Up limited, error threshold: 0.01%, constant stiffness) are in very good agreement with the reference solutions from *Zienkiewicz et al. (1975)* and *Prandtl (1921)*, both for associative (Figure 4) and non-associative (Figure 5) plasticity.

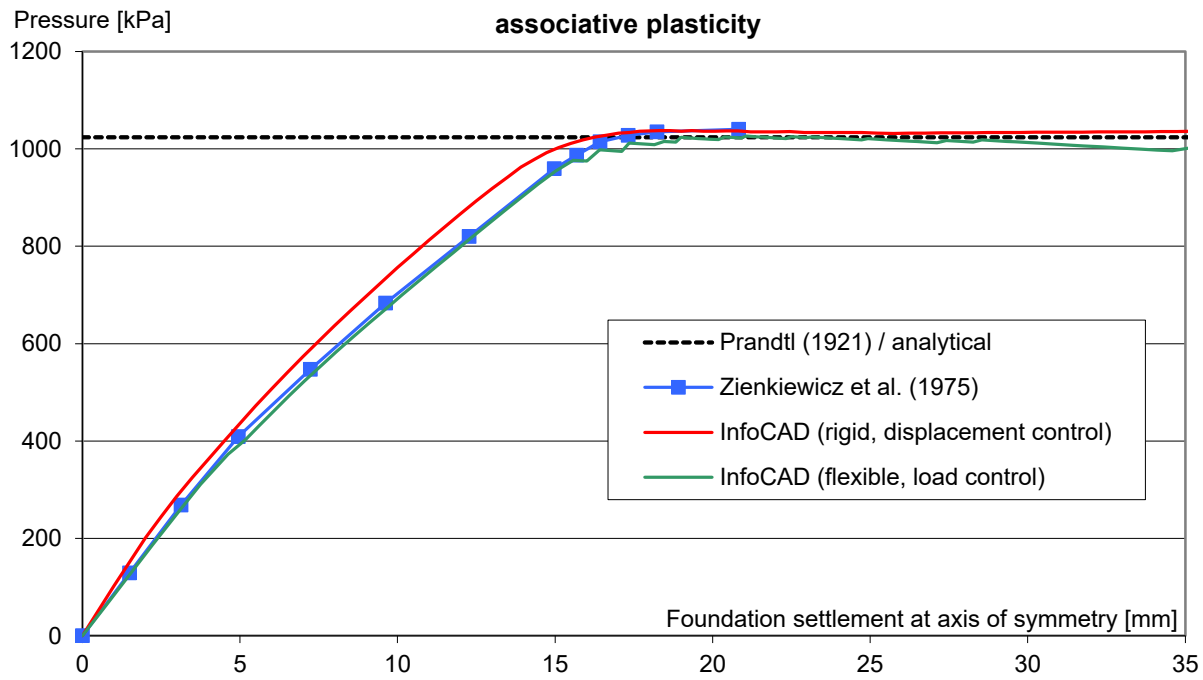


Figure 4: Load-deformation curve (associative flow rule:  $\varphi = \psi = 20^\circ$ )

The influence of the foundation stiffness is negligible here (deviation at the maximum bottom pressure is about 5%).

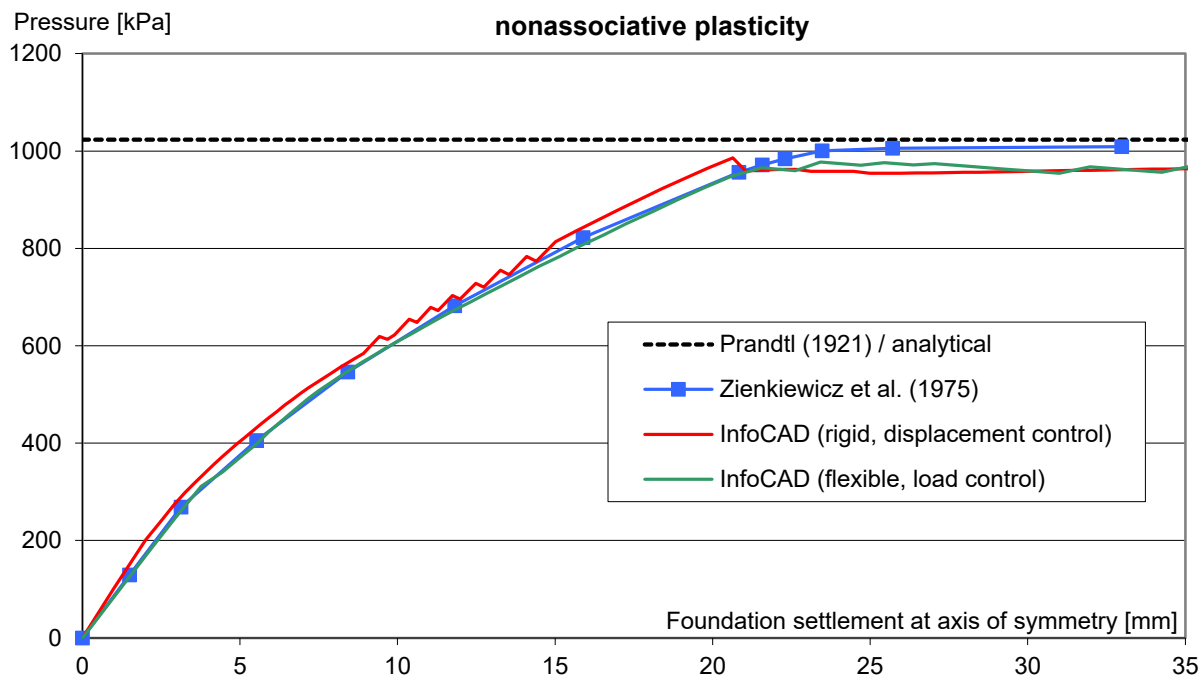


Figure 5: Load-deformation curve (non-associative flow rule:  $\varphi = 20^\circ, \psi = 0^\circ$ )



### Embankment Failure

To calculate the safety factor (FoS) or utilization factor (1 / FoS) of an embankment, the rule according to *Fellenius* (1927) [*Phi-c Reduction*] is used in *InfoCAD*. The soil parameters (friction angle  $\phi$  and cohesion  $c$ ) are successively reduced (see Eq. 1 ) until the failure of the soil occurs, characterized by the sliding of the sliding body on a sliding surface (slope failure).

$$\begin{aligned} \text{Cohesion}_{\text{Failure state}} &= \text{Cohesion} / \text{FoS} \\ \text{Friction angle}_{\text{Failure state}} &= \arctan( \tan(\text{Friction angle}) / \text{FoS} ) \end{aligned} \tag{Eq. 1}$$

On the basis of the problem described below and shown in Figure 6, taken from the publication by *Griffiths & Lane* (1999), the results calculated with *InfoCAD* are verified on the one hand and on the other hand possible analysis settings and internal program adjustments made when using the *Phi-c Reduction* are explained. The embankment failure simulation can be performed with any of the three implemented nonlinear analysis methods and is activated in the *Nonlinear System Analysis* settings (Serviceability).

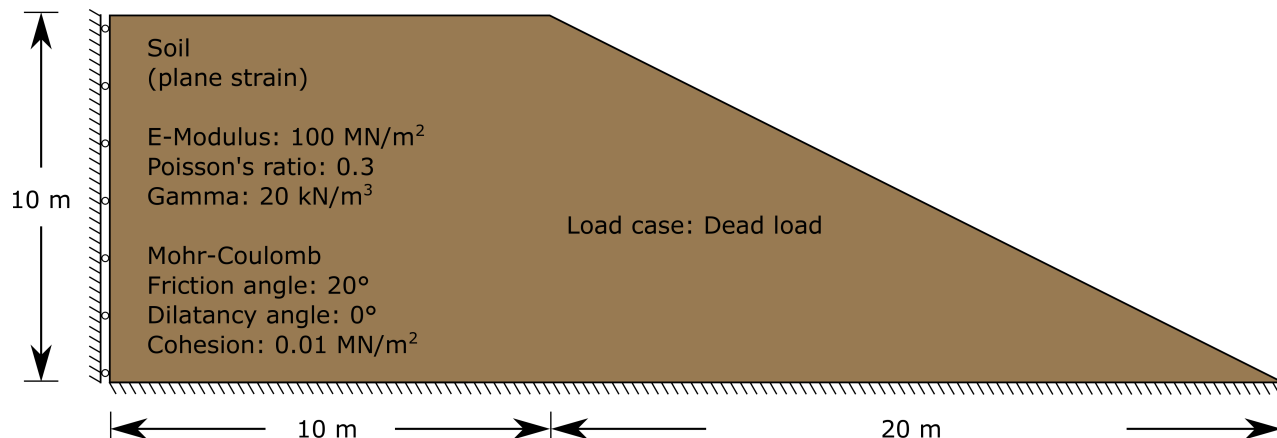


Figure 6: Dimensions (thickness  $d=1\text{m}$ ), boundary conditions and material parameters

The results obtained with both available analysis methods, the Newton and arc length methods, are compared here. Whereas with the arc length method (iterations per arc length 200; initial arc length 0.2; limited; tolerance 0.01%; constant stiffness) the dead weight is increased incrementally until the entire load is applied (load factor = 1), with the Newton method (max. iterations per load step 200; tolerance 0.01%; constant stiffness) the load is applied immediately. In addition, to show a possible influence of the discretization on the calculation of the factor of safety (FoS), the analysis was performed with three different FE meshes ( tetrahedron elements). The numerically determined safety factors are included in Table 1.

Number of elements	Analysis method	
	Newton	Arc length method
1822	1.35	1.325
18922	1.35	1.325
55085	1.35	1.325

Table 1: Influence of discretization and analysis method on the determination of the safety factor

Newton and the arc length method provide a factor of safety of 1.35 and 1.325, independent of the discretization. The values correspond very well both with the numerical reference solution of *Griffiths & Lane* (1999) [FoS = 1.35 (last convergent state)] and with the analytical solution of *Bishop & Morgenstern* (1960) [FoS = 1.38]. In *InfoCAD*, in addition to the factor, the reduced soil properties are also logged, see Figure 7. When using the arc length method or the Newton time-step method, the log also contains the entire history of the *Phi-c Reduction* (all increments of the interval nesting).

**Phi-c Reduction: Load case 1 (Serviceability)**

```
Factor of Safety (FoS) according to Fellenius : 1.35

Reduced parameters (Failure)
Nr. Friction angle [°] Dilatancy angle [°] Cohesion [MPa]
-----
1 15.1 0.0 0.0074
```

Figure 7: Log entry

In the same way as in the foundation failure analysis, the sliding surface occurring within the soil can also be clearly recognized here using the equivalent plastic strains (Figure 8). It should be noted that all *Phi-c Reduction* results (deformations, stresses, strains, etc.) presented in *InfoCAD* show the condition when the embankment is still stable and not the condition when the embankment has already failed.

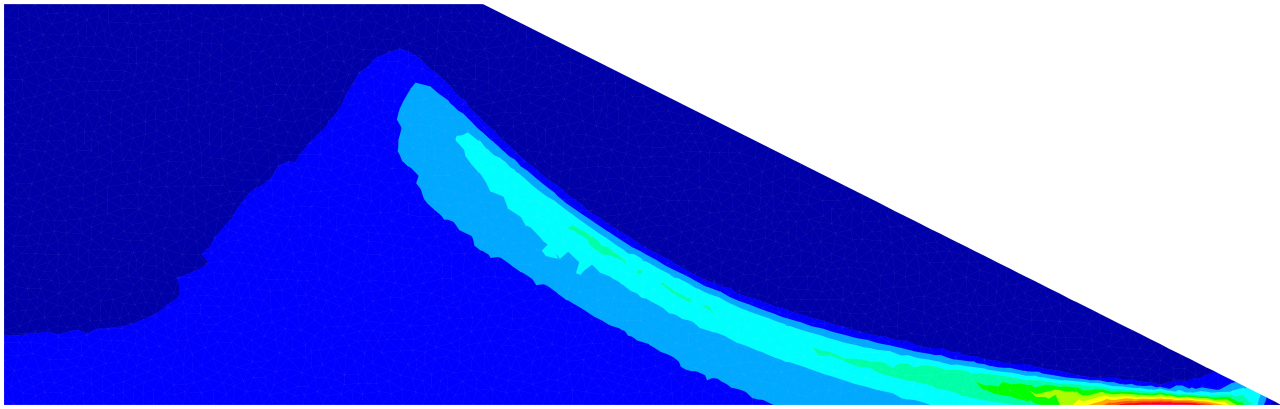


Figure 8: Equivalent plastic strains  $\epsilon_v$  (maximum values form sliding surface)

It should also be noted that the load-time curve is adjusted internally in the program when the Newton time-step method is used for embankment failure calculation. The internal adjustment affects all loads with assigned *load-time curves*. Here, a load-time curve entered by the user as desired is converted into a linear load-time curve (Figure 9). The duration at which the maximum load factor ( $f(t)=1$ ) is reached is to be defined by the user in the analysis settings.

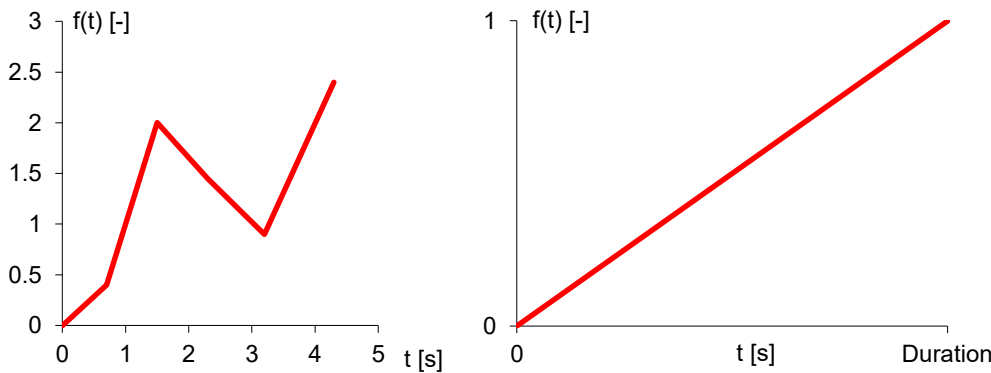
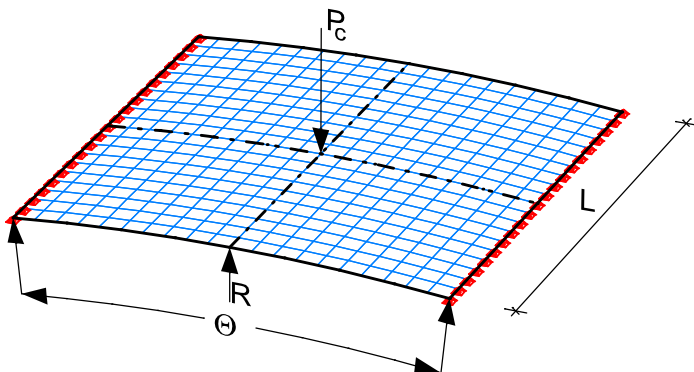


Figure 9: Load-time curve when using the nonlinear analysis method Newton time steps. Left: Random load-time curve (entered in InfoCAD), right: load-time curve after automatic adjustment.

### Snap Through Problem of a Cylindrical Shell

The cylindrical shell subjected to a central point load as shown below was first numerically investigated by Sabir and Lock in 1972. Their results are used to verify the arc length method implemented in InfoCAD. For the comparison calculation, the structure with jointed supports on two sides was discretized with 20x20 shell elements.



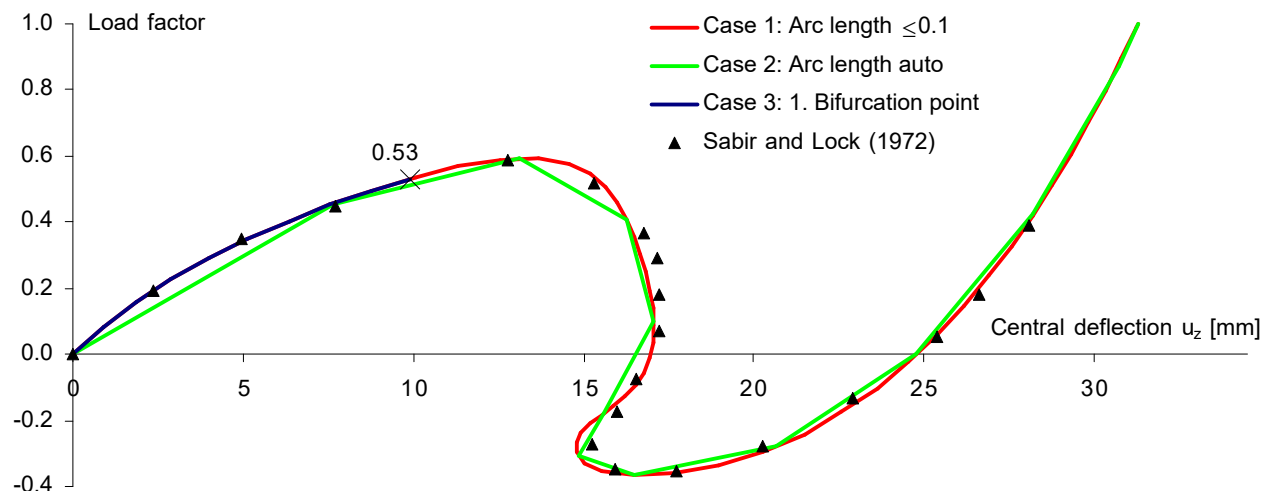
- Parameters:
- $P_c = 1 \text{ kN}$
  - $R = 2540 \text{ mm}$
  - $\Theta = 0,2 \text{ rad}$
  - $L = 508 \text{ mm}$
  - $t = 6.35 \text{ mm}$
  - $E_s = 3102.64 \text{ MN/m}^2$
  - $\nu = 0.3$

Discretization with supports, load and parameters

The calculation was based on the third order theory assuming linear-elastic material behavior (Error threshold 0.01 %). In order to illustrate the influence of the arc length on the numerical result as well as to show two possibilities to control the end of the calculation, three FE simulations with different specifications for the non-linear system analysis were performed.

Settings:	Case 1	Case 2	Case 3
Control:	Load factor = 1	Load factor = 1	1. Bifurcation point
Initial arc length:	0.1	auto	0.1
Up limited:	yes	no	yes
Iterations per arc length:	3	3	3

As illustrated in the figure below, all load-displacement curves calculated with the arc length method are in good agreement with the results of *Sabir* and *Lock*. (identical result curve for case 1 and 3 until first branch point is reached, load factor = 0.53) In addition, it becomes clear that a sufficiently small arc length is required for a detailed resolution of the complete equilibrium path.



Load-displacement curves

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# Structural Analysis for Fire Scenarios

## Area of Application

The structural analysis for fire scenarios allows the analysis of two- and three-dimensional beam and area structures within the *2D Frames*, *3D Frames* and *Finite Elements* program systems. Steel, reinforced concrete, timber and composite sections according to the following standards can be considered:

- EN 1992-1-2:2004+A1:2019 (Eurocode 2)
- EN 1993-1-2:2005+AC:2009 (Eurocode 3)
- EN 1994-1-2:2005+A1:2014 (Eurocode 4)
- EN 1995-1-2:2004+AC:2009 (Eurocode 5)

Steel sections, polygon sections and combinations of these as well as area sections can be used as section types. The following materials can be selected:

- Concrete according to DIN 1045-1, EN 1992-1-1, DIN EN 1992-1-1, OENORM B 4700 and SIA 262
- Construction steel according to DIN 18800 and EN 10025-2
- Steel with user-defined properties
- Timber according to EN 338:2016 and EN 14080:2013 (beams only)
- Freely defined material

The analysis can usually be divided up into the following steps:

### 1. Dimensioning the structure for standard actions

- Structural definition with statically effective sections.
- Determine the internal forces for the decisive load situations.
- Description of the action combinations according to the relevant national standard.
- Stress checks for structural steel members, determination of the reinforcement required for reinforced concrete members.

### 2. Thermal analysis to determine the section temperatures

- Definition of the determinant sections for fire scenarios (*Thermal analysis* function in the Section dialog).
- Determination of the effects of fire on the affected section boundaries.
- Calculation of the temperature profile in the section (using the *Thermal analysis* dialog or the *Section Temperatures* function in the *Analysis* menu).

### 3. Nonlinear system analysis for fire scenarios

- Specification of a decisive load case. Existing load cases are grouped together and weighted using the *Load group* load type.
- Addition of the *fire scenario* load type to the load case.
- Selection of this load case from the *Fire scenario* tab in the calculations settings for nonlinear system analysis.
- Selection of the reinforcement determined under item 1 (above), if relevant.
- Execution of the framework or FEM calculations to determine the load-bearing capacity for a fire scenario.

When different fire scenarios are to be investigated, the section boundaries can be allocated different fire effects under item 2. The individual fire scenarios are numbered in this case. Finally, a check load case should be generated under item 3 for each fire scenario. The respective numbers mentioned above are used in the *fire scenario* load type.



## Calculation of Section Temperatures

This section describes the thermal analysis of the section temperatures due to the effects of fire.

### Basics

The calculation of the temperature distribution for transient situations in solid bodies is carried out by solving the thermal conductivity equation as set out by Fourier. For the two-dimensional case considered here, this has the following form:

$$\rho \cdot c_p \cdot \frac{\partial \theta}{\partial t} = \frac{\partial}{\partial y} \cdot \left( \lambda \frac{\partial \theta}{\partial y} \right) + \frac{\partial}{\partial z} \cdot \left( \lambda \frac{\partial \theta}{\partial z} \right) \quad (1)$$

with

$t$  Time [s]

$y, z$  Local coordinates [m]

Functions of  $y, z, t$

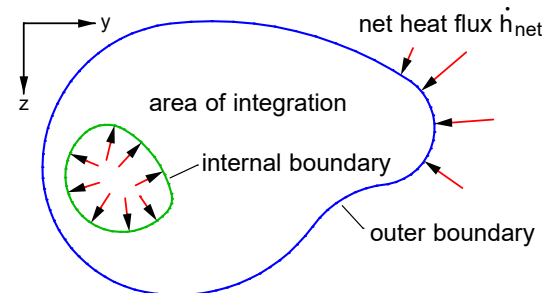
$\theta$  Temperature [°C]

$\rho$  Apparent density [kg/m<sup>3</sup>]

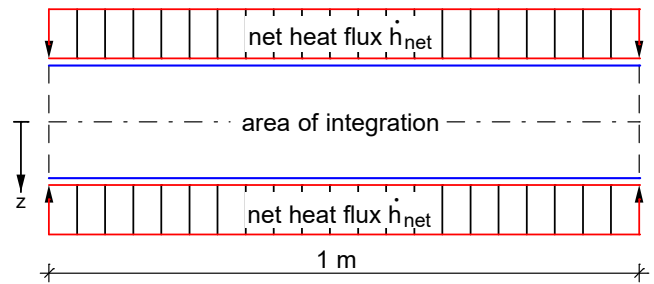
$c_p$  Specific thermal capacity [J/(kg K)]

$\lambda$  Thermal conductivity [W/(m K)]

For area elements the thermal conductivity equation is reduced to the one-dimensional case with the coordinate  $z$ . A horizontal thermal conductivity cannot be taken into account. For this a solid model has to be used. Heat sources within the integration area are ignored for area sections. As all factors are dependent on time, the initial value problem needs to be solved in consideration of the boundary conditions. The temperature distribution in the area of integration at the time  $t = 0$  is taken as the initial condition.



Beam section



Area section

For the problem being considered here, the thermal actions will be primarily determined by the net heat flux  $\dot{h}_{net}$  [W/m<sup>2</sup>] normal to the boundaries. This, in turn, comprises a convection and radiation component.

$$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r} \quad (2)$$

The convective component is calculated as follows:

$$\dot{h}_{net,c} = \alpha_c (\Theta_g - \Theta_m) \quad (3)$$

with

$\alpha_c$  Coefficient of heat transfer [W/(m<sup>2</sup> K)]

$\Theta_g$  Gas temperature in the region of the component under load [°C]

$\Theta_m$  Temperature at the boundary of the component [°C]

In analysing of the convective component on the internal boundary of a section hole, during one time step, a mean gas temperature is assumed.

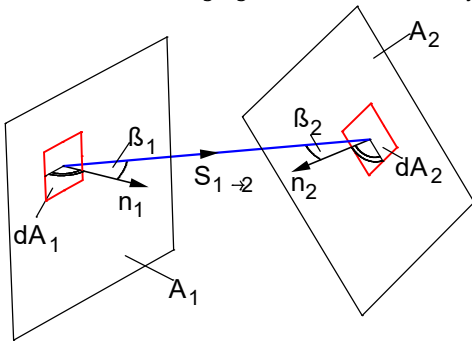
The net heat flux between two surfaces due to radiation is calculated as follows:

$$\dot{h}_{\text{net,r}} = \Phi \cdot \varepsilon_1 \cdot \varepsilon_2 \cdot \sigma \cdot [(\Theta_1 + 273)^4 - (\Theta_2 + 273)^4] \quad (4)$$

with

$\Phi$	Configuration factor
$\varepsilon_1, \varepsilon_2$	Emissivity of the component surfaces [-]
$\sigma$	Stephan-Boltzmann constant [= $5.67 \cdot 10^{-8} \text{ W}/(\text{m}^2 \text{ K}^4)$ ]
$\Theta_1, \Theta_2$	Temperature at the component surfaces [°C]

The configuration factor is determined by the geometry of the surface, the distance between the surfaces and shadowing effects. The following figure shows the underlying parameters.



The configuration factor can be determined as follows:

$$\Phi_{1 \rightarrow 2} = \frac{1}{\pi \cdot A_1} \cdot \int_{A_1} \int_{A_2} \frac{\cos \beta_1 \cdot \cos \beta_2}{s_{1 \rightarrow 2}^2} \cdot dA_1 \cdot dA_2 \quad (5)$$

In the case of radiation due to fire follows:

$\Phi = 1$	
$\varepsilon_1 = \varepsilon_m$	emissivity of the component surface [-]
$\varepsilon_2 = \varepsilon_f$	emissivity of the flame (here $\varepsilon_f = 1$ ) [-]
$\Theta_1 = \Theta_r$	effective radiation temperature of the fire (here $\Theta_r = \Theta_g$ ) [°C]
$\Theta_2 = \Theta_m$	surface temperature at the component boundary [°C]

The gas temperature  $\Theta_g$  of a region of a fire is assumed to be known and is generally defined according to particular temperature-time curves.

The nonlinear boundary problem set by equations (1) to (5) cannot be solved in a closed form. Instead, the finite element method is used to solve the problem. In this case, Galerkin's method followed by partial integration is used to convert the boundary problem described above into a system of variation equations.

The program system uses compatible elements with a fully quadratic approach. This results in a high degree of accuracy, which is largely independent of the net geometry.

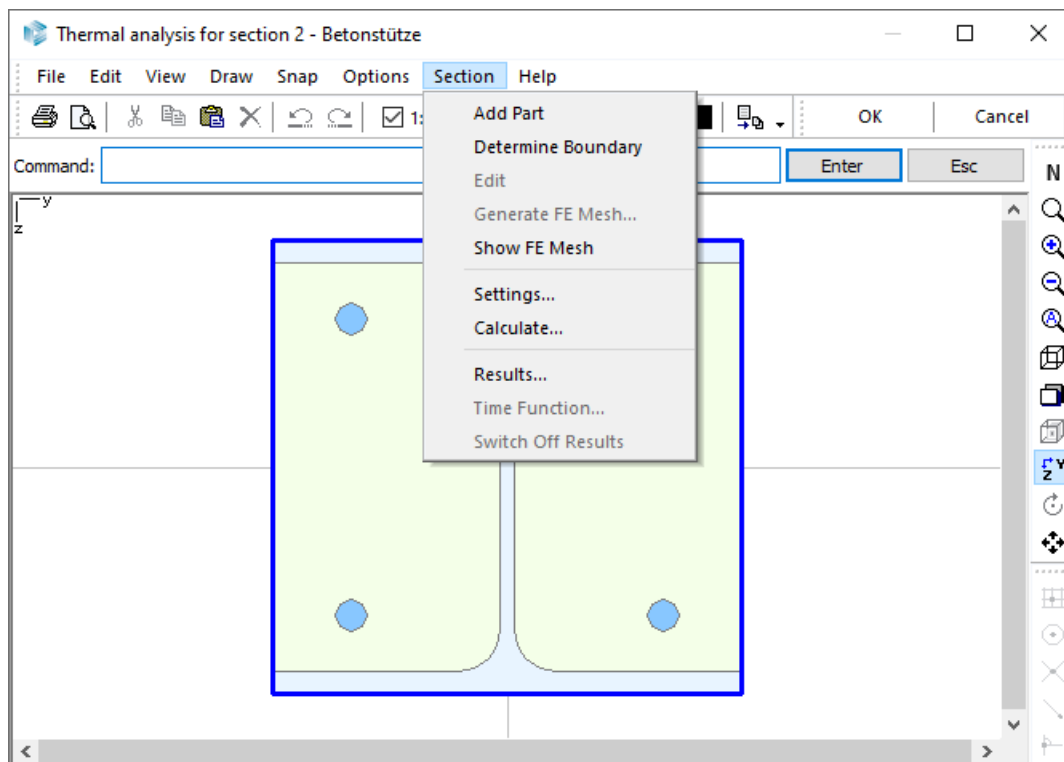
The element approach automatically satisfies the so-called 'adiabatic boundary conditions', meaning that thermal energy is only exchanged where boundary conditions are explicitly specified.

The nonlinear initial value problem is solved using time-step integration in conjunction with an incremental Newton-Raphson algorithm. After successful calculation, the temperature profiles for all times considered are available for further calculation.

## Thermal Section

The input window shown below is accessed by way of the sections dialog and then the *Thermal analysis* function. The functions which are described below can be found in the *Section* and in the context menus:

<i>Add part</i>	Allows the section to be assembled from new and existing parts.
<i>Determine boundary</i>	Determines the boundary of a section assembled from parts and highlights it with a red line.
<i>Edit</i>	Starts the dialog for editing a selected section part or for determining the fire actions for the marked part of the section boundary.
<i>Generate FE mesh...</i>	Starts the dialog for specifying the mesh size of the element mesh used for thermal calculation.
<i>Show FE mesh</i>	Shows the element mesh for the thermal calculation. This mesh is automatically generated before each analysis, even when the display is switched off.
<i>Settings...</i>	Starts the dialog with the settings for thermal analysis.
<i>Calculate...</i>	Starts the thermal analysis in its own window and shows the temperature distribution for the last calculated time step.
<i>Results...</i>	Starts the dialog for selecting results calculated for different times.
<i>Time function...</i>	Starts the dialog with the illustration of a time function of the current result for a selected section point.
<i>Switch off results</i>	Switches off the representation of results.



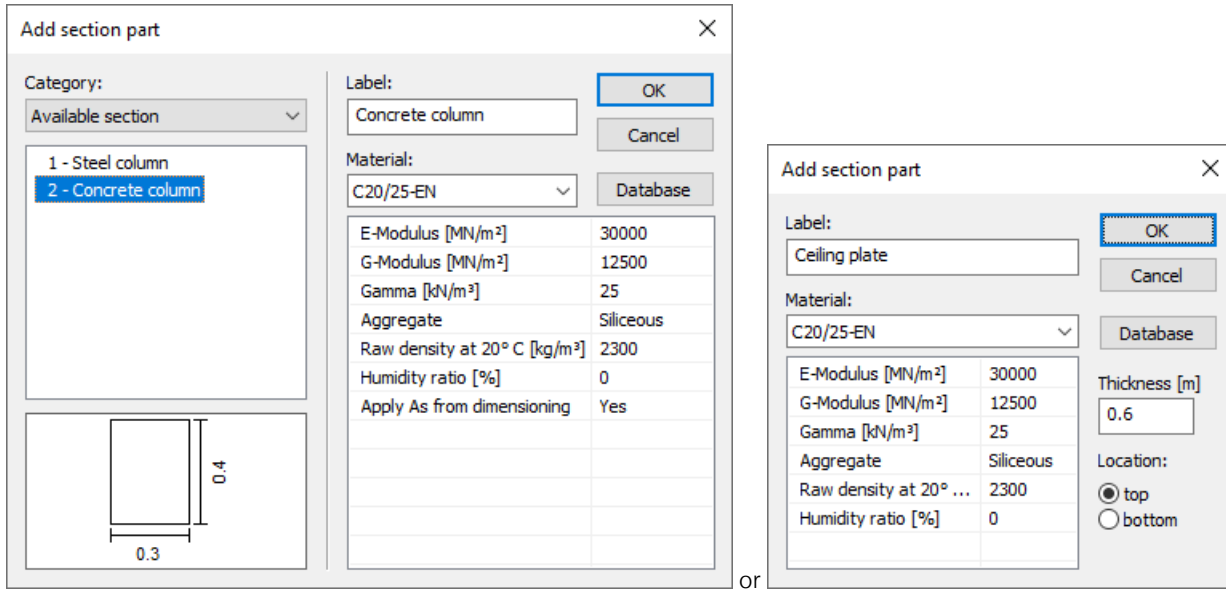
The thermal section can be assembled from any sectional components and, along with the representation of statically relevant parts, can also include, for example, thermal insulations.

The beam axes for the static analysis (fire scenario) are located at the origin (0; 0) of the thermal cross-section.

The central area of the area elements goes through  $z = 0$ .

## Addition of Section Parts

The thermal section is defined by adding section parts. During this process, section parts added later overlap parts that were added previously. The order in which section parts are added therefore determines the geometry of the final section. When adding a cross-section with a hole, first a cross-section part with the shape of the outer polygon and the selected material type is created and then a cross-section part with the shape of the hole and the material type *Null*. Note, however, that a part in the background may not be completely overlapped by another part.



For area sections section parts of desired thickness can be added above or below existing section parts. The width of the section parts is always 1 m.

For beam sections the following categories of section parts may be added:

### Category

- *Reinforcement bar*: is added as a section part and represented as a structural element and therefore influences temperature distribution (however, only makes sense for larger diameters).
- *New definition*: new definition of a polygonal section part which can also contain reinforcement. This is not represented as a structural element and has no influence on the temperature profile.
- *Steel profile*: steel profile from the section library.
- *Available section*: a section already defined in the project.

### Material

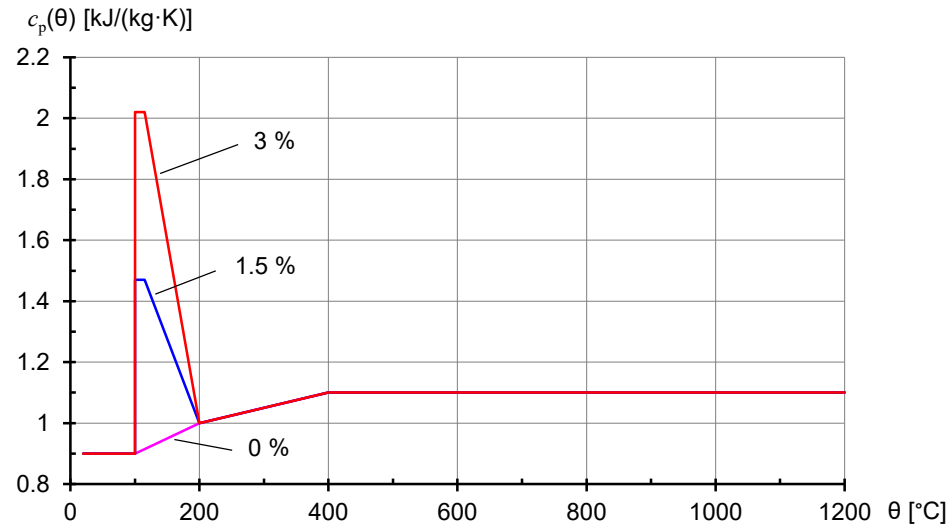
The section material together with its thermal and mechanical properties.

The *Apply As from dimensioning* option is used to specify that the selected reinforcing will be applied to the current section. This reinforcing is selected on the *Fire Scenario* tab in the nonlinear system analysis *Calculation Settings*. It is assumed that these reinforcing steel layers have no influence on the temperature distribution in the section and so the concrete temperature at their locations is assigned to them. The material can be saved in the *Igraph.dat* database for use in other structures.

The following specific material properties apply to the thermal analysis:

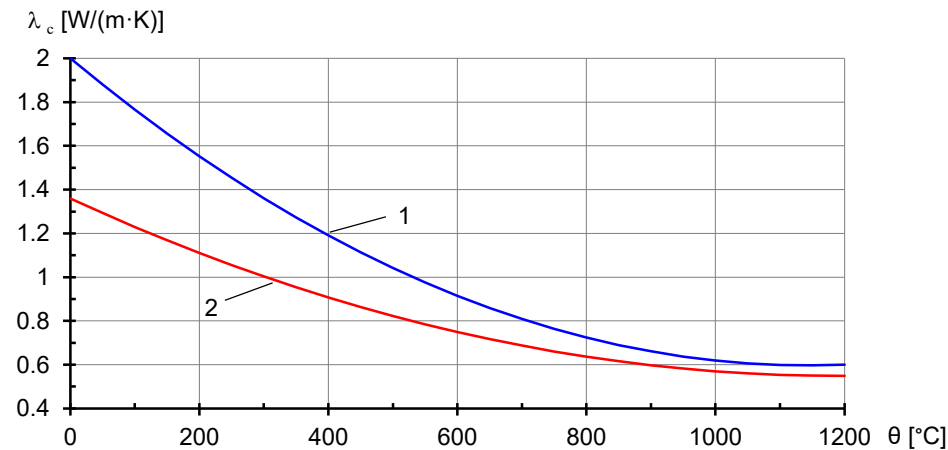
### Concrete

All concrete section parts are automatically assigned the temperature-dependent *specific thermal capacity*, the *thermal conductivity* and the *raw density* as specified in EN 1992-1-2. According to DIN EN 1992-1-2/NA:2010-12 the upper limit function of the *thermal conductivity* in 3.3.3 (2) is used for DIN concrete. The lower boundary value is used for other kinds of concrete. The *humidity ratio* and *raw density at 20° C* can be specified by the user. For the ÖNBeton material user-defined progressions of the thermal properties can be described.

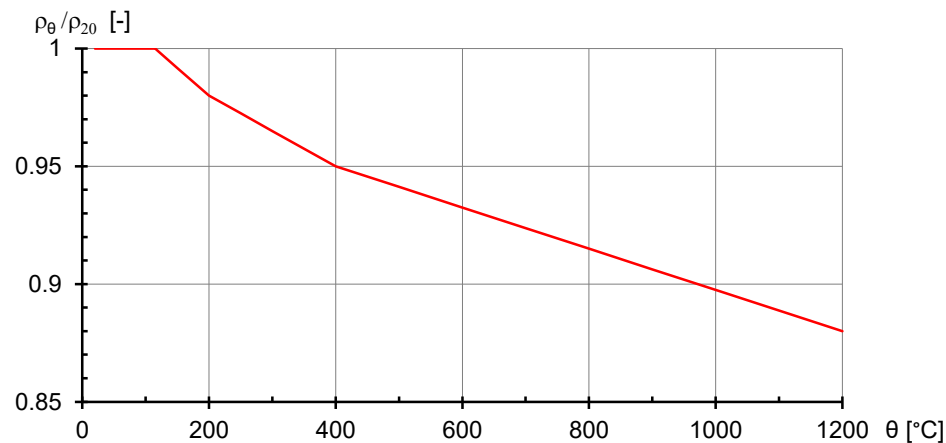


The specific heat capacity of siliceous and calcareous concrete  $c_p(\theta)$  depending on the temperature (moisture content  $u = 0, 1.5$  and  $3$  % by weight)

Moisture contents between the values specified will be linearly interpolated.



Temperature-dependent thermal conductivity  
1: upper limit (DIN concrete); 2: lower limit

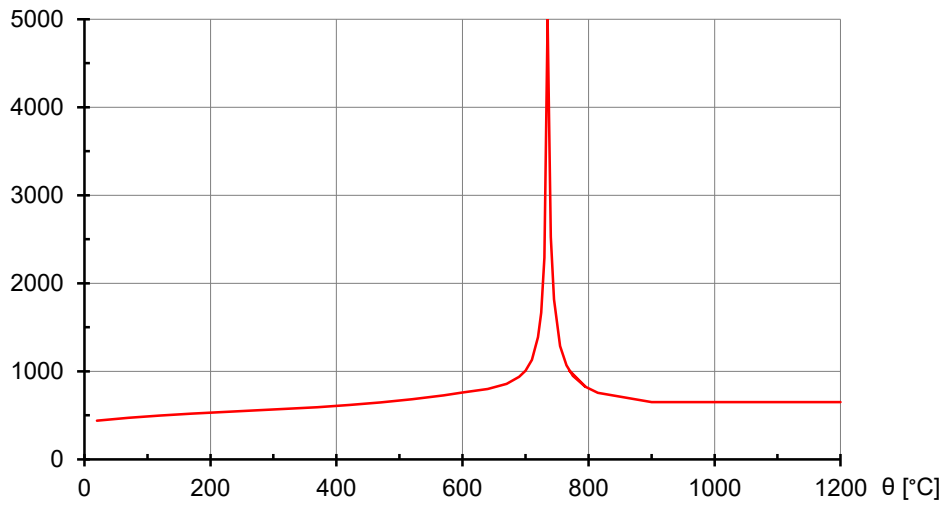


Temperature-dependent raw density ratio of concrete

**Steel**

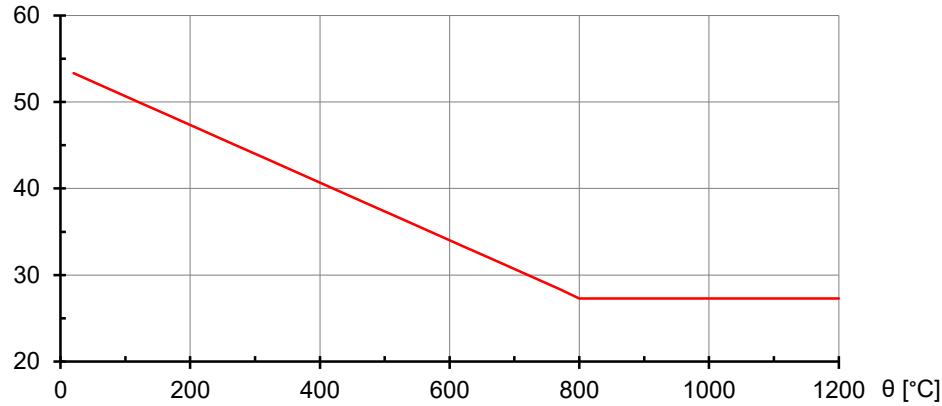
All steel sections are automatically assigned the temperature-dependent *specific thermal capacity* and the *thermal conductivity* as specified in EN 1993-1-2.

$c_a$  [J/(kg·K)]



The temperature-dependent specific thermal capacity for carbon steel

$\lambda_a$  [W/(m·K)]

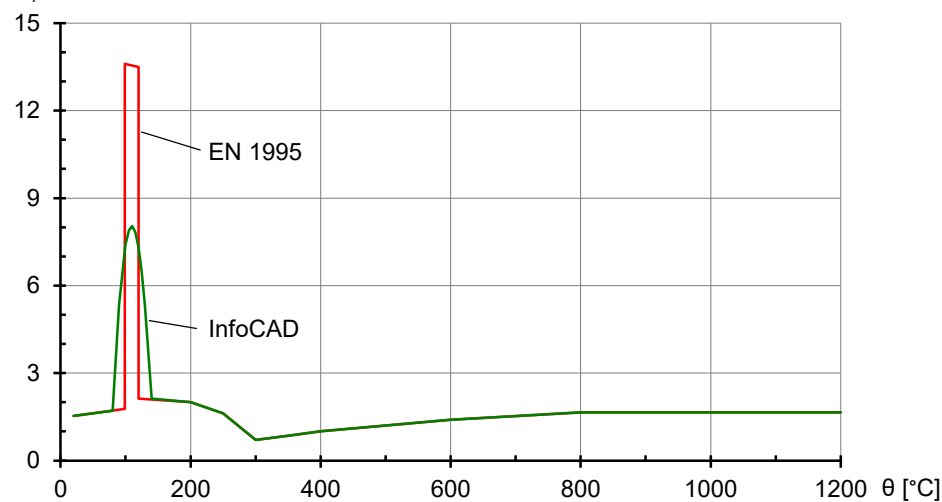


The temperature-dependent conductivity for carbon steel

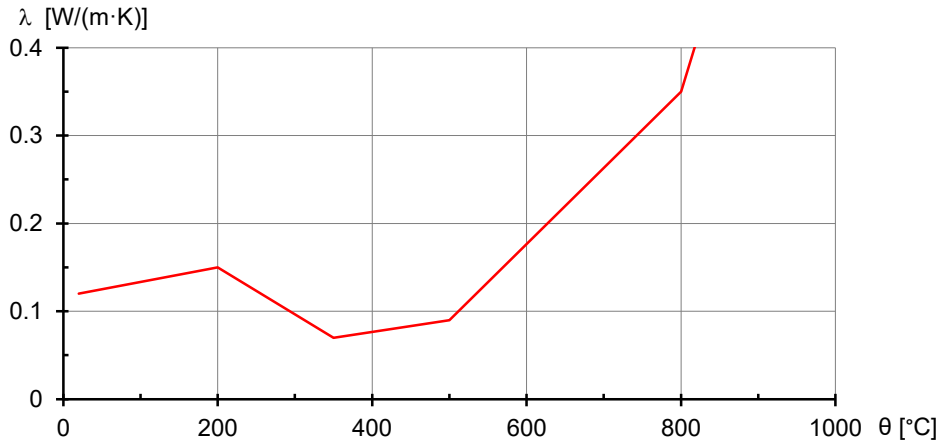
**Timber**

All timber section parts are automatically assigned the temperature-dependent *specific thermal capacity*, the *thermal conductivity* and the *raw density* as specified in EN 1995-1-2.

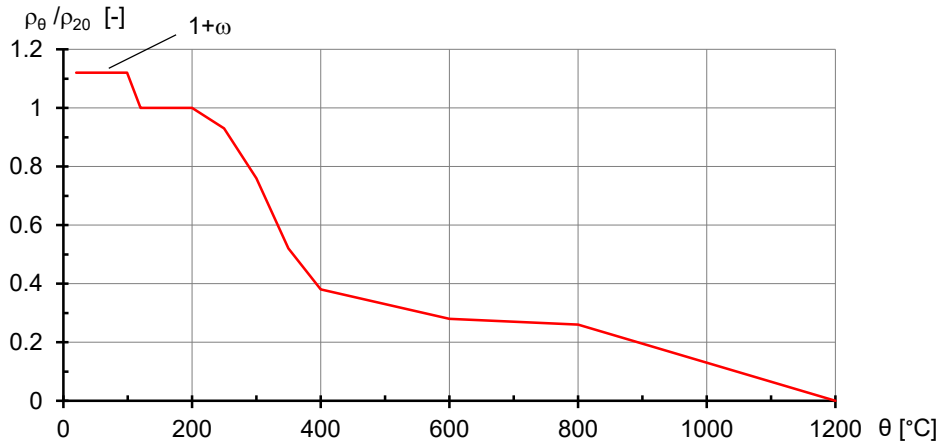
$c_p$  [kJ/(kg·K)]



The temperature-dependent specific thermal capacity for timber



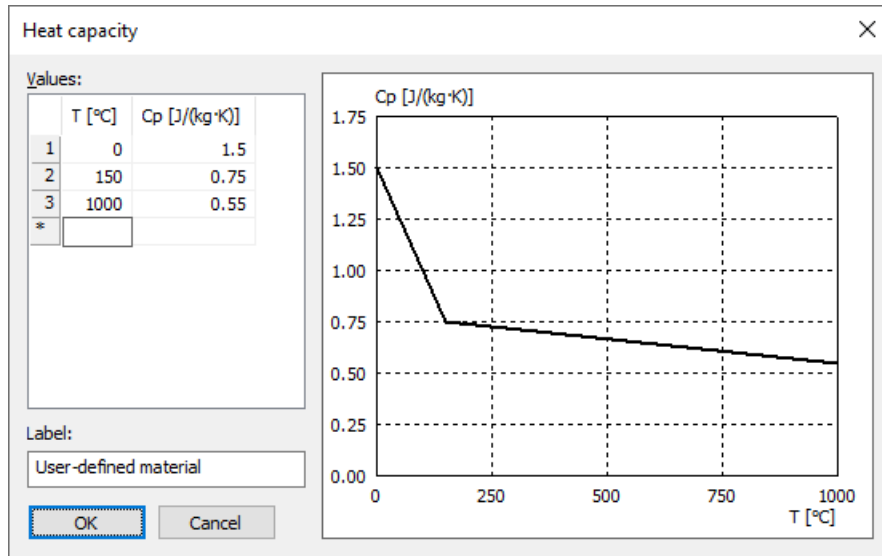
The temperature-dependent conductivity for timber



The temperature-dependent raw density ratio for timber with an initial moisture content  $\omega$  of 12 %

**User-defined material**

User-defined section parts are used to specify, for example, insulating materials, where any temperature-dependent *specific thermal capacity, apparent density and conductivity* can be specified.



User-defined thermal conductivity

**'Null' Material type**

Sections with this material type are excluded from analysis, which allows openings to be 'added' subsequently to a section.

## Edit Section Parts

E-Modulus [MN/m <sup>2</sup> ]	30000
G-Modulus [MN/m <sup>2</sup> ]	12500
Gamma [kN/m <sup>3</sup> ]	25
Aggregate	Siliceous
Raw density at 20° C [kg/m <sup>3</sup> ]	2300
Humidity ratio [%]	0
Apply As from dimensioning	Yes

The *Edit* function allows the material properties and thermal parameters of a selected section part to be changed. In addition, the geometry and reinforcement of section polygons can be modified.

## Edit Section Boundaries

The section boundaries are specified after all the section parts have been added. This is automatically carried out using the '*Determine boundary*' command. Each of the resulting section boundaries can then be allocated the following properties. Edges at holes have the standard feature '*Inner radiation and convection*'.

### Mesh width

The section boundary is divided with the chosen mesh width [mm]. If the mesh width '0' is chosen and a fire curve is assigned to the boundary the boundary is divided into pieces of about 20 mm.

### Conditions

In order to be able to investigate different fire situations, different *fire scenarios* can be defined. The name and number of these scenarios can be changed by the user as required. The thermal calculations for each fire scenario are carried out separately. During calculation, actions whose number corresponds with the fire scenario number will be applied to the boundaries. Boundaries that have not been allocated actions for a particular fire scenario number are treated adiabatically.

The following properties can be specified for each fire scenario:

- Fire curve, e.g., *ETK (Standard Temperature-Time Curve)*
- *Coefficient of heat transfer [W/(m<sup>2</sup> K)]*
- *Emissivity [-]*

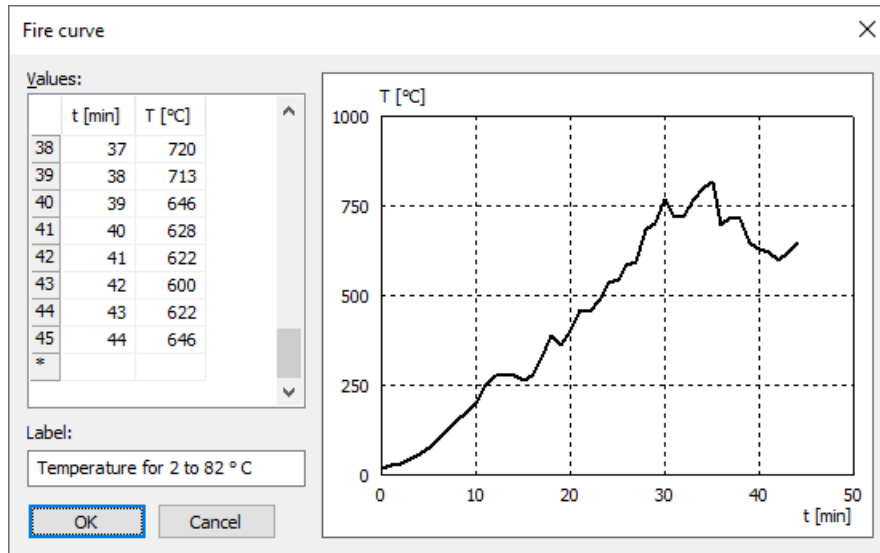
The convective coefficient of heat transfer can be taken from EN 1991-1-2, Appendix B.

The emissivity of concrete and construction steel surfaces should be determined according to EN 1992-1-2 or EN 1993-1-2, Chapter 2.2 (2) for a default value of 0.7. According to Appendix C of the standard, the emissivity for stainless steel should be set to 0.4.

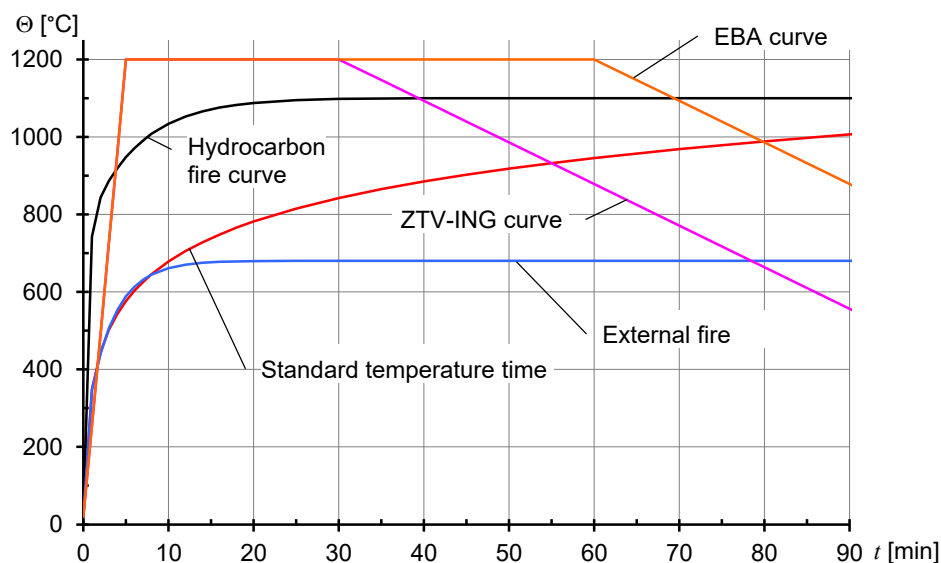


The following fire curves can be used to determine the fire progression.

- *Constant ambient temperature*: The ambient temperature can be set (colored green).
- *EBA curve*: Fire curve according to EBA-Guideline (colored orange).
- *ETK (Standard Temperature-Time Curve)*: Fire curve according to EN 1991-1-2 or prEN 13501-2 (colored red).
- *External fire curve*: Fire curve according to EN 1991-1-2 (colored turquoise).
- *Hydrocarbon fire curve*: Fire curve according to EN 1991-1-2 (colored black).
- *User-defined fire curve*: Fire curve defined by the user (colored blue).
- *ZTV-ING curve*: Fire curve according to ZTV-ING Part 5 (colored purple).



User-defined fire curve

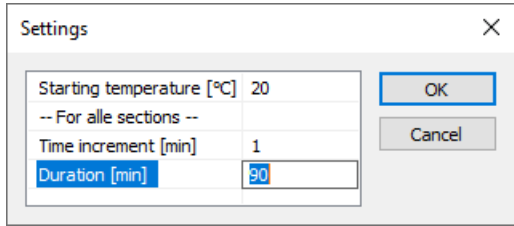


Fire curves according to EN 1991-1-2, ZTV-ING Part 5 resp. EBA-Guideline

## Generate FE Mesh

The section geometry is determined by the added section parts and the boundary. The finite element mesh for the thermal analysis is then generated by the program based on this geometry. This mesh can be displayed by use of the 'Show FE mesh' command. For beam sections the option 'Generate FE-Mesh' allows the selection of the mesh width [mm] and the mesh angle [°] (smallest internal angle of elements). The FE-mesh affects the thermal analysis and also the subsequent mechanical analysis, because both use the same element mesh.

### Settings for Thermal Analysis



The initial temperature is used to set the initial conditions for the time-step calculation at time  $t = 0$ . This applies to the current section. At a temperature of 20°C there is no thermal expansion according to the standard.

The time increment and duration apply for both the thermal and the mechanical analysis. For reasons of compatibility, both parameters are applied globally.

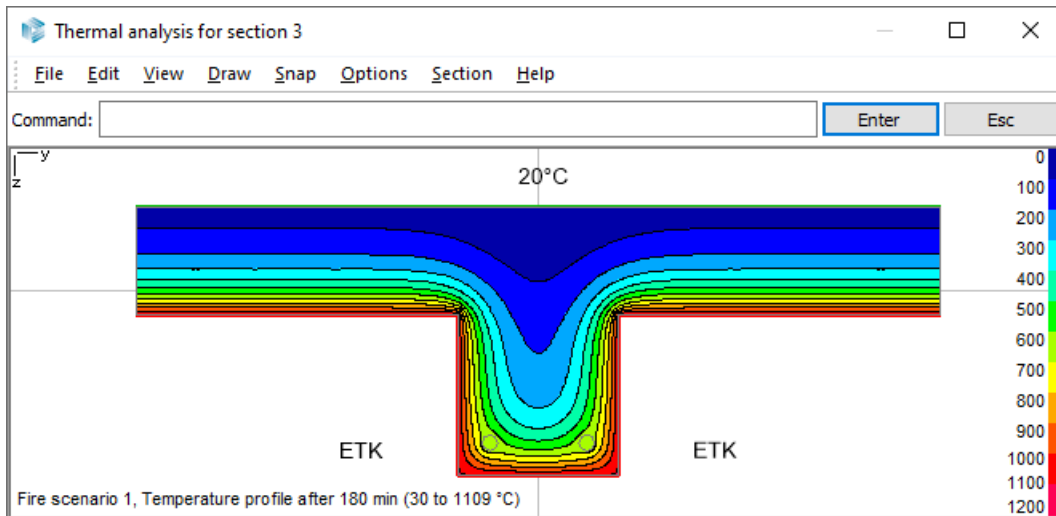
The size of the time step influences calculation accuracy; however, because of the computing time required, this should be set to a value smaller than 1 min in exceptional situations only.

### Calculation of the Temperature Profile

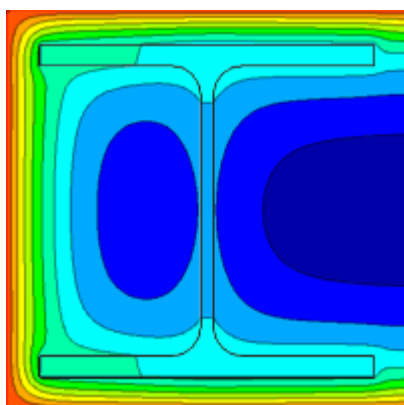
The *Calculate* menu item in the editing window starts the calculation of the temperature profile for the current section over the required period of time. The *Section Temperatures* function in the main window's *Analysis* menu is used to start the collective calculation for all sections.

### Display Results

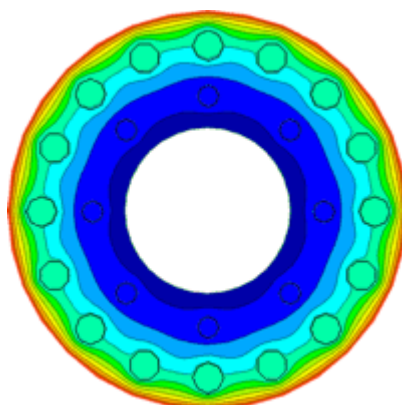
After calculating results, the temperature distribution for the last time step calculated will be shown. The results dialog can be used to show the temperature profile for each time investigated.



Temperature distribution from the standard temperature-time curve (ETK)



Composite section fired from three sides with one adiabatic boundary



Spun concrete column with steel reinforcing

## Nonlinear Structure Analysis for Fire Scenarios

Points to be considered when applying nonlinear systems to fire scenarios are explained below. The general theoretical principles are explained in the *Nonlinear Structural Analysis* chapter.

### Basics

The nonlinear representation of fire is to be carried out in accordance with the '*Advanced calculation method*' of the EN 1992-1-2, EN 1993-1-2 and EN 1995-1-2 standards. This procedure assumes that the mechanical and thermal analyses can be carried out independently of one another. The basis for these calculations is the temperature profile previously determined as part of the analysis of the thermal section. This basis corresponds with the '*Calculation of section temperatures*' chapter.

As the most important factors are dependent on time and temperature, a time-step calculation is always carried out with '*Nonlinear system analysis for fire scenarios*'. In this situation, all sections for which a thermal analysis is available will be treated according to the procedure specified in EN 1992-1-2, EN 1993-1-2 and EN 1995-1-2. The thermal section will be used for all section-related integrations. Should no temperature profile be available for a thermal section, then the fire analysis is to be carried out at an assumed normal temperature (20°C) as laid out in EN 1992-1-2, EN 1993-1-2 and EN 1995-1-2. The properties of the basic section will be applied for all other sections. Section parts of the thermal section that are made neither from steel nor reinforced concrete nor timber are ignored in the mechanical analysis. As only very little is known about the shear load performance in fire scenarios – in particular for reinforced concrete sections – calculations are currently based on the shear and torsional stiffness of the basic sections.

The program calculates according to nonlinear elasticity theory, where loading and unloading follow the same path. A possible strength increase of concrete or timber in the cool-down period is not considered.

On the resistance side, the material property partial safety factors for fire scenarios are set to  $\gamma_{M,fi} = 1.0$  within the program.

On the (user-defined) action side, the following action combinations are to be considered for fire scenarios according to EN 1990 (Eurocode 0):

$$\sum_{j \geq 1} G_{k,j} + P + A_d (\psi_{1,1} \text{ or } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i}$$

where  $A_d$  specifies the thermal expansion and the thermal effects on the material properties. The strain approach is:

$$\varepsilon_s = \varepsilon - \varepsilon_{th}$$

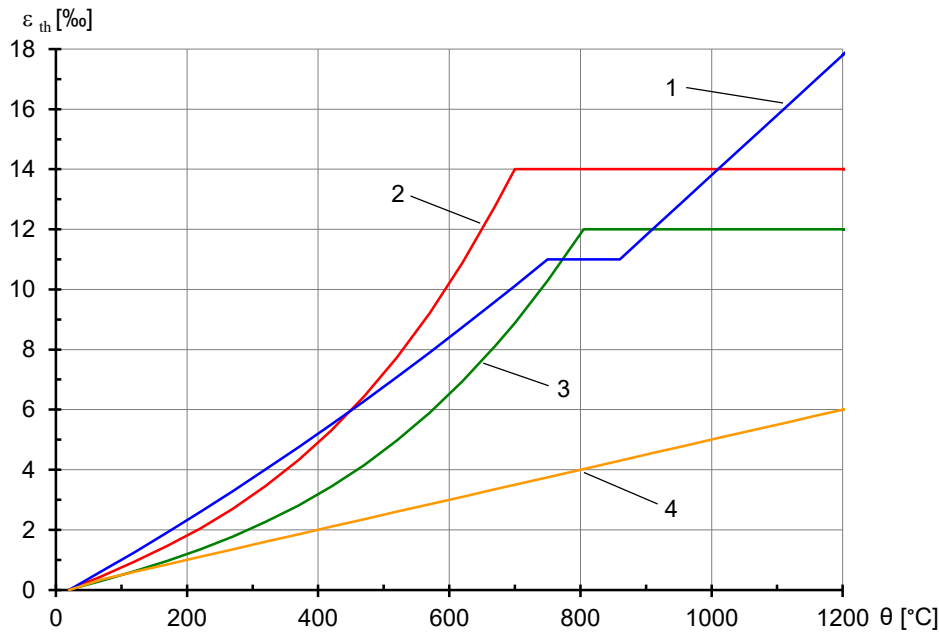
with

$\varepsilon$  Total strain

$\varepsilon_s$  Stress-creating strain

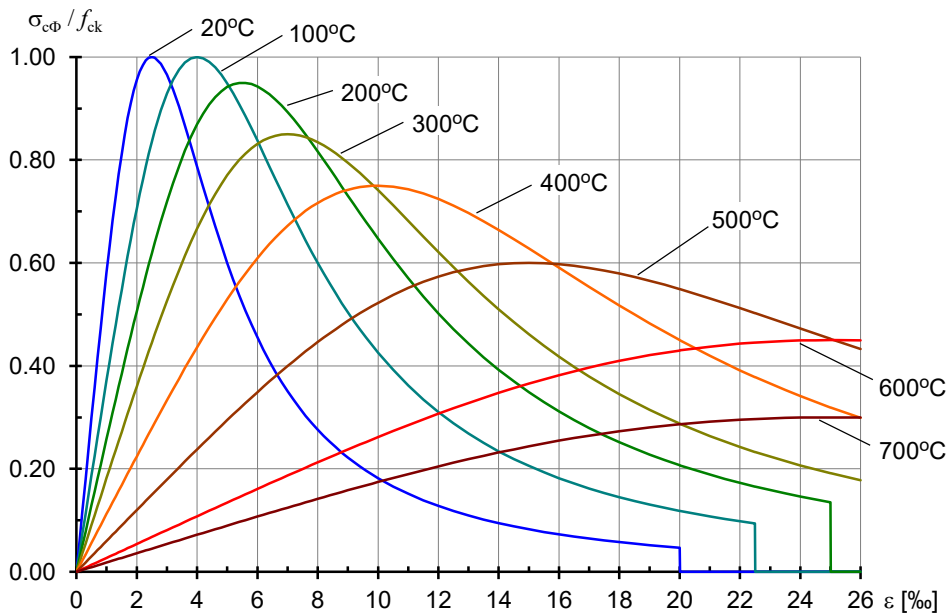
$\varepsilon_{th}$  Thermal strain

The following diagram shows the variation in thermal expansion for steel and concrete against temperature.



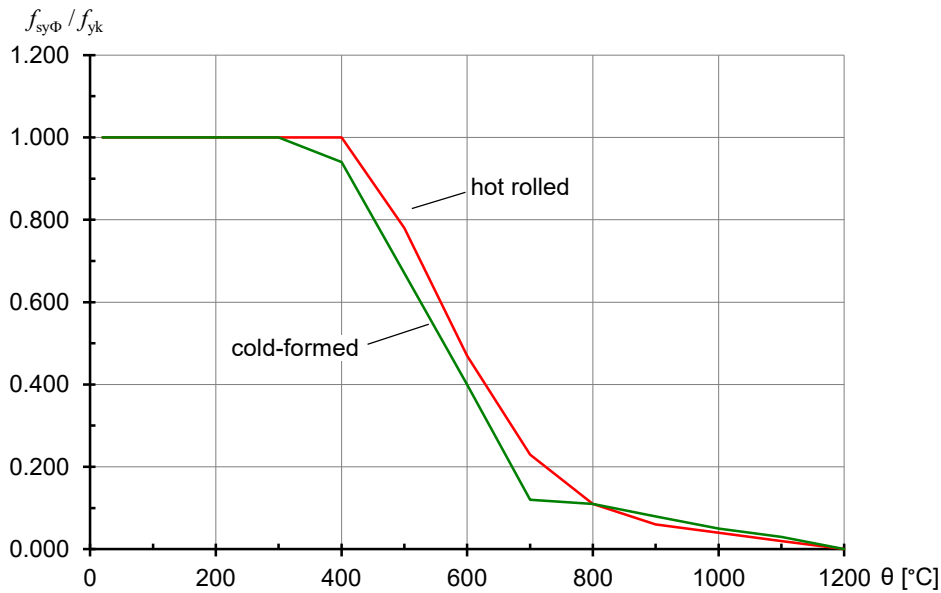
Thermal expansion (1:steel; 2:concrete with siliceous aggregate; 3:with calcareous aggregate; 4: timber)

The dependency of the stress-strain curves for concrete on temperature is clearly shown in the following diagram. Any tensile strength for concrete is ignored for fire scenarios according to EN 1992-1-2 (section 3.2.2.2).



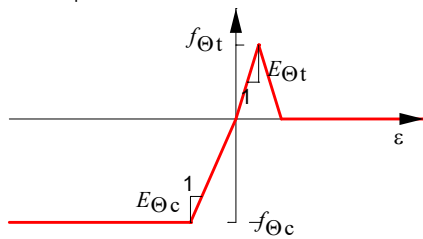
Related stress-strain curves for concrete with siliceous aggregate

The stress-strain curves for steel are also temperature dependent. They are derived from Figure 3.3 and Table 3.2a of EN 1992-1-2. The reduction in strength at higher temperatures is demonstrated in the following diagram.

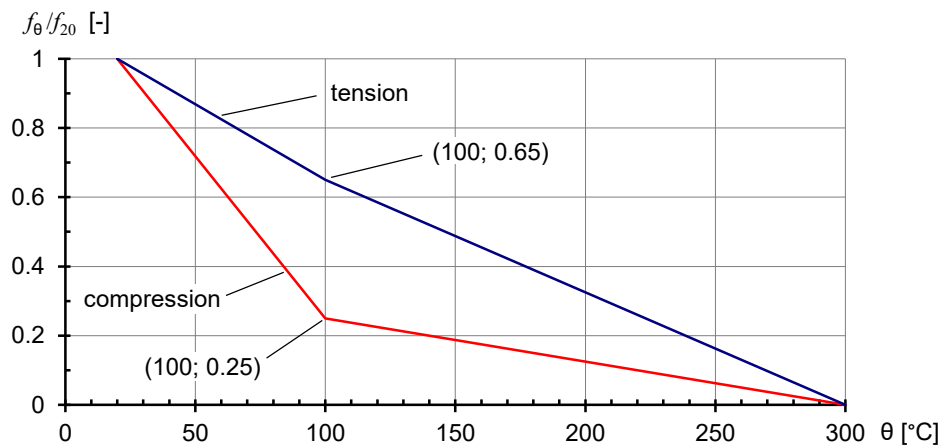


Reduction in strength of reinforcing and construction steel at higher temperatures

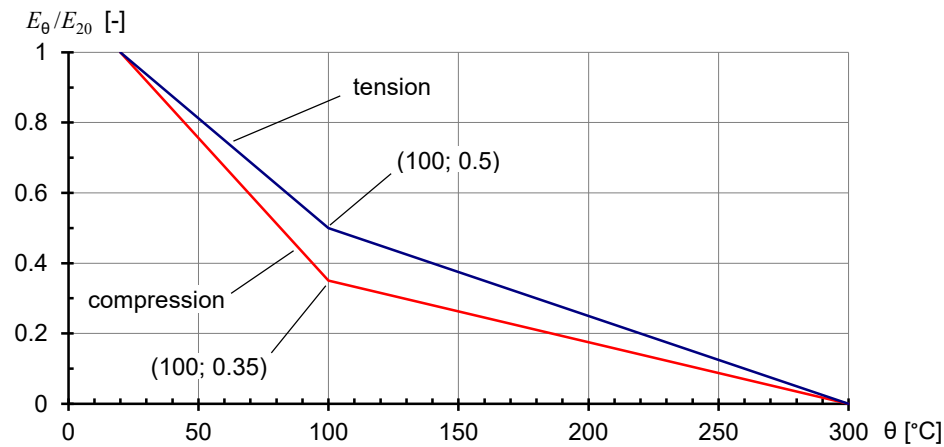
The stress-strain behavior of timber at increased temperatures is not specified in EN 1995-1-2. At the same time the consideration of the nonlinear material behavior is postulated. Therefore the following stress-strain curve with softening in the tensile zone and a bi-linear curve in the compression zone is used for frameworks. The strengths  $f_{\theta t}$ ,  $f_{\theta c}$  and the moduli of elasticity  $E_{\theta t}$ ,  $E_{\theta c}$  correspond to EN 338:2016 and EN 14080:2013 with consideration of the temperature-dependent decrease of strength. With this especially for construction members stressed by bending the resistance is underestimated compared to the design procedure according to EN 1995-1-2, Chapter 4.2. The reason for that are the lower compressive and tensile strengths ( $f_{\theta t}$ ,  $f_{\theta c}$ ) of the advanced calculation method compared to the average bending strength  $f_m$  used by the simplified check.



Stress-strain curve of timber



Decrease of strength of timber for increased temperatures



Decrease of elasticity modulus of timber for increased temperatures

Framework theory assumes that sections remain planar. The nonlinear thermal strain distribution over the section results in a similar nonlinear distribution of stress-creating strains. The stresses calculated here are integrated to internal forces over the section using the element mesh. The nonlinear strain state is varied as long as necessary during the equilibrium iteration through the addition of linear strain states until equilibrium is achieved. This method is implemented in the program as the standard algorithm.

An alternative procedure is based on the so-called 'Equivalent Temperature'. In this case, the nonlinear temperature distribution determined in the thermal calculations is linearized using a regression level before the analysis is carried out. While this approach fully complies with the framework theory, the influence of the stiffness distribution of the whole framework on the position of the regression level cannot be determined; therefore, this method does not satisfactorily take into account the thermal effects, in particular for reinforced concrete structures.

Area structures are calculated using a sandwich model analogous to the non-linear analysis under normal temperature (see 'Non-linear system analysis > area elements'). Thereby the biaxial structural behavior of reinforced concrete is taken into account. For area structures made of steel the Raghava yield criterion with the temperature dependent strengths is used as a basis. Area structures made of timber cannot be analyzed.

## Load Case Definition

A decisive load case must be defined for nonlinear analysis of fire scenarios. The following procedure for doing this is recommended:

- Grouping and weighting existing load cases using the *load group* load type.
- Selection of the second-order theory if geometrical nonlinearity should be considered in addition to physical nonlinearity.
- Addition of the *fire scenario* load type to the load case. The numbering of the fire scenario is based on the numbering in the *Thermal Analysis* dialog.

When different fire scenarios are to be investigated, a new load case must be created for each scenario.

## Analysis Settings

The following settings are made on the *Fire Scenario* tab in the *Settings* for the nonlinear analysis of the menu item *Analysis - Settings*.

### Consider the following load cases

The load cases to be analyzed and described above are to be entered in the left-hand selection field.

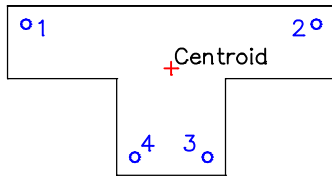
### Constantstiffness

The iteration is done with a constant stiffness matrix. If the switch is not set, then the 'Newton Raphson method' is used.

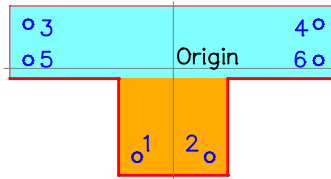
### Start reinforcement

Reinforcement is incorporated in the *analysis for fire scenarios* in a different manner to the analysis under standard load conditions. The initial reinforcement according to the dimensions of the reinforced concrete corresponds with the reinforcing steel layers in the basic section. The embedded steel layers in the thermal section are generated by copying or

redefining the section parts. This means that matching with the initial reinforcement in the basic section can only be carried out by comparing coordinates. In the example below, layers 1 to 4 are geometrically assigned when the 'Apply As from dimensioning' option is activated. It must be ensured that the centroid of the basic section coincide with the origin of the thermal section.



Basic section



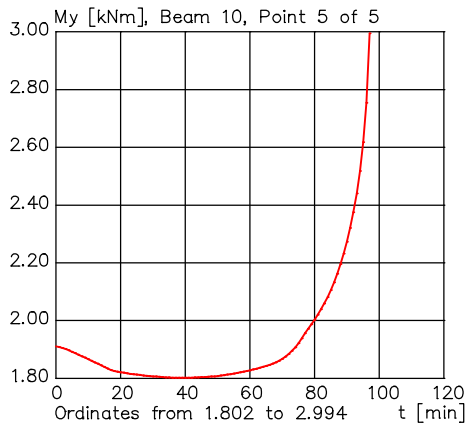
Thermal section

For the calculation of area elements the reinforcing steel layers of the basic section are always used. They are assigned to the thermal section depending on their height. Therefore, the center line of the thermal concrete section should be located on the zero line ( $z=0$ ) and the thickness of the basic section should coincide with the thickness of the thermal concrete section.

## Results

The deformations, internal forces and support reactions for the selected time steps are saved in the database for the check load cases selected in the settings dialog.

Additionally, for beams with thermal section the stress distribution of the section is stored at the result markers.



Variation of the bending moment  $M_y$  at a beam node over time

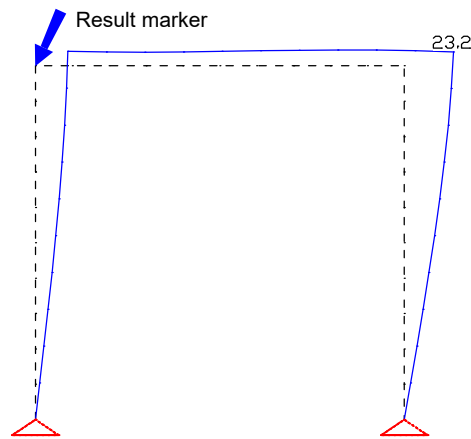
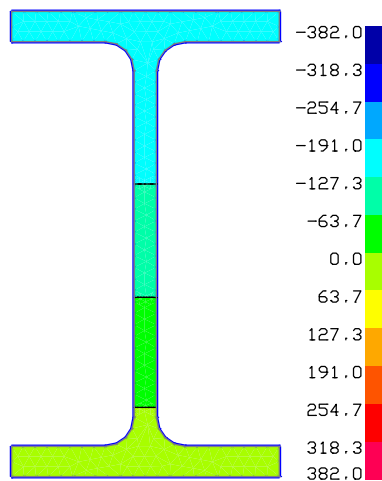
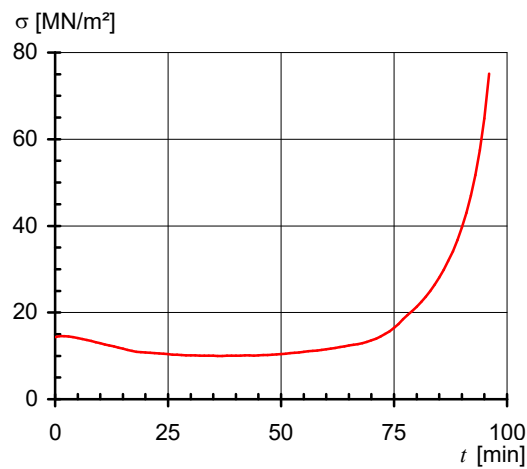


Illustration of the node displacements  $u$  [mm] for  $t = 90$  min



Distribution of the stress  $\sigma_x$  [MN/m<sup>2</sup>] at the result marker for  $t = 90$  min



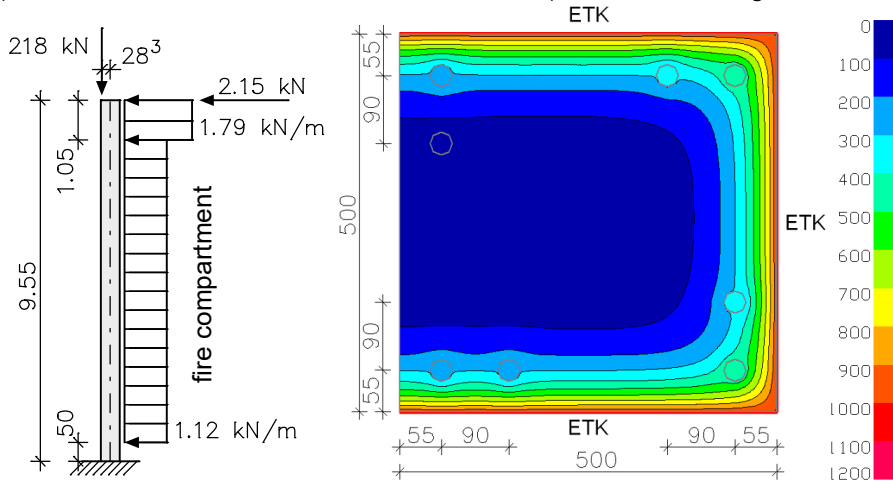
Stress curve  $\sigma_x$  at the result marker in the middle of the cross-section bottom edge

## Examples

The following example analyses are intended to demonstrate the range of applications possible with this program system and at the same time provide results that are validated experimentally.

### Gable Column

The following gable column was analyzed by *Richter*. The underlying static system together with the associated material parameters is shown below. The beam division corresponds to the load figure.



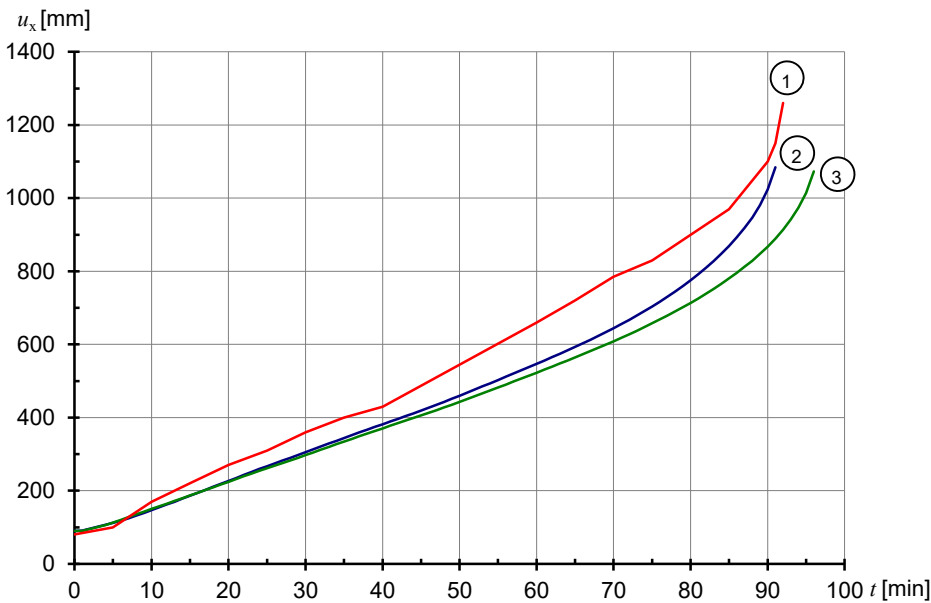
The system with loads and dimensions, section view and temperature profile after 90 min.

Material parameters:

Concrete: C30/37-EN, raw density at 20°C 2300 kg/m<sup>3</sup>, humidity ratio 0 %, Siliceous aggregate,  $\alpha_c = 25 \text{ W/(m}^2 \text{ K)}$ ,  $\epsilon_m = 0.7$

Reinforcing steel: BSt 500 S (A), 8 Ø 28 (hot-rolled), Axis distance from edge  $u = 55 \text{ mm}$

The calculations were carried out assuming flame exposure from three sides according to the standard temperature-time curve (ETK). Geometrical and physical nonlinearities were accounted for in the time-step analysis. Depending on the type of model used to represent the reinforcement, system failure occurred after 91 min (curve 2) or after 96 min (curve 3). For curve 3 the existing reinforcement was represented as structural elements. This influenced the temperature distribution, as can be readily seen on the temperature profile shown. In both cases the required R90 fire resistance class could be met.



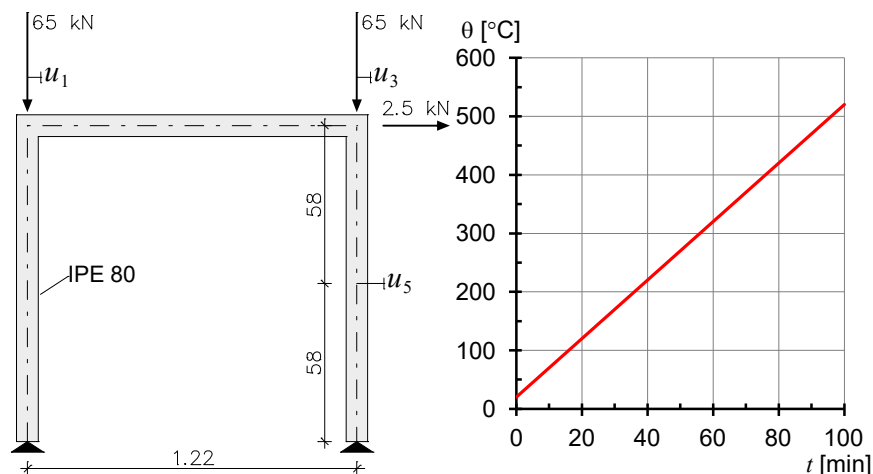
Horizontal Load-displacement curves for the column head

- 1: Richter: Structural Design for Fire Safety, Institute for Building Materials, Concrete Construction and Fire Protection, Techn. University of Braunschweig (IBMB TU Braunschweig), Example 6
- 2: InfoGraph: Temperature profile without consideration of reinforcement
- 3: InfoGraph: Temperature profile with consideration of reinforcement



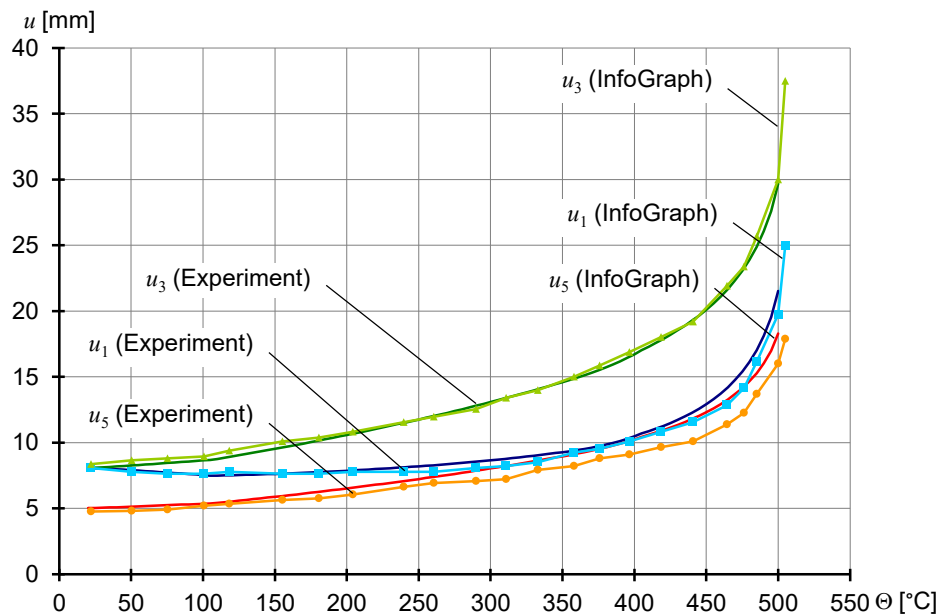
## Steel Frame

The large model frame shown below (Trial EGR 1c) was experimentally and analytically researched by *Rubert* and *Schaumann* in 1985. In the fire tests, a uniform temperature increase rate of approx. 5 K/min was achieved. According to the test report, deviations from the ideal geometry were negligible. All material characteristics were determined experimentally. As these are not fully documented, the load utilization  $1/\nu_u = 0.55$  and the system slenderness ratio  $\lambda_{sys} = 0.93$  were taken account of when calibrating the system at normal temperature. Using a jointed foot support and a tenfold division of the columns resulted in a slightly increased load utilization factor of 0.57. This difference was eliminated by applying a very small elastic foot restraint (8.5 kNm/rad).



The system with loads and dimensions, section view and temperature profile.

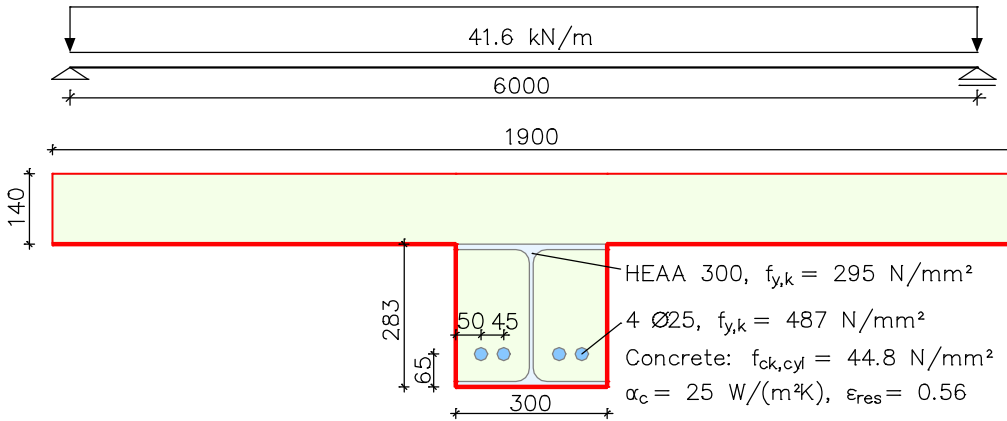
The temperature-displacement curves measured in the test for the points 1, 3 and 5 are shown in the following diagram alongside the calculated results. The curves show a very good correlation over the whole temperature range.



Experimental and comparative theoretical temperature-displacement curves

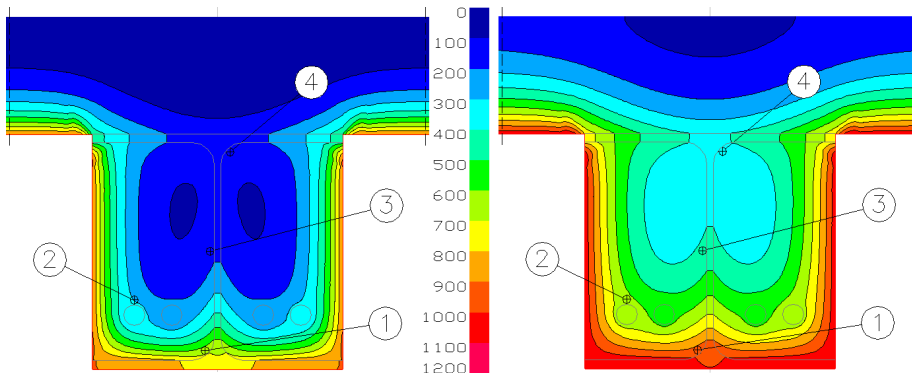
## Composite Girder

The results presented here were taken from Upmeyer's dissertation completed at the University of Hannover in 2001 and describe the results of *fire tests on concrete-encased composite girders*. The static system consisting of 10 beams is shown below together with the associated material parameters.

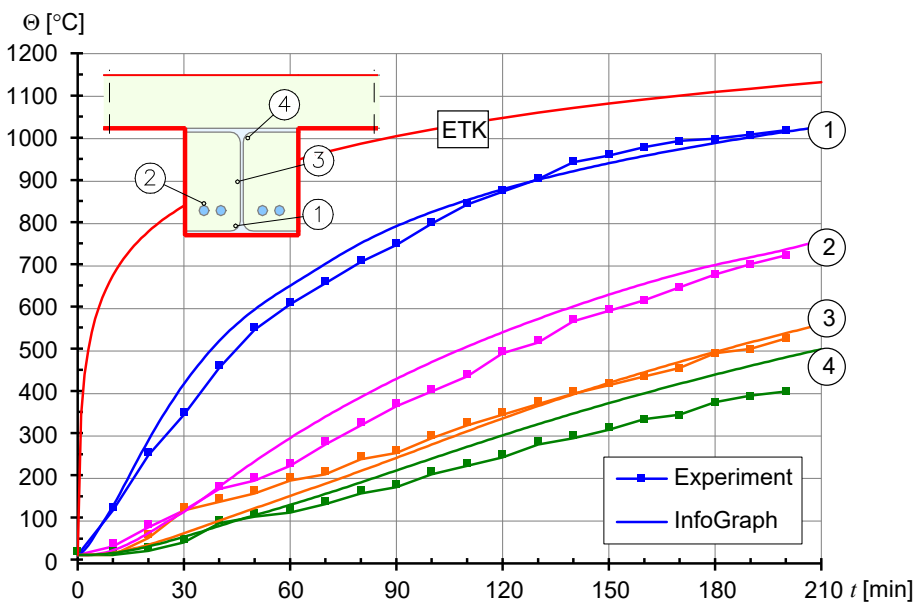


System with sections, loads and dimensions [mm]

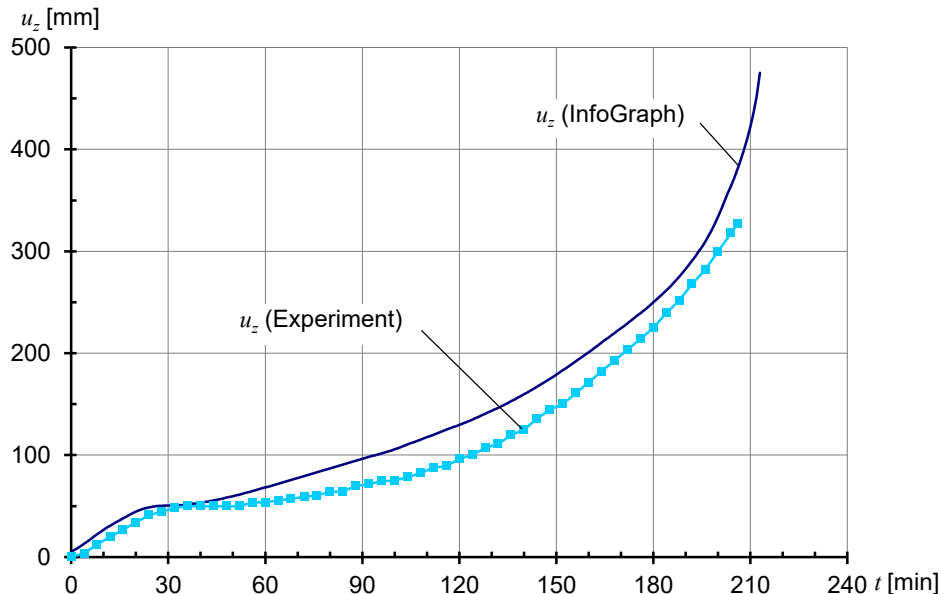
Component temperatures measured at four points are available along with the curves of displacement against time for the girder mid-point. The calculated development of the temperatures correlates strongly with the experimentally measured values. A single significant difference occurs at measuring point 4. This could either come from a variation in the moisture content of the concrete or in inaccuracy of the thermal conductivity resulting from bolts in the cold plate region of the girder which were not included in the model. In addition the exact position of the thermal elements is not known. This is particularly noticeable at measuring point 1 because of the large temperature gradient.



Temperature profile after 60 min and 150 min [°C]



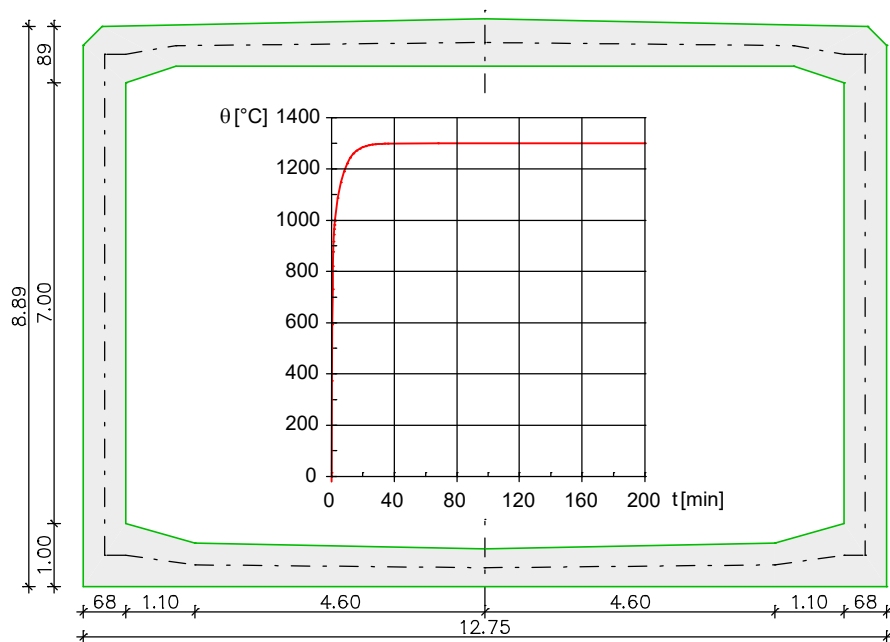
The good correlation between the calculated and measured curves for displacement against time and the time of failure are shown in the next diagram.



Midspan bending over time

## Tunnel Structure

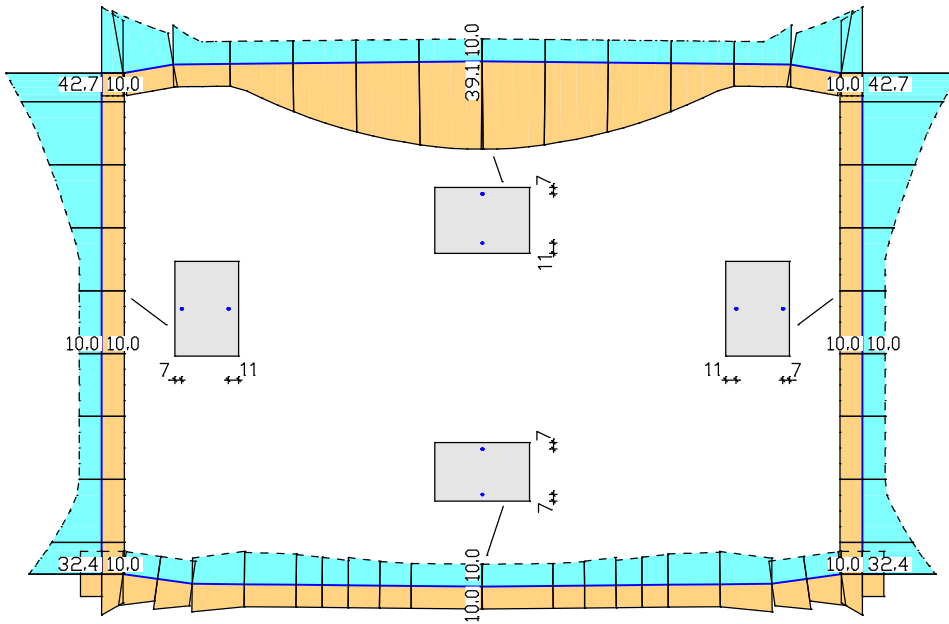
The following description of a tunnel structure are taken from the 'Fire Resistance of Fiber, Reinforced and Prestressed Concrete' ('Brandbeständigkeit von Faser-, Stahl- und Spannbeton'). Here, the internal forces and deformation after 180 min of fire exposure were investigated.



System with dimensions and temperature curve

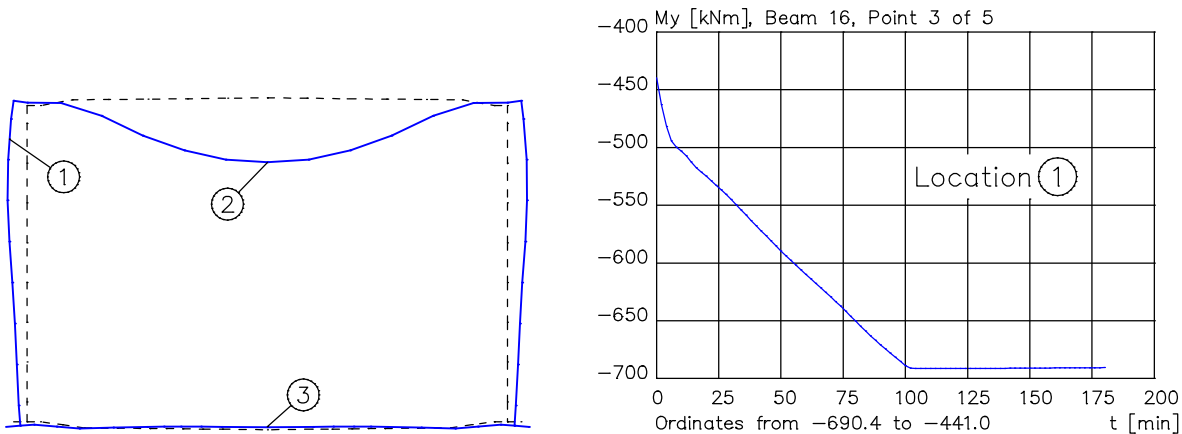
The base of the tunnel structure is flexibly bedded. The extensive section, material and load parameters are not relevant for the purposes of this description. Alongside the loads caused by fire, the action combination consists of dead load, buoyancy and fill forces as well as pressure from surrounding water and earth and traffic loads. The assumed temperature curve is shown above. The thermal section is geometrically identical with the basic section. Therefore the existing reinforcement could be automatically taken over. The reinforcing is distributed as shown below.

The handling of a similar model is also shown in the video (German language) "[https://download.infograph.de/video\\_de/Stahlbetonrahmen%20im%20Brandfall.mp4](https://download.infograph.de/video_de/Stahlbetonrahmen%20im%20Brandfall.mp4)".

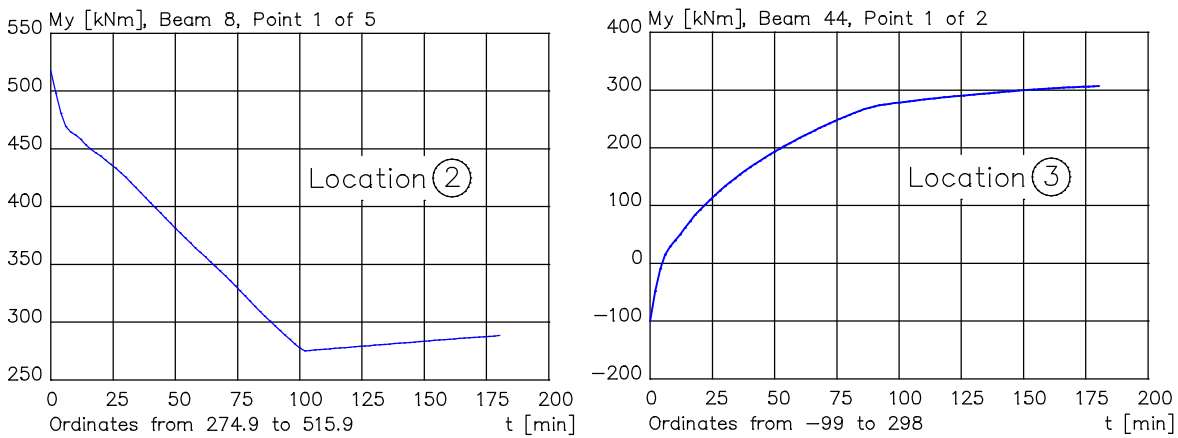


Distribution of reinforcement [cm²/m]

The combined loads result in negative moments being created in the regions around the frame corners. This means that the temperature of the tensile reinforcement is within the normal range. The lower reinforcement in the region around the horizontal frame member (location 2) is decisive for the load-bearing capacity. Because of the large overlap of 11 cm, after 180 min a temperature of approx. 250 °C was reached. This did not result in any significant reduction in strength.



Diagrams showing exaggerated deformation of the frame after 180 min and the moment curve over time



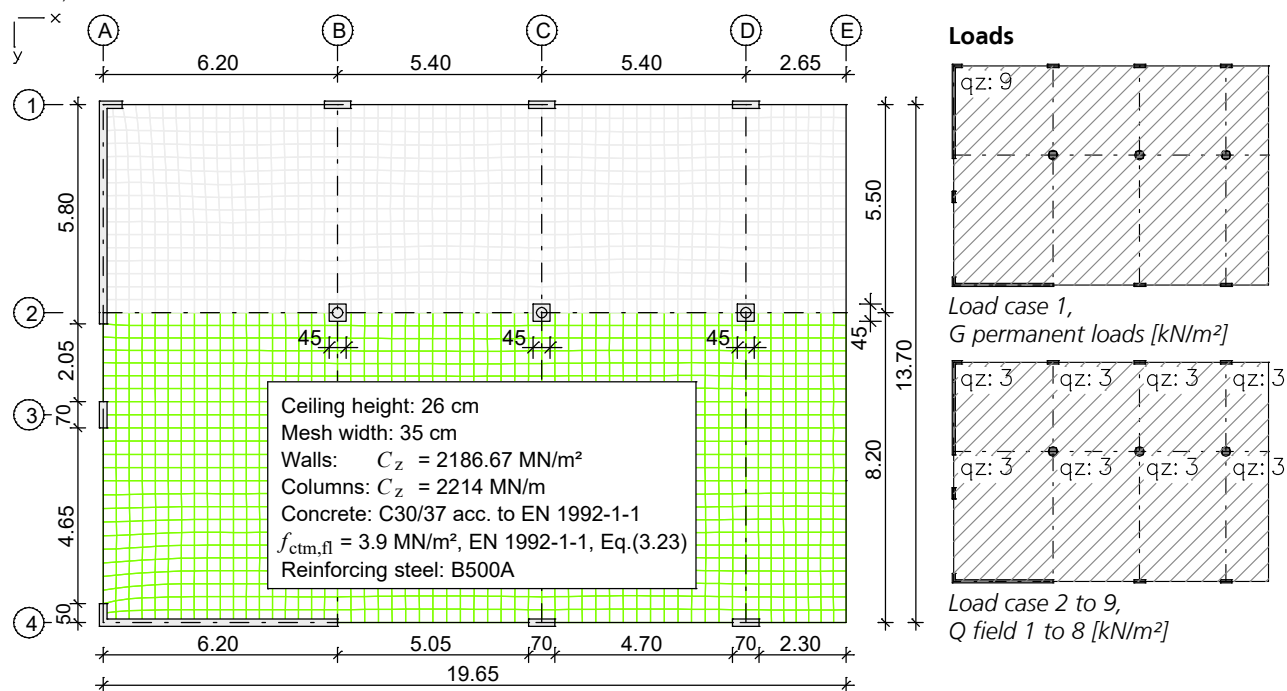
Moment curves over time

The moment curves show that after approx. 100 min a plastic joint forms at location 1 in the tunnel structure. The system as a whole is, however, still stable after 180 min. However, a different load situation would result in concrete flaking off in the region around the horizontal frame member (location 2). The higher temperatures would then extend further into the section and failure of the span reinforcement would lead to failure of the whole system.

## Ceiling Slab Under Fire Exposure

This example shows a ceiling slab, which has been analyzed before in the manual chapters *EN 1992-1-1 Design* and *Nonlinear structural analysis*. It shall be verified that the slab resists a fire exposure of 90 minutes.

In the **first calculation step** a thermal analysis of the ceiling slab is carried out for fire exposure (standard temperature-time curve) at the bottom of the slab.



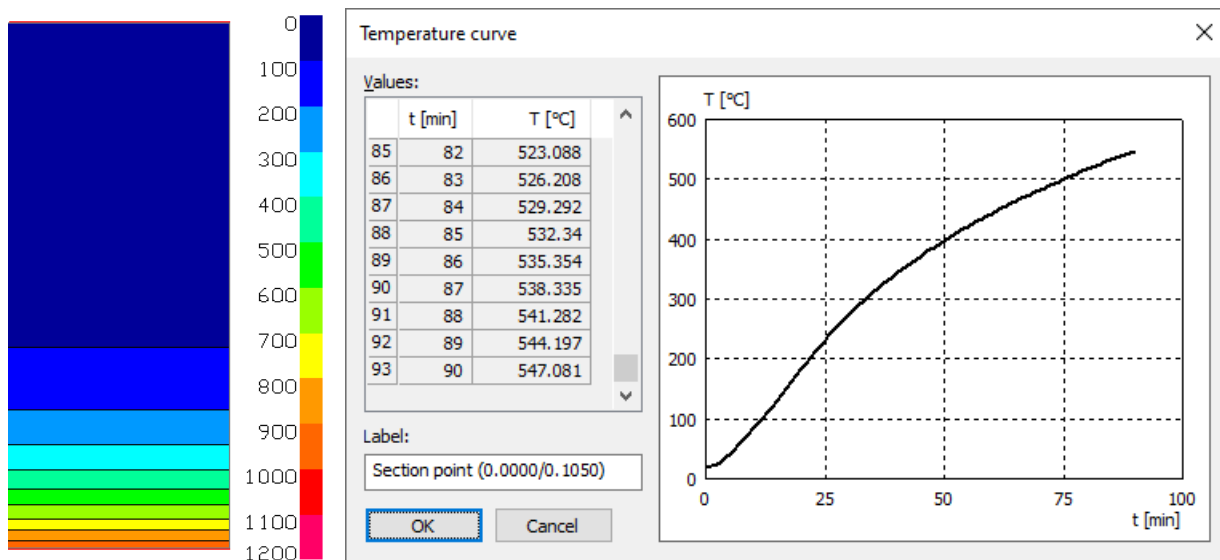
Element system and loads from "Beispiele zur Bemessung nach Eurocode 2 – Band 2: Ingenieurbau"

### Thermal section 1 - Ceiling slab

Starting temperature = 20 °C; Time increment = 1 min; Duration = 90 min

1. Part - (Section 1)		
	Material	C30/37-EN
	E-Modulus [MN/m²]	33000
	G-Modulus [MN/m²]	13800
	Gamma [kN/m³]	25
	Aggregate	Limestone-containing
	Raw density at 20° C [kg/m³]	2300
	Humidity ratio [%]	3
	Apply As from dimensioning	Yes
1. Fire scenario - (Section 1)		
	ETK (Standard temperature-time curve)	
	Emissivity	0.7
	Coefficient of heat transfer	25 W/(m²·K)

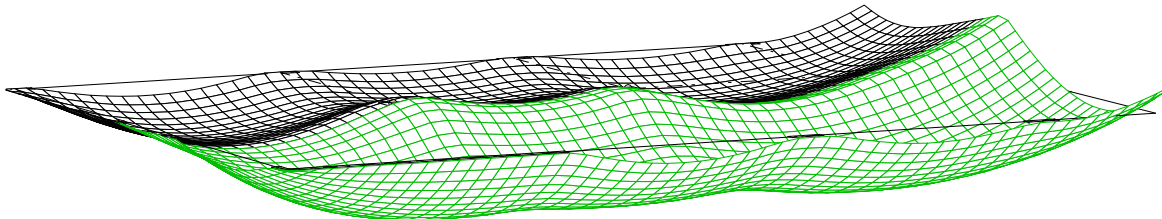
The following figure documents the results.



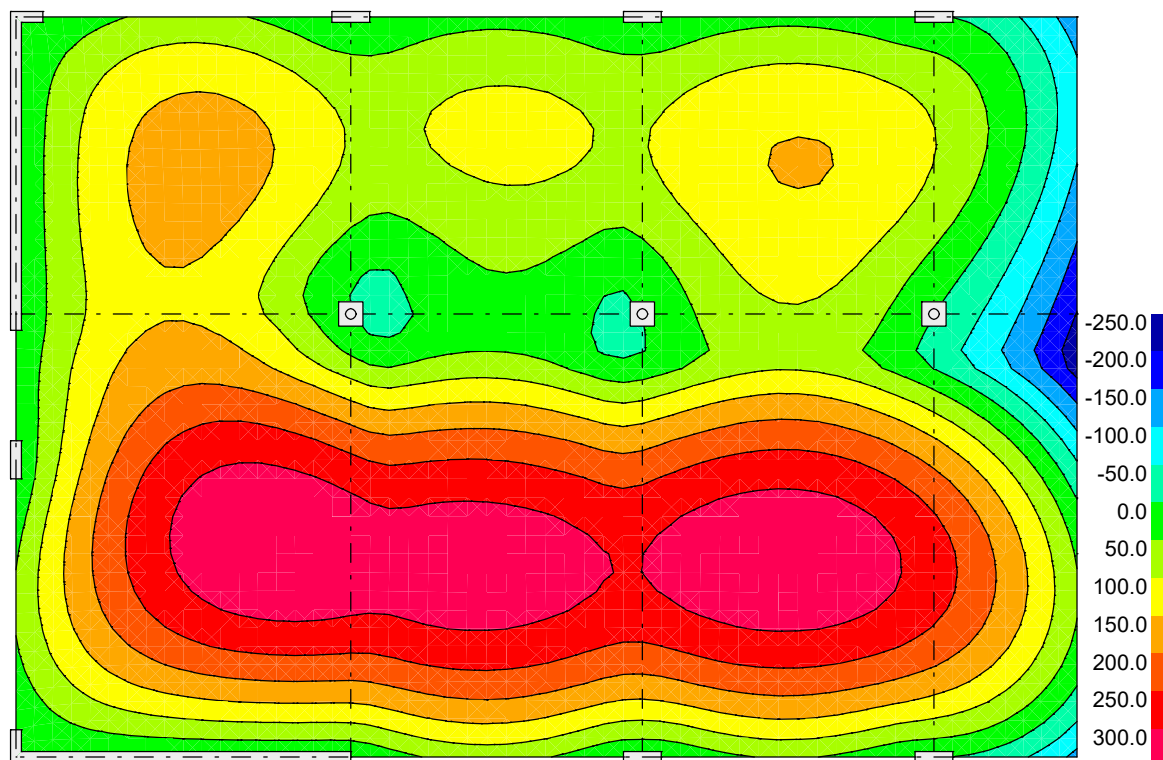
Temperature curve after 90 min at the section and over the time at the location of the lower reinforcement layer

In the **second calculation step** a nonlinear analysis for the fire scenario is carried out. As in the above-mentioned example, the longitudinal reinforcement from a previously performed design is taken as a basis.

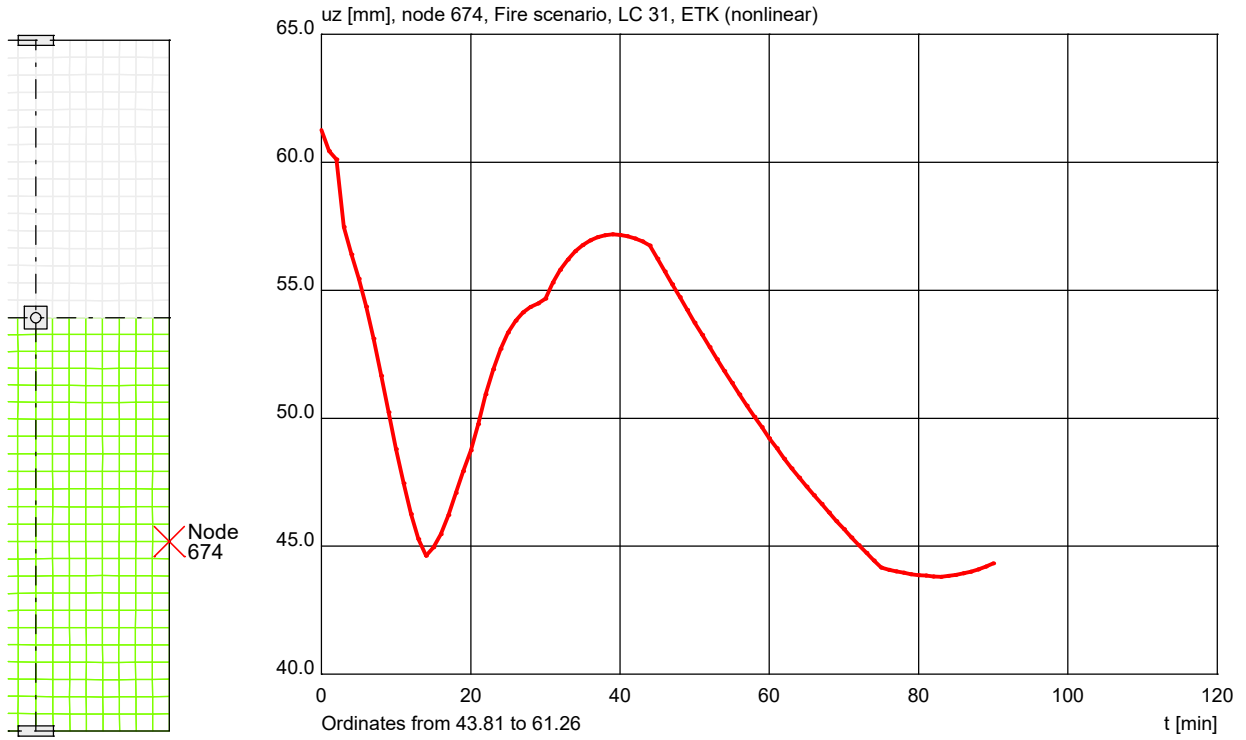
In consideration of the nonlinear material behavior in fire scenarios the load-bearing capacity can be verified for 90 minutes for the accidental combination with one fold dead load and 50% of the traffic load, because equilibrium could be reached in every time-step.



Deformation figure after 90 min with 5-fold superelevation



Color gradient of deformations  $u_z$  [mm] after 90 min under fire exposure (ETK)



Deformations  $u_z$  of node 674 under fire exposure (ETK)

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Part 1-2: General rules – Structural fire design.  
Publisher: DIN Deutsches Institut für Normung e.V., Berlin.  
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Publisher: CEN European Committee for Standardization, Brussels.  
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Publisher: Bundesanstalt für Straßenwesen (BASt). Date: 3/2012.

# Thermal Analysis of Solid Structures

## Area of Application

The *Thermal analysis* allows the analysis of stationary and instationary temperature distributions in solid structures of tetrahedron elements within the *Finite Elements* program systems. Any three-dimensional geometry can be analyzed. The material properties are described in the section *Thermal material properties*.

The analysis can be divided up into the following steps:

- The structure geometry is described with model object *Solid*.
- Properties like material and presets for the mesh generation are assigned to each solid model object.
- The thermal material properties are defined in the section dialog.
- After selection of the surface of a solid a surface property can be assigned to it.
- In the dialog *Thermal action* thermal boundary conditions are assigned to the surfaces.
- After the definition of all properties which are relevant for the structure the finite element model is generated with the mesh generator *Tetrahedrons from Solid*.
- Subsequently the thermal analysis is carried out.
- Additionally a static analysis can be performed considering the thermal strains due to a thermal action (load type *Thermal action*).

Multiple independent *Thermal actions* can be defined.

## Basics

The calculation of the temperature distribution for transient situations in solid bodies is carried out by solving the thermal conductivity equation as set out by Fourier. This has the following form:

$$\rho \cdot c_p \cdot \frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x} \cdot \left( \lambda \frac{\partial \theta}{\partial x} \right) + \frac{\partial}{\partial y} \cdot \left( \lambda \frac{\partial \theta}{\partial y} \right) + \frac{\partial}{\partial z} \cdot \left( \lambda \frac{\partial \theta}{\partial z} \right) + \dot{q} \quad (1)$$

with

$t$  Time [s]

$y, z$  Local coordinates [m]

Functions of  $y, z, t$

$\theta$  Temperature [°C]

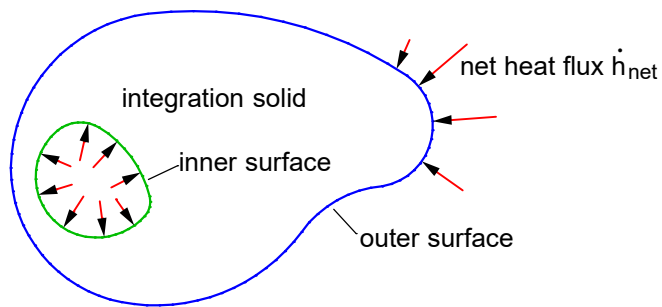
$\rho$  Apparent density [kg/m<sup>3</sup>]

$c_p$  Specific thermal capacity [J/(kg K)]

$\lambda$  Thermal conductivity [W/(m K)]

$\dot{q}$  Heat source (power density) [W/m<sup>3</sup>]

As all factors are dependent on time, the initial value problem needs to be solved in consideration of the boundary conditions. The temperature distribution in the integration solid at the time  $t = 0$  is taken as the initial condition.



For the problem being considered here, the thermal actions will be primarily determined by the net heat flux  $\dot{h}_{net}$  [W/m<sup>2</sup>] normal to the surfaces. This, in turn, comprises a convection and radiation component.

$$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r} \quad (2)$$

The convective component is calculated as follows:

$$\dot{h}_{net,c} = \alpha_c (\Theta_g - \Theta_m) \quad (3)$$

with

$\alpha_c$  Coefficient of heat transfer [W/(m<sup>2</sup> K)]

$\Theta_g$  Gas temperature in the region of the component under load [°C]

$\Theta_m$  Temperature at the surfaces of the component [°C]

The net heat flux between two surfaces due to radiation is calculated as follows:

$$\dot{h}_{net,r} = \Phi \cdot \varepsilon_1 \cdot \varepsilon_2 \cdot \sigma \cdot [(\Theta_1 + 273)^4 - (\Theta_2 + 273)^4] \quad (4)$$

with

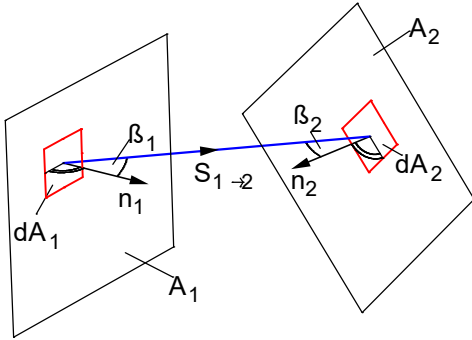
$\Phi$  Configuration factor

$\varepsilon_1, \varepsilon_2$  Emissivity of the component surfaces [-]

$\sigma$  Stephan-Boltzmann constant [= 5.67 · 10<sup>-8</sup> W/(m<sup>2</sup> K<sup>4</sup>)]

$\Theta_1, \Theta_2$  Temperature at the component surfaces [°C]

The configuration factor is determined by the geometry of the surface, the distance between the surfaces and shadowing effects. The following figure shows the underlying parameters.



The configuration factor can be determined as follows:

$$\varphi_{1 \rightarrow 2} = \frac{1}{\pi \cdot A_1} \cdot \int_{A_1} \int_{A_2} \frac{\cos \beta_1 \cdot \cos \beta_2}{s_{1 \rightarrow 2}^2} \cdot dA_1 \cdot dA_2$$

In the case of radiation due to fire follows:

$$\Phi = 1$$

$$\varepsilon_1 = \varepsilon_m \quad \text{emissivity of the component surface [-]}$$

$$\varepsilon_2 = \varepsilon_f \quad \text{emissivity of the flame (here } \varepsilon_f = 1) \text{ [-]}$$

$$\Theta_1 = \Theta_r \quad \text{effective radiation temperature of the fire (here } \Theta_r = \Theta_g) \text{ [}^\circ\text{C]}$$

$$\Theta_2 = \Theta_m \quad \text{surface temperature at the component boundary [}^\circ\text{C]}$$

The gas temperature  $\Theta_g$  is assumed to be known and is generally defined according to particular temperature-time curves.

The nonlinear boundary problem set by equations (1) to (4) cannot be solved in a closed form. Instead, the finite element method is used to solve the problem. In this case, Galerkin's method followed by partial integration is used to convert the boundary problem described above into a system of variation equations.

The program system uses compatible elements with a fully quadratic approach. This results in a high degree of accuracy, which is largely independent of the mesh geometry.

The element approach automatically satisfies the so-called *adiabatic boundary conditions*, meaning that thermal energy is only exchanged where boundary conditions are explicitly specified.

The nonlinear initial value problem is solved using time-step integration in conjunction with an incremental Newton-Raphson algorithm. After successful calculation, the temperature profiles for all times considered are available for further calculation.

## Input Data

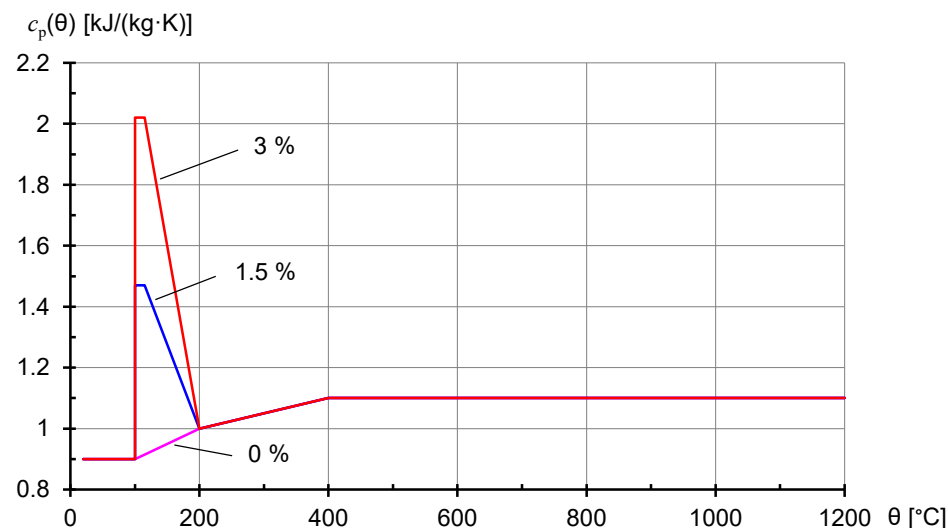
### Thermal Material Properties

The used curves of the temperature dependent material properties are described subsequently. The necessary user-defined values are to be specified in the section dialog.

All material values outside the temperature range between 20 °C and 1200 °C are assumed to be constant.

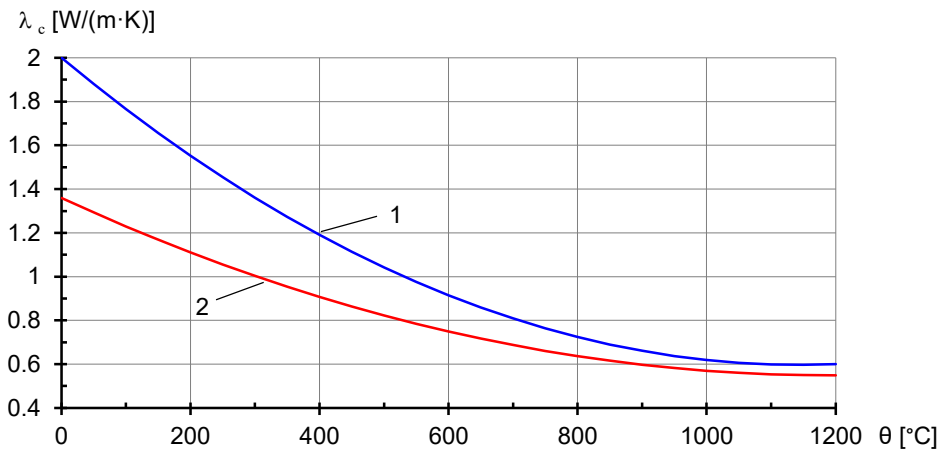
#### Concrete

All concrete types are assigned the temperature-dependent *specific heat capacity*, the *thermal conductivity* and the *raw density* as specified in EN 1992-1-2. According to DIN EN 1992-1-2/NA:2010-12 the upper limit function of the *thermal conductivity* in 3.3.3 (2) is used for DIN concrete. The lower boundary value is used for other kinds of concrete. The *humidity ratio* and *raw density at 20° C* can be specified by the user.

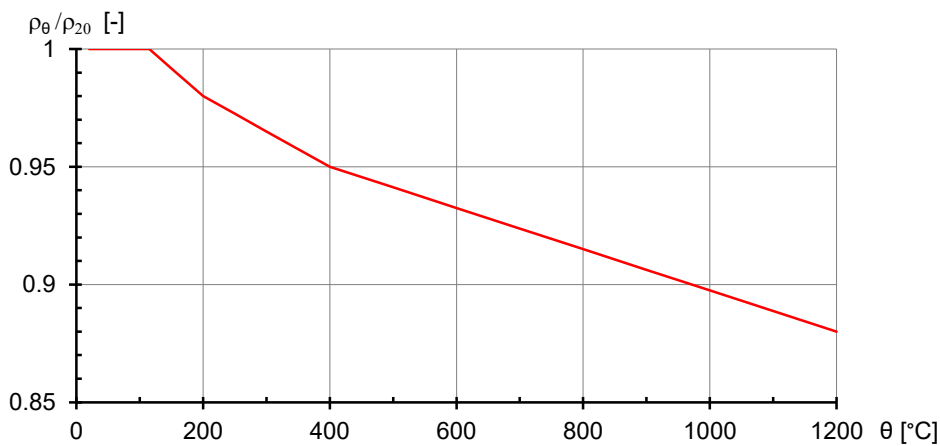


The specific heat capacity of siliceous and calcareous concrete  $c_p(\theta)$  depending on the temperature (moisture content  $u = 0, 1.5$  and  $3$  % by weight)

Moisture contents between the values specified will be linearly interpolated.



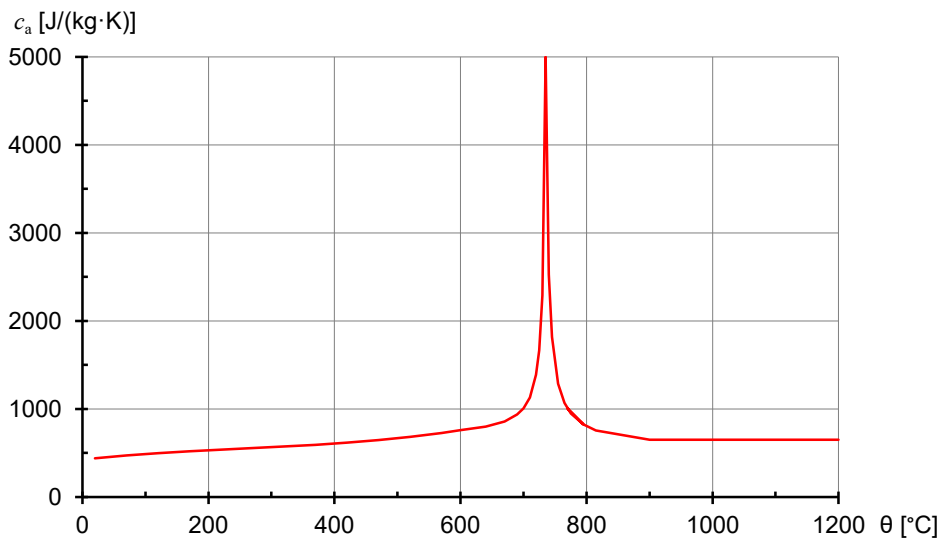
Temperature-dependent thermal conductivity  
 1: upper limit (DIN concrete); 2: lower limit



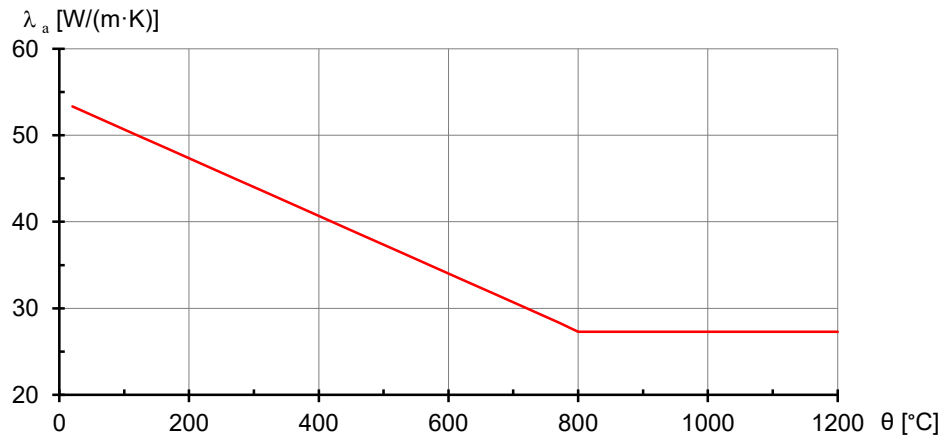
Temperature-dependent raw density ratio of concrete

**Steel**

All steel types are assigned the temperature-dependent *specific heat capacity* and the *thermal conductivity* as specified in EN 1993-1-2.



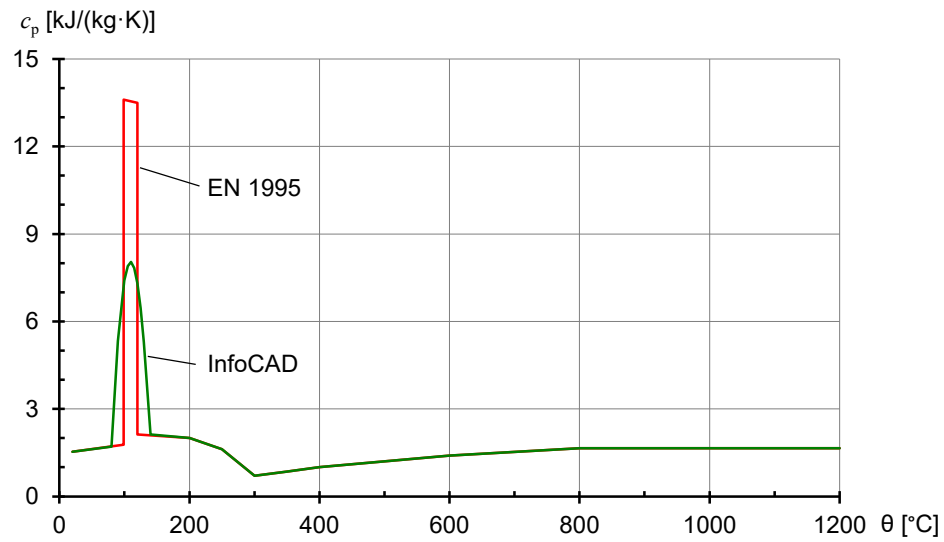
The temperature-dependent specific heat capacity for carbon steel



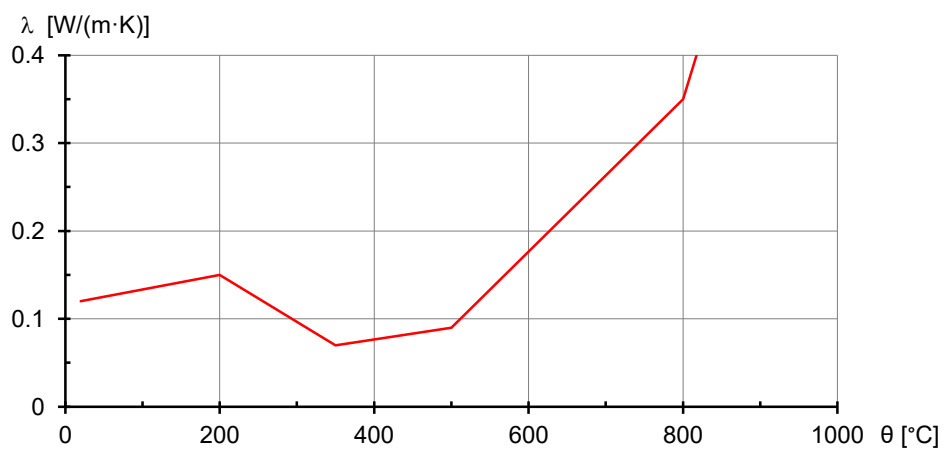
The temperature-dependent conductivity for carbon steel

### Timber

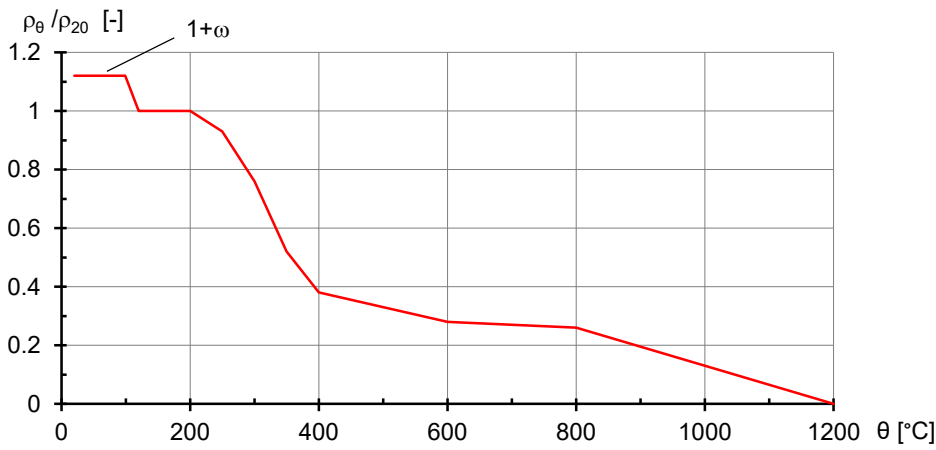
All timber types are assigned the temperature-dependent *specific thermal capacity*, the *thermal conductivity* and the *raw density* as specified in EN 1995-1-2.



The temperature-dependent specific heat capacity for timber



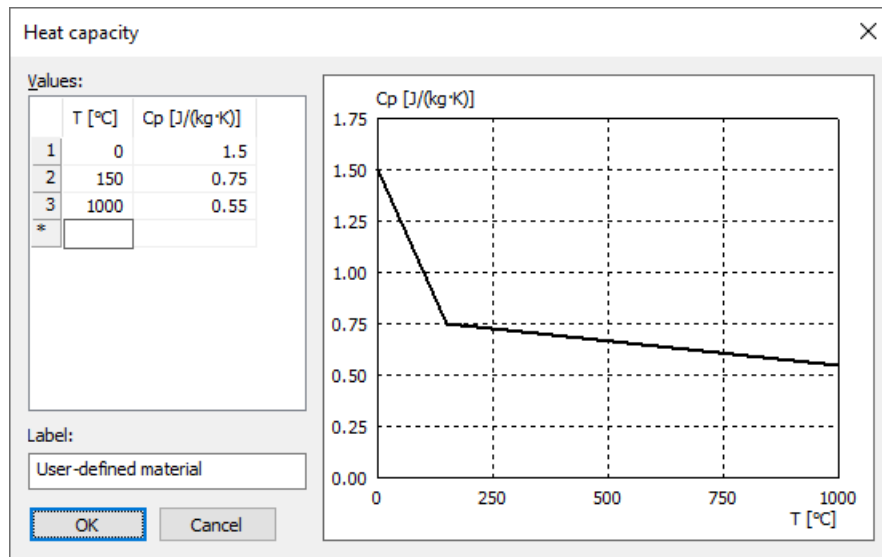
The temperature-dependent conductivity for timber



The temperature-dependent raw density ratio for timber with an initial moisture content  $\omega$  of 12 %

### User-defined material

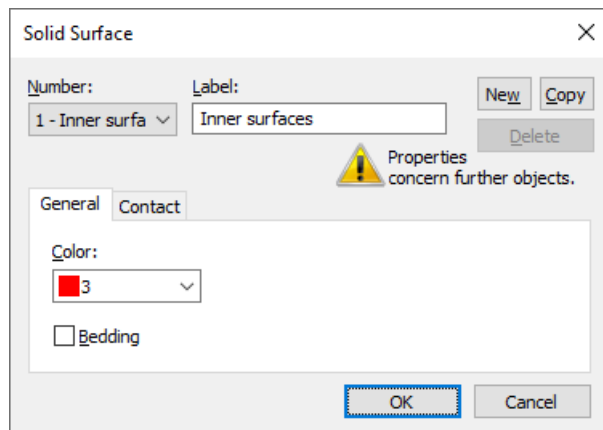
User-defined section parts are used to specify, for example, insulating materials, where any temperature-dependent *specific heat capacity*, *apparent density* and *thermal conductivity* can be specified.



User-defined thermal conductivity

### Solid Surface

With the context menu surface properties can be assigned to each surface of a solid model object. At contact faces the thermal conduction proceed in the same way as in a monolithic structure, if they have identical element dimensions and touch each other at the initial state.



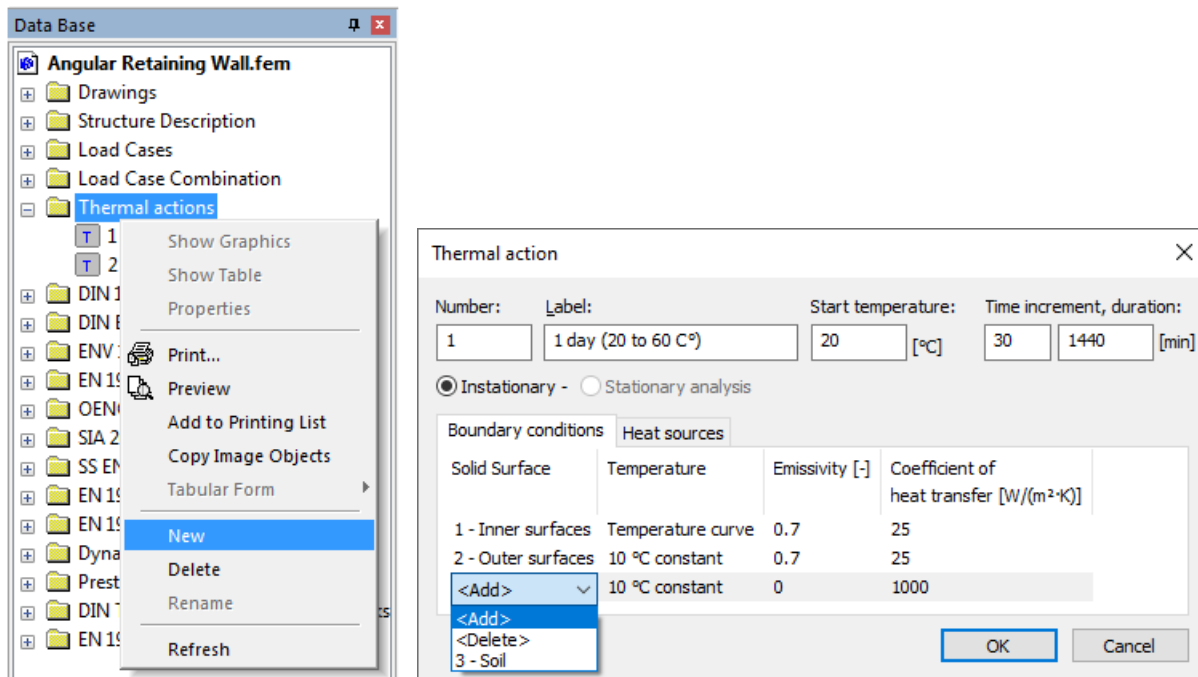


## Thermal Actions

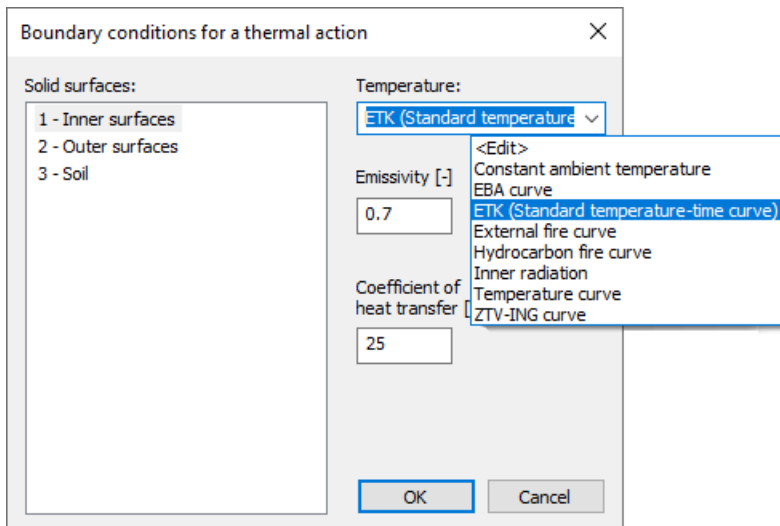
New thermal actions are created using the context menu of the data base. The analysis is performed separately for each thermal action.

In the dialog of the actual thermal action the following properties are defined:

- *Instationary* or *Stationary analysis*: For an instationary analysis additionally the start temperature [°C], the time increment and duration of the analysis [min] are to be specified.
- For each solid surface the boundary conditions described below can be assigned.
- Additionally a heat source can be defined for each section. For the power density [W/m<sup>3</sup>] of the heat source of the material a constant value or a user-defined time-dependent curve can be specified.



### Boundary conditions of solid surfaces



The thermal action on a surface is defined by the following boundary conditions:

- Temperature curve of the action, e.g. *ETK (Standard Temperature-Time Curve)*
- *Emissivity [-]*
- *Coefficient of heat transfer [W/(m<sup>2</sup> K)]*

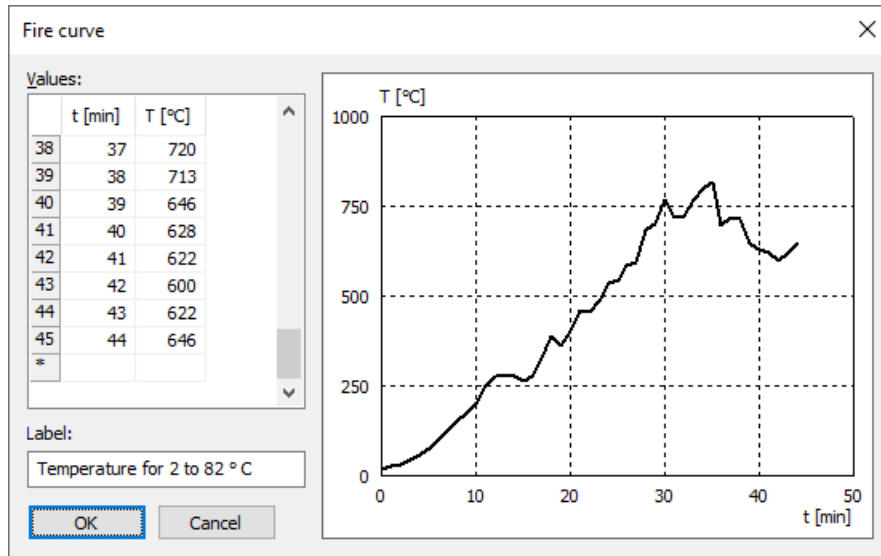
Surfaces which have no boundary conditions specified in this thermal action behave adiabatic. To surfaces which irradiate each other the property *inner radiation* can be assigned.

The convective coefficient of heat transfer can be taken from EN 1991-1-2, Appendix B.

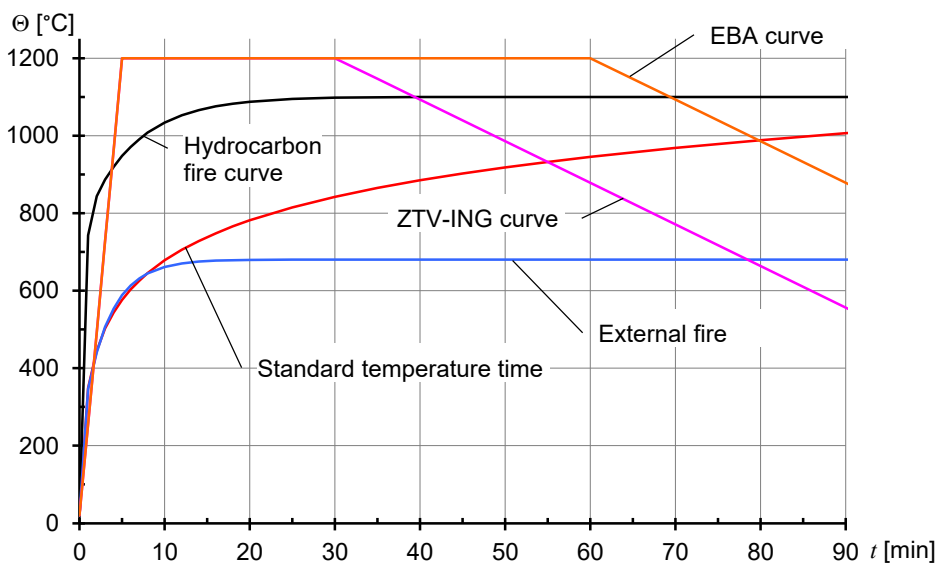
The emissivity of concrete and construction steel surfaces should be determined according to EN 1992-1-2 or EN 1993-1-2, Chapter 2.2 (2) for a default value of 0.7. According to Appendix C of the standard, the emissivity for stainless steel should be set to 0.4.

The following temperature curves can be used to determine the progression of the action.

- *Constant ambient temperature*: The ambient temperature can be set.
- *EBA curve*: Fire curve according to EBA-Guideline.
- *ETK (Standard Temperature-Time Curve)*: Fire curve according to EN 1991-1-2 or prEN 13501-2.
- *External fire curve*: Fire curve according to EN 1991-1-2.
- *Hydrocarbon fire curve*: Fire curve according to EN 1991-1-2.
- *Inner radiation*: Surfaces which irradiate or are affected by radiation.
- *User-defined temperature curve*: Temperature curve defined by the user.
- *ZTV-ING curve*: Fire curve according to ZTV-ING Part 5.



User-defined temperature curve



Fire curves according to EN 1991-1-2, ZTV-ING Part 5 resp. EBA-Guideline

## Thermal Analysis

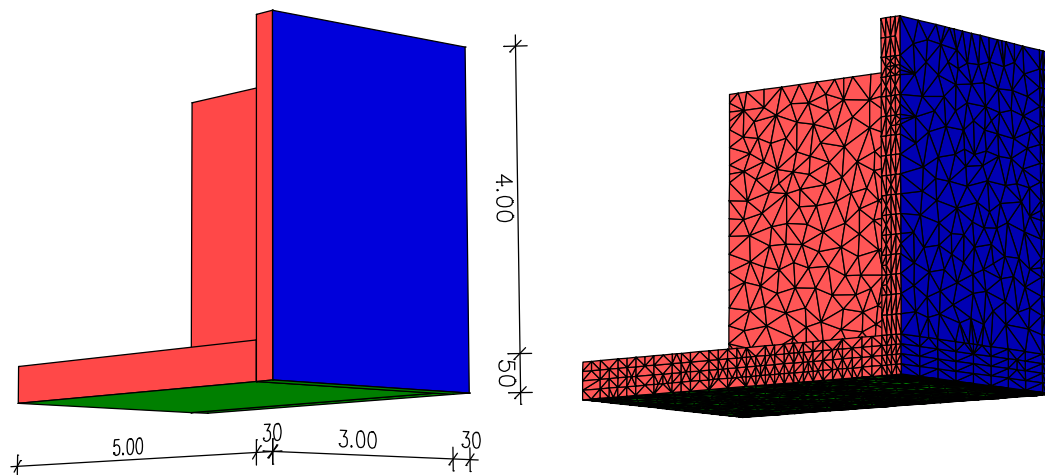
With the function *Thermal analysis* of the analysis menu all defined thermal actions are calculated.

## Examples

The following examples shall demonstrate possible applications of the program and also be used for validation of the attained results.

### Instationary Temperature Progression in an Angular Retaining Wall

This example simulates the temperature progression during one day in an angular retaining wall.



Solid model

Tetrahedron elements

#### Material properties

No.	Type	E-Modu. [MN/m <sup>2</sup> ]	G-Modu. [MN/m <sup>2</sup> ]	Poiss. ratio	alpha.t [1/K]	gamma [kN/m <sup>3</sup> ]
1	C35/45-EN	34000	14200	0.20	1.000e-05	25.000

#### Thermal properties

Material	Raw density [kg/m <sup>3</sup> ]	Humidity ratio [%]	Heat capacity [J/(kg·K)]	Heat conductivity [W/(m·K)]
1 - C35/45-EN	2300	1.5		

#### Bedding

No.	Start point [MN/m <sup>3</sup> ]			End point [MN/m <sup>3</sup> ]			Bedding width [m]		
	kbx	kby	kbz	kbx	kby	kbz	bx	by	bz
1	10	10	100						

The bedding performs in the direction of the axes of the local element or surface system.

#### Solid surfaces

Number	Label	Color	Bedding	Slave of	Factor Contact-stiffness	Coefficient of friction	Influence radius [m]
1	Air side	Red					
2	Outside	Blue					
3	Soil	Green	X				

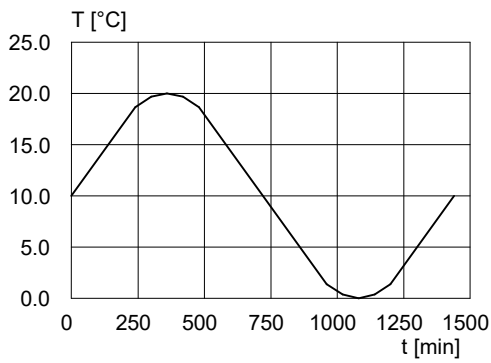
**Thermal action 1 - 1 day (0 to 20 °C)**

Instationary analysis. Starting temperature = 10 °C; Time increment = 30 min; Duration = 1440 min

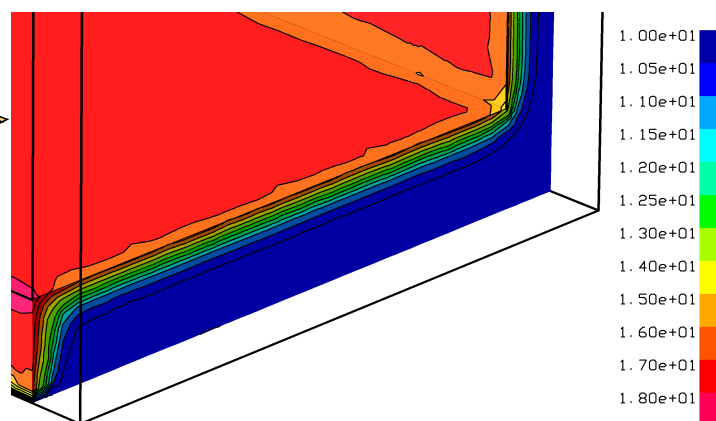
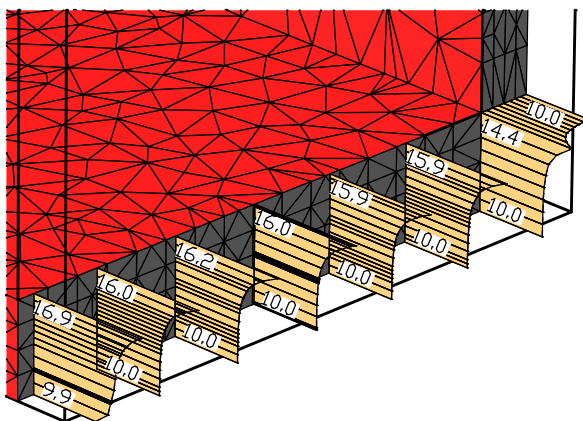
**Boundary conditions**

Solid Surface	Temperature	Emissivity [-]	Coefficient of heat transfer [W/(m <sup>2</sup> ·K)]
1 - Air side	Temperature curve	0.7	25
2 - Outside	10 °C constant	0	1000
3 - Soil	10 °C constant	0	1000

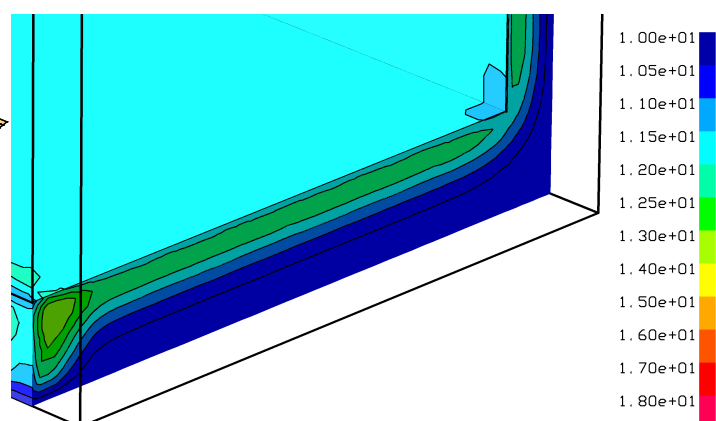
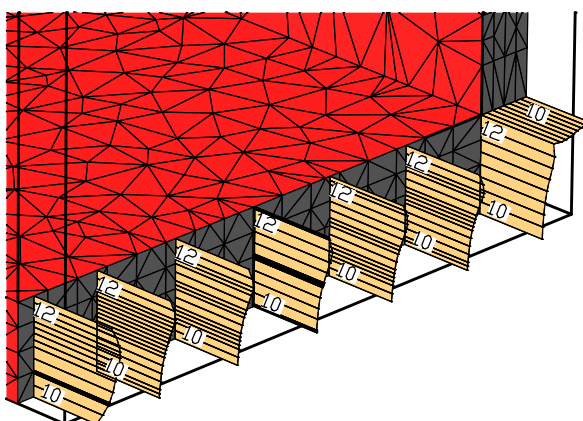
Temperature curve 1 - Days temperature 0 to 20 °C



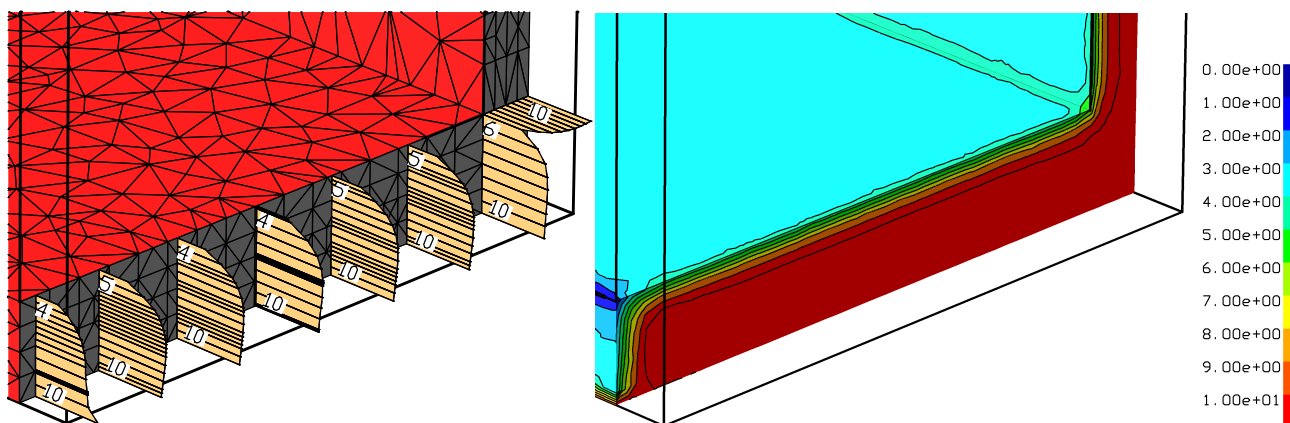
Pt.	t [min]	T [°C]
1	0	10
2	240	18.6603
3	300	19.6593
4	360	20
5	420	19.6593
6	480	18.6603
7	960	1.33975
8	1020	0.340742
9	1080	0
10	1140	0.340742
11	1200	1.33975
12	1440	10



Temperature distribution [°C] after 6 hours at the intersection face between wall and foundation slab

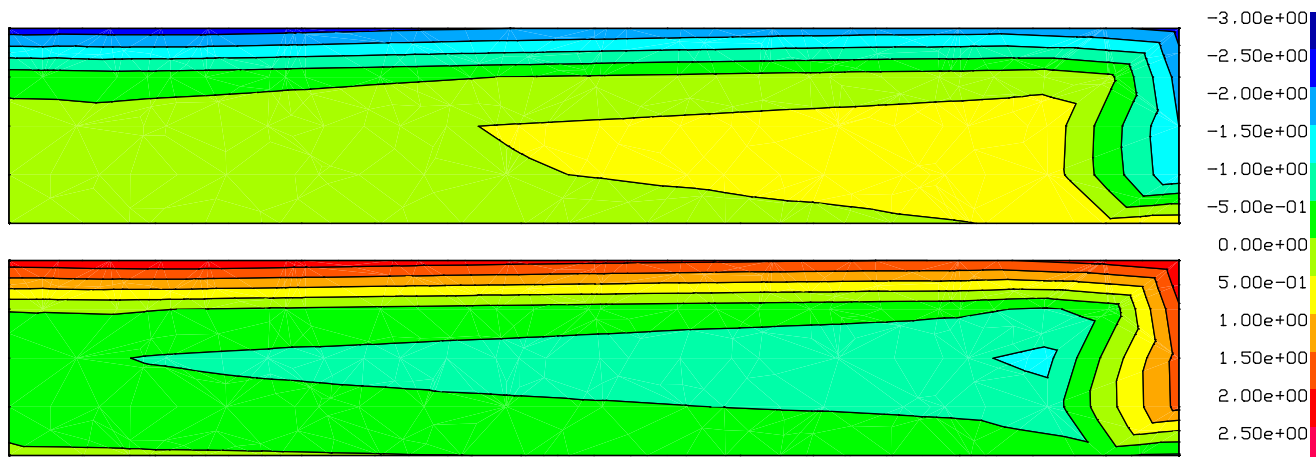
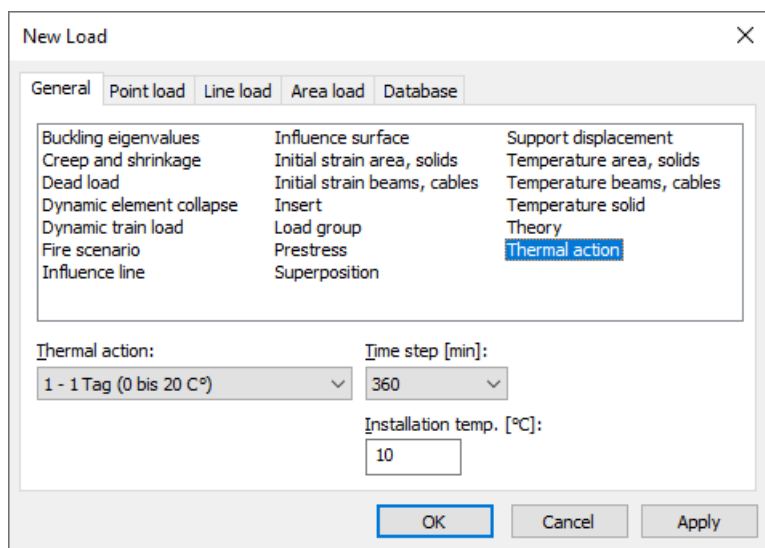


Temperature distribution [°C] after 12 hours at the intersection face between wall and foundation slab



Temperature distribution [°C] after 18 hours at the intersection face between wall and foundation slab

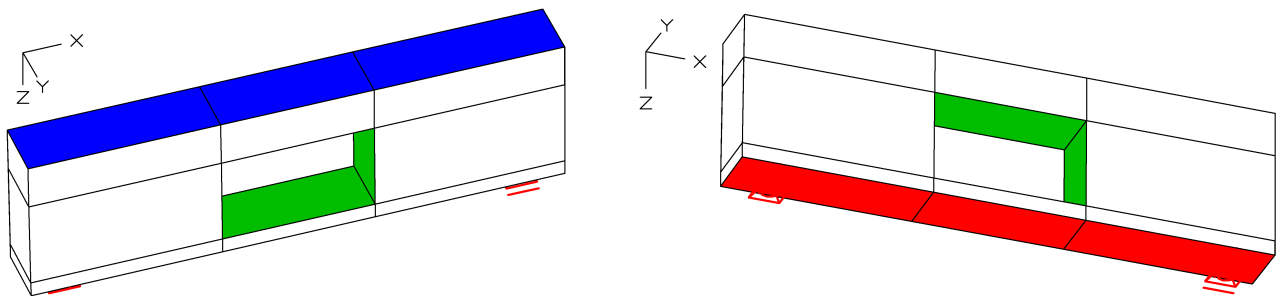
The stresses were determined at the time of the maximum and minimum air temperature (6/18 h). A installation temperature of 10 °C was assumed.



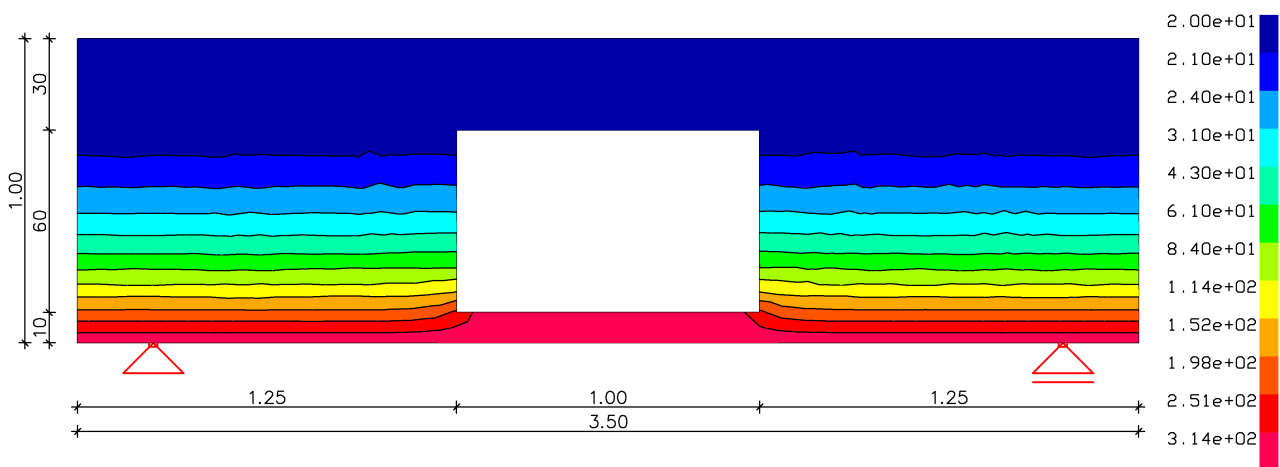
Stresses [MN/m<sup>2</sup>] after 6 and 18 hours perpendicular to the intersection face in the center of the foundation slab.

## Comparative Calculation With and Without Radiation

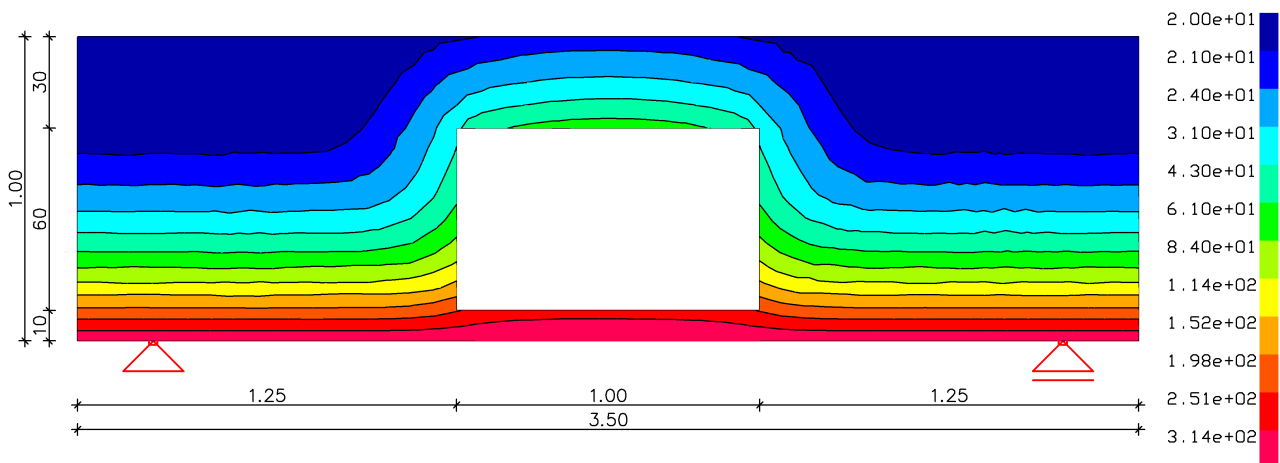
This example illustrates the effect of inner radiation on the temperature distribution. The pictured 40 cm thick concrete body C20/25 (EN 1992-1-1, raw density 2300 kg/m<sup>3</sup>, humidity ratio 2 %) has depending on the radiation considerably different temperature distributions after 10 hours under temperature exposure. The radiation is applied at the four inner surfaces of the aperture. All defined surfaces have an emissivity of 0.7. The coefficient of heat transfer is 25 W/(m<sup>2</sup>·K). Surfaces to which no boundary conditions were assigned (white) behave adiabatic.



Solid model with surfaces 1. constant temperature 400 °C (red); 2. radiation (green); 3. outer surface 20 °C (blue).



Temperature  $T$  [K] without inner radiation (adiabatic) after 10 h



Temperature  $T$  [K] with inner radiation after 10 h

## Stationary Temperature Distribution

In the following example a stationary temperature distribution with heat source is calculated. The example was taken from the textbook *Methode der finiten Elemente*, Schwarz (1984), p. 285.

### Task definition

- Heat source  $\dot{q} = 20 \text{ W/m}^3$  in G
- $T = 0 \text{ }^\circ\text{C}$  at AB
- Adiabatic surface at BD, DE, EF, LM, MA
- Coefficient of heat transfer  $\alpha = 2 \text{ W/(m}^2\cdot\text{K)}$ ,  $T = 0 \text{ }^\circ\text{C}$  at FHIKL
- Conductivity  $1.0 \text{ W/(m K)}$

### Solid surfaces

	Number	Label	Color	Bedding	Slave of	Factor Contact-stiffness	Coefficient of friction	Influence radius [m]
1	1	T = 0; Alpha = 2	Green					
2	2	T = 0; Alpha = oo	Blue					

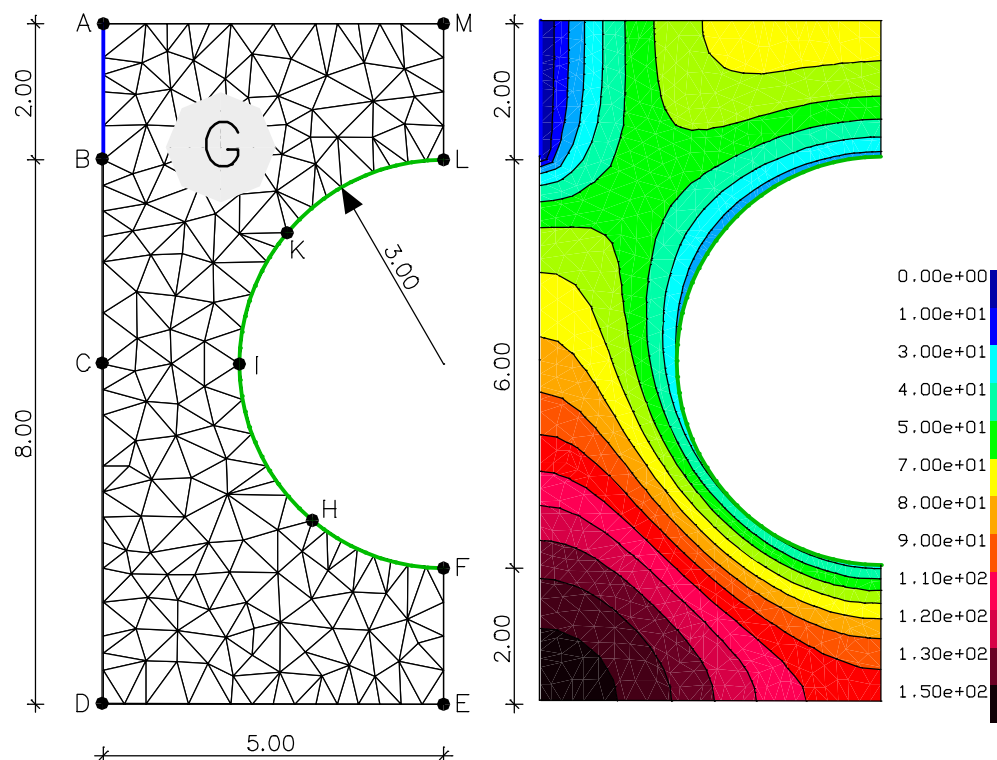
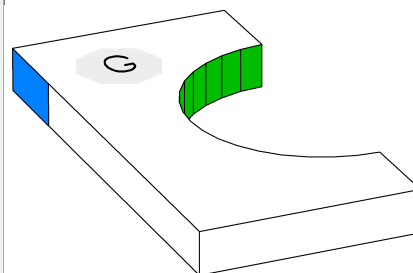
**Thermal action** ✕

Number:  Label:  Start temperature:  [°C] Time increment, duration:  [min]

Instationary -  Stationary analysis

Boundary conditions Heat sources

Solid Surface	Temperature	Emissivity [-]	Coefficient of heat transfer [W/(m²·K)]
1 - T = 0; Alpha = 2	0 °C constant	0	2
2 - T = 0; Alpha = oo	0 °C constant	0	10000



Element mesh, dimensions [m] and temperature distribution [K]

## References

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Nationaler Anhang – National festgelegte Parameter – (National Annex – Nationally Determined Parameters –)  
Eurocode 1: Einwirkungen auf Tragwerke – Teil 1-2: Allgemeine Einwirkungen - Brandeinwirkungen auf Tragwerke.  
(Eurocode 1: Actions on Structures – Part 1-2: General Actions – Actions on Structures exposed to Fire).  
Publisher: DIN Deutsches Institut für Normung e.V., Berlin. Beuth Verlag, Berlin 2010.
- DIN EN 1992-1-2/NA:2010-12  
Nationaler Anhang – National festgelegte Parameter – (National Annex – Nationally Determined Parameters –)  
Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken –  
Teil 1-2: Allgemeine Regeln - Tragwerksbemessung für den Brandfall.  
(Eurocode 2: Design of Concrete Structures – Part 1-2: General Rules – Structural Fire Design)  
Publisher: DIN Deutsches Institut für Normung e.V., Berlin. Beuth Verlag, Berlin 2010.
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Teil 1-2: Allgemeine Regeln - Tragwerksbemessung für den Brandfall.  
(Eurocode 3: Design of Steel Structures – Part 1-2: General Rules – Structural Fire Design)  
Publisher: DIN Deutsches Institut für Normung e.V., Berlin. Beuth Verlag, Berlin 2010.
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Nationaler Anhang – National festgelegte Parameter – (National Annex – Nationally Determined Parameters –)  
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Eurocode 4: Design of composite Steel and Concrete Structures – Part 1-2: General Rules – Structural Fire Design  
Publisher: CEN European Committee for Standardization, Brussels. Beuth Verlag, Berlin 2010.
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Eurocode 5: Design of Timber Structures – Part 1-2: General Rules – Structural Fire Design  
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Zusätzliche Technische Vertragsbedingungen und Richtlinien für Ingenieurbauten, Teil 5 – Tunnelbau.  
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