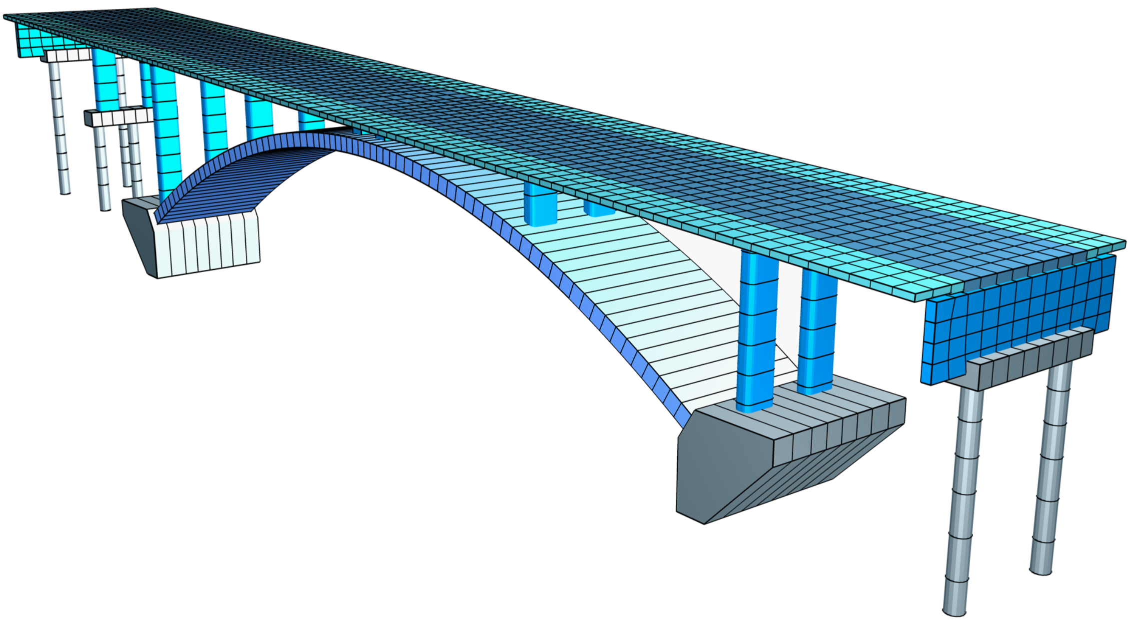
Bridge Checks

according to EN 1992-2

with National Annexes
Austria
Germany
Great Britain
Sweden



InfoGraph
Software für die Tragwerksplanung

Title image: Federal motorway A 44, bridge construction near Hessisch Lichtenau.
Courtesy of MEHLHORN und VIER Ingenieurgesellschaft mbH, Kassel, Germany.

Contents

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.

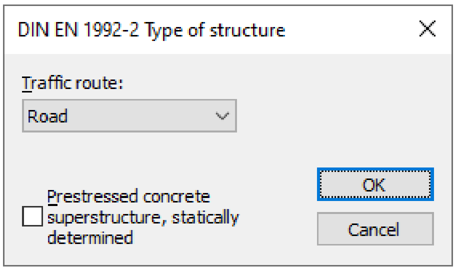
The EN 1992-2 guidelines are primarily cited for the following comments. Reference to the relevant national settings is only made if they contain different or complementary rules. The passages in question are marked by a vertical line left of the text.

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:

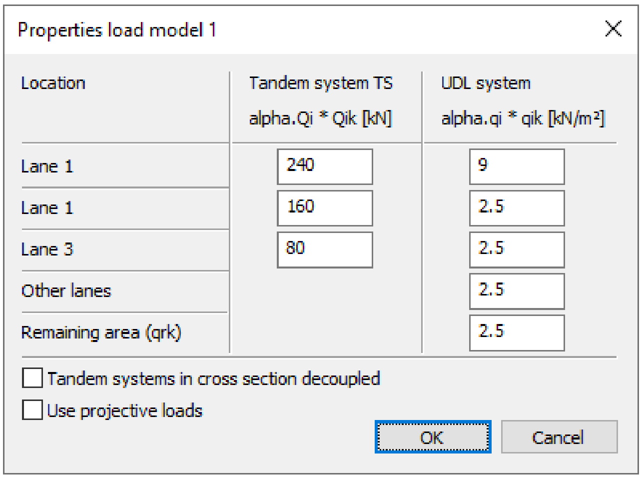
a) Double-axle (TS tandem system)

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 *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.



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.

The video "*https://download.infograph.de/video_de/LM%201%20eingeben.mp4*" (German language) shows the definition of load model 1.

Off

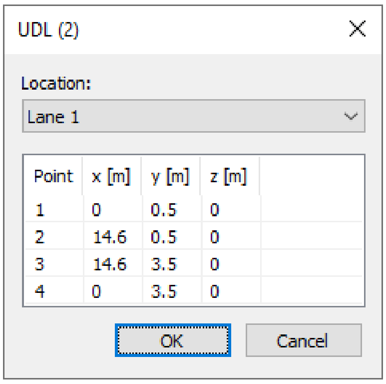
Exit the load model 1 screen.

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

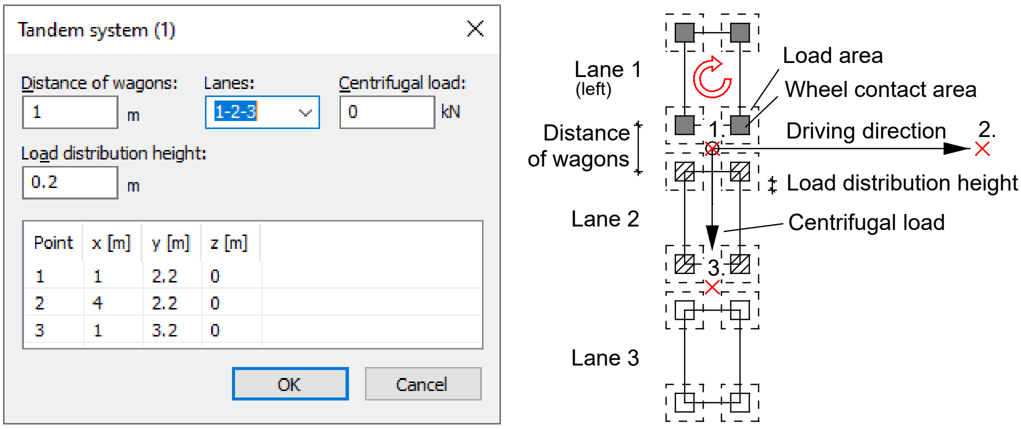
Consecutive input of rectangular or triangular load areas of the UDL load. The partial areas of the UDL load can act simultaneously. The input for UDL2 (lane 2) is preset. You can select a different lane or remaining area from the context menu of the load area.



UDL 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



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).

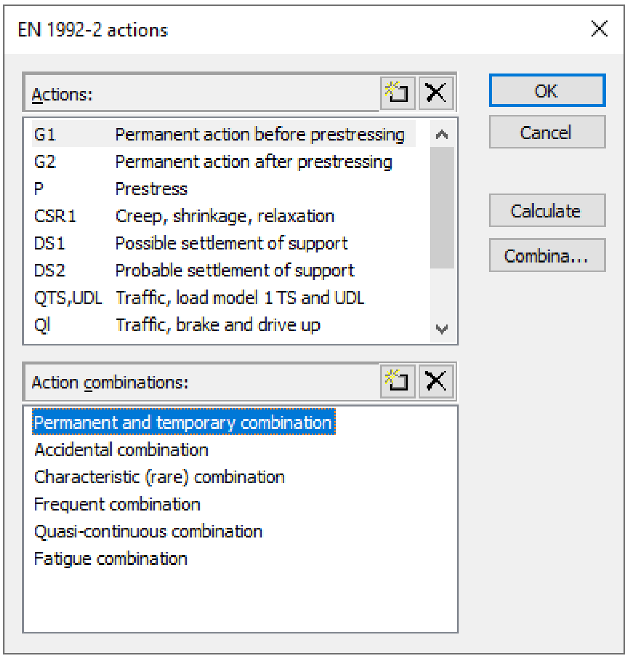
Point

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, breaking 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.

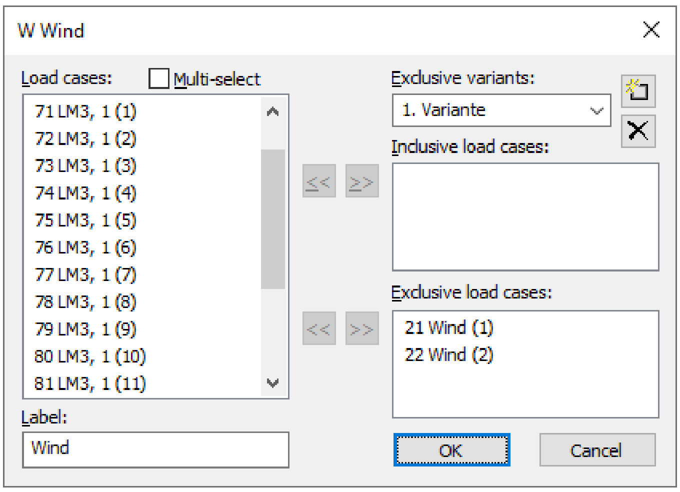
Situation	Ultimate limit state	EN 1992-1-1	EN 1992-2
Permanent & temp. Accidental Earthquake	Longitudinal reinforcement Lateral reinforcement Torsion reinforcement	6.1 6.2 6.3	3.1.6
Characteristic (rare)	Robustness reinforcement		6.1 (110)
Frequent	Fatigue simplified	6.8.6 (2)	
Fatigue	Concrete Reinforcing steel Prestressing steel	6.8.7 (1) 6.8.4 6.8.4	NN.3.2 NN.2.1 NN.3.1
Situation	Serviceability Limit State	EN 1992-1-1	EN 1992-2
Characteristic (rare)	Concrete compressive stress Reinforcing steel stress Prestressing steel stress	 7.2 (5) 7.2 (5)	7.2 (102)
Frequent	Decompression class XC2-XS3 Crack width, prestressing with bond		7.3.1 7.3.1
Quasi-continuous	Concrete compressive stress Crack width, reinforced concrete and prestressing without bond Deformations	7.2 (2) 7.4	 7.3.1

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_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.

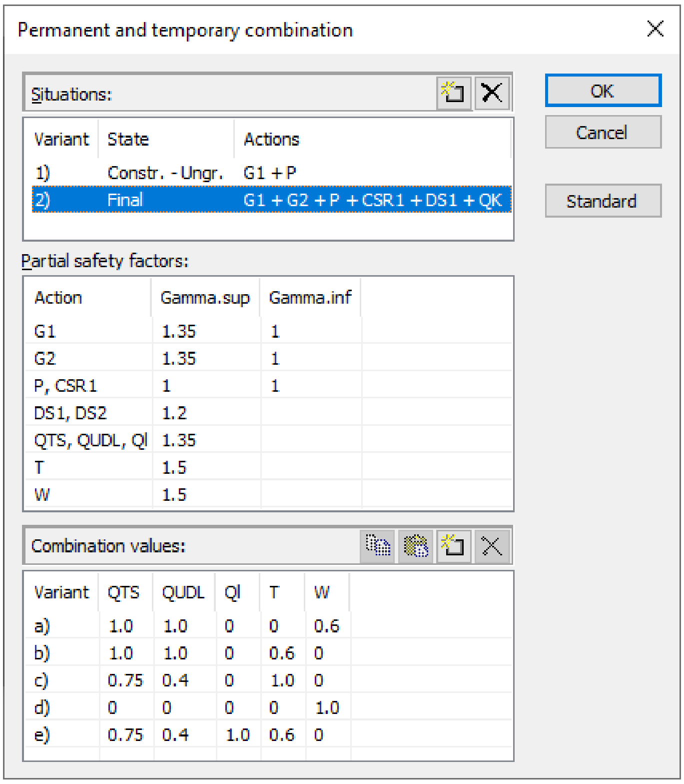
CS as constant reduction of prestress
As an alternative to defining CS load cases, you can allow for the effect of creep and shrinkage by defining a constant percentage reduction of prestress.

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.



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 γ_{sup} and γ_{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:

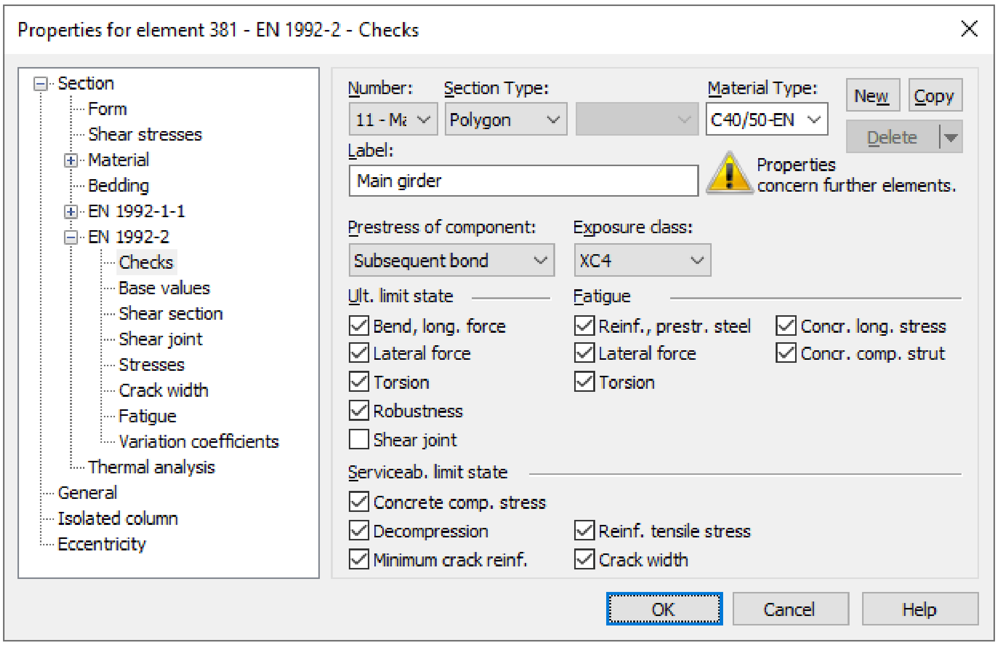
In the design situations due to earthquakes, according to the NDP to 5.2.4 (1) and (3), the safety factors of the permanent and temporary design situation generally apply.

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.



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

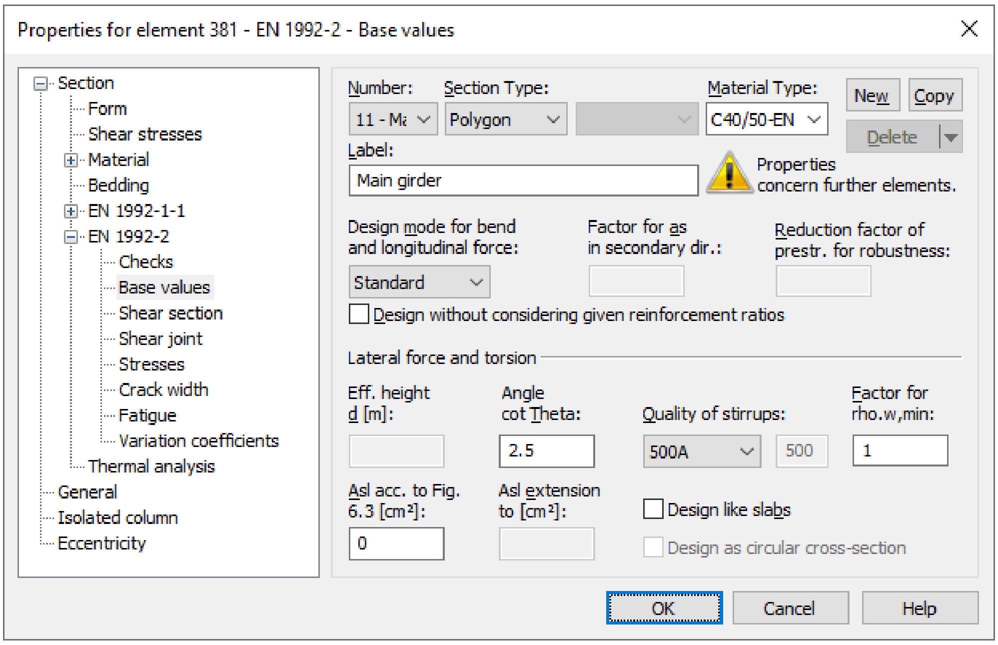
For components with internal prestressing, both the prestressing steel stresses an the stresses of the longitudinal reinforcement are checked.

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.



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 $1.0 \le \cot\Theta \le 1.75$ 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 $1.0 \le \cot\Theta \le 3.0$ 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 [cm²].

Quality of the stirrups

- *420S*: Reinforcing rod with f_{yk} = 420 MN/m².
- *500A*: Reinforcing rod with f_{yk} = 500 MN/m².
- *500M*: Reinforcing meshes with f_{yk} = 500 MN/m².
- *General information*: Freely definable steel quality [MN/m²].

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 $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$. The corresponding inputs are made on the *Shear Section* dialog page.

Laying measure cv,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 $z = d - 2\,c_{v,l}$ 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

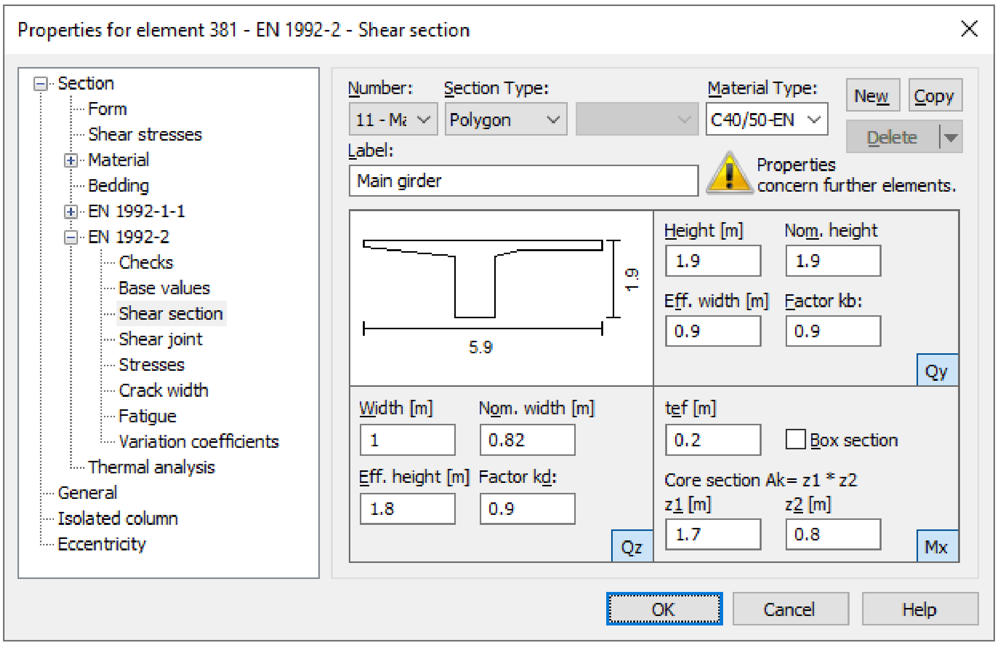
DIN EN 1992-2:
The lever arm z for lateral force design of area elements is normally assumed to be $0.9 \cdot d$. Alternatively, the lever arm from bending design can be used. The program determines the maximum lever arm at each check location for both reinforcement directions depending on the design situation and limits the value to the range of
$0.1 \cdot d \le z \le \max(\, d - 2c_{v,l};\, d - c_{v,l} - 30\text{ mm})$.
When the check is performed for the resulting force q_r the minimum from z_x and z_y is used.

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 σ_2 is calculated basically in state I from the longitudinal stress σ_{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.

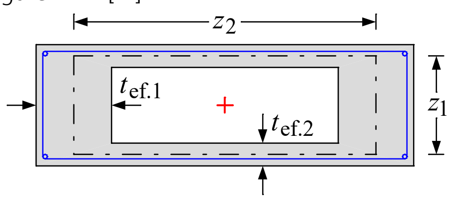
Core section Ak = z1 * z2

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

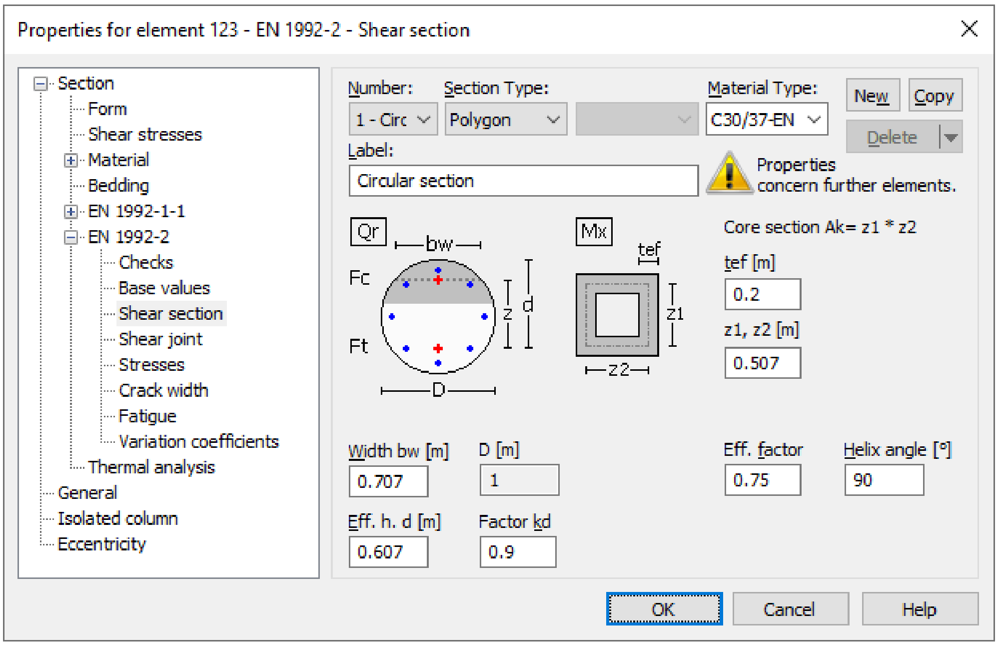
DIN EN 1992-2:
The inner lever arm z can be defined in the following ways:

- *kd * effective height, kb * 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 * 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 kd
Factor for calculating the inner lever arm z from the effective height d in the verification for Q_r.

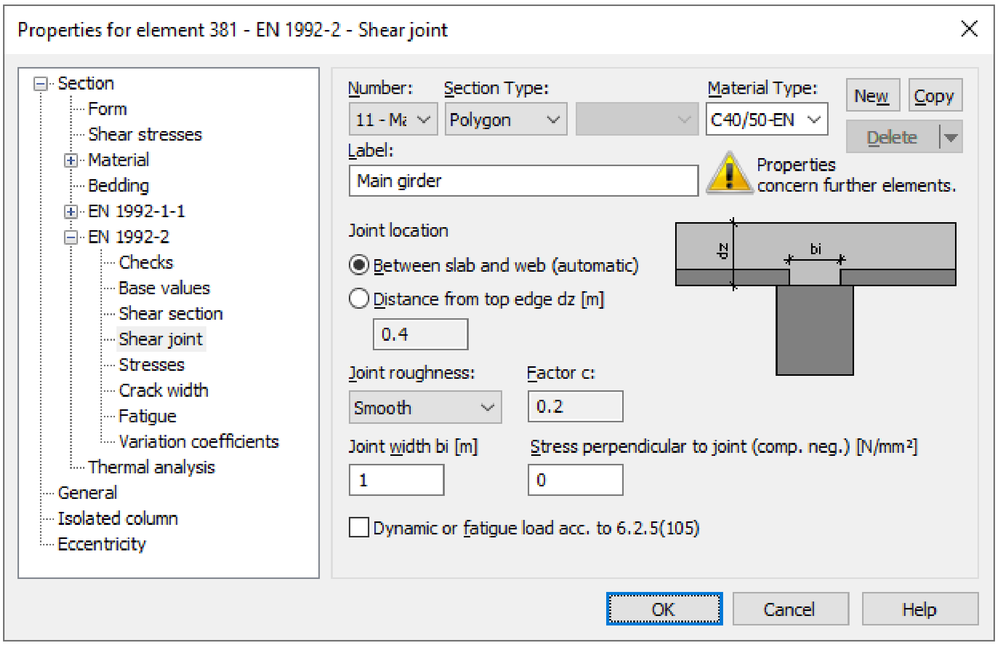
Efficacy factor
According to Bender et al. (2010), p. 422, the efficacy factor α_k is stress-dependent ($0.715 \le \alpha_k \le 0.785$) and can be assumed with the mean value $\alpha_k = 0.75$.

Helix inclination
Angle between shear force reinforcement and component axis. When entering an inclination of 90°, 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 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.



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 bi
Width of the joint over which shear forces are transferred between existing and new concrete [m].

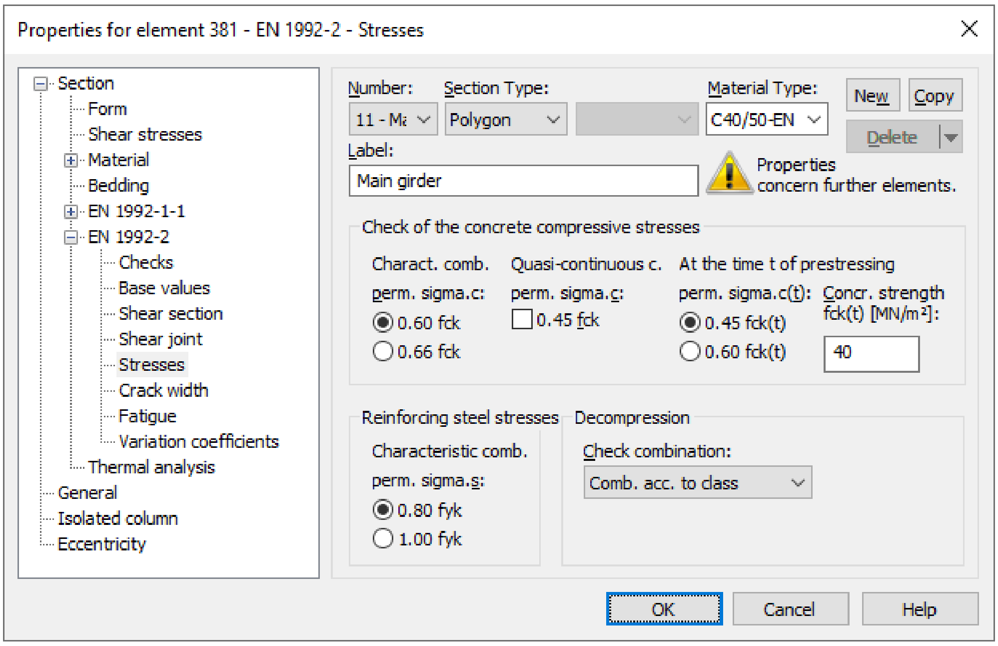
Stress perpendicular to joint (comp. neg.)

Stress σ_n caused by the minimum normal force perpendicular to the joint which can act simultaneously with the lateral force [N/mm²]. 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 σ_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 $0.45{\cdot}f_{ck}$, linear creep can be assumed according to 7.2 (3). If this is not the case, non-linear creep must be taken into account.

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 $0.45{\cdot}f_{ck(t)}$, 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/m²].

Reinforcing steel stresses

According to EN 1992-1-1, Chapter 7.2 (5), the tensile stresses in the reinforcement may not exceed the value $0.8{\cdot}f_{yk}$ 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.

Decompression, Stress

DIN EN 1992-2, OENORM B 1992-2:

Decisive stress for the decompression check for area elements (σ_1, σ_x, σ_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.

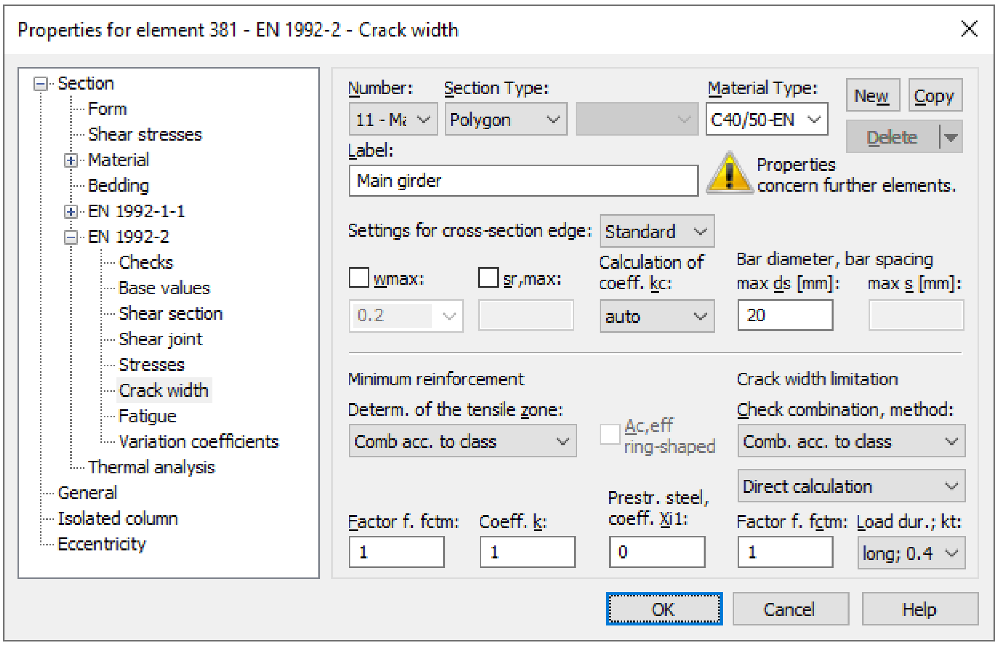
Av

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:

Standard	The standard properties are used for the unspecified edges and directions.
Top, bottom, 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 1992-2, 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:

auto	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).
web	kc is calculated according to Eq. (7.2).
chord	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, centrical 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 centrical 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 centrical 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 fctm

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 wether crack formation will occur within the first 28 days, a tensile strength of at least 2.9 MN/m² 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 wether crack formation will occur within the first 28 days, a tensile strength of at least 3.0 MN/m² for normal concrete and 2.5 MN/m² 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.

Ac,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 Xi1

The bond coefficient ξ_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 ξ_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.

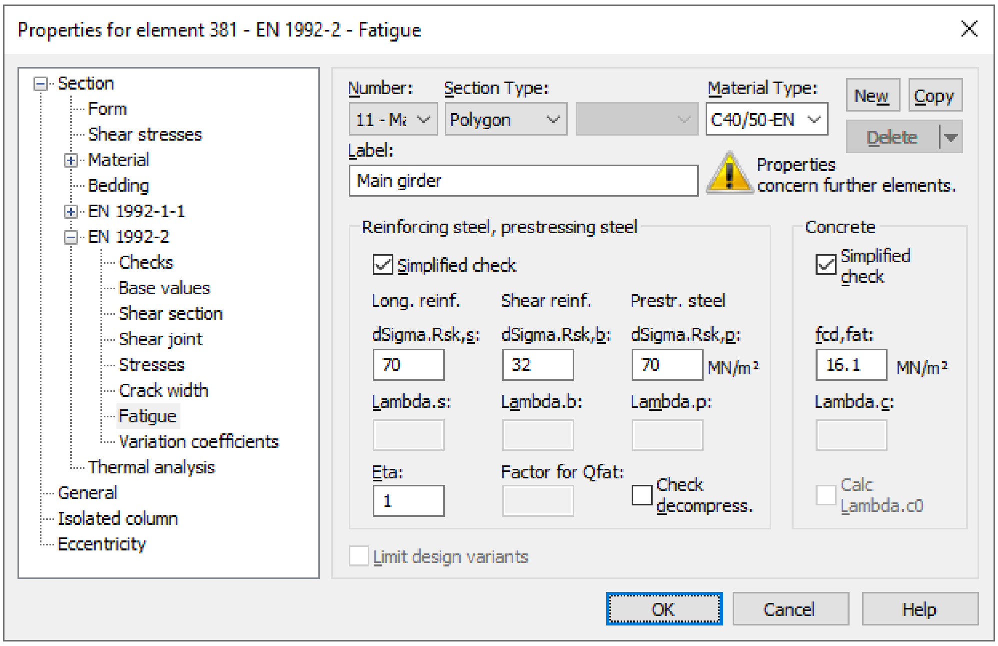
OENORM B 1992-1-1:
The method is applicable to single-layer reinforcement with a bar spacing according to Table 10AT or 11AT. These are valid for concrete covers 25 mm $\leq c_{nom} \leq$ 40 mm with bar diameters 8 mm $\leq d_s \leq$ 20 mm.

Load duration; kt

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



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/m²]. 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/m²]. 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 λ_s for determining the damage-equivalent stress range $\Delta\sigma_{s,equ}$ 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 η 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).

fcd,fat

Concrete compressive strength before onset of cyclic load according to EN 1992-1-1, Chapter 6.8.7 (1), Eq. (6.76) [MN/m²]. In general, the following applies:

$$f_{\mathrm{cd,fat}} = k_1 \cdot \beta_{\mathrm{cc}}(t_0) \cdot f_{\mathrm{cd}} \cdot \left(1 - \frac{f_{\mathrm{ck}}}{250}\right) \tag{6.76}$$

with

$$\beta_{\mathrm{cc}}(t_0) = e^{s\,(1-\sqrt{28/t_0})}$$

s Coefficient depending on the cement type.

t_0 Time of the initial stressing of the concrete.

k_1 $= 0.85$

$f_{\mathrm{cd,fat}}$ 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 $f_{\mathrm{cd,fat}}$, f_{cd} is determined with $\alpha_{\mathrm{cc}} = 1.0$ in Eq. (3.15).

k0

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 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 $\Delta\sigma_{\mathrm{s.equ}}$ 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:

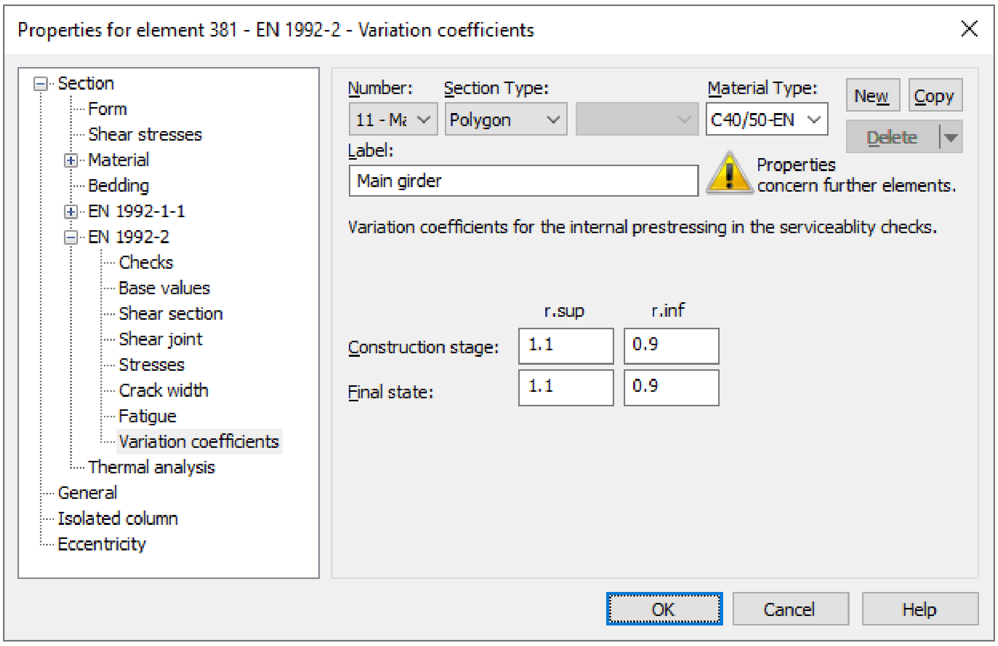
This option is applicable at superstructures for reinforcing and prestressing steel without welding joints or couplings when decompression is checked under the frequent combination.

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 $P_{\mathrm{k,sup}}$ and $P_{\mathrm{k,inf}}$ 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



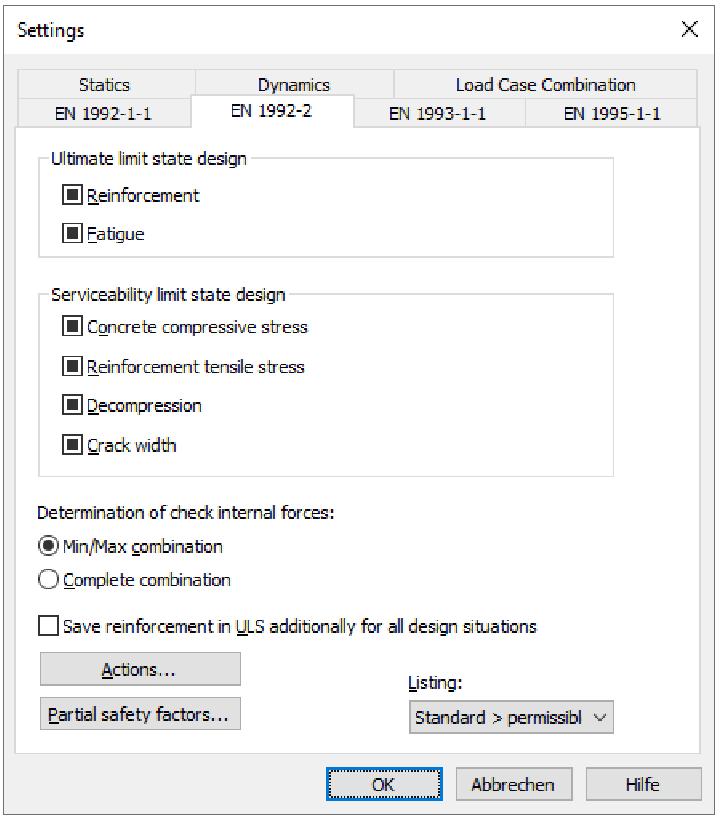
The coefficients used to take into account the variation of prestressing force are defined in EN 1992-1-1 depending on the prestressing type. In the dialog, values are suggested according to Chapter 5.10.9 (1)P for subsequent bond. In the action combinations, situations are declared as construction stage 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 ε_{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 ξ_1 specified for the section to be checked.

Prestressed Structures

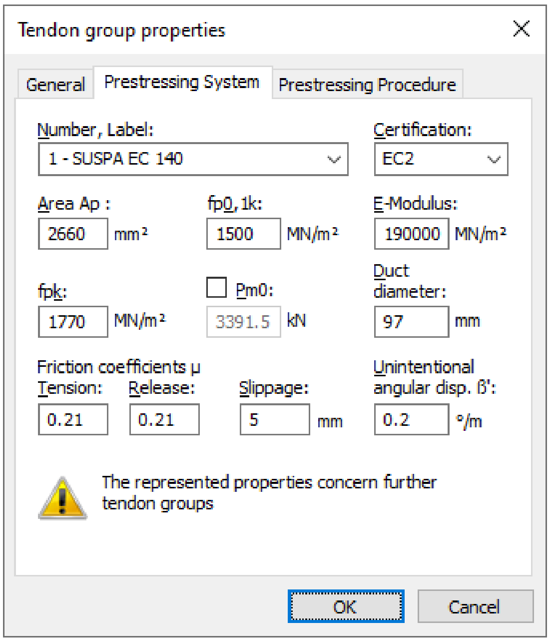
Internal Prestressing

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.

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.

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 *Igraph.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 Ap

Section area A_p of a tendon [mm²].

ßs, ß02

Yield strength or $\beta_{0.2}$ limit of the prestressing steel according to DIN 4227 [MN/m²].

fp0,1k

Characteristic value of the 0.1% strain limit of the prestressing steel per DIN 1045-1, OENORM, SIA 262 and EC2 [MN/m²].

E-Modulus

E-modulus of the prestressing steel [MN/m²].

ßz

Tensile strength of the prestressing steel according to DIN 4227 [MN/m²].

fpk

Characteristic value of the tensile strength of the prestressing steel per DIN 1045-1, OENORM, SIA 262 and EC2 [MN/m²].

Pm0

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:
$P_{m0} = A_p \cdot 0.85 f_{p0,1k}$ or $A_p \cdot 0.75 f_{pk}$ according to DIN 1045-1, Eq. (49).

Certification as per DIN 4227:
$P_{m0} = A_p \cdot 0.75 ß_s$ or $A_p \cdot 0.55 ß_z$ according to DIN 4227-1, Tab. 9, Row 65.

Certification as per EC2:
$P_{m0} = A_p \cdot 0.85 f_{p0,1k}$ or $A_p \cdot 0.75 f_{pk}$ according to EN 1992-1-1, Eq. (5.43).

Certification as per OENORM:
$P_{m0} = A_p \cdot 0.80 f_{p0,1k}$ or $A_p \cdot 0.70 f_{pk}$ according to OENORM B 4750, Eq. (4) and (5), and OENORM B 1992-1-1, Chapter 8.9.6.

Certification as per SIA 262:
$P_{m0} = A_p \cdot 0.7 f_{pk}$ 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 μ for prestressing and release.

Slippage

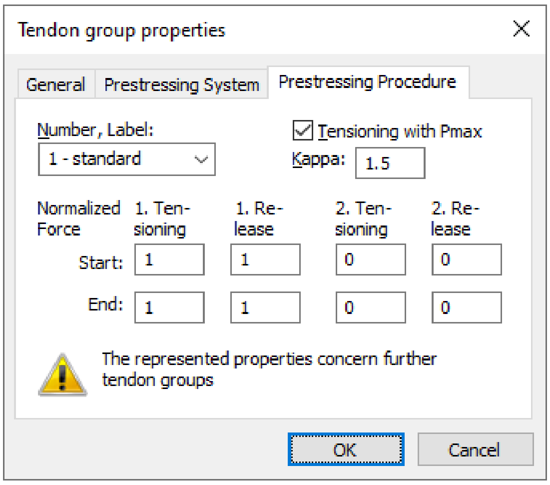
Slippage at the prestressing anchor [mm].

Unintentional deviation angle ß'

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.



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 κ 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.

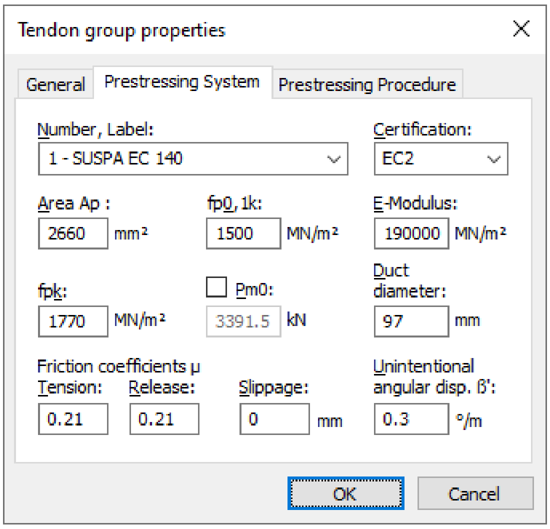
The differences between tensioning with P_{m0} and P_{max} are described in the following examples.

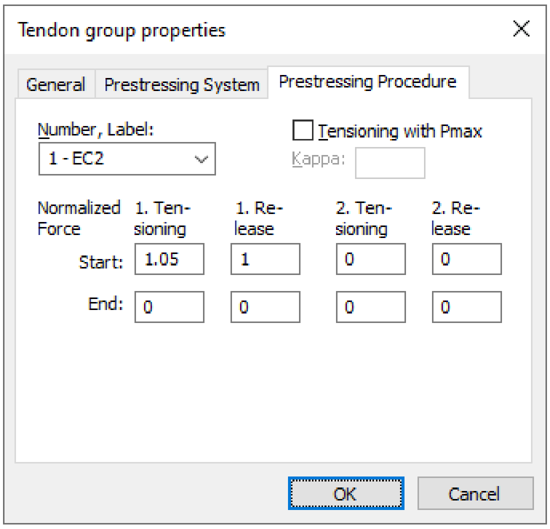
The user is responsible for checking the permissibility of the maximum force during the stressing process.

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.





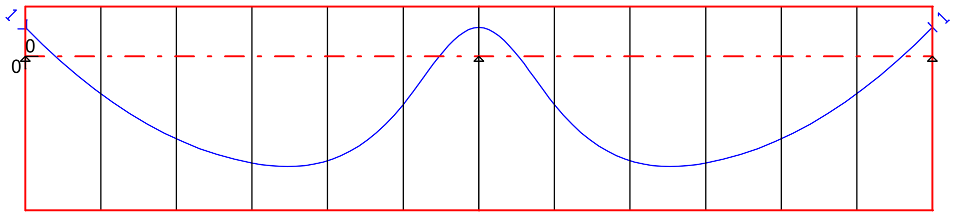
The permissible prestressing forces ar defined by:

$P_{max} = min(A_p \cdot 0.80\, f_{pk}\, ,\, A_p \cdot 0.90\, f_{p0.1k}\,)$ = 3591.0 kN

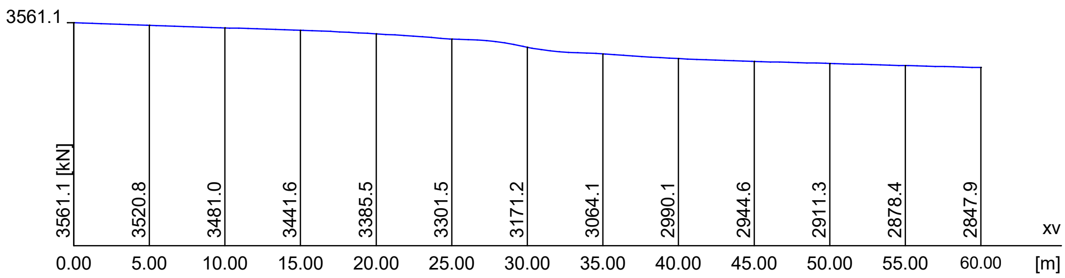
$P_{m0} = min(A_p \cdot 0.75\, f_{pk}\, ,\, A_p \cdot 0.85\, f_{p0.1k}\,)$ = 3391.5 kN

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}.

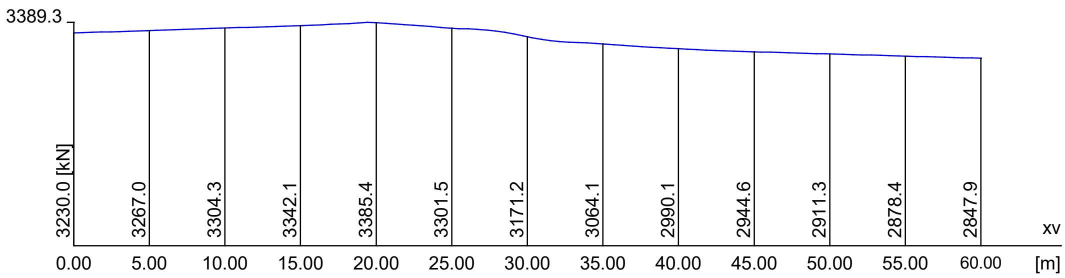
The second prestressing force curve results after tensioning and release with the factors 1.05 and 1.0, i.e. the maximum prestressing force that remains in the tendon after it is fixed into place is 3389.3 kN < P_{m0}.



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

μ Friction coefficient according to the general certification from the building authorities.

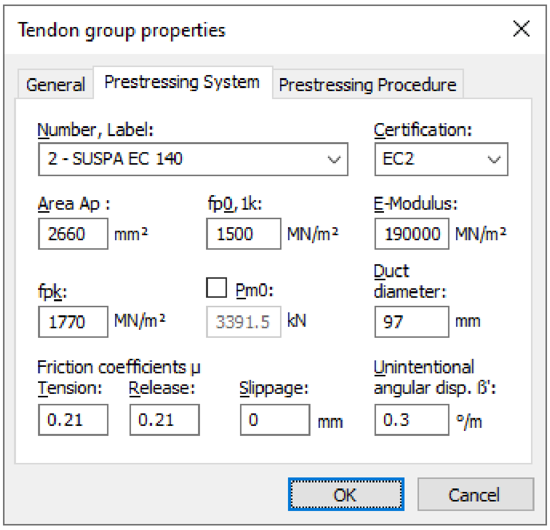
γ $\Phi + k{\cdot}x$

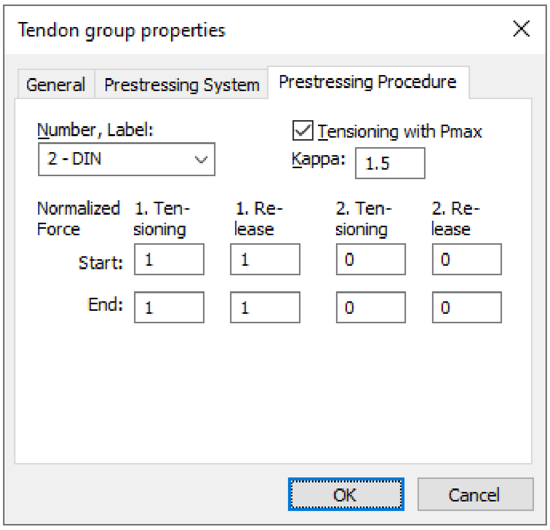
Φ = sum of planned deviation angle over the length x,
k = unintentional deviation angle per unit of length (ß' in the dialog),
x = the distance between the prestressed anchor and the fixed anchor in the case of one-sided prestressing or the influence length of the respective anchor in the case of two-sided prestressing.

κ Allowance value for ensuring an overstressing reserve with $1.5 \le \kappa \le 2$ for tendons with supplemental bond according to the German standard and $\kappa = 1$ for all other cases.

The program uses the specified allowance value κ 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.



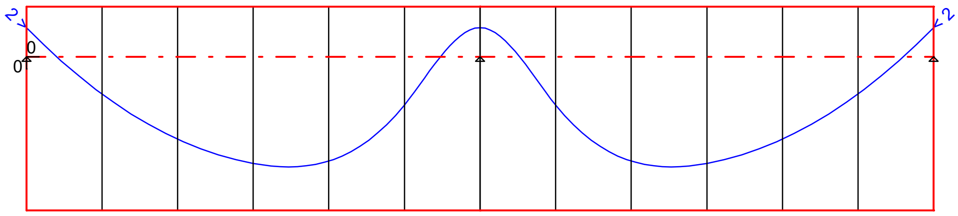


The program will determine the permissible prestressing forces as follows:

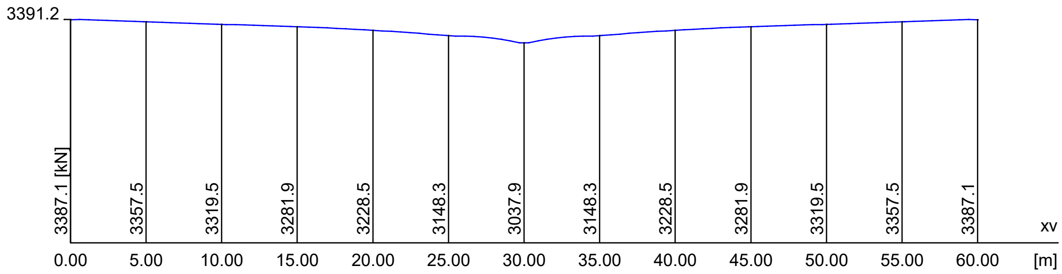
$$P_{max} = e^{-\mu\gamma(\kappa-1)} \cdot min(A_p \cdot 0.80 f_{pk} , A_p \cdot 0.90 f_{p0.1k}) = 0.9457 \cdot 3591 = 3395.9 \text{ kN}$$

$$P_{m0} = min(A_p \cdot 0.75 f_{pk} , A_p \cdot 0.85 f_{p0.1k}) = 3391.5 \text{ kN}$$

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.

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.

γ_P Partial safety factor of prestressing force, γ_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):

$P_{k,sup} = r_{sup} \cdot P_{m,t}(x)$ Upper characteristic value according to Equation (5.47).

$P_{k,inf} = r_{inf} \cdot P_{m,t}(x)$ 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 $r_{sup} = r_{inf} = 1$.

For internal prestressing, the recommended country-specific values are:

- For tendons with immediate bond or without bond: $r_{sup} = 1.05$ and $r_{inf} = 0.95$.
- For tendons with subsequent bond: $r_{sup} = 1.10$ and $r_{inf} = 0.90$.

DIN EN 1992-2:
In the construction stages the following values may be used according to NCI for 5.10.9 (1)P:

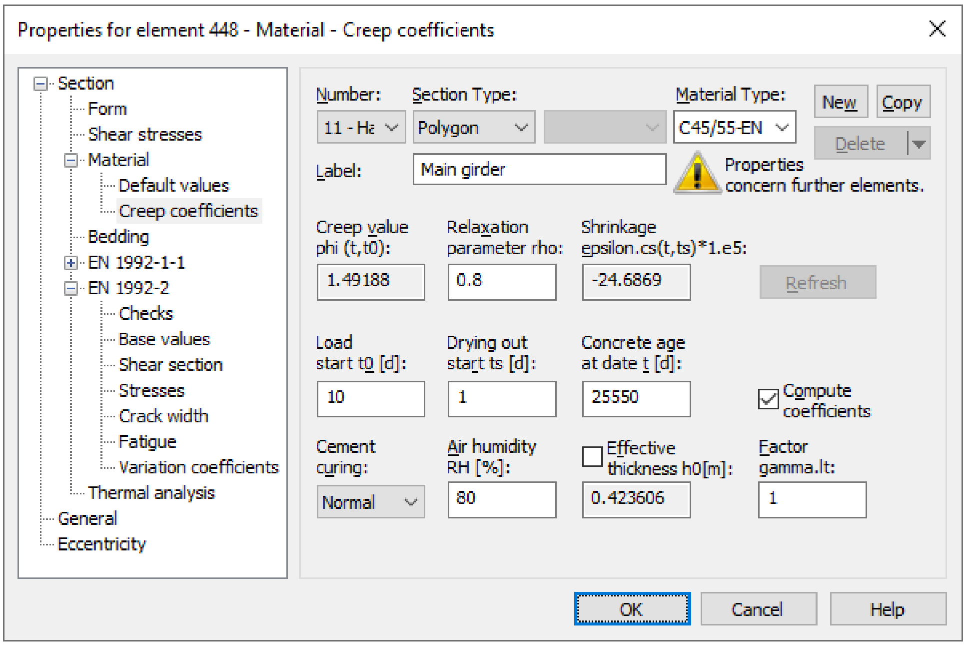
- For straight-lined tendons with bond and tendons without bond: $r_{sup} = r_{inf} = 1.00$.
- For curved tendons with bond: $r_{sup} = 1.05$ und $r_{inf} = 0.95$.

OENORM B 1992-1-1:

- For tendons with immediate bond or without bond: $r_{sup} = r_{inf} = 1.00$.
- For tendons with subsequent bond: $r_{sup} = 1.05$ und $r_{inf} = 0.95$.

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 γ_{lt} 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.

$$\sigma_b(t) = \frac{E_b}{1+\rho \cdot \varphi}\left(\varepsilon_b(t) - \varphi \cdot \varepsilon_{b,0} - \varepsilon_{b,S}\right)$$

Explanation of the individual terms:

$\sigma_b(t)$	Concrete stress from creep and shrinkage at time t.
E_b	E-modulus of the concrete.
ρ	Relaxation coefficient according to Trost for time t (normally ρ = 0.80).
φ	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 (σ_{p0}/f_{pk}). $\sigma_{p0} = \sigma_{pg0}$ may be used as the initial stress, with σ_{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. Nx, My). 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	Nx	My	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	Nx	My	Combination
min Nx	-15	40	L1
max Nx	-10	50	L1+L3
min My	-15	30	L1+L4
max My	-10	70	L1+L2+L3

Results of min/max combination

Set	Nx	My	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.

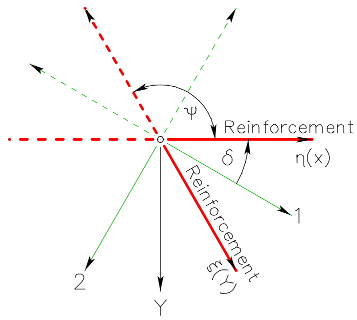
Orthogonal area reinforcement

Slabs	$m_x \pm \lvert m_{xy}\rvert$
	$m_y \pm \lvert m_{xy}\rvert$
Plain stress	$n_x \pm \lvert n_{xy}\rvert$
elements	$n_y \pm \lvert n_{xy}\rvert$
Shells	$m_x \pm \lvert m_{xy}\rvert$ and $n_x \pm \lvert n_{xy}\rvert$
	$m_y \pm \lvert m_{xy}\rvert$ and $n_y \pm \lvert n_{xy}\rvert$

Oblique area reinforcement

The bending design of slabs with oblique reinforcement assemblies is carried out according to Kuyt or Rüsch. 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):

$$m_{1,2} = \frac{1}{2}\cdot(m_x + m_y) \pm \frac{1}{2}\sqrt{(m_x - m_y)^2 + 4m_{xy}^2}$$

with $m_1 \geq m_2$

The angle δ 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:

$$m_\eta = \frac{1}{\sin^2\psi}\left[m_1\sin^2(\delta+\psi) + m_2\cos^2(\delta+\psi) \pm \left| m_1\sin\delta\,\sin(\delta+\psi) + m_2\cos\delta\,\cos(\delta+\psi)\right|\right]$$

$$m_\xi = \frac{1}{\sin^2\psi}\left[m_1\sin^2\delta + m_2\cos^2\delta \pm \left| m_1\sin\delta\,\sin(\delta+\psi) + m_2\cos\delta\,\cos(\delta+\psi)\right|\right]$$

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.

$$\sum_{j\geq 1}\gamma_{G,j}\cdot G_{k,j}\text{"+"}\ \gamma_P\cdot P\ \text{"+"}\ \gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10)$$

$$\sum_{j\geq 1}\gamma_{G,j}\cdot G_{k,j}\text{"+"}\ \gamma_P\cdot P\ \text{"+"}\ \gamma_{Q,1}\cdot\psi_{0,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10a)$$

$$\sum_{j\geq 1}\xi_j\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\ \gamma_P\cdot P\ \text{"+"}\ \gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10b)$$

For the coefficient ξ 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:

$$\sum_{j\geq 1}\gamma_d\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\ \gamma_P\cdot P \qquad (6.10aSS)$$

$$\sum_{j\geq 1}\xi_j\cdot\gamma_d\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\ \gamma_P\cdot P\ \text{"+"}\ \gamma_d\cdot\gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_d\cdot\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10bSS)$$

Assuming reliability class 3, factor γ_d is set to 1. (see Section A, Article 11 and 14). The coefficient ξ is set to the value of $\xi = 0.89$.

- Combination for accidental design situations

$$\sum_{j\geq 1}G_{k,j}\text{"+"}\ P\ \text{"+"}\ A_d\ \text{"+"}\ (\psi_{1,1}\ \text{or}\ \psi_{2,1})\cdot Q_{k,1}\text{"+"}\sum_{i>1}\psi_{2,i}\cdot Q_{k,i} \qquad (6.11b)$$

$\psi_{1,1}\cdot Q_{k,1}$ 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

$$\sum_{j\geq 1}G_{k,j}\text{"+"}\ P\ \text{"+"}\ A_{Ed}\ \text{"+"}\sum_{i\geq 1}\psi_{2,i}\cdot Q_{k,i} \qquad (6.12b)$$

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.

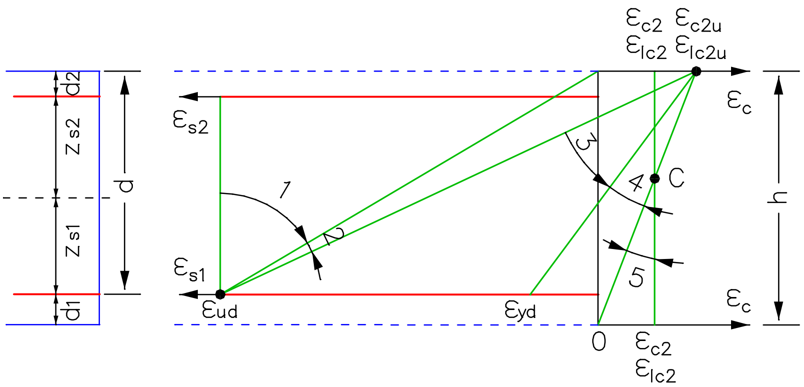
  DIN EN 1992-2:

  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 $\varepsilon_{ud} = 0.9\ \varepsilon_{uk}$ 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 ≤ 0.45 for concrete strength classes up to C50/60.

≤ 0.35 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:

$A_{s,min} = 0.10 \mid N_{Ed} \mid / f_{yd}$ or $0.002\ A_c$, depending on which value is greater (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:

$A_{s,min} = 0.15\ N_{Ed} / f_{yd} \geq 0.003\ A_c$ (9.12DE)

OENORM B 1992-1-1:

$A_{s,min} = 0.13\ N_{Ed} / f_{yd} \geq 0.0026\ A_c$ (30AT)

SS EN 1992-1-1:

$A_{s,min} = 0.002\ A_c$ (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):

$$A_{s,min} = M_{rep} / (z_s \cdot f_{yk}) \tag{6.101a}$$

with

M_{rep} Crack moment without allowance for prestressing force and under the assumption that the edge tensile stress corresponds to f_{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 $f_{ctk,0.05}$ 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 $z = d - c_{v,l} - 30$ mm and $z = d - 2c_{v,l}$. Note that $c_{v,l}$ 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

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100\ \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}]\ b_w \cdot d \tag{6.2a}$$

with at least

$$V_{Rd,c} = (\ v_{min} + k_1 \cdot \sigma_{cp}\ )\ b_w \cdot d \tag{6.2b}$$

For lightweight concrete applies:

$$\begin{aligned} V_{lRd,c} &= [C_{lRd,c} \cdot \eta_1 \cdot k \cdot (100\ \rho_l \cdot f_{lck})^{1/3} + k_1 \cdot \sigma_{cp}]\ b_w \cdot d \\ &\geq (\eta_1 \cdot v_{l,min} + k_1 \cdot \sigma_{cp})\ b_w \cdot d \end{aligned} \tag{11.6.2}$$

where

f_{ck} , f_{lck}	is the characteristic concrete strength [N/mm²].
k	$= 1 + \sqrt{(200 / d)} \leq 2.0$ with d specified in mm.
ρ_l	$= A_{sl} / (b_w \cdot d) \leq 0.02$.

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].

σ_{cp} $= N_{Ed} / A_c < 0.2 f_{cd}$ [N/mm²].

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 [mm²].

$V_{Rd,c}$, $V_{lRd,c}$ is the design value of the lateral force resistance [N].

η_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

κ_1 $= 0.0525$ for $d < 600$ mm

$= 0.0375$ for $d > 800$ mm

For 600 mm $< d \leq 800$ mm can be interpolated.

Components with computationally necessary lateral force reinforcement

The angle θ between the concrete struts and the component axis perpendicular to the lateral force must be limited:

$$1 \leq \cot\theta \leq 2.5 \tag{6.7N}$$

DIN EN 1992-2:

$$1.0 \leq \cot\theta \leq (1.2 + 1.4\, \sigma_{cp} / f_{cd}) / (1 - V_{Rd,cc} / V_{Ed}) \leq 1.75 \tag{6.107aDE}$$

with

$$V_{Rd,cc} = c \cdot 0.48 \cdot f_{ck}^{1/3}\, (1 - 1.2\, \sigma_{cp} / f_{cd}) \cdot b_w \cdot z \tag{6.7bDE}$$

where

c $= 0.5$

σ_{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/mm².

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 η_1 according to Eq. (11.1).

OENORM B 1992-1-1:

$0.6 \le \tan\theta \le 1.0$ (3AT)

If the section is completely under pressure, then θ in the area

$0.4 \le \tan\theta \le 1.0$ (4AT)

may be selected.

SS EN 1992-1-1:
According to Article 15 and differing from Equation (6.7N), for prestressed components the condition $1.0 \le \cot\theta \le 3.0$ applies.

For components with lateral force reinforcement perpendicular to the component axis, the lateral force resistance V_{Rd} is the smaller value from

$V_{Rd,s} = (A_{sw} / s) \cdot z \cdot f_{ywd} \cdot \cot\theta$ (6.8)

and

$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$ (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.
ν_1	is a reduction coefficient for concrete strength when shear cracks occur.
α_{cw}	is a coefficient for taking into account the stress state in the compression chord.

The recommended values are:

ν_1	$= \nu$		
	with		
	$\nu = 0.6\,(1 - f_{ck} / 250)$	(f_{ck} in N/mm²)	(6.6N)
α_{cw}	$= 1$	for non-prestressed components	
	$= (1 + \sigma_{cp} / f_{cd})$	for $0 < \sigma_{cp} \le 0.25\,f_{cd}$	(6.11aN)
	$= 1.25$	for $0.25\,f_{cd} < \sigma_{cp} \le 0.5\,f_{cd}$	(6.11bN)
	$= 2.5\,(1 - \sigma_{cp} / f_{cd})$	for $0.5\,f_{cd} < \sigma_{cp} \le 1.0\,f_{cd}$	(6.11cN)

where

σ_{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:

$(A_{sw,max} \cdot f_{ywd}) / (b_w \cdot s) \le ½\,\alpha_{cw} \cdot \nu_1 \cdot f_{cd}$ (6.12)

The additional tensile force in the longitudinal reinforcement due to lateral force according to Eq. (6.18) is

$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot\Theta - \cot\alpha)$. (6.18)

DIN EN 1992-2:

ν_1	$= \eta_1 \cdot 0.75$ for concrete up to C50/60-EN-D
	$= \eta_1 \cdot 0.75 \cdot \min(1.0;\ 1.1 - f_{ck} / 500)$ from C55/67-EN-D as per DIN EN 1992-1-1, NDP 6.2.3 (3).
η_1	$= 1.0$ for normal concrete and as per Eq. (11.1) for lightweight concrete.
α_{cw}	$= 1.0$

Equation (6.12) is not applied.

BS EN 1992-2:

$\nu_1 \quad = \nu \cdot (1 - 0.5 \cos \alpha)$

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):

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) \qquad (9.4)$$

where

ρ_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.

α is the angle between the lateral force reinforcement and the component axis.

The recommended value for the minimum reinforcement is:

$$\rho_{w,min} = 0.08 \sqrt{f_{ck}} / f_{yk} \qquad (9.5N)$$

DIN EN 1992-1-1:

$$\rho_{w,min} = 0.16 f_{ctm} / f_{yk} \qquad (9.5aDE)$$

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:

$$\rho_{w,min} = 0.256 f_{ctm} / f_{yk} \qquad (9.5bDE)$$

OENORM B 1992-1-1:

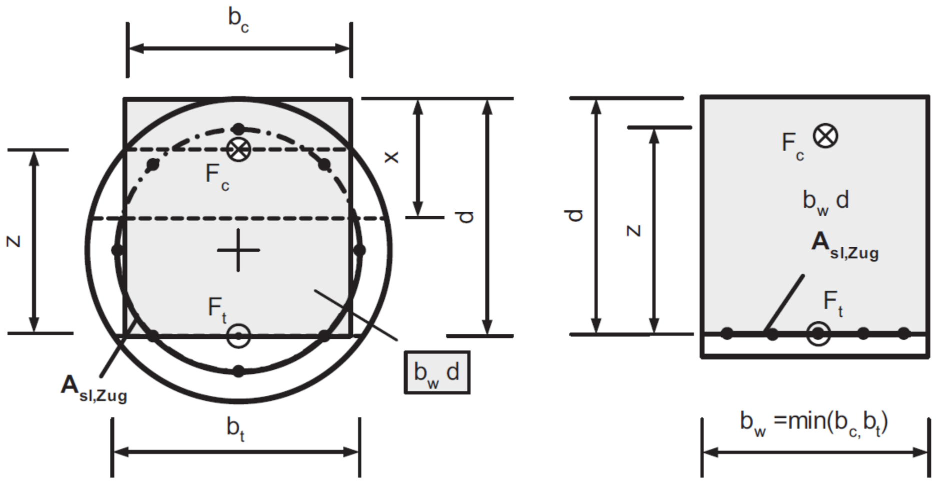
$$\rho_{w,min} = 0.15 f_{ctm} / f_{yd} \qquad (24AT)$$

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

α_k is an efficacy factor, which is stress-dependent ($0.715 \le \alpha_k \le 0.785$) 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-ection width.

z is the inner lever arm.

Θ is the inclination of the conrete compressive struts.

α 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.

α_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 Θ. The reinforcements determined in this manner are to be added together.

$$V_{Ed,T+V} = V_{Ed,T} + V_{Ed} \cdot t_{ef} / b_w \quad \text{(NA.6.27.1)}$$

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:

$$T_{Ed} / T_{Rd,c} + V_{Ed} / V_{Rd,c} \leq 1.0 \tag{6.31}$$

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:

$$T_{Ed} \leq \frac{V_{Ed} \cdot b_w}{4.5} \tag{NA.6.31.1}$$

$$V_{Ed}\left[1 + \frac{4.5\, T_{Ed}}{V_{Ed} \cdot b_w}\right] \leq V_{Rd,c} \tag{NA.6.31.2}$$

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:

$$T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max} \leq 1.0 \tag{6.29}$$

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

$$T_{Rd,max} = 2\, \nu \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t_{ef,i} \cdot \sin \Theta \cdot \cos \Theta \tag{6.30}$$

with α_{cw} according to Equation (6.9) and ν 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:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \tag{NA.6.29.1}$$

In Equation (6.30) $\nu = \eta_1 \cdot 0.75$ is used for box sections and $\nu = \eta_1 \cdot 0.525 \cdot \min(1.0;\ 1.1 - f_{ck} / 500)$ in all other cases according to the NPD for 6.2.2 (6) with η_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:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \tag{9AT}$$

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 σ_2 are to be determined in state I from the average longitudinal stress σ_{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 σ_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 fcd 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 σ_1 being determined in cross-section areas under longitudinal pressure.

- $\sigma_1 \leq f_{ctk;0.05}$
  To determine the principal compressive stress σ_2 according to state I, the longitudinal compressive stress σ_{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 θ_V and θ_T are assumed as in the shear design. The compressive strength is reduced to $\alpha_{cw} \cdot \nu_1 \cdot f_{cd}$ with α_{cw} as per NDP to EN 1992-2, Chapter 6.2.3 (103), and ν_1 as per NCI to EN 1992-2, Chapter 6.3.2 (104).

- Selection of "*Check* σ_2 *in state I*" in the cross-section dialog
  According to DIN Technical Report 102:2003, the principal compressive stress σ_2 is calculated basically in state I from the longitudinal stress σ_{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 peformed 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:

$$v_{Edi} \leq v_{Rdi} \qquad (6.23)$$

Design value of the shear stress in the joint v_{Edi}:

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) \tag{6.24}$$

where

β 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 β 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		β
		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}:

$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} (\mu \cdot \sin \alpha + \cos \alpha) \leq 0.5 \cdot \nu \cdot f_{cd} \tag{6.25}$$

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, μ 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.

σ_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 σ_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.

α the angle of inclination of the transverse reinforcement. This is set at 90° by the program.

ν 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:

$$\rho_{\text{min}} = 0.2 \cdot f_{\text{ctm}} / f_{\text{yk}} \geq 0.001$$

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 concret 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.

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} \cdot Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \tag{6.15b}$$

- Fatigue combination for checks with damage equivalent stress ranges according to EN 1992-1-1, Chapter 6.8.3.

$$\left(\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} \cdot Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \right) \text{ "+" } Q_{fat} \tag{6.69}$$

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 ε_{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 $tan\ \Theta_{fat} = \sqrt{tan\ \Theta} \le 1$ acc. to Eq. (6.65). Where Θ 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/m² for unwelded reinforcing bars and 35 MN/m² 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:

$$\gamma_{F,fat} \cdot \Delta\sigma_{s,equ}(N^*) \le \Delta\sigma_{Rsk}(N^*) / \gamma_{s,fat} \qquad (6.71)$$

with

$\gamma_{F,fat}$	= 1 according to Chapter 2.4.2.3 and 6.8.4 (1).
$\gamma_{s,fat}$	= 1.15 for reinforcing and prestressing steel according to Chapter 2.4.2.4.
$\Delta\sigma_{Rsk}(N^*)$	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.
$\Delta\sigma_{s,equ}(N^*)$	Damage equivalent stress range with $\Delta\sigma_{s,equ} = \lambda_s \cdot \Delta\sigma_s$ according to EN 1992-2, Eq. (NN.101) and Eq. (NN.106).
λ_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_{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 η 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 η and λ 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:

$$\frac{\sigma_{c,\max}}{f_{cd,fat}} \le 0.5 + 0.45 \frac{\sigma_{c,\min}}{f_{cd,fat}} \begin{array}{l} \le 0.9 \text{ for } f_{ck} \le 50 \text{ MN/m}^2 \\ \le 0.8 \text{ for } f_{ck} > 50 \text{ MN/m}^2 \end{array} \tag{6.77}$$

with

$\sigma_{c,max}$ the maximum compressive stress at a fibre under the frequent load combination (compression measured positive).

$\sigma_{c,min}$ the minumum 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.

$f_{cd,fat}$ $= k_1 \cdot \beta_{cc}(t_0) \cdot f_{cd} \cdot (1 - f_{ck} / 250)$ with $\beta_{cc}(t_0)$ as per Eq. (3.2) and f_{cd} as per Eq. (3.15) (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:

$$14 \cdot (1 - E_{cd,max,equ}) / \sqrt{(1 - R_{equ})} \ge 6 \tag{NN.112}$$

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).

γ_{sd} a coefficient which is assumed to be $\gamma_{sd} = 1$.

DIN EN 1992-2:
Coefficient γ_{sd} corresponds to coefficient $\gamma_{Ed,fat}$. Eq. (NN.112) is equivalent to Eq. (NA.NN.12).

$$E_{cd,max,equ} + 0.43 \cdot \sqrt{(1 - R_{equ})} \le 1{,}0 \tag{NA.NN.12}$$

The upper and lower stress of the damage equivalent stress range shall be calculated with the equation (NN.113).

$$\sigma_{cd,max,equ} = \sigma_{c,perm} + \lambda_c\,(\sigma_{c,max,71} - \sigma_{c,perm})$$
$$\sigma_{cd,min,equ} = \sigma_{c,perm} - \lambda_c\,(\sigma_{c,perm} - \sigma_{c,min,71}) \qquad \text{(NN.113)}$$

with

$\sigma_{c,perm}$ Compressive stress under the fatigue combination without load model 71.

$\sigma_{c,max71}$, $\sigma_{c,min,71}$ Minimum and maximum compressive stress under the fatigue combination with load model 71 and the dynamic coefficient Φ.

λ_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 (α = 90°), 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 θ for lateral force and torsion as well:

$$\sigma_{cd,T} = \frac{T_{Ed}}{2 \cdot A_k \cdot t_{ef}} \cdot (\cot\Theta + \tan\Theta)$$

$$\sigma_{cd,V} = \frac{V_{Ed}}{b_w \cdot z} \cdot (\cot\Theta + \tan\Theta)$$

$$\sigma_{cd,max} = \begin{cases} \max \sigma_{cd,T} + \text{cor.}\ \sigma_{cd,V} \\ \max \sigma_{cd,V} + \text{cor.}\ \sigma_{cd,T} \end{cases}$$

$$\sigma_{cd,min} = \begin{cases} \min \sigma_{cd,T} + \text{cor.}\ \sigma_{cd,V} \\ \min \sigma_{cd,V} + \text{cor.}\ \sigma_{cd,T} \end{cases}$$

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 $\nu \cdot \eta_1$ as per Chapter 6.2.2 (6). In case of combined stressing from lateral force and torsion, the reduction factor $\nu \cdot \alpha_{cw} \cdot \eta_1$ with α_{cw} as per Eq. (6.9) applies. The coefficient η_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:

$\nu_1 = 0.75 \cdot \eta_1$ in case of pure lateral force as per NDP 6.2.3 (103).

$\nu_2 = (1.1 - f_{ck} / 500) \leq 1.0$ acc. to DIN EN 1992-1-1, NCI 6.8.7 (3).

$\nu \cdot \alpha_{cw} \cdot \eta_1 = 0.525 \cdot \eta_1$ 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:

$$\text{for}\quad \frac{V_{\mathrm{Ed,min}}}{V_{\mathrm{Ed,max}}} \geq 0 : \frac{|V_{\mathrm{Ed,max}}|}{|V_{\mathrm{Rd,c}}|} \leq 0.5 + 0.45 \cdot \frac{|V_{\mathrm{Ed,min}}|}{|V_{\mathrm{Rd,c}}|} \begin{cases} \leq 0.9 \text{ for concrete up to } \mathit{C50/60} \\ \leq 0.8 \text{ for concrete } \mathit{C55/67} \text{ or higher} \end{cases} \tag{6.78}$$

$$\text{for}\quad \frac{V_{\mathrm{Ed,min}}}{V_{\mathrm{Ed,max}}} < 0 : \frac{|V_{\mathrm{Ed,max}}|}{|V_{\mathrm{Rd,c}}|} \leq 0.5 - \frac{|V_{\mathrm{Ed,min}}|}{|V_{\mathrm{Rd,c}}|} \tag{6.79}$$

where

$V_{\mathrm{Ed,max}}$ is the design value of the maximum lateral force under the frequent action combination.

$V_{\mathrm{Ed,min}}$ is the design value of the minimum lateral force under the frequent action combination at the cross-section where $V_{\mathrm{Ed,max}}$ occurs.

$V_{\mathrm{Rd,c}}$ 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 Rüsch 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 Rüsch 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

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} \tag{6.14b}$$

- Combination for frequent situations

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} \cdot Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \tag{6.15b}$$

- Combination for quasi-continuous situations

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \tag{6.16b}$$

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 ε_{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 · 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 ΔF_{tp} in the prestressing steel is to be calculated as follows:

$$\Delta F_{tp} = \xi_1^2 \cdot A_p \cdot \varepsilon(y_p) \cdot E_p \qquad (13AT)$$

where

ξ_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 σ_c as defined in EN 1992-2, Chapter 7.2 (102) is to be limited to $0.60\,f_{\mathrm{ck}}$ 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 $0.45\,f_{\mathrm{ck}}$, 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 $0.60\,f_{\mathrm{c(t)}}$ when entering the average prestressing value. If the concrete compressive stress exceeds the value $0.45\,f_{\mathrm{c(t)}}$, the nonlinearity of the creep must be taken into account. $f_{\mathrm{c(t)}}$ 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 $0.45\,f_{\mathrm{c(t)}}$ or $0.60\,f_{\mathrm{c(t)}}$ for this situation depending on the user's specification. The value for $f_{\mathrm{c(t)}}$ 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 $0.8\,f_{\mathrm{yk}}$ or $1.0\,f_{\mathrm{yk}}$ 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 $1.0\,f_{\mathrm{yk}}$ 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 $0.75\,f_{\mathrm{pk}}$ under the characteristic action combination.

DIN EN 1992-1-1:

The check is carried out for the quasi-continuous combination with the limit $0.65\,f_{\mathrm{pk}}$. In addition, the stresses are checked against the minimum of $0.9 \cdot f_{\mathrm{p0,1k}}$ and $0.8 \cdot f_{\mathrm{pk}}$ under the characteristic combination.

For situations before prestressing and for tendons without bond, the stress $\sigma_{\mathrm{pm0}}\,(\mathrm{x})$ 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 users'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 σ_1 or one of the longitudinal tensile stresses σ_x or σ_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, centrical 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} / ζ with ζ as per SS EN 1992-1-1, Arcticle 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:

$$A_{s,min} \cdot \sigma_s = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} \tag{7.1}$$

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 $f_{ct,eff}$.

σ_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/m² 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/m² for normal concrete and 2.5 MN/m² 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:

$$k_c = 0.4\,(1 - \sigma_c / (k_1 \cdot h \;/\; h^*) / f_{ct,eff}) \le 1 \tag{7.2}$$

For flanges of box girders and T sections:

$$k_c = 0.9 \cdot F_{cr} / A_{ct} / f_{ct,eff} \ge 0.5 \tag{7.3}$$

σ_c is the average concrete stress in the analyzed part of the section with

$$\sigma_c = N_{Ed} / (b \cdot h) \tag{7.4}$$

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:

$k_1 = 1.5$ for compressive normal force

$k_1 = 2\, h^* / (3h)$ 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 ϕ_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 ϕ_s^* as an input value for Table 7.2N:

$\phi_s = \phi_s^* \cdot f_{ct,eff} \,/\, 2.9 \cdot k_c \cdot h_{cr} \,/\, (2(h\text{-}d))$ for bending (7.6N)

$\phi_s = \phi_s^* \cdot f_{ct,eff} \,/\, 2.9 \cdot h_{cr} \,/\, (8(h\text{-}d))$ for central tension (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 ϕ_s^* and the permissible crack width w_{max} are used to determine the permissible reinforcing steel stress σ_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 σ_s.

When determining the minimum reinforcement under centrical tension, the steel stress σ_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 ϕ_s^* for Table 7.2DE is determined using the following equations:

$\phi_s = \phi_s^* \cdot f_{ct,eff} \,/\, 2.9 \cdot k_c \cdot k \cdot h_{cr} \,/\, (4(h\text{-}d)) \geq \phi_s^* \cdot f_{ct,eff} \,/\, 2.9$ for bending (7.6DE)

$\phi_s = \phi_s^* \cdot f_{ct,eff} \,/\, 2.9 \cdot k_c \cdot k \cdot h_{cr} \,/\, (8(h\text{-}d)) \geq \phi_s^* \cdot f_{ct,eff} \,/\, 2.9$ for centrical tension (7.7DE)

The steel stress σ_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 σ_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:

$\phi_s = \phi_s^* \cdot f_{ct,eff} \,/\, 2.9 \cdot k_c \cdot k \cdot h_{cr} \,/\, (4(h\text{-}d)) \geq \phi_s^* \cdot f_{ct,eff} \,/\, 2.9$ (21AT)

For centrical tension $h_{cr} = h / 2$ for each member side is applied.

For members under centrical 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 $h\text{-}d$. 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.

ξ_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 ξ_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/mm². 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/mm² according to Chapter 9.2.2.1.

SS EN 1992-1-1:
The value is specified as $\sigma_{ct,p} = f_{ctk} / \zeta$ with ζ 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 ϕ (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:

  $\rho_{p,eff} = (A_s + \xi_1^2 \cdot A_p') / A_{c,eff}$ (7.10)

  ξ_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

$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - k_t \cdot f_{ct,eff} / \rho_{p,eff} (1 + \alpha_e \cdot \rho_{p,eff})] / E_s \geq 0.6\ \sigma_s / E_s$ (7.9)

Where

α_e $= E_s / E_{cm}$

σ_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:

$\sigma_s = \sigma_{s2} + 0.4 f_{ct,eff} (1/\rho_{p,eff} - 1/\rho_{tot})$ (NA. 7.5.3)

σ_s is limited to f_{yk} in the program

σ_{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

$s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot \phi / \rho_{p,eff}$ (7.11)

Where

ϕ 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:

$k_1 \cdot k_2 = 1$, $k_3 = 0$ 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:

$k_3 = 0$ and $k_4 = 1 / (3.6 \cdot k_1 \cdot k_2) \leq \rho_{p,eff} \cdot \sigma_s / (3.6 \cdot k_1 \cdot k_2 \cdot f_{ct,eff})$ (22AT)

$s_{r,max} = \phi / (3.6 \cdot \rho_{p,eff}) \leq \sigma_s \cdot \phi / (3.6 \cdot f_{ct,eff})$. (23AT)

SS EN 1992-1-1:

$k_3 = 7 \cdot \phi / c$ (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

$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm})$ (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 σ_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:

  $$\sigma_s = \sigma_{s2} + 0{,}4 f_{ct,eff} (1/\rho_{p,eff} - 1/\rho_{tot}) \quad \text{(NA. 7.5.3)}$$

  σ_{s2} = Reinforcing steel stress from strain state II
- Compare the value entered in the dialog ($\max. s$) with the table value ($\text{perm. } s$), which is derived from the calculated steel stress σ_s and the permissible crack width w_k. In the log, the location with the largest quotient ($\max. s$ / $\text{perm. } s$) 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 mm ≤ c_{nom} ≤ 40 mm with bar diameters 8 mm ≤ d_s ≤ 20 mm.

The user is responsible for the evaluation of these requirements.

Determining the Effective Area Ac,eff

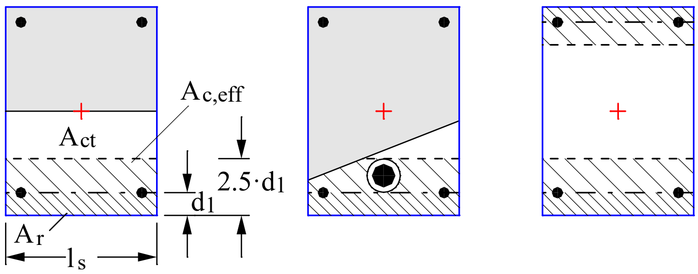
According to EN 1992-1-1, Figure 7.1, the effective area of reinforcement $A_{c,eff}$ defines the area of a rectangular, uniaxially stressed concrete section in which the model assumptions in Book 466 of the German Committee for Reinforced Concrete (DAfStb) are applicable. Although the program can apply 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:

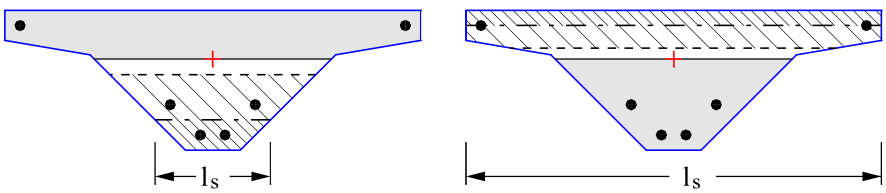
- Determine tensile zone A_{ct} in state I: When calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the exposure class. In the case of prestressed cross-sections, the specified variation coefficients 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 $A_c / 2$.

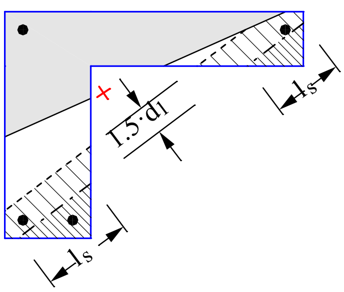
The following illustrations show the effective areas determined by the program for a few representative situations. The last case (edge beam) deviates from the model assumptions in Book 466 to such a degree that it is questionable as to whether it should be used.



Effective area of the reinforcement at a rectangular section under uniaxial bending, normal force with double bending and centrical tension



Effective area of the reinforcement at a bridge section under uniaxial bending



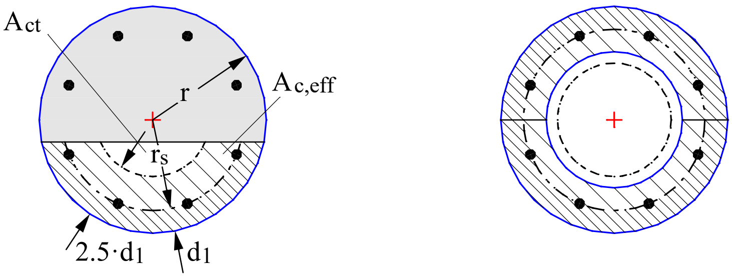
Effective area of the reinforcement at an edge beam under uniaxial bending

Ring-shaped determination of Ac,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 centrical 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

σ_x Longitudinal stress from bending and longitudinal force in accordance with the rules for stress analysis indicated above.

τ_{xy} Edge shear stress in the y direction from lateral force and torsion.

τ_{xz} Edge shear stress in the z direction from lateral force and torsion.

The angle between σ_1 and σ_x is derived with:

$$\tan 2\vartheta_1 = \frac{-2\tau}{\sigma_x}$$

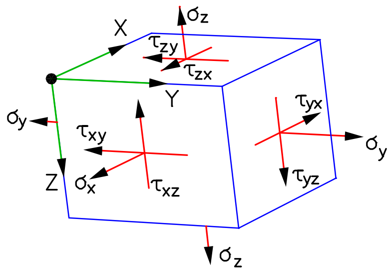
with

$\tau = \sqrt{\tau_{xy}^2 + \tau_{xz}^2}$ as the resulting edge shear stress.

The maximum value for σ_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:



$$\begin{bmatrix} \sigma_x & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_y & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & \sigma_z \end{bmatrix}$$

with

σ_x, σ_y Longitudinal stress from bending and normal force.

σ_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 σ_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

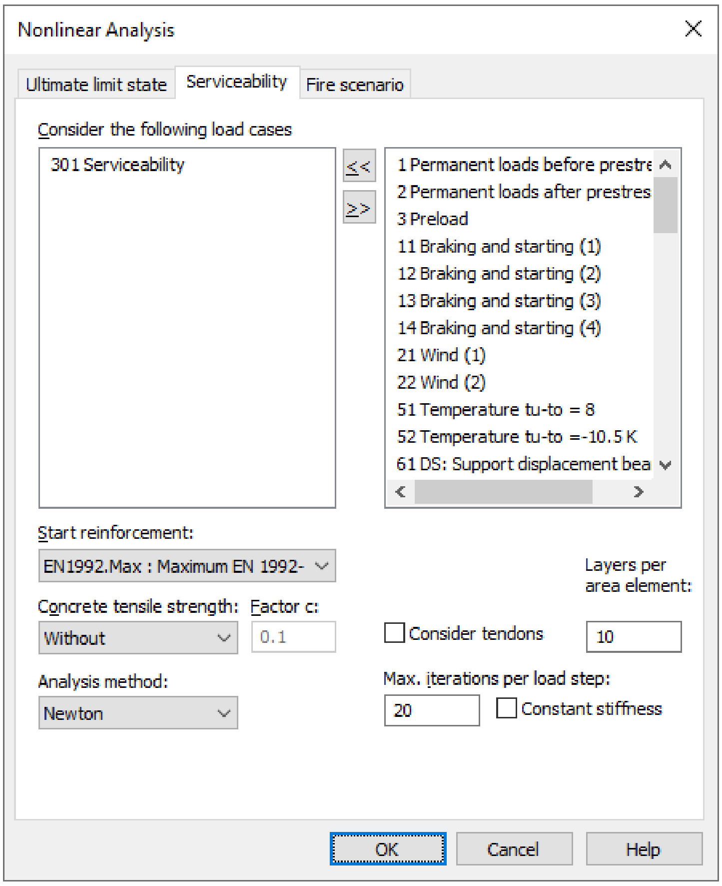
According to EN 1992-1-1, Chapter 7.4.1, the deformations of a component or structure may not impair its proper functioning or appearance.

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 ψ_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

σ_p, $\Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m²].
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

σ_x	Longitudinal stresses in the decompression and concrete compressive stress checks [MN/m²].
σ_1, σ_x, τ_{max}	DIN EN 1992-2: Diagonal principal tensile stresses and related longitudinal and shear stresses [MN/m²].
$\Delta\sigma_1$	DIN EN 1992-2: Stress ranges of principal stresses in the fatigue check under lateral forces [MN/m²].
σ_2	DIN EN 1992-2: Principal compressive stresses of box girders [MN/m²].
σ_s, $\Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m²].
σ_p, $\Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m²].
σ_{cd}, $\Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete [MN/m²].
$\sigma_{c,perm}$	Permanent stress during fatigue check for concrete under longitudinal pressure [MN/m²].
$\Delta\sigma_{sb,y}$, $\Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from Q_y and Q_z [MN/m²].
$\Delta\sigma_{sb,T}$, $\Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and longitudinal torsion reinforcement [MN/m²].
σ / σ_{perm}	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

Stresses for area elements

σ_r	Concrete stress in the tendon direction in the decompression check [MN/m²].
σ_x, σ_y, σ_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/m²].
σ_2	Principal compressive stresses in the concrete compressive stress check [MN/m²].
σ_1, σ_x, σ_y,	DIN EN 1992-2: Diagonal principal tensile stresses and related longitudinal and
τ_{xy}, τ_{xz}, τ_{yz}	shear stresses [MN/m²].
σ_{sx}, $\Delta\sigma_{sx}$	Stresses and stress ranges for reinforcing steel in the x direction [MN/m²].
σ_{sy}, $\Delta\sigma_{sy}$	Stresses and stress ranges for reinforcing steel in the y direction [MN/m²].
σ_p, $\Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m²].
$\sigma_{cd,x}$, $\Delta\sigma_{cd,x}$,	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/m²].
$\sigma_{cx,perm}$,	Permanent stress during fatigue check for concrete under longitudinal pressure in x-
$\sigma_{cy,perm}$	and y-direction [MN/m²].
$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m²].
σ / σ_{perm}	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

Bending reinforcement

A_s	Bending reinforcement [cm²] for beams and design objects.
a_{sx}, a_{sy}	Bending reinforcement [cm²/m] for area elements in the x and y direction.

Reinforcement from lateral force

a_{sbx}, a_{sby}, a_{sb}	Stirrup reinforcement [cm²/m²] of area elements from q_x, q_y and q_r.
$A_{sb.y}$, $A_{sb.z}$	Stirrup reinforcement [cm²/m] of beams and design objects from Q_y and Q_z.
A_{sl} for $a_{sb}=0$	Longitudinal reinforcement [cm²] of area elements.
z_y, z_z	Inner lever arm [m] for lateral force Q_y and Q_z.
ΔF_{tdy}, ΔF_{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 [cm²/m] of beams and design objects from M_x.
$A_{sl.T}$	Torsional longitudinal reinforcement [cm²] 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

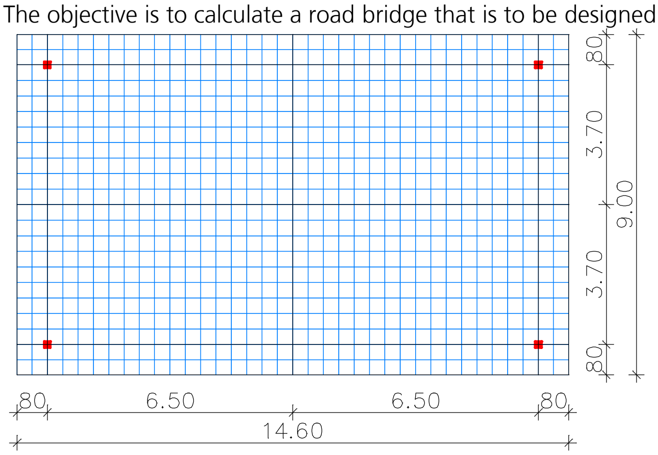
> Bauer, Thomas / Müller, Michael, Straßenbrücken in Massivbauweise nach DIN-Fachbericht
> (Road Bridges in Solid Construction According to DIN Technical Report)
> **Beispiele prüffähiger Standsicherheitsnachweise (Examples of Verifiable Stability Safety Checks).**
> Stahlbeton- und Spannbetonüberbau 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:

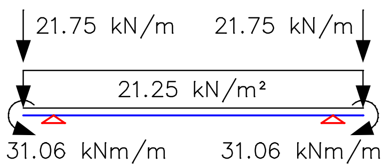
- Section-dependent check specifications
- Selection of the structure type
- Load model 1
- Combination of actions
- Performing checks

The Task

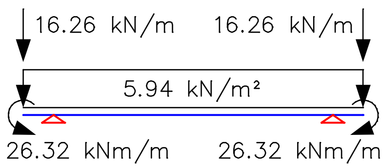
The objective is to calculate a road bridge that is to be designed as a single-span, reinforced concrete slab.



Concrete C30/37-EN, slab thickness 0.85 [m]



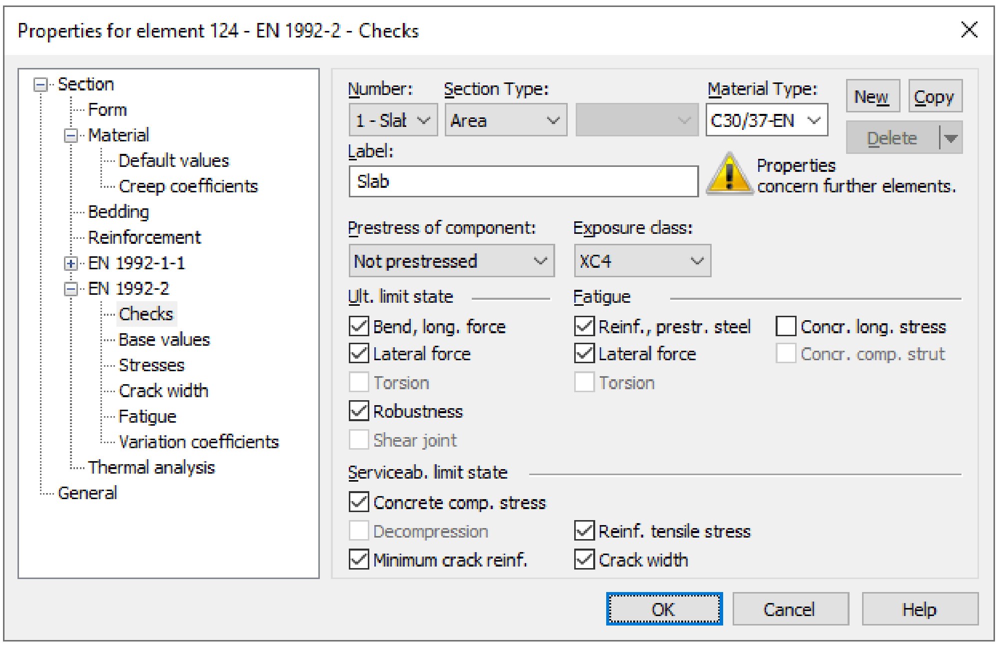
Bridge cross section with load case 1 (dead load)



Bridge cross section with load case 2 (additional dead load)

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:

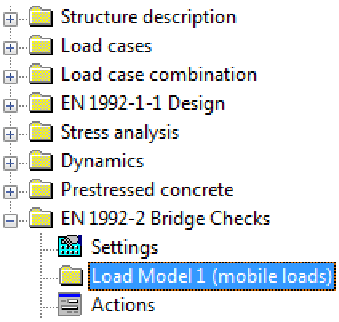


Load model 1

Load model 1 consists of two parts:

(a) Twin axle (TS tandem system)
(b) Uniformly distributed area loads UDL

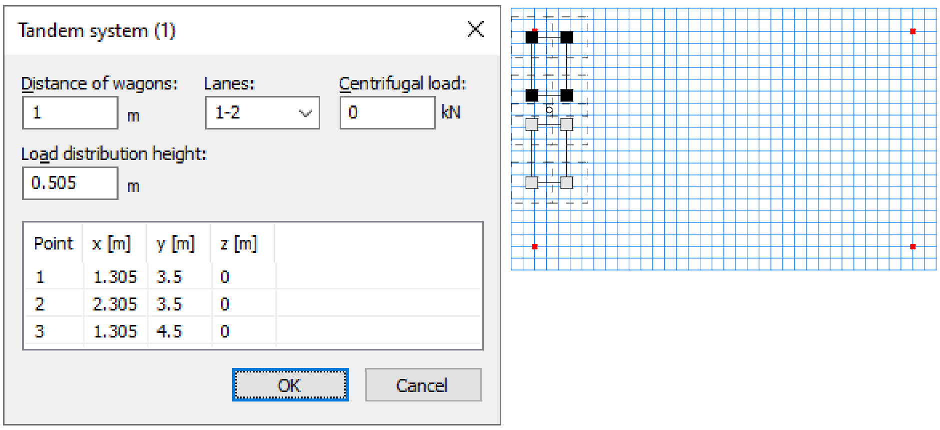
These loads should be applied in both the longitudinal and lateral directions of the bridge in the least favorable position. In the lateral direction, the load positions are determined by dividing the roadway into computational lanes.
Since the decisive lane division is not always known in advance, you can define different load position variants.



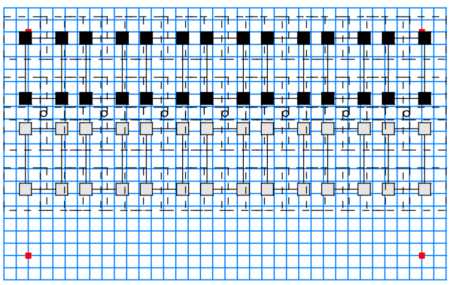
In the *EN 1992-2 Bridge Checks* folder right-click *Load model 1* and then select *New* from the context menu.

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.

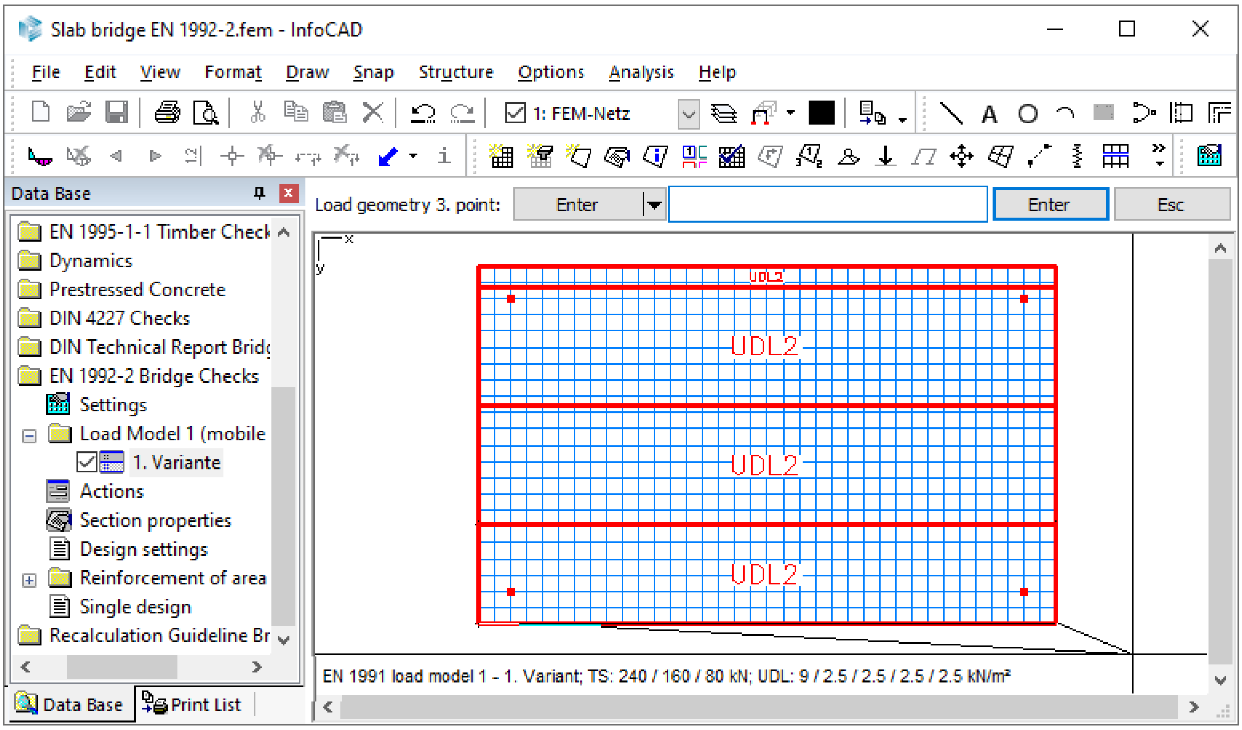


The properties of the tandem system position are set in the dialog.
The Lane 1 is defined left of the traffic direction and indicated by a dark hatching pattern. The area resulting from the load distribution height is shown in dashes.

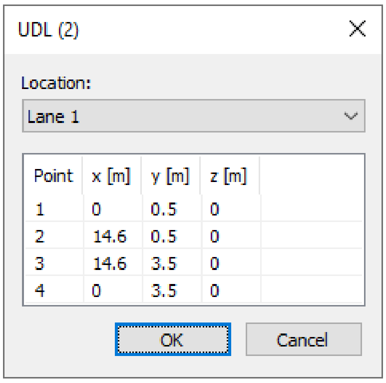


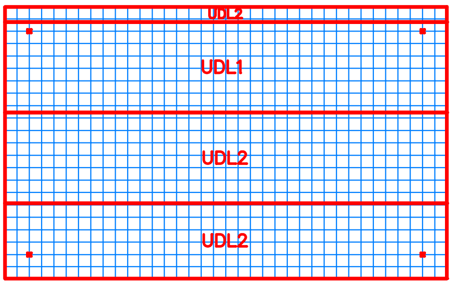
Once the tandem system is selected, you can either copy or generate the desired load positions. The usual snap functions such as *Mid-point* or *Endpoint* can be used for this purpose.

Now click *UDL New* to define the load areas of the UDL system in consecutive order.



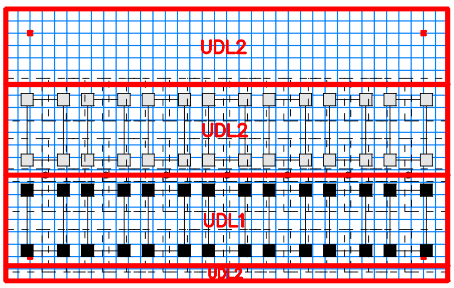
Initially, all the load areas are accepted as UDL2 (lanes 2). Double-click the second UDL area and specify *Lane 1* for this.





Completed UDL areas (TS hidden)

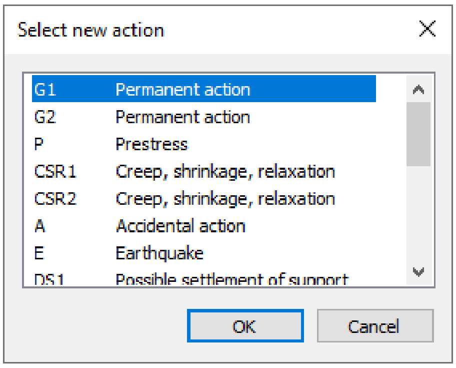
This done, the 1st variant of the load model has been defined. To enter the 2nd variant, right-click *Load model 1* in the database again and select *New* from the context menu.
All of the TS and UDL positions can be copied to the 2nd variant via the clipboard and then modified (in this case: rotated by 180°).



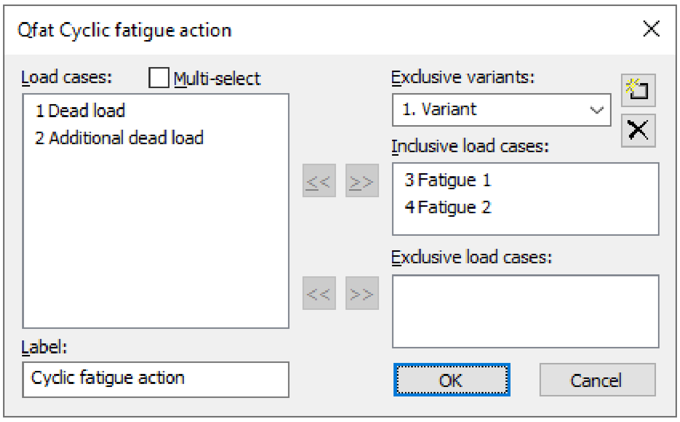
Load model 1 is now fully described and can be calculated. This is done in conjunction with the load cases as part of the FEM calculation. The results of all load positions are individually saved and grouped in the
Load model 1 (Mobile loads) folder.

Combination of actions

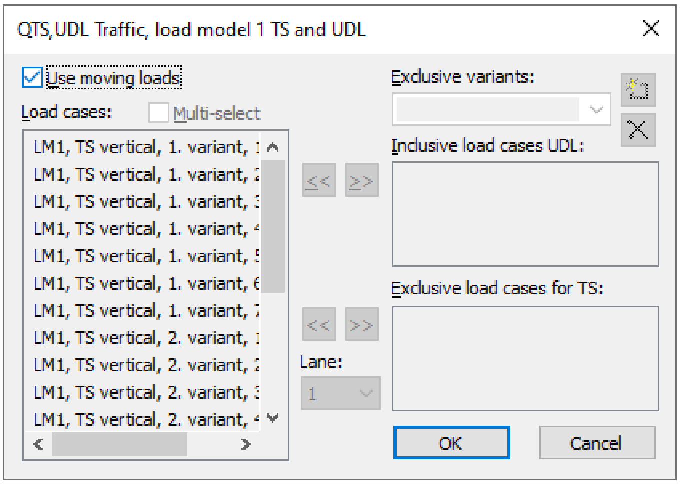
To describe the actions and their combinations, click the *Actions* option in the database and then select *Insert*.



The actions to be considered are selected in the dialog. You can now assign load cases to them.



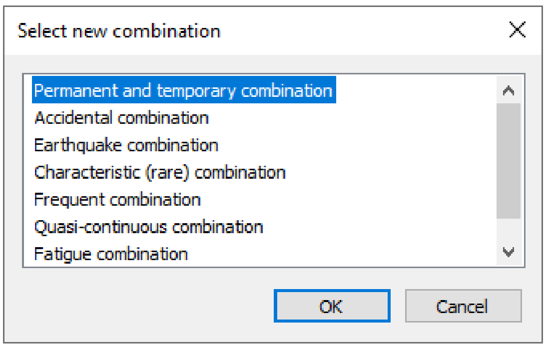
For variable actions, you can add multiple load cases to form inclusive and exclusive groups. If required, you can also define multiple mutually exclusive variants.



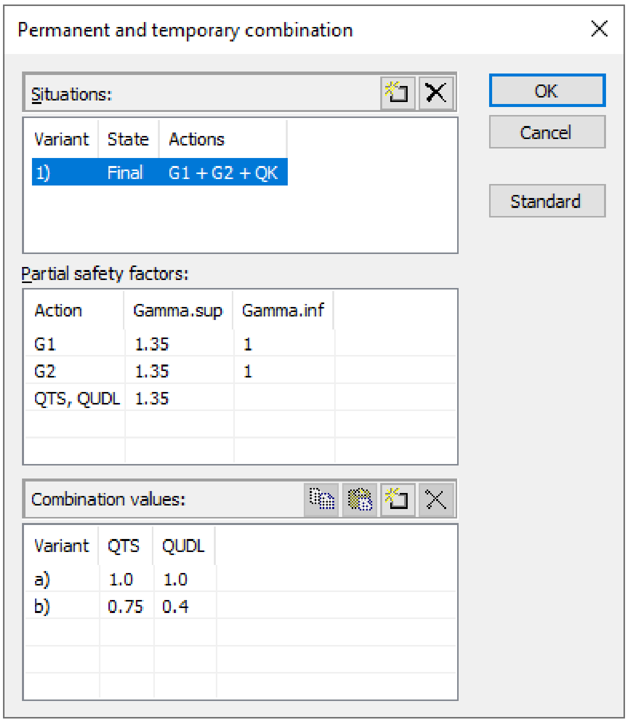
The variable actions QTS and QUDL are taken directly from load model 1.

The *Use moving loads* option must be enabled for this to work.

Alternatively, you can also use freely-definable load cases or combinations.

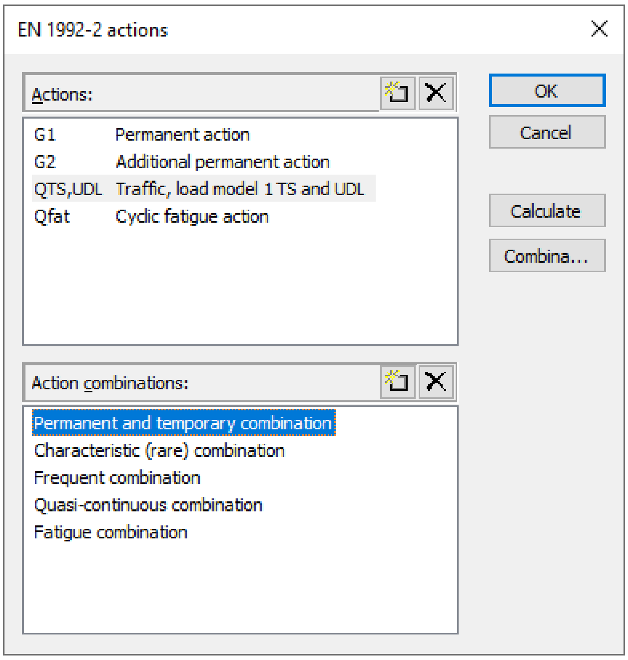


Once the respective loads have been assigned to the actions, you can add the combinations required for the checks.

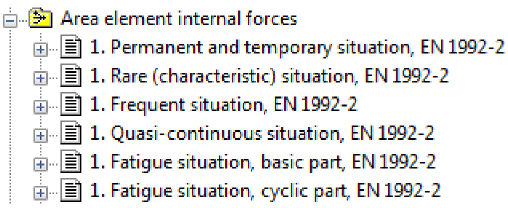


The following variants are derived using the safety factors and combination coefficients according to EN 1990.

You can accept the suggestion by clicking *OK* in order to continue selecting more combinations.



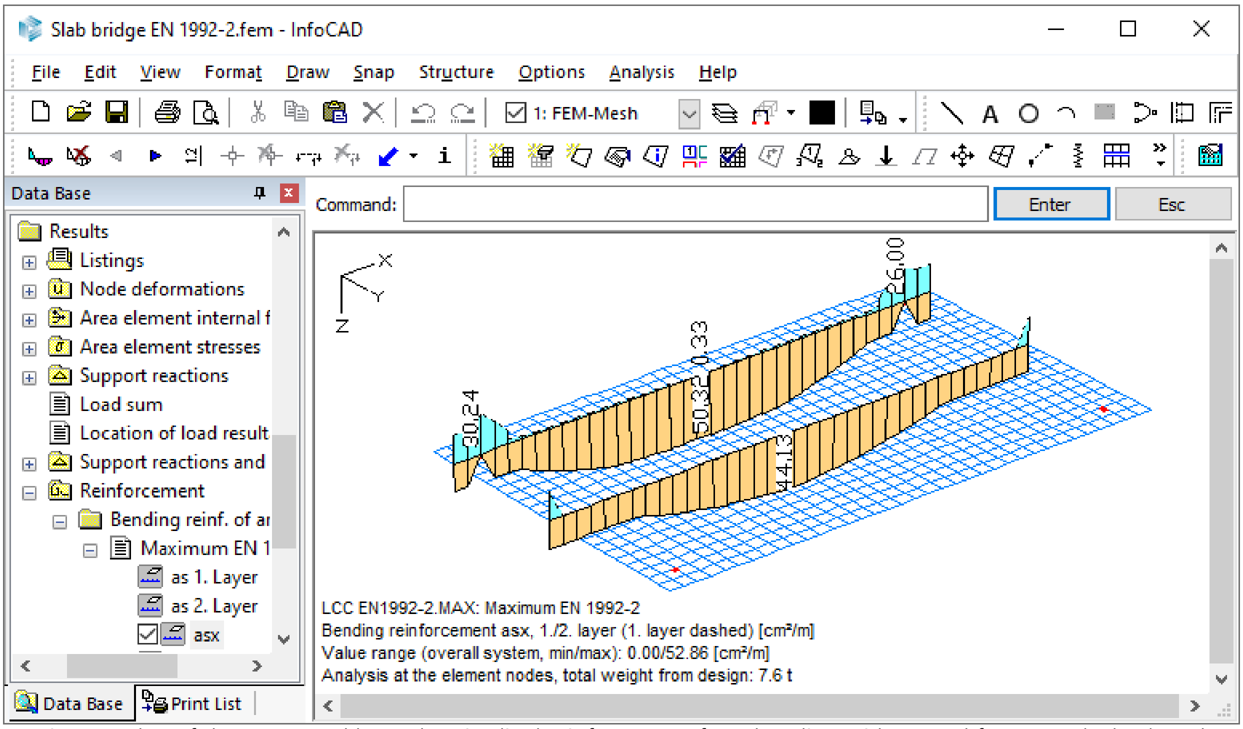
To determine the extremal internal forces from the defined combinations, click *Calculate*.
The program combines the results of the load cases, combinations and load models that are used and then makes them available in the usual manner:



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.    Prestress         Reinforcem.  Fatigue      Cr.  De-   Stress
     class     of component      M R B Q T S  B Q T P C V  wi.  co.   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.
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.

                   Den-                              Asl [cm²]            Red.
 Se. Concr.        sity     Dsn.   fyk    cot    Dsn.   Pic. 6.3    Factor   pre-
                   [kg/m³]  M,N   [MPa]  Theta  S.C.  given   max   rhow  as  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] bw	bw.nom	Eff. width bn [m]	kb	Height[m] h	h.nom	Eff.height d [m]	kd	Torsion section [m] 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].
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 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 = Centrical 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 max	Coefficient Xi1	k	kt	Fact.fctm As	wk	Comb. As	wk	Method kc	wk	RS
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] s	b	p	Lambda s	b	p	c	Eta	Factor 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/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 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 reinforement at ultimate limit state
In case of dominant bending, compression reinforcement is marked with "*".
For section areas acc. to 6.1 (5) the conrecte 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.	Reinforcem. Lay.	Type	Reinf. for x-direction nsd [kN/m]	msd [kNm/m]	req.asx [cm²/m]	Reinf. for y-direction nsd [kN/m]	msd [kNm/m]	req.asy [cm²/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
				B	0.00	1618.41	0.37	0.00	-64.40	1.86
			2	M	0.00	1198.82	9.82	0.00	21.95	10.08
				R	0.00	778.28	45.58!	0.00	-23.88	0.00
				B	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
				B	0.00	1616.96	0.32	0.00	-55.35	1.60
			2	M	0.00	1197.75	9.82	0.00	13.24	10.08
				R	0.00	775.27	45.44!	0.00	-29.79	0.00
				B	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
				B	0.00	1591.86	0.20	0.00	-34.74	1.00
			2	M	0.00	1179.15	9.82	0.00	39.02	10.08
				R	0.00	762.99	44.86!	0.00	-18.35	0.00
				B	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
				B	0.00	1593.34	0.24	0.00	-41.19	1.19
			2	M	0.00	1180.25	9.82	0.00	47.67	10.08
				R	0.00	766.00	45.01!	0.00	-12.42	0.00
				B	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
Asl Req. longitudinal reinf. as per Fig. 6.3 for req. asb [cm²]
qr Lateral force for design = (qx²+qy²)½ [kN/m]
req.asb Req. stirrup reinforcement [cm²/m²]
Exceedings as per Eq. (6.12) are marked by "!"
req.Asl Req. longitudinal reinf. as per Fig. 6.3 [cm²] for req. asb

Element No.	Loc.	qx [kN/m]	qy [kN/m]	n [kN/m]	qr [kN/m]	qr/ vRd	Angle	req. asb [cm²/m²]	Asl f. asb=0 [cm²]	req. Asl [cm²]
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/m²]
dSigma.s,zul = dSigma.Rsk,s / gamma.s,fat [MN/m²]
(simplified) = dSigma.Rsk,s as per 6.8.6 (1) [MN/m²]

Element No.	Lo.	Lay.	Check in x-direction as [cm²/m] giv.	req.	dSigma.s [MN/m²] equ.	per.	Check in y-direction as [cm²/m] giv.	req.	dSigma.s [MN/m²] equ.	per.
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/m²]
dSigma.s,per = dSigma.Rsk,b / gamma.s,fat [MN/m²]
(simplified) = dSigma.Rsk,b as per 6.8.6 (1) [MN/m²]
```

Element No.	Se.	Loc.	asb [cm²/m²] giv.	req.	dSigma.s [MN/m²] 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 No.	Se.	C.	Loc.	Check in x-direction nsd [kN/m]	msd [kNm/m]	wk [mm]	wmax [mm]	Check in y-direction nsd [kN/m]	msd [kNm/m]	wk [mm]	wmax [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/m²]
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 No.	Se.	Loc.	Stress	min Sigma [MN/m²]	Sigma,per [MN/m²]	Side t	b	Period	Situation
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)

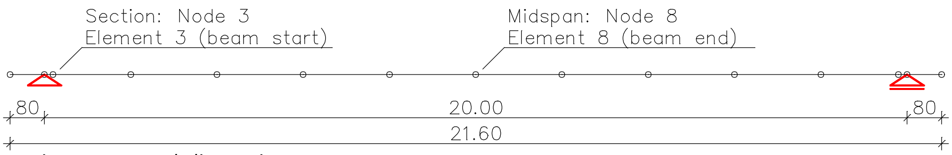
Element No.	Se.	Lo.	Lay.	Check in x-direction as [cm²/m]	Sigma.s [MN/m²]	per. [MN/m²]	Check in y-direction as [cm²/m]	Sigma.s [MN/m²]	per. [MN/m²]
124	1	1	1	0.37	-45.68	400.00	22.08	29.73	400.00
			2	50.29	329.30	400.00	10.08	29.45	400.00
124	1	2	1	0.32	-46.25	400.00	22.08	25.55	400.00
			2	50.24	329.33	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.

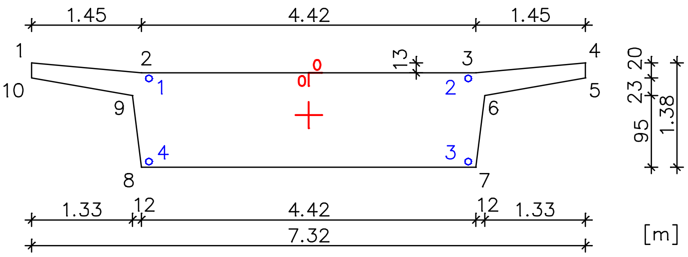


Static system and dimensions

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 α = 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

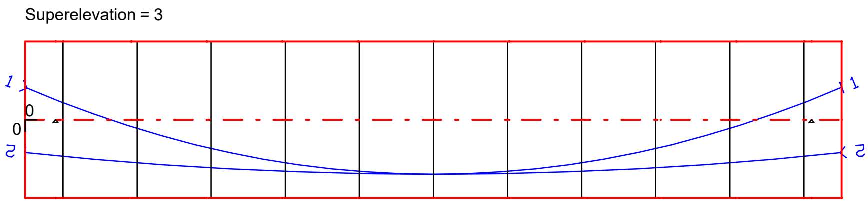
Because of their low influence on the selected checks, actions arising from centrifugal loads, lateral impacts, wind loads, the 'unloaded train' load model, starting and braking, derailment, actions on rails, traffic loads on service sidewalks and soil pressure are ignored.

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 mm²
E-modulus of the prestressing steel	190000 MN/m²
0.1% strain limit (yield strength) of the prestressing steel $f_{p0.1k}$	1500 MN/m²
Tensile strength of the prestressing steel f_{pk}	1770 MN/m²
Permissible prestressing force of a tendon P_{m0}	3391.5 kN
Friction coefficients when prestressing and releasing μ	0.21
Unintentional deviation angle of a tendon β'	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 zv [cm] at the base points

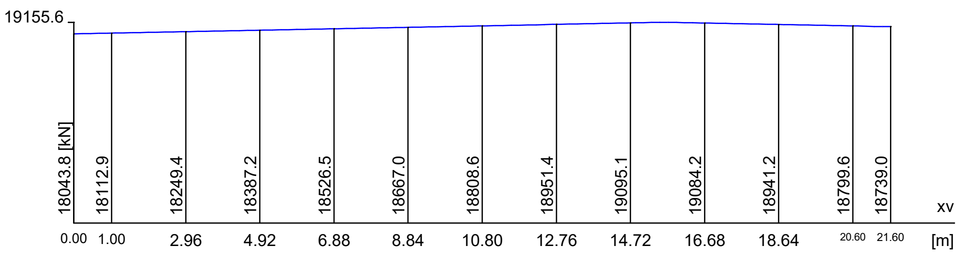
xv	0.00	1.00	2.96	4.92	6.88	8.84	10.80	12.76	14.72	16.68	18.64	20.60	21.60
1	-28.6	-15.1	7.6	25.3	37.9	45.5	48.0	45.5	37.9	25.3	7.6	-15.1	-28.6
2	28.8	32.2	37.9	42.3	45.5	47.4	48.0	47.4	45.5	42.3	37.9	32.2	28.8

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.
Pm0 = 3391.5 kN, Ap = 2660.0 mm², µa = 0.21, Angle ß' = 0.30 °/m
E-Modulus= 190000 MN/m², Ah = 7389.8 mm², 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]: 139.6



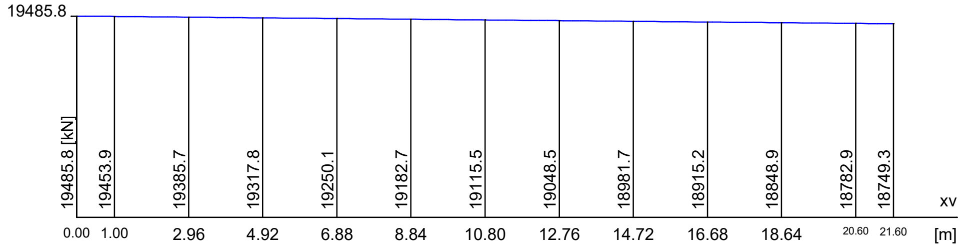
Prestressing curve of tendon group 1 in the longitudinal section

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 mm², μa = 0.21, Angle ß' = 0.30 °/m
E-Modulus= 190000 MN/m², Ah = 7389.8 mm², 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

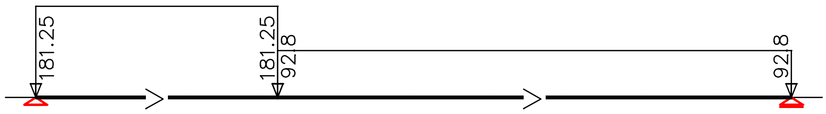


Prestressing curve of tendon group 2 in the longitudinal section

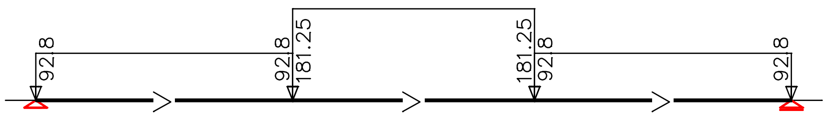
Loads

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_∞). The specifications RH=80%, t_0=10 d, t_s=1 d, cement hardening = normal result in the following coefficients: $\varphi_{t\infty} = 1.57$; $\varepsilon_{t\infty} = -23.7 \cdot 10^{-5}$; $\rho = 0.8$ 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 Δx_{neg} (T)
Load case 32	Temperature Δx_{pos} (T)

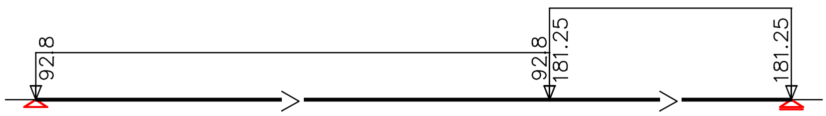
Load model 71



Load case 11: Load model 71, left
The dynamic coefficient Φ of 1.16 is already considered in the load ordinates.

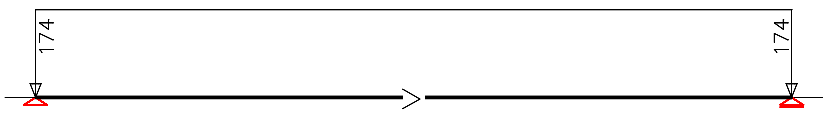


Load case 12: Load model 71, middle



Load case 13: Load model 71, right

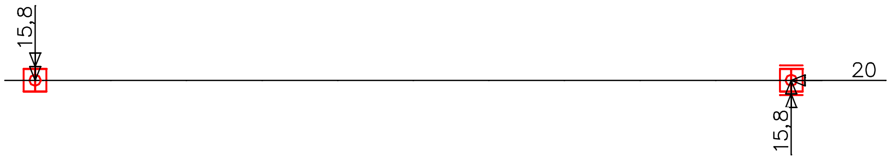
Load model SW/2



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 ΔT_{Mz} and ΔT_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

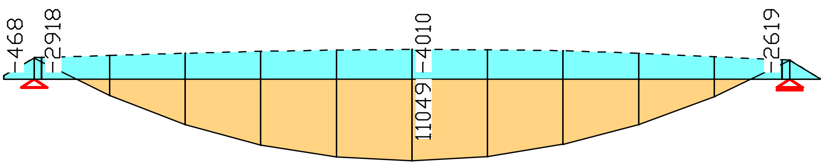
Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
P, CSR1, CSR2	1	1
T	1	0
Q1	1	0
Q2	1	0
Qfat	1	0

Fatigue combination, combination coefficients

Variant	T	Q1	Q2
a)	0.6	0	0

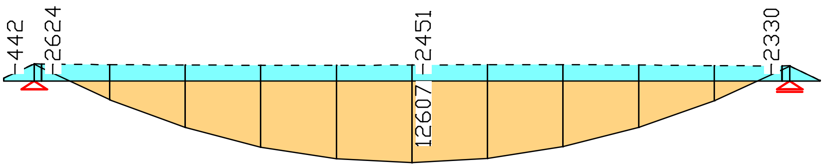
Below you will find an example of the curve of bending moment M_y for design situations in the ultimate limit states.

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 Pm,t of prestressing.

Se.	Prestressing of component	Const.period r.sup	Const.period r.inf	Final state r.sup	Final state 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.
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.	Density [kg/m^3]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. S.C.	Asl [cm^2] Pic. 6.3 given	Asl [cm^2] max	Factor rhow	Factor 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] bw	Width [m] bw.nom	Eff. width bn [m]	Eff. width kb	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	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].
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 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 = Centrical 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 max	Coefficient Xi1	Coefficient k	Coefficient kt	Fact.fctm As	Fact.fctm wk	Comb. As	Comb. wk	Method kc	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)	per.sigma.c(t)	per.sigma.c		per.sigma.s	Decompression	
	[MN/m²]	(CC, QC)	(CC)	(QC)	(CC)	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

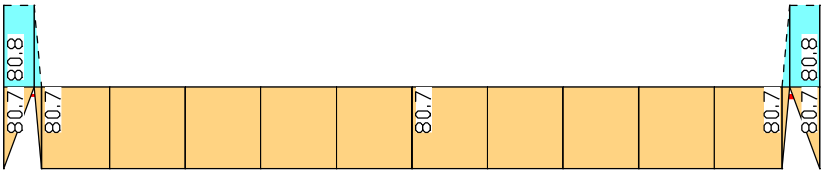
The concrete compressive stresses are calculated according to fig. 3.2.

```
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).
```

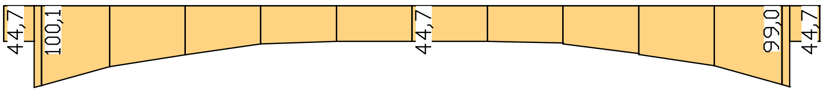
Se.	Check	fcd,fat	dSigma.Rsk [MN/m²]			Lambda				Eta	Factor
	SPCVQT	[MN/m²]	s	b	p	s	b	p	c		Qfat
1	xxx.x.	13.68	162.00	73.00	120.00	0.65	0.65	0.70	0.90	2.00	1.00

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) [cm²] (upper reinforcement with dashed lines).



Necessary lateral force reinforcement A_{sbz} to satisfy the fatigue check [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 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 Xi1*Ap which was used to reduce req.As
    Xi1     Bond coefficient for prestressing steel as per Eq. (7.5)
(B) Design of reinforement at ultimate limit state
    In case of dominant bending, compression reinforcement is marked with "*".
    For section areas acc. to 6.1 (5) the conrecte 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' [cm²]	req.As [cm²]
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
			2	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
			3	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
			4	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' [cm²]	req.As [cm²]
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.
          Exceddings 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	Asb.y [cm²/m]	Qz/ VRd	Angle	Asb.z [cm²/m]	Asl [cm²]	Q/VRd+ Mx/TRd	Asb.T [cm²/m]	Asl.T [cm²]
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/m²]
dSigma.s,zul = dSigma.Rsk,s / gamma.s,fat [MN/m²]
(simplified) = dSigma.Rsk,s as per 6.8.6 (1) [MN/m²]
```

Beam No.	Se.	Loc.	Steel Type	No.	As [cm²] giv.	req.	dSigma.s [MN/m²] equ.	per.
3	1	1	S	1	0.00	.	.	140.87
			S	2	0.00	.	.	140.87
			S	3	40.35	.	1.31	140.87
			S	4	40.35	.	1.31	140.87
			P	1	159.60	.	0.17	104.35
			P	2	159.60	.	0.35	104.35
8	1	1	S	1	0.00	.	.	140.87
			S	2	0.00	.	.	140.87
			S	3	40.35	.	31.64	140.87
			S	4	40.35	.	31.65	140.87
			P	1	159.60	.	12.54	104.35
			P	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 [cm²/m]
Asb giv.     Given reinforcement from preceded dimensioning  [cm²/m]
Asb req.     Requ. reinf. for compl. with the permiss. stress range [cm²/m]
dSigma.s,equ Damage equivalent fatigue stress range [MN/m²]
dSigma.s,per = dSigma.Rsk,b / gamma.s,fat [MN/m²]
(simplified) = dSigma.Rsk,b as per 6.8.6 (1) [MN/m²]
```

Beam No.	Se.	Loc.	Rei. Type	Asb [cm²/m] giv.	req.	dSigma.s [MN/m²]	[MN/m²]
3	1	1	BY	0.00	.	.	63.48
			BZ	44.73	100.08	63.48	63.48
8	1	1	BY	0.00	.	.	63.48
			BZ	44.73	44.73	11.00	63.48

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/m²]
Sigma.cd,max  Absolute value of damage equiv. max. compressive stress [MN/m²]
Sigma.cd,per  = fcd,fat * (1 - 0.43 * (1 - Sigma.cd,min/Sigma.cd,max)½) [MN/m²]
t, b          Position of the edge point: above, below of centre
```

Beam No.	Se.	Loc.	|Sigma.cd|min [MN/m²]	|Sigma.cd|max [MN/m²]	Sigma.cd,per [MN/m²]	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/m²]
         (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/m²]	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/m²] top	bottom	Compr.depth[mm] dp	dp,min	Tendon group	Period	Situation
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/m²]
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/m²]	Sigma.x,per [MN/m²]	Se.-Pnt.	Side t	b	Period	Situation
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

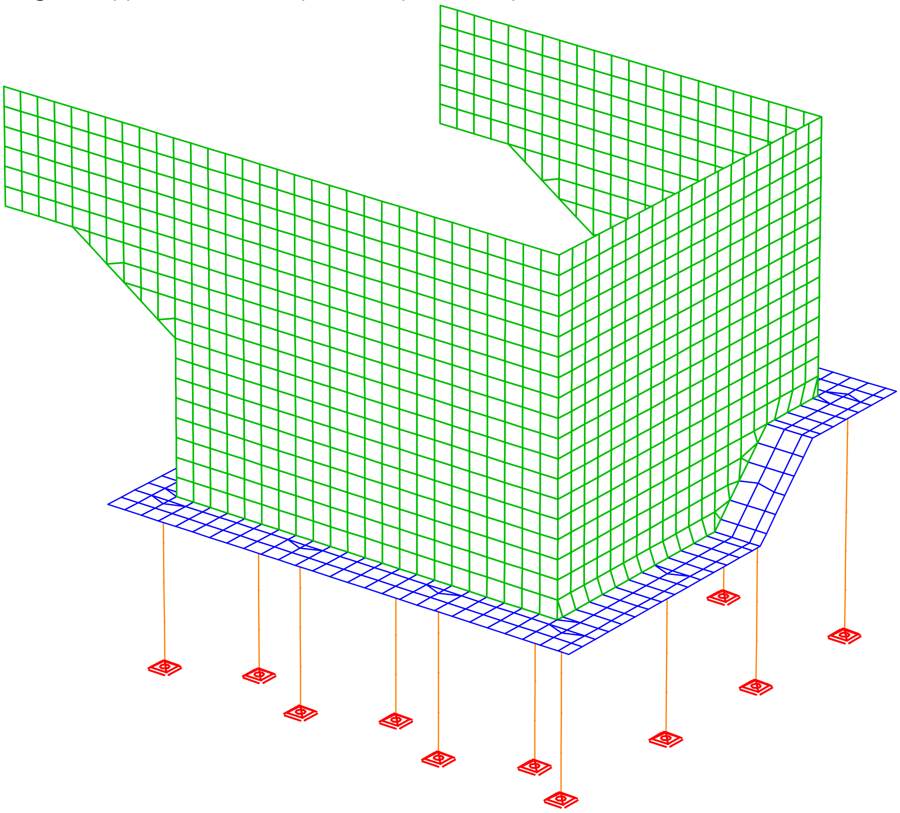
For the check, a cracked concrete section is assumed.
For tendon groups without bond and/or for situations before grouting, the prestressing steel stress is checked acc. to Eq. (5.43).

```
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.	Steel Type	No.	As [cm²]	Sigma.s [MN/m²]	per. [MN/m²]	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

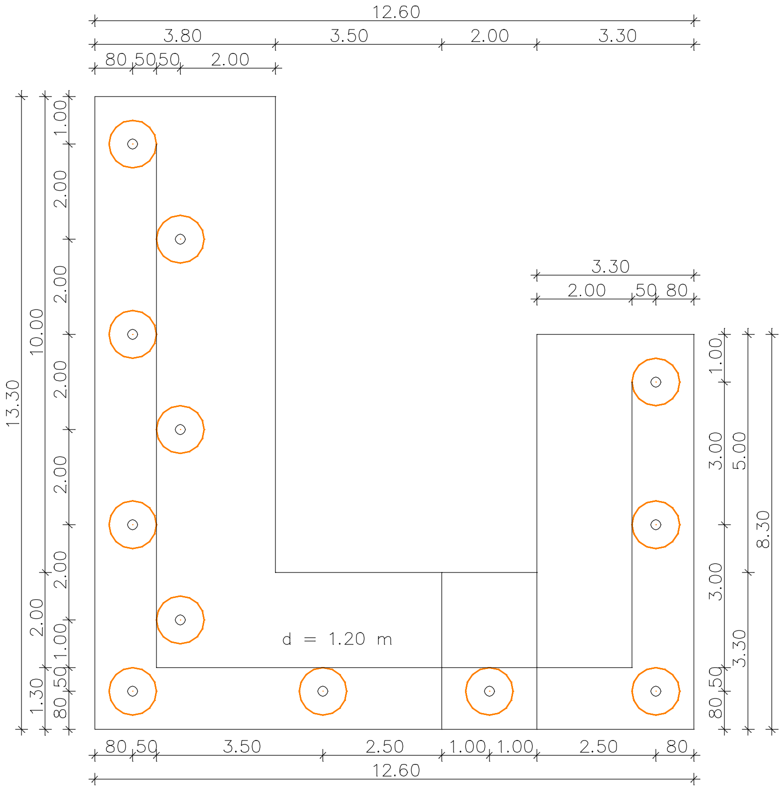
Concrete	C 30/37-EN (areas), C 40/50-EN (piles)
Reinforcing steel	BSt 500/550, axis distance from edge 8 cm
Bedding	50 MN/m³ horizontal bedding module

Section

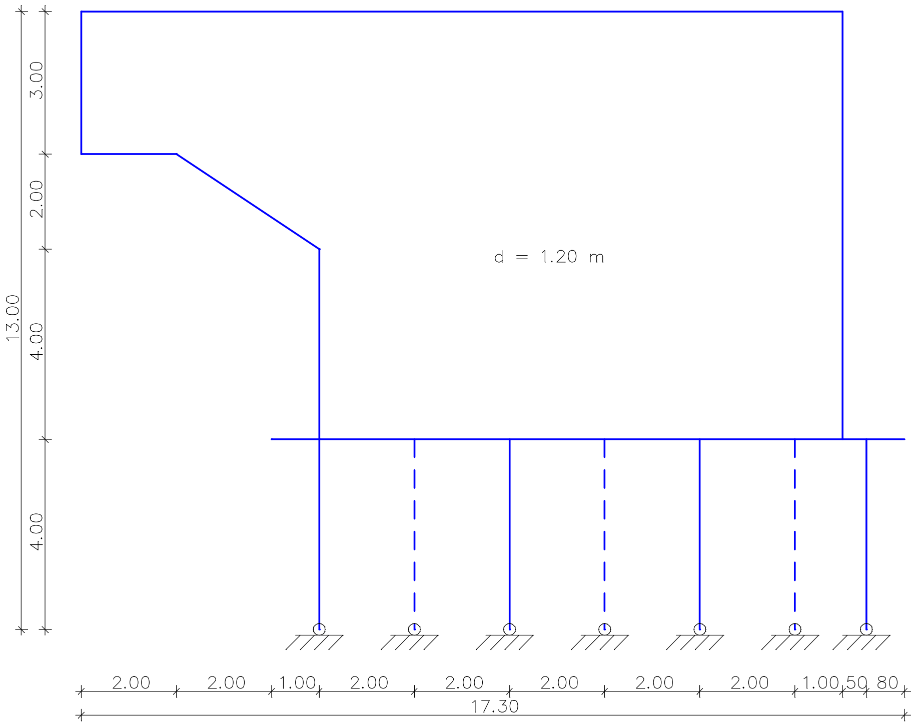
Wing walls	d = 1.2 m (section 1)
Support block	d = 1.5 m (section 2)
Pile head plate	d = 1.2 m (section 3)
Piles	∅ = 1.0 m (section 4)
Exposure class	XC4

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.

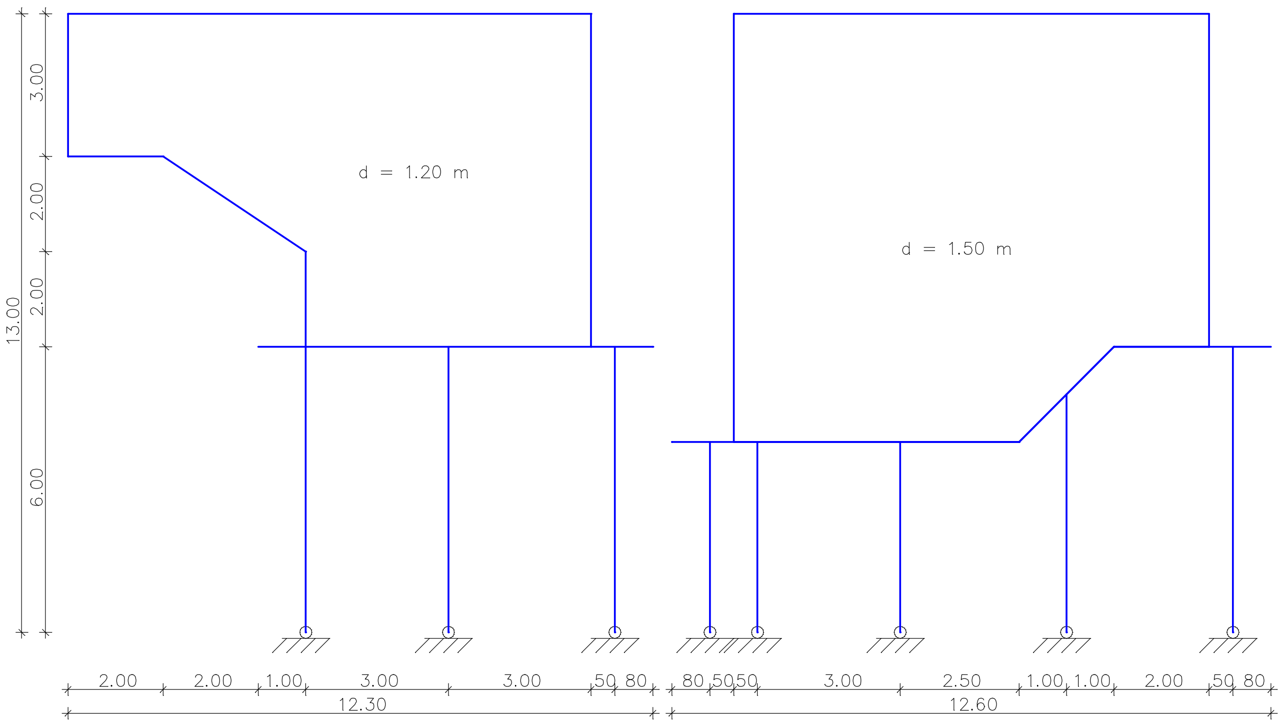
Dimensions



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

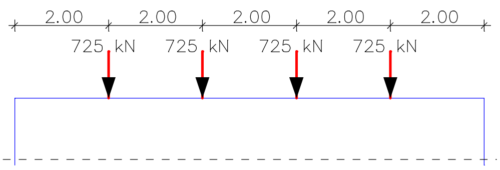
Due to the scope of the project, the actions were limited to the decisive loads.

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

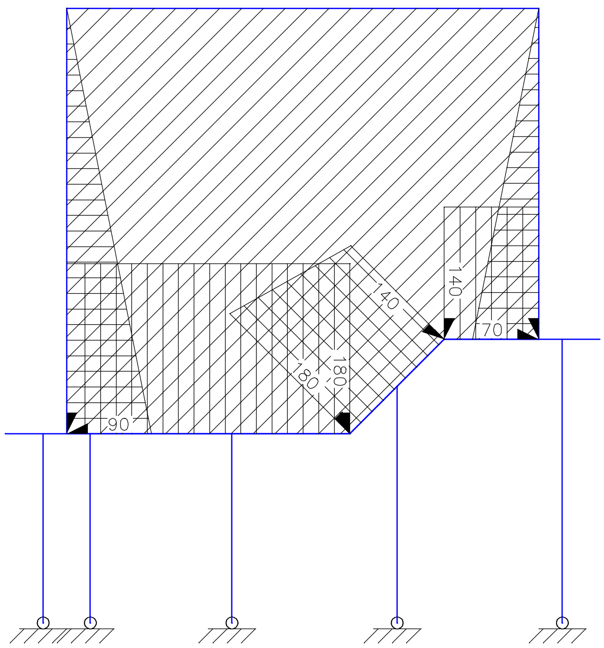
Load data load case 1: G: Dead load

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

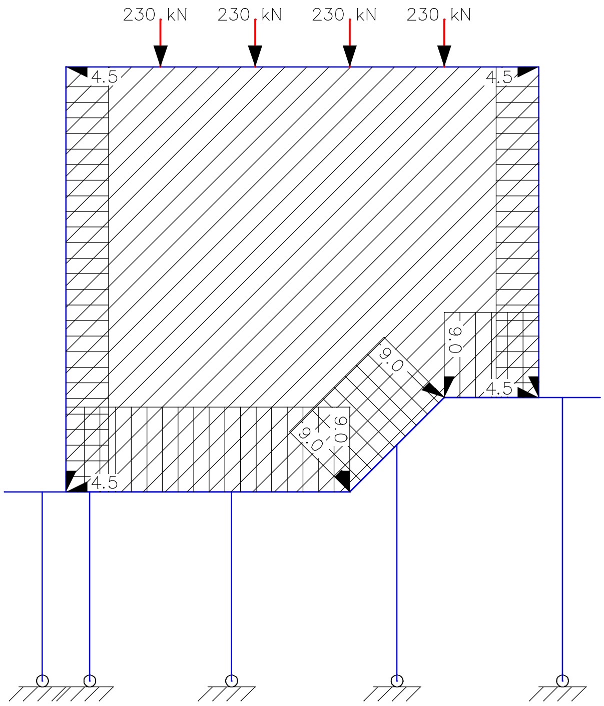
Load case 1, G: Dead load



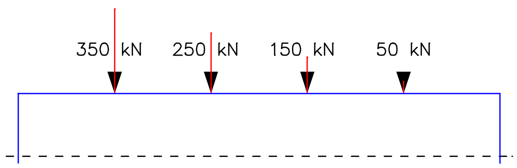
Load case 3, G: Superstructure



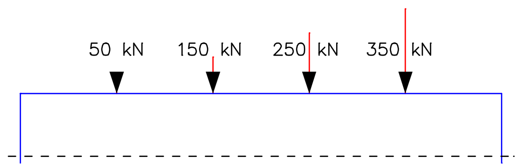
Load case 2, G: Soil pressure



Load case 4, Q: UDL



Load case 5, Q: TS position 1



Load case 6, Q: TS position 2

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	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

Quasi-continuous combination, situations

Variant	State	Actions
1)	Final	G1 + G2

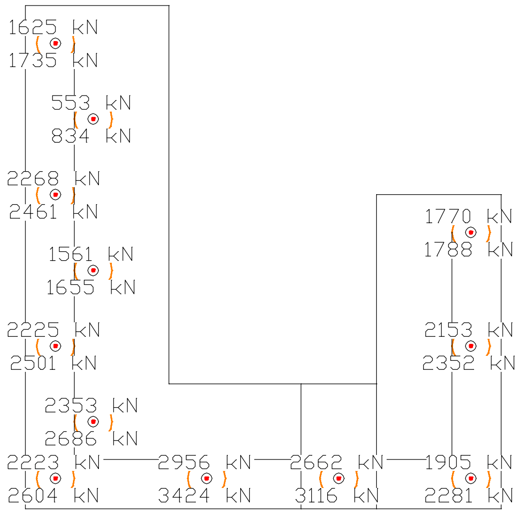
Quasi-continuous combination, safety coefficients

Action	Gamma.sup	Gamma.inf
G1	1	1
G2	1	1
QTS, QUDL	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.
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.
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.	Density [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. S.	Dsn. C.	Asl [cm²] Pic. 6.3 given	Asl [cm²] max	Factor rhow	Factor 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.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] bw	bw.nom	Eff. width bn [m]	kb	Height[m] h	h.nom	Eff.height d [m]	kd	Torsion. section [m] z1	z2	tef	B.
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. [1])

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).
[1] 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]	kd	Torsion section [m] z1	z2	tef	R.
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].
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 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 = Centrical 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 max	Coefficient Xi1	k	kt	Fact.fctm As	wk	Comb. As	wk	Method kc	wk	RS
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

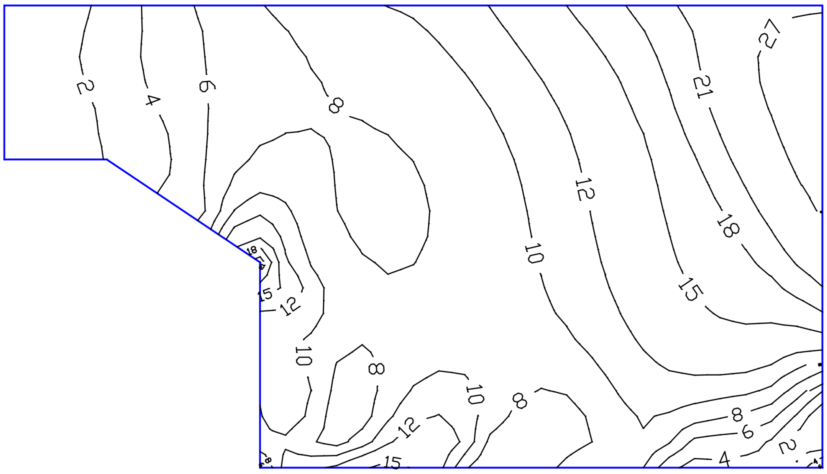
Settings for the check of concrete and steel stresses

```
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.
```

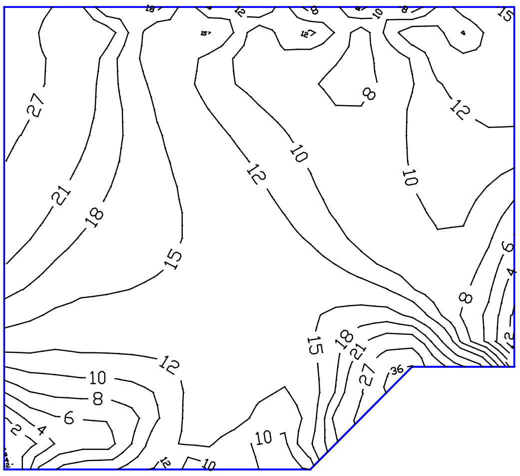
Se.	fck(t) [MN/m²]	per.sigma.c(t) (CC, QC)	per.sigma.c (CC)	(QC)	per.sigma.s (CC)	Decompression Comb.	Stress
1	.	.	0.60 fck	.	0.80 fyk	.	.
2	.	.	0.60 fck	.	0.80 fyk	.	.
4	.	.	0.60 fck	.	0.80 fyk	.	.

Results

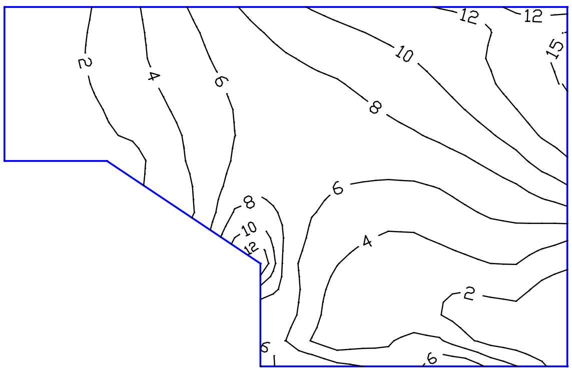
The following figures show the curve of the required longitudinal reinforcement.



Long wing wall

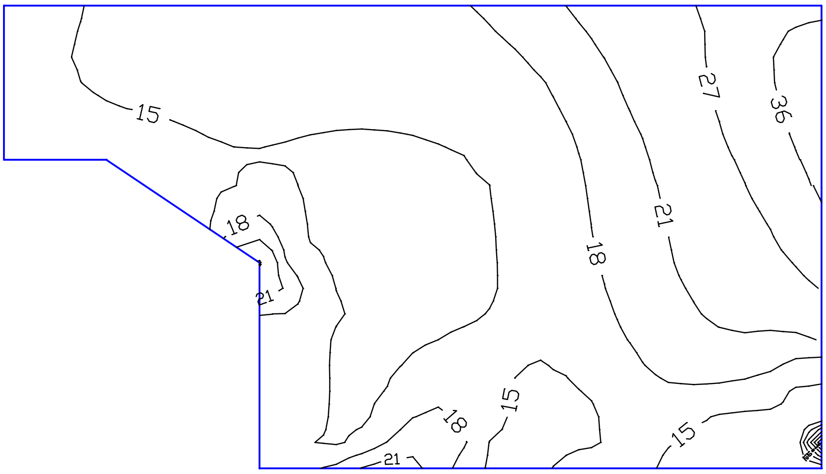


Support block

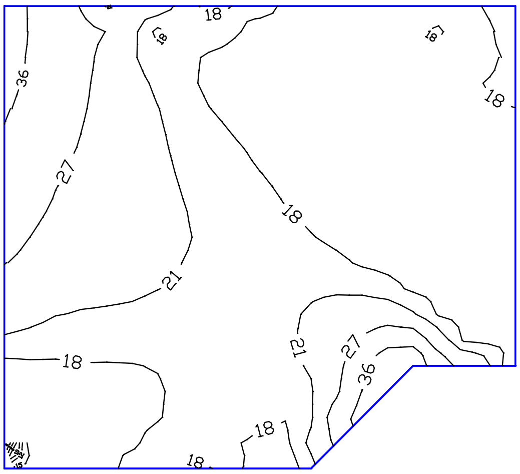


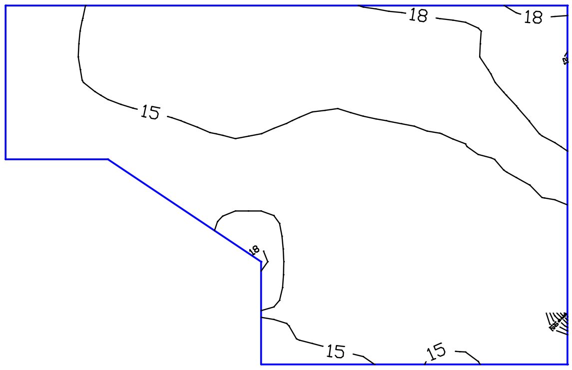
Short wing wall

Required horizontal internal longitudinal reinforcement for the load-bearing capacity [cm²/m]



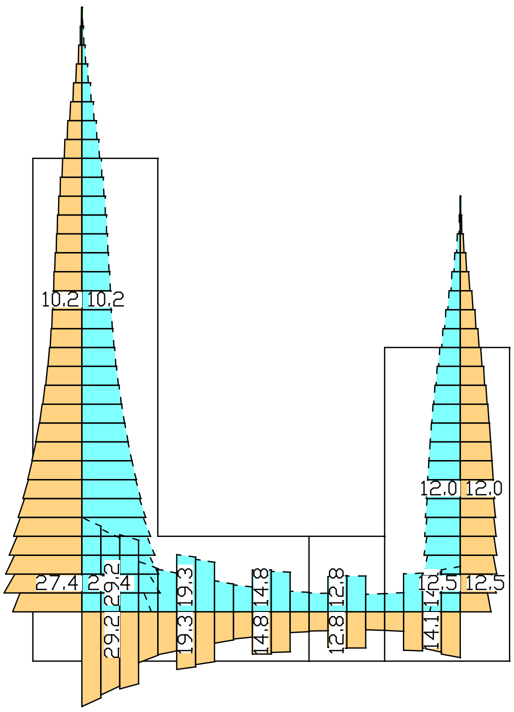
Long wing wall

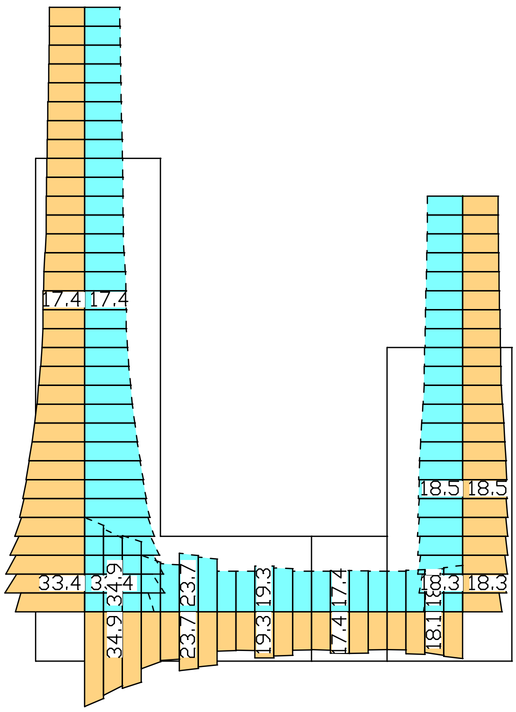




Support block *Short wing wall*

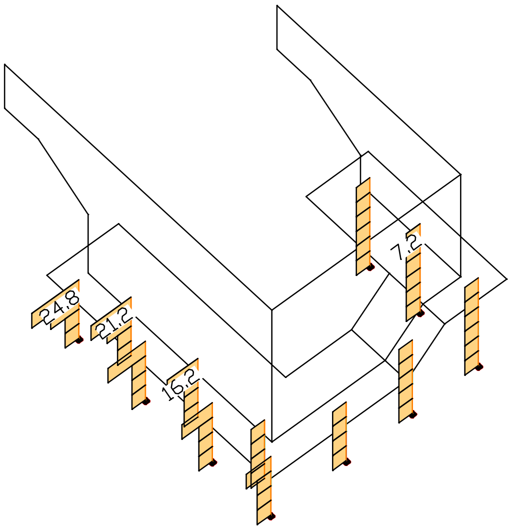
Maximum required horizontal internal longitudinal reinforcement [cm²/m]



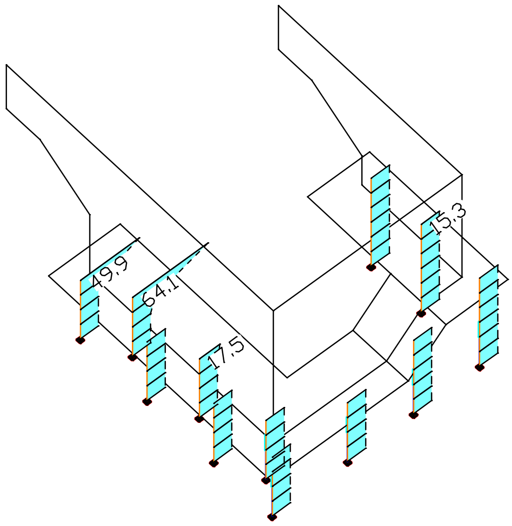


Reinforcement for the load-bearing capacity *Maximum required reinforcement*

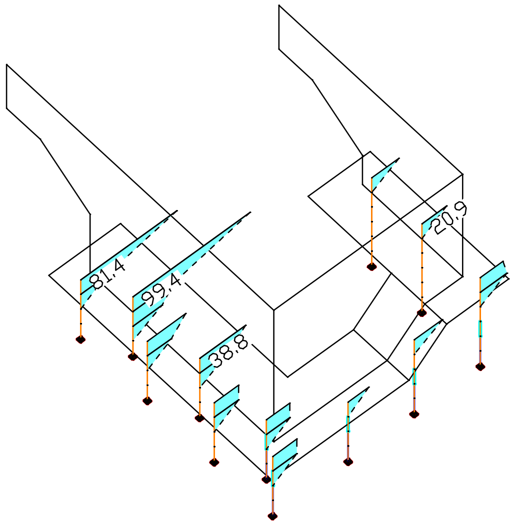
Horizontal longitudinal reinforcement along the upper edge [cm²/m]



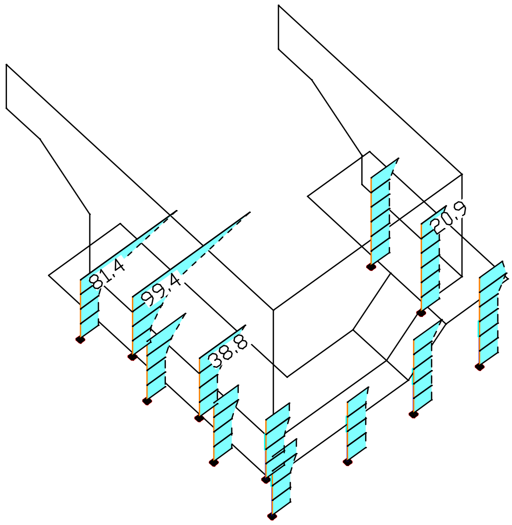
Lateral force reinforcement [cm²/m]



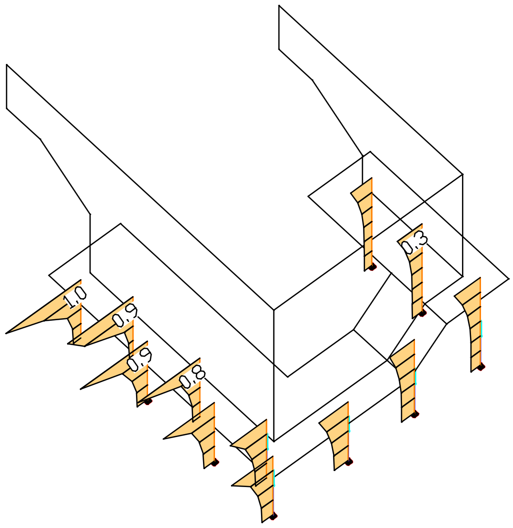
Load-bearing capacity reinforcement [cm²]



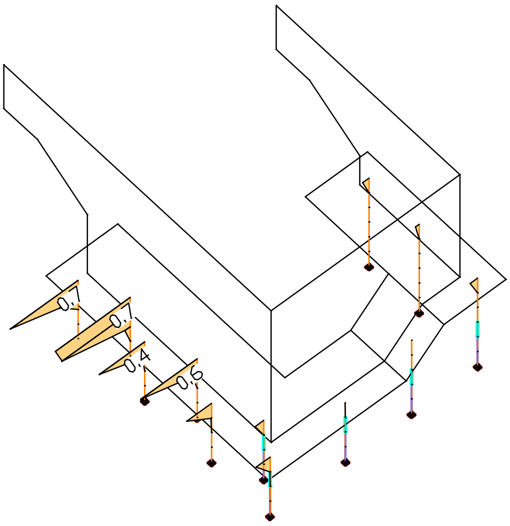
Crack check reinforcement [cm²]



Maximum required reinforcement [cm²]



Concrete compressive stresses $\sigma_c / \sigma_{c.perm}$ *[MN/m²]*



Reinforcing steel stresses $\sigma_s / \sigma_{c.perm}$ *[MN/m²]*

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