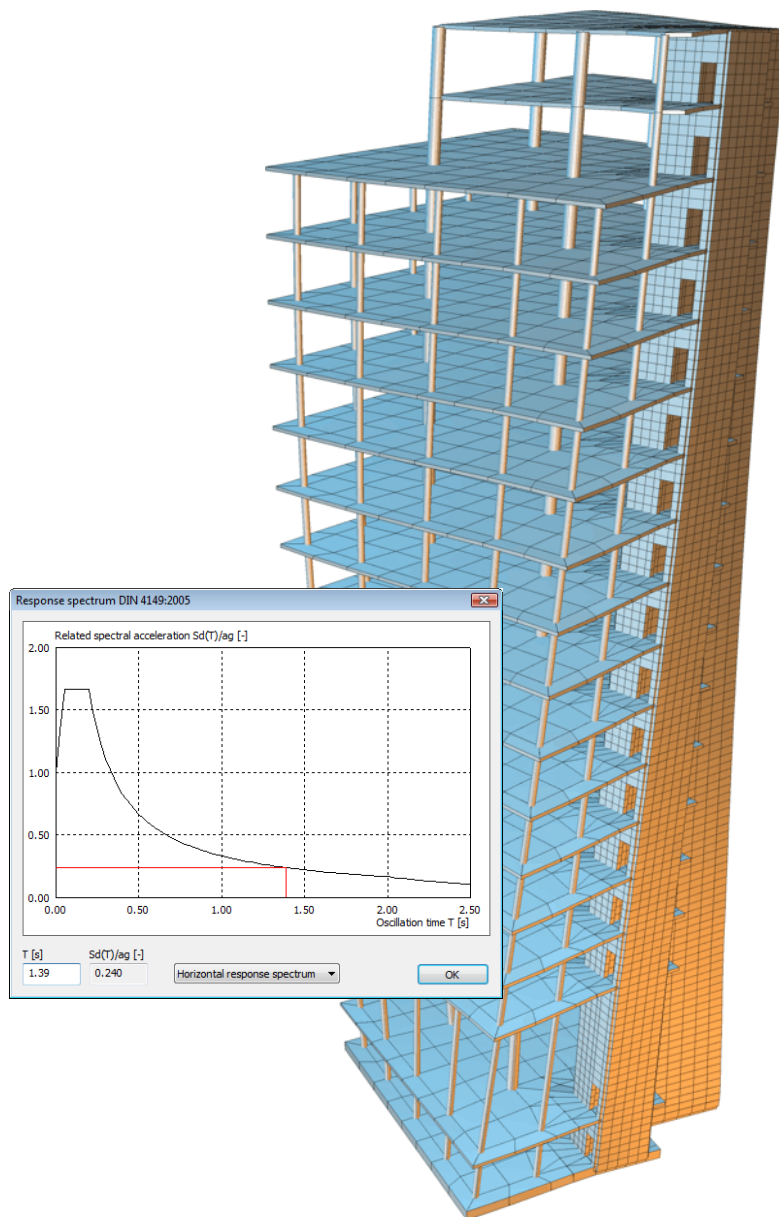


Earthquake Checks according to

EN 1998-1
DIN EN 1998-1
DIN 4149
OENORM B 4015
SIA 261



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Title image: Eigenmode of a high-rise with a response spectrum based on DIN 4149:2005
Courtesy of Nees & Otten, Beratende Ingenieure GmbH, Germany.

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Dynamics

Use this extension module to directly perform dynamic analyses for existing FEM-projects with the following features:

- Dynamic analysis of 3D and 2D beam, cable and shell structures as well as solid models of any geometry or load.
- Determination of eigenvalues and eigenvectors using subspace iteration. User-defined adjustment of the size of the iteration vector space and the iteration accuracy.
- Representation of point and distributed mass distributions. Simultaneous system load by exciters of different frequencies.
- Solution of the equations of motion in modal coordinates with subsequent mode superposition for periodic or instationary load-time functions. You can specify the integration range and the size of the time steps. For each time step, all deformation and load values are available. These can be passed on to post-processing programs for load case combination, design or stress determination.
- Direct integration of the equations of motion for any load-time function. Rayleigh's system damping and point dampers can be represented through the definition of a complete damping matrix. This analysis method can also be used for systems with non-linear properties such as failure of tension supports, non-linear springs, 2nd or 3rd order theory, cable dynamics and dynamic element collapse. This analysis method can also be used for systems with non-linear properties such as failure of tension supports, non-linear springs, 2nd or 3rd order theory, cable dynamics and dynamic element collapse.
- Determination of the system response (U_{re} , U_{im} , $abs(U)$, $abs(DU)$, $abs(DDU)$, *phase*) to periodic excitation for a chosen stress-frequency range or for an excitation spectrum. Soil acceleration as well as direct structure loads can be taken into account.
- Evaluation of the response spectra according to EN 1998-1 (Eurocode 8), DIN EN 1998-1:2021, DIN 4149:1981, DIN 4149:2005, OENORM B 4015 or SIA 261 and superposition of the internal forces and displacements according to statistical laws. Application of user-defined response spectra.
- Determination of static horizontal loads and calculation of the corresponding equivalent mass m^* and of the transformation coefficient Γ for a nonlinear pushover analysis according to EN 1998-1:2013 or DIN EN 1998-1:2021.

The dynamics settings as well as the contents of the log are controlled via the *Settings* option of the *Analysis* menu. Point masses are usually defined in the graphical view. Other inputs related to dynamics are performed in the database.

The sections of the manual relevant for the earthquake check are provided below.

Basics

Equations of Motion

The general equations of motion for a finite element system look as follows in matrix form:

$$M \cdot \ddot{u}(t) + C \cdot \dot{u}(t) + K \cdot u(t) = p(t) \quad (1)$$

inertial forces damping forces elastic forces excitation forces

with

M Mass matrix
 C Damping matrix
 K Stiffness matrix
 $u(t)$ Deformation vector

$\dot{u}(t) = \frac{\partial u}{\partial t}$ Velocity vector

$\ddot{u}(t) = \frac{\partial^2 u}{\partial t^2}$ Acceleration vector

$p(t)$ Load vector

Load vector

This vector is composed of the following components:

$$p(t) = (p - M \ddot{u}_k) f(t)$$

with

p Load vector
 \ddot{u}_k Vector of the node acceleration
 $f(t)$ Time function of the load

All force and displacement values are functions of time t . Equation (1) describes a system of linked linear differential equations which normally must be solved collectively. Ignoring inertial and damping forces and assuming a time-independent load results in the static case of application.

Integration of the Equations of Motion

The equations of motion are integrated based on the general integration scheme defined by Newmark and Wilson. This provides reliable results regardless of the size selected for the time step.

The process is based on the following assumptions:

$$\dot{u}(t + \Delta t) = \dot{u}(t) + \frac{1}{2}[\ddot{u}(t) + \ddot{u}(t + \Delta t)] \Delta t \quad (2)$$

$$u(t + \Delta t) = u(t) + \dot{u}(t)\Delta t + \frac{1}{4}[\ddot{u}(t) + \ddot{u}(t + \Delta t)]\Delta t^2 \quad (3)$$

In addition, the equation of motion (1) is used to calculate displacements, velocities and accelerations at the time $t + \Delta t$. The time step solution is carried out according to the following scheme:

Analysis specifications:

1. Determination of mass, damping and stiffness matrices.
2. Specification of $u(t)$, $\dot{u}(t)$, $\ddot{u}(t)$.
3. Selection of the time step Δt .

Calculation of the integration constants:

4. Calculation of the effective stiffness matrix and triangulation.

Time step integration:

5. Calculation of the effective loads at time $t + \Delta t$.
6. Determination of the displacements at time $t + \Delta t$ by means of reverse resolution.
7. Determination of the velocities and accelerations at time $t + \Delta t$ from equations (2) and (3).

Undamped Natural Frequency Problem

If the damping and excitation forces are ignored in equation (1), the result is:

$$M \ddot{u}(t) + K u(t) = 0 \tag{4}$$

The approach $u(t) = x e^{i\omega t}$ yields the eigenvalue problem

$$(-\omega^2 M + K) x = 0 \tag{5}$$

with ω as the angular frequency

and, using a different notation:

$$K \Phi = M \Phi \lambda \tag{6}$$

In this case:

λ Matrix of the eigenvalues

$$\lambda = \begin{bmatrix} \omega_1^2 & & & \\ & \omega_2^2 & & \\ & & \ddots & \\ & & & \omega_i^2 \end{bmatrix}$$

Φ Matrix of the eigenvectors

The eigenvalue problem at hand is solved using the method of subspace iteration. Using this approach, the subspace of the eigenvectors in question is iterated simultaneously. The eigenvectors have the following transformation properties with respect to the mass and stiffness matrix:

$$\Phi^T K \Phi = \Phi^T M \Phi \lambda \tag{7}$$

$$\Phi^T M \Phi = I \tag{8}$$

In order to solve then eigenvalue problem, a stiffness matrix according to the linear elasticity theory is needed. Therefore nonlinear effects (e.g. compression beams, tension beams, nonlinear springs, loss of tensile support, cable) are not taken into account.

The eigenvalue analysis determines the eigenvalues and eigenvectors for cable structures on the basis of a prescribed cable force state.

A singularity of the Jacobi matrix can occur for dynamic analyses and causes the calculation to terminate.

```
Calculating projection of the stiffness matrix
Calculating projection of the mass matrix
Solving subspace

** Errors **

FEM-F-9915: Error in jacobi-iteration, subspace is singular

Abort of program, 0 Warning(s), 1 Error(s).
```

In most cases, one of the following erroneous inputs is the cause for this:

- Unrealistically high values were input for the element stiffnesses.
- For beams with polygon section no realistic torsional moment of inertia was provided; instead, the default $I_x = 10^{-6} \text{ m}^4$ was used.

In general, be aware that when performing dynamic analyses usable values for the resonance frequency of a structure can only be determined if realistic stiffnesses are used.

Modal Analysis

The equations (7) and (8) can be used to perform a transformation of the equations of motion to the space of eigenvectors. Taking

$$u(t) = \Phi x(t) \quad (9)$$

and performing a left-handed multiplication of the equation of motion (1) with Φ^T yields:

$$\ddot{x}(t) + \Phi^T C \Phi \dot{x}(t) + \lambda x(t) = \Phi^T p(t) \quad (10)$$

This leads to a system of equations (10) in modal coordinates whose terms are unlinked, with the exception of the damping term.

If the following approach is used for the damping matrix

$$C = \alpha M + \beta K, \quad (11)$$

then, using the equations (7) and (8), the equation of motion is unlinked with regard to:

$$\ddot{x}_i(t) + 2\omega_i \xi_i \dot{x}_i(t) + \omega_i^2 x_i(t) = r_i(t) \quad (12)$$

with

ω Angular frequency

$\xi_i = D$ or $(\alpha + \beta \omega_i^2) / 2\omega_i$ Modal damping measure

D Lehr's damping measure

$r_i(t) = \phi_i^T p(t)$ Modal load vector

ϕ_i i-th eigenvector

i Index of the eigenvalues, eigenvectors

The integration of the equations (12) can be carried out either analytically or with a numerical integration scheme.

The approach (11) is generally applicable for structures with natural damping only. But it is not adequate for realistically modeling point dampers.

An essential advantage of the modal analysis is that in general the complete eigenvector space does not need to be taken into account. Instead, due to the natural damping that is always present for buildings, only the first n eigenmodes ($n \ll n_u$) have a significant impact on the structure response. This greatly reduces the complexity of the analysis when compared with direct integration of equations of motion.

System Excitation Caused by Soil Acceleration

A load resulting from soil acceleration can be replaced with a load of the unsupported structure nodes by transforming the reference system.

The effective structure load is described by the following load vector:

$$p(t)_{\text{eff}} = p_{\text{eff}} f(t) = -M (T_s \ddot{u}_b) f(t) = -M \ddot{u}_k f(t) \quad (17)$$

with

$f(t)$ Excitation as a function of time

\ddot{u}_b Vector of the soil acceleration

T_s Transformation matrix

\ddot{u}_k Vector of the node acceleration

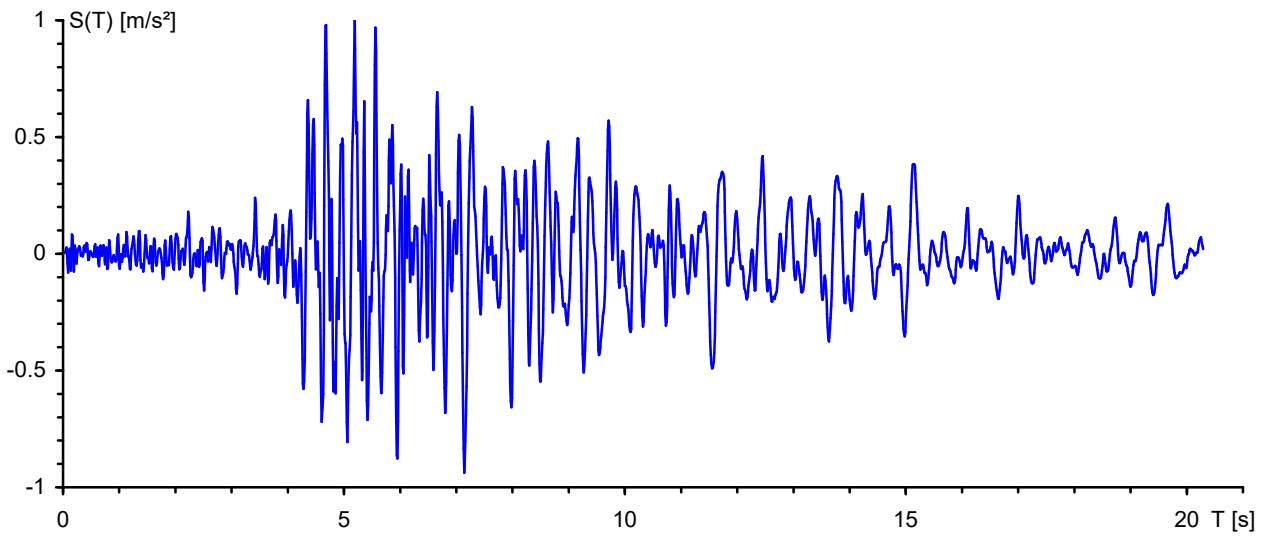
This relationship ignores damping forces resulting from the soil acceleration as well as the link between the supported and free elements of the mass matrix. These influences are, however, of low significance when compared with the inertial forces.

The transformation matrix T_s expresses the static influence of the column displacement on the other system nodes. For structures supported in a statically determined fashion or for stiff soil layers, this is described by strictly kinematic relationships.

The deformations and velocities are calculated in the accompanying coordinate system. The accelerations relate to absolute coordinates.

Response Spectrum Method

The response spectrum method is used to calculate the deformations and internal forces of structures stressed by earthquakes. In this case, instead of analyzing the support point excitation of the system, the maximum response of the significant eigenmodes is calculated with the help of an acceleration response spectrum. This can be taken from a design standard or can be determined through integration of the equation of motion for special cases.



Soil acceleration from earthquakes standardized to $\ddot{u}_b = 1.0 \text{ m/s}^2$

The modified equation of motion (12) in modal coordinates:

$$\ddot{x}_i(t) + 2\omega_i \xi_i \dot{x}_i(t) + \omega_i^2 x_i(t) = -\varphi_i^T M^{-1} t_s \ddot{u}_b(t) = -\beta_i \ddot{u}_b(t) \quad (18)$$

The solution of which is derived by solving:

$$x_i(t) = \beta_i S_d^*(t) \quad (19)$$

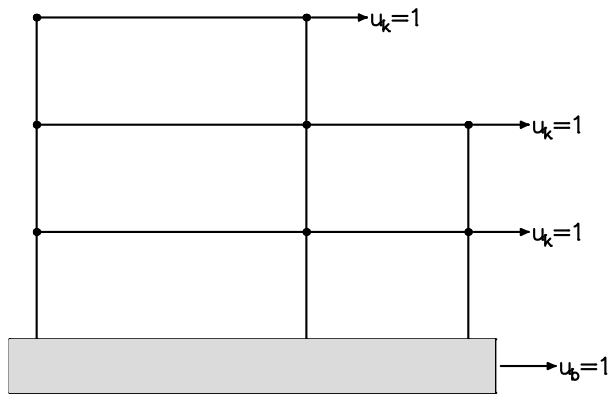
by using the DUHAMEL integral

$$S_d^*(t) = \frac{1}{\omega_i \sqrt{1-\xi_i^2}} \int_0^t \ddot{u}_b(\tau) e^{-\xi_i \omega_i (t-\tau)} \sin[\omega_i \sqrt{1-\xi_i^2} (t-\tau)] d\tau \quad (20)$$

In this case

- β_i Fraction factor
- ω_i Angular frequency of the i -th eigenmode
- ξ_i Lehr's damping measure of the i -th eigenmode
- $\ddot{u}_b(t)$ Soil acceleration from earthquake

The vector t_s expresses the proportionate node displacements that result from applying a unit deformation of the supported system nodes. The program assumes that these are the kinematic displacements of the entire system (see 'System excitation from soil acceleration').

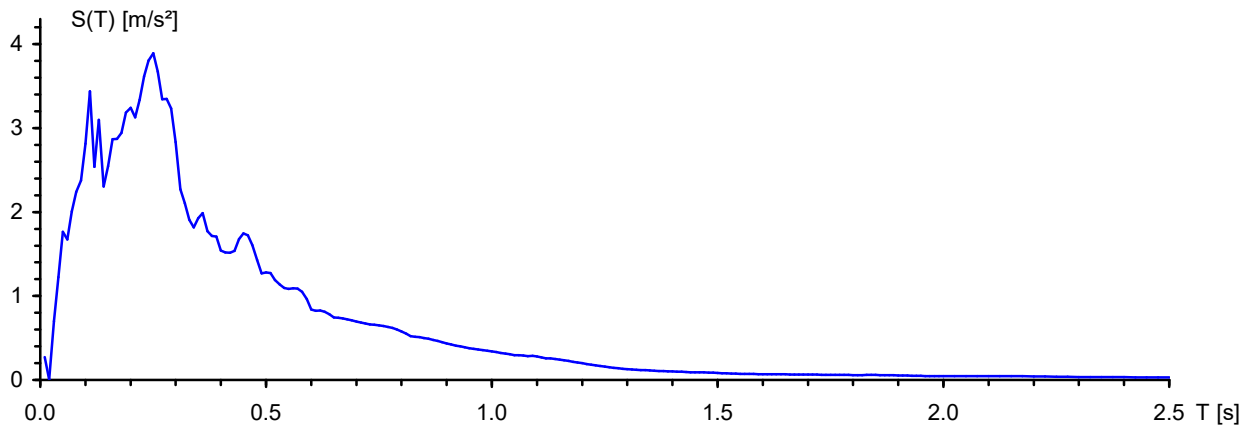


The ordinates of the displacement, velocity and acceleration response spectra result from the maximum ordinates of $S_d^*(t)$ for different oscillation times.

$$S_d = S_v(1/\omega_i) = S_a(1/\omega_i^2) = \max | S_d^*(t) | \quad (21)$$

These are generally standardized and smoothed (see also the response spectra of the various standards).

The following illustration shows an unsmoothed, elastic acceleration response spectrum calculated on the basis of the soil acceleration shown above. A damping of 5% was used.



The static pseudo loads of the corresponding eigenmodes required for further calculation are determined from the acceleration response spectrum in the following manner:

$$H_{E,i} = -\beta_i S_{a,i} M \varphi_i \quad (22)$$

Deformations and internal forces result for each eigenmode that can optionally be statistically superposed before being used further.

An important parameter for evaluating whether sufficient modal components have been considered is the so-called effective modal mass. It results to the following under consideration of (8):

$$M_{\text{eff}} = \sum_i \beta_i^2 \quad (23)$$

If all eigenmodes are considered, this corresponds to the total mass of the analyzed structure. The general requirement is that the effective modal mass should amount to at least 90% of the total mass (see SIA 261, EC 8, etc.). This requirement is already fulfilled for conventional buildings by considering the initial significant eigenmodes.

For structures with many torsion-flexible beams and rotatory mass distribution or quasi-stiff partial systems, meeting this requirement can be problematic due to the fact that a large number of eigenmodes must be calculated. Therefore, you should consider changing the calculation model or the selected mass distribution in this case.

The additional masses which are not included in the net mass of the structure as well as the masses due to traffic which are to be observed according to the standard, should be specified by the user using appropriate *Masses from loads* or *Point masses*.

ZPA method

The ZPA method (Zero Period Acceleration) is one way of taking missing effective masses into account. It is also called static correction. Here additional static load cases are calculated, which capture the missing effective masses with a spectral value to be specified. Assuming that the higher-order modes that are not detected have no dynamic amplification, this method can be useful.

In general, the static correction load vector p_c can be calculated as follows for each direction of acceleration:

$$p_c = p - \sum_i r_i (M \cdot \varphi_i) \tag{24}$$

With the modal load of r_i according to equation 12 and the approach from equation 18 for the earthquake load follows:

$$r_i(t) = \varphi_i^T \cdot p(t) = -\varphi_i^T \cdot M \cdot \ddot{u}_b(t) \cdot t_s = -\beta_i \cdot \ddot{u}_b(t)$$

The acceleration $\ddot{u}_b(t)$ is defined due to a selected spectral value $S_{a,ZPA}$ (usually $S_a(T=0)$) of the response spectrum. From equation 24 results:

$$p_c = -M \cdot \ddot{u}_b \cdot t_s + \sum_i \beta_i \cdot \ddot{u}_b \cdot (M \cdot \varphi_i) = S_{a,ZPA} \cdot M \cdot \left(\sum_i (\beta_i \cdot \varphi_i) - t_s \right) = S_{a,ZPA} \cdot M \cdot \psi$$

The expression ψ in the equation can be seen as a kind of eigenmode of the mass portion, which is not yet taken into account. The effective masses of the ZPA portion of all k nodes in the mass matrix are:

$$M_{eff,ZPA} = \sum_k (M_k \cdot \psi_k)$$

The results of the additional static load cases (correction loads) are also treated like other eigenmodes in the statistical superposition. The SRSS method is used as the combination rule, because there is no resonant frequency for the ZPA portion.

Response Spectrum EN 1998-1:2013 (Design Spectrum)

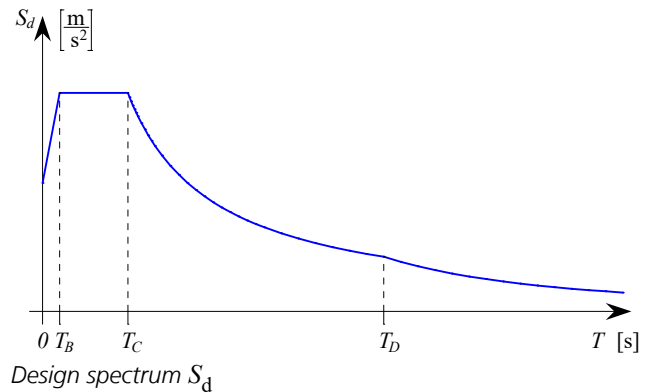
The ordinate values S_d required for the design spectrum according to the basic standard EN 1998-1 (EC8) result from the following equations:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases}$$

$$T_D \leq T: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases}$$



The following parameters are required to determine the design spectrum:

- γ_I Importance factor according to Section 4.2.5.
- a_g Design value of the soil acceleration for subsoil class A with $a_g = \gamma_I \cdot a_{gR}$. The reference peak soil acceleration a_{gR} is to be specified by the user in the setting dialog (a_x, a_y, a_z). The factor for the vertical soil acceleration must be considered by the user according to Table 3.4. For spectra of type 1 the vertical soil acceleration is to be reduced to 90 % and for spectra of type 2 to 45 % of the horizontal soil acceleration.
- S, T_B, T_C, T_D Parameters according to Section 3.2.2. The design spectrum for the vertical earthquake action is internally calculated according to Table 3.4 with the parameters $S=1.0, T_B=0.05, T_C=0.15$ and $T_D=1.0$.
- q Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to the standard. For the vertical earthquake action, q is set to 1.0 in the calculation.
- β Coefficient for the lower limit value for the horizontal design spectrum with $\beta = 0.2$.

Statistical Combination Method

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. If the following requirement is fulfilled, then according to the standard the SRSS method is selected as superposition specification during automatic assignment, otherwise the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

Response Spectrum DIN EN 1998-1:2021 (Design Spectrum)

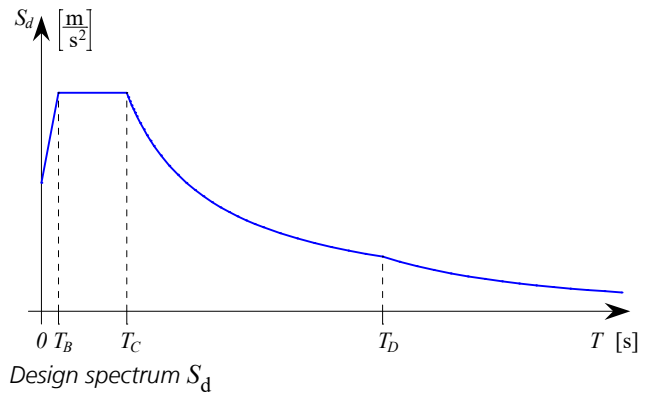
The ordinate values S_d required for the design spectrum according to DIN EN 1998-1 result from the following equations:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases}$$

$$T_D \leq T: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases}$$



The following parameters are required to determine the design spectrum:

γ_I Importance factor according to Table NA.5.

a_g Design value of the soil acceleration for subsoil circumstance A-R with $a_g = \gamma_I \cdot a_{gR}$ and $a_{gR} = S_{aP,R} / 2.5$ with $S_{aP,R}$ according to Figure NA.1 of the standard. The reference peak soil acceleration a_{gR} is to be specified by the user in the setting dialog (a_x, a_y, a_z). The factor for the vertical soil acceleration must be considered by the user according to Table NA.3. According to this the vertical soil acceleration is to be reduced to 70 % of the horizontal soil acceleration.

S, T_B, T_C, T_D Parameters according to NDP by 3.2.2.1(4), 3.2.2.2(2)P. Any deviations according to NA.E.1 and NA.E.2 for higher return periods shall be indicated by the user accordingly. The design spectrum for the vertical earthquake action is internally calculated according to Table NA.3 with the parameters $S=1.0, T_B=0.05, T_C=0.20$ and $T_D=1.20$.

q Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to the standard. For the vertical earthquake action, q is set to 1.0 in the calculation.

β Coefficient for the lower limit value for the horizontal design spectrum with $\beta = 0.0$.

Statistical Combination Method

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. If the following requirement is fulfilled, then according to the standard the SRSS method is selected as superposition specification during automatic assignment, otherwise the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

Response Spectrum DIN 4149:2005

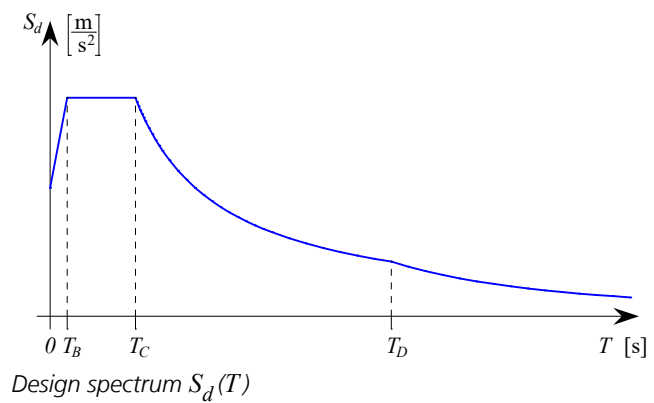
The ordinates of the design spectrum $S_d(T)$ are determined by the following expressions:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \left[1 + \frac{T}{T_B} \left(\frac{\beta_0}{q} - 1 \right) \right]$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \cdot \frac{\beta_0}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \cdot \frac{\beta_0}{q} \cdot \frac{T_C}{T}$$

$$T_D \leq T: \quad S_d(T) = a_g \cdot \gamma_I \cdot S \cdot \frac{\beta_0}{q} \cdot \frac{T_C T_D}{T^2}$$



The following parameters are required to determine the design spectrum:

- T Vibration period ($T = 1 / f$).
- a_g Design value of the soil acceleration as a function of the earthquake zone according to DIN 4149, Table 2. It is to be specified by the user in the setting dialog (a_x, a_y, a_z). The factor of 0.7 for the vertical soil acceleration according to Section 5.4.1 (4) must be considered by the user.
- γ_I Importance coefficient according to Table 3.
- β_0 Amplification factor of the spectral acceleration with the reference value $\beta_0 = 2.5$ for 5% viscous damping.
- S, T_B, T_C, T_D Parameters according to Table 4 or 5.
- q Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to Sections 8 through 12. For the vertical earthquake action, q is set to 1.0 in the calculation according to Chapter 8.3.3.2.2.

Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. If the following requirement is fulfilled for all of the eigenmodes to be analyzed, then, according to the standard the SRSS method is selected as superposition specification during automatic assignment; otherwise, the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

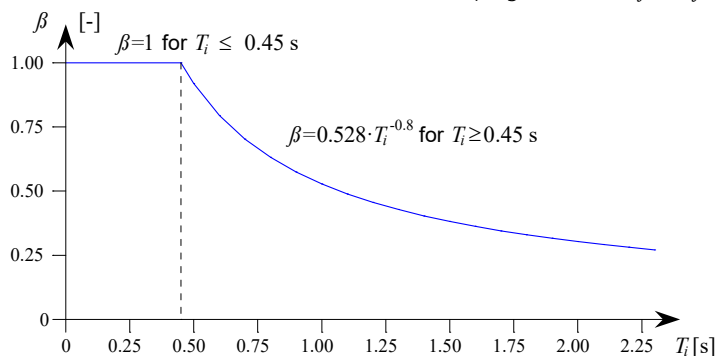
Response Spectrum DIN 4149:1981

The ordinate values of the design spectrum $S_{d,i}$ of the i -th eigenmode required for the response spectrum according to DIN 4149:1981 result from the following equation:

$$S_{d,i} = \beta a_0 \chi \alpha \tag{27}$$

with

$\beta = \beta(T_i)$ Coefficient of the standardized response spectrum (DIN 4149:1981, Part 1, Figure 2). An possible increase of the coefficient due to insufficient damping ($D < 0.05$: $\beta^* = \beta \cdot 1.3$) can be determined, if necessary.



T_i	Vibration period of the i -th resonant frequency.
a_0	Soil acceleration [m/s^2] as a function of the earthquake zone (Section 7.2.1). It is to be specified by the user in the setting dialog (a_x, a_y, a_z). The factor of 0.5 for the vertical soil acceleration must be considered by the user (Section 7.3).
χ	Factor for the subsoil (Section 7.2.2).
α	Reduction factor as a function of the subsoil class and the earthquake zone (Section 7.2.3).

Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method. DIN 4149:1981 stipulates the SRSS method.

Response Spectrum OENORM B 4015

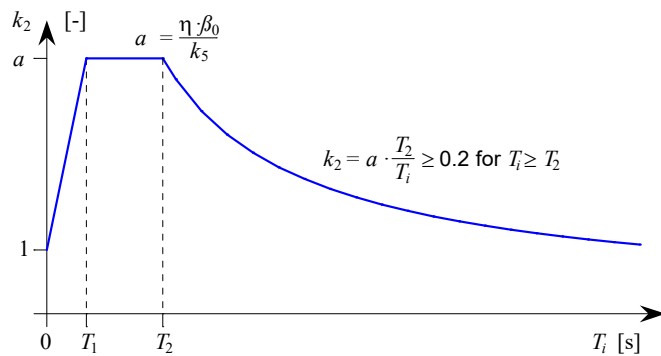
The seismic coefficients $k_{E,i}$ of the i -th eigenmode required according to OENORM B 4015 result from the following equation:

$$k_{E,i} = \varepsilon \cdot k_1 \cdot k_2 \cdot k_{3,i} \cdot k_4$$

with

ε	Earthquake coefficient as a function of the soil acceleration a_0 and the gravitational acceleration g with $\varepsilon = a_0/g$. Make sure to use a factor of 0.67 for the effective vertical soil acceleration.
k_1	Foundation coefficient (Table 9).
k_2	Building reaction coefficient.
$k_{3,i}$	Distribution coefficient.
k_4	Factor for taking the building type into account (in general 1, for bridges according to Section 10.2).

For determining the building reaction coefficient k_2 the following parameters are required:



T_i	Vibration period of the i -th resonant frequency.
T_1, T_2	Reaction parameter [s] (Table 9).
η	Factor for determining the building damping with $\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$
ξ	Lehr's damping measure.
β_0	Reaction parameter as a function of the subsoil (Table 9).
k_5	Building ductility (ratio of the limit deformation and the elastic deformation, Table 12). If the value is greater than 1.5, η is assumed to be 1 or greater.

Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method: If the following condition is fulfilled, according to OENORM B 4015 the SRSS method is selected as the superposition specification during automatic assignment, otherwise the CQC method is selected.

$$f_i / f_j < 1/(1 + 10\xi) \text{ with } f_i \leq f_j \text{ and } j = 1 \text{ to } n$$

Response Spectrum SIA 261

The ordinate values S_d required for the response spectrum according to SIA 261 result from the following equations:

$$0 \leq T \leq T_B: \quad S_d = \gamma_F \frac{a_{gd}}{g} S \left[0.67 + \left(\frac{2.5}{q} - 0.67 \right) \frac{T}{T_B} \right]$$

$$T_B \leq T \leq T_C: \quad S_d = 2.5 \gamma_F \frac{a_{gd}}{g} \frac{S}{q}$$

$$T_C \leq T \leq T_D: \quad S_d = 2.5 \gamma_F \frac{a_{gd}}{g} S \frac{T_C}{T q}$$

$$T_D \leq T: \quad S_d = 2.5 \gamma_F \frac{a_{gd}}{g} S \frac{T_C T_D}{T^2 q} \geq 0.1 \gamma_F \frac{a_{gd}}{g}$$

Design spectrum S_d

The following parameters are required to determine the design spectrum:

- γ_f Importance factor according to Table 25.
- a_{gd} Soil acceleration according to Section 16.2.1.2 as a function of the earthquake zone. The factor of 0.7 for the vertical soil acceleration according to Section 16.2.4.2 must be considered by the user.
- g Gravitational acceleration.
- S, T_B, T_C, T_D Reaction parameter according to Table 24.
- q Behavior coefficient that takes into account the ability of a structure to resist the earthquake action in the nonlinear area with post-limit stiffness. The behavior coefficient depends on the plastic deformation and energy dissipation capacity of the structure and is to be determined according to the standards SIA 262 to 267. For the vertical earthquake action, q is set to 1.5 in the calculation.

Statistical Combination Method

As the response spectrum only records maximum amplitudes, a superposition of the displacements and internal forces is carried out according to the SRSS method.

Horizontal Elastic Response Spectrum EN 1998-1:2013

The ordinate values S_e required for the horizontal elastic response spectrum according to the basic standard EN 1998-1 (EC8) result from the following equations:

$$0 \leq T \leq T_B: \quad S_e(T) = a_g \cdot S \left[1 + \frac{T}{T_B} (\eta \cdot 2.5 - 1) \right]$$

$$T_B \leq T \leq T_C: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$$

$$T_C \leq T \leq T_D: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C}{T} \right]$$

$$T_D \leq T \leq 4 s: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C T_D}{T^2} \right]$$

Elastic response spectrum S_e

The following parameters are required to determine the horizontal elastic response spectrum needed for the pushover analysis:

γ_I	Importance factor according to Section 4.2.5.
a_g	Design value of the soil acceleration for subsoil class A with $a_g = \gamma_I \cdot a_{gR}$. The reference peak soil acceleration a_{gR} is to be specified by the user in the setting dialog (a_x, a_y).
S, T_B, T_C, T_D	Reaction parameter according to Section 3.2.2.
η	Damping correction coefficient with the reference value $\eta = 1$ for 5 % viscous damping.
	$\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$
ξ	Lehr's damping measure.

Horizontal Elastic Response Spectrum DIN EN 1998-1:2021

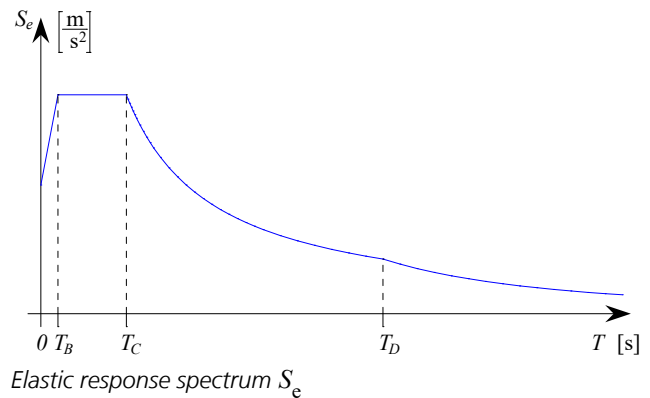
The ordinate values S_e required for the horizontal elastic response spectrum according to DIN EN 1998-1 result from the following equations:

$$0 \leq T \leq T_B: \quad S_e(T) = a_g \cdot S \left[1 + \frac{T}{T_B} (\eta \cdot 2.5 - 1) \right]$$

$$T_B \leq T \leq T_C: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$$

$$T_C \leq T \leq T_D: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C}{T} \right]$$

$$T_D \leq T \leq 4 \text{ s}: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C T_D}{T^2} \right]$$

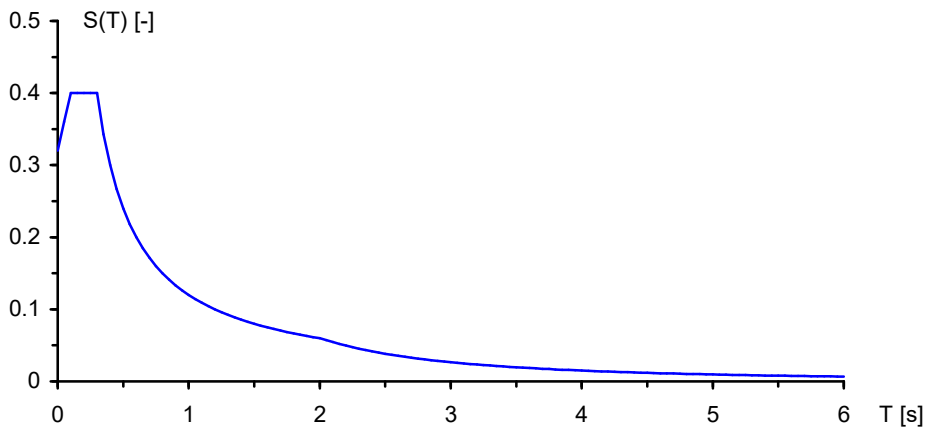


The following parameters are required to determine the horizontal elastic response spectrum needed for the pushover analysis:

γ_I	Importance factor according to Table NA.5.
a_g	Design value of the soil acceleration for subsoil class A with $a_g = \gamma_I \cdot a_{gR}$ and $a_{gR} = S_{aP,R} / 2.5$ with $S_{aP,R}$ according to Figure NA.1 of the standard. The reference peak soil acceleration a_{gR} is to be specified by the user in the setting dialog (a_x, a_y).
S, T_B, T_C, T_D	Parameters according to NDP by 3.2.2.1(4), 3.2.2.2(2)P. Any deviations according to NA.E.1 and NA.E.2 for higher return periods shall be indicated by the user accordingly.
η	Damping correction coefficient with the reference value $\eta = 1$ for 5 % viscous damping.
	$\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$
ξ	Lehr's damping measure.

Alternative Response Spectrum

The curve of the alternative response spectrum can be defined freely. The alternative acceleration response spectrums ($S_{a,x}$, $S_{a,y}$, $S_{a,z}$) are the arithmetic product of the specified soil accelerations (a_x , a_y , a_z) and the user-defined standardized response spectrums (*standard* or *vertical*).



Example of an standardized alternative response spectrum

Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to the SRSS or CQC method.

Statistical Combination Methods

As the response spectrum only records maximum amplitudes, the superposition of the displacements and internal forces is carried out according to one of the following two statistical laws:

$$E = \sqrt{\sum_{i=1}^n E_i^2} \quad \text{(SRSS method)}$$

$$E = \sqrt{\sum_{i=1}^n \sum_{j=1}^n E_i \cdot \rho_{ij} \cdot E_j} \quad \text{(CQC method)}$$

with

E_i Displacements and internal forces of the i -th eigenmodes.

n Number of eigenmodes.

$$\rho_{ij} = \frac{8\xi^2 \cdot (1+r) \cdot r^{1.5}}{(1-r^2)^2 + 4\xi^2 \cdot r(1+r)^2}$$

$r = f_i / f_j = T_j / T_i$ Frequency ratio.

ξ Lehr's damping measure.

The CQC method is only applicable if $\xi > 0$.

In the standard case the superposition specification is chosen automatically depending on the standard. If the following requirement, e.g. according to DIN 4149, is fulfilled for all of the eigenmodes to be analyzed, then, according to the standard the SRSS method is selected as superposition specification during automatic assignment; otherwise, the CQC method is selected.

$$T_i \leq 0.9 T_j \text{ with } T_i \leq T_j$$

Given a symmetrical 3D structure, two congruent oscillation times often result ($T_i = T_j$). Therefore, the automatic selection of the superposition specification always chooses the CQC method for such systems. If you wish to use a different method, you must select it manually.

Earthquake Action

Within the checks in the ultimate limit state of the different standards an earthquake action (A_{Ed}) is to be taken into account.

$$\sum_{j \geq 1} G_{k,j} \cdot P \cdot A_{Ed} \cdot \sum_{i \geq 1} \Psi_{2,i} \cdot Q_{k,i}$$

In general, to check the horizontal earthquake action two directions of the building section that are orthogonal to one another are analyzed.

These are to be combined in the form:

$$(E_{Edx} \oplus 0.3 \cdot E_{Edy}) \text{ or } (0.3 \cdot E_{Edx} \oplus E_{Edy})$$

Here the internal forces E_{Edx} or E_{Edy} can be determined through statistical superposition. The two following combinations illustrate this procedure.

Load case combination 1, Edx "+" 0.3 Edy

1. Variable exclusive action	Factor
Response spectrum overlaid (x)	1.000
Response spectrum overlaid (x)	-1.000
2. Variable exclusive action	Factor
Response spectrum overlaid (y)	0.300
Response spectrum overlaid (y)	-0.300

Load case combination 2, 0.3 Edx "+" Edy

1. Variable exclusive action	Factor
Response spectrum overlaid (x)	-0.300
Response spectrum overlaid (x)	0.300
2. Variable exclusive action	Factor
Response spectrum overlaid (y)	-1.000
Response spectrum overlaid (y)	1.000

The random torsion actions according to DIN 4149:2005, Section 6.2.3.3 or EN 1998-1, Section 4.3.2 are not included in the static pseudo loads that are calculated by the program. If necessary, you can specify them by providing additional load cases for the earthquake actions.

Characteristic value of the sensitivity to story drift

The analysis of the load-bearing capacity according to DIN 4149:2005, Section 7.2.2 or EN 1998-1, Section 4.4.2.2, involves checking the characteristic value of the sensitivity to story drift (see example *Earthquake*). If this value exceeds 0.1, the reactions from the earthquake action should be increased, if necessary. Alternatively, you can carry out a calculation according to the second-order theory. The value of θ may not exceed 0.3. This value cannot be automatically calculated by the program.

Pushover Analysis EN 1998-1:2013 and DIN EN 1998-1:2021

The pushover analysis is a nonlinear method for the verification of earthquake safety of structures and can be used alternatively to the response spectrum method. Here, the displacement of the control node d_n is compared with the total earthquake force at foundation level F_b (capacity curve), which occurs under constant vertical loads and monotonically increasing horizontal loads. The method is suitable for largely regular structures whose basic eigenmode in the direction considered is not determined by torsional vibrations (Section 4.3.3.4.2.7, EN 1998-1). The procedure is described in Section 4.3.3.4.2 and Annex B, EN 1998-1. The method explained there is based on the N2 method developed by Fajfar et al.

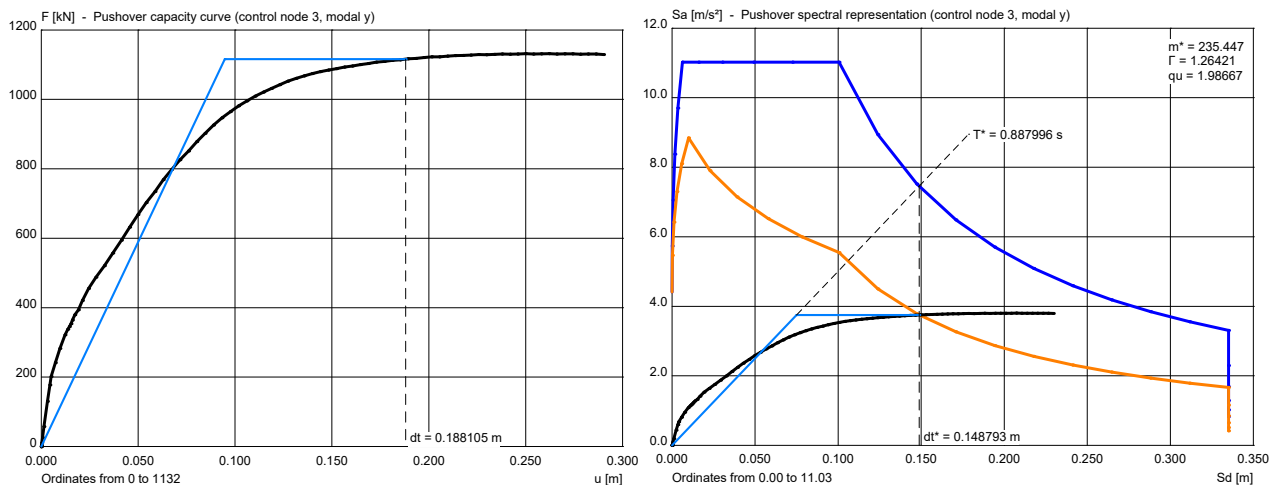
Essentially, the implemented method is divided into two calculation steps:

1. Dynamics:

Selection of the control node according to Section 4.3.3.4.2.3, definition of the elastic response spectrum, input of the soil accelerations, determination of the dynamic eigenmodes ϕ_i and their normalization (displacement of the control node $\phi_n = 1$), provision of selected horizontal load distributions as well as the calculation of the associated equivalent mass m^* , the transformation coefficient Γ and the ductility coefficient q_u . The basic modal form considered for the modal horizontal load distribution corresponds to the eigenmode with the largest effective modal mass in the desired direction.
2. Nonlinear structural analysis:

Definition of vertical and horizontal load cases with associated load time function, input of load groups for the distributions (*modal $\pm x$, constant $\pm x$* and possibly *modal $\pm y, constant \pm y$*), nonlinear system analysis with the Newton time step or the arc length method.

After the two calculation steps, the capacity curve in the multi-mass system and in the spectral representation are available. Their linearization is carried out iteratively in accordance with Section B.5, EN 1998-1.



By evaluating the elastic or inelastic response spectrum, this results in the target displacement d_t , which is also shown. To verify the earthquake stability the calculated target displacement should be held by an appropriate ductility of the structure.

Input

The tabular input is made in the *Dynamics* folder of the database.

Analysis Settings

This is where the desired calculation mode and the corresponding control values are set.

Analysis mode

- **Eigenvalues**
 - Eigenvalues
 - Effect of the masses
- **Response spectrum**
 - Eigenvalues
 - Effect of the masses
 - Soil acceleration
 - Coefficients as per EN 1998-1:2013, DIN EN 1998-1:2021, DIN 4149:2005, DIN 4149:1981, OENORM B 4015, SIA 261 (design spectrum)
 - Alternative response spectrum
 - Store static pseudo loads
- **Pushover analysis**
 - Eigenvalues
 - Effect of the masses
 - Soil acceleration
 - Coefficients as per EN 1998-1:2013, DIN EN 1998-1:2021 (horizontal elastic response spectrum)
 - Vertical distribution of horizontal loads (Modal x, Modal y, Constant x, Constant y)
 - Control node (Node)

Eigenvalues

Number of eigenvalues: Up to 999 eigenvalues and vectors can be calculated.

Max. vector space: Size of the vector space to be used during the eigenvalue iteration. The number of iteration vectors must always be greater than or equal to the number of the selected eigenvalues.

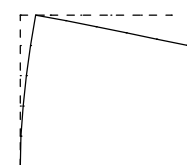
Iteration accuracy: Size of the maximum relative error [%].

Stiffness according 2nd order (LC): Calculation of eigenvalues with a stiffness matrix, which take into account the geometrical nonlinear part of the second-order theory. Therefore the internal forces of the statical load case, which is selected here, are used.

Effect of the Masses

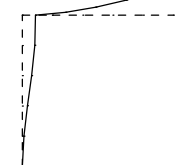
Selection of the mass portions implemented in the mass matrix.

All masses



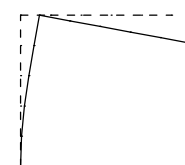
Eigenmode 1: 6.43 Hz

All masses



Eigenmode 2: 17.48 Hz

Masses only in x-direction



Eigenmode 1: 8.56 Hz

Attention: With the elimination of mass portions maybe differing eigenmodes and eigenfrequencies result!

Soil Acceleration

a_x, a_y : Vector with the standard value of the horizontal acceleration of the soil as a function of the earthquake zone [m/s²].

a_z : Principle value of the vertical acceleration of the soil. In general this does not need to be considered.

Apply components separately: Static pseudo load cases are applied separately for each direction.

Reference system angle Alpha: Rotation of the soil accelerations a_x and a_y about the global z axis [°].

ZPA-Method: Use of the ZPA method.

ZPA-T: Oscillation time T [s] to calculate the spectral value $S_{a,ZPA}$.

Coefficients as per EN 1998-1:2013 (Design Spectrum)


Ground type: Define the parameter values according to the basic standard EN 1998-1, Table 3.2 or 3.3.

Parameters S, T_B, T_C, T_D : Coefficients for determining the design spectrum for the horizontal earthquake action. The parameters are suggested in accordance with the ground type and the type of the response spectrum. The design spectrum for the vertical earthquake action is internally calculated according to Table 3.4 with the coefficients $S=1.0, T_B=0.05, T_C=0.15$ and $T_D=1.0$.

Lehr's damping measure: Measure ξ for determining the factor that accounts for building damping η . For the design spectrum according to Section 3.2.2.5 5% viscous damping is assumed.

Importance factor γ_I : Factor according to Section 4.2.5.

Behavior coefficient (horiz.) q : The capacity of a structure to resist the earthquake action in the nonlinear zone with post-limit stiffness can be accounted for with the behavior coefficient q . It is used for the horizontal earthquake action. For vertical earthquake actions q is internally set to 1.0 by the program.

The button  opens a dialog which shows the related spectral acceleration $S_d(T) / a_g$ (with $a_g = \gamma_I \cdot a_{gR}$).

Combination method:

- *Standard*: The program automatically selects one of the two following combination methods (see *Basics > Response spectrum EN 1998-1:2013*).
- *SRSS method*: Square root of the sum of the squares.
- *CQC method*: Complete quadratic combination.

Coefficients as per DIN EN 1998-1:2021 (Design Spectrum)

Subsoil circumstances 475 years: Define the parameter values according to DIN EN 1998-1, Table NA.1 or NA.2.

Parameters S, T_B, T_C, T_D : Coefficients for determining the design spectrum for the horizontal earthquake action. The parameters are suggested in accordance with the subsoil circumstances and the spectral acceleration $S_{aP,R}$. The design spectrum for the vertical earthquake action is internally calculated according to Table NA.3 with the coefficients $S=1.0, T_B=0.05, T_C=0.20$ and $T_D=1.20$.

Lehr's damping measure: Measure ξ for determining the factor that accounts for building damping η . For the design spectrum according to Section 3.2.2.5 5% viscous damping is assumed.

Importance factor γ_I : Factor according to Table NA.5.

Behavior coefficient (horiz.) q : The capacity of a structure to resist the earthquake action in the nonlinear zone with post-limit stiffness can be accounted for with the behavior coefficient q . It is used for the horizontal earthquake action. For vertical earthquake actions q is internally set to 1.0 by the program.

The button  opens a dialog which shows the related spectral acceleration $S_d(T) / a_g$ (with $a_g = \gamma_I \cdot a_{gR}$).

Combination method:

- *Standard*: The program automatically selects one of the two following combination methods (see *Basics > Response spectrum DIN EN 1998-1:2021*).
- *SRSS method*: Square root of the sum of the squares.
- *CQC method*: Complete quadratic combination.

Coefficients as per DIN 4149:2005


Subsoil circumstances: Define the subsoil parameters according to DIN 4149:2005, Table 4 or 5.

Subsoil parameters S, TB, TC, TD : Coefficients for determining the design spectrum.

Lehr's damping measure: Measure ξ for determining the damping correction coefficient η . For the design spectrum according to Section 5.4.3, 5% viscous damping is assumed.

Importance coefficient γ_I : Factor according to Table 3.

Behavior coefficient (horiz.) q : Coefficient to take into account the energy dissipation capacity of a structure according to Sections 8 to 12. For vertical earthquake actions q is internally set to 1.0 by the program.

The button  opens a dialog which shows the related spectral acceleration $S_d(T) / a_g$.

Combination method:

- *Standard:* The program automatically selects one of the two following combination methods (see *Basics > Response spectrum DIN 4149:2005*).
- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.


Coefficients as per DIN 4149:1981

Subsoil factor $kappa$: Factor to account for the subsoil (DIN 4149:1981, Section 7.2.2).

Factor for regard of the building class $alpha$: Reduction factor as a function of the subsoil class and the earthquake zone (Section 7.2.3).

Increase coefficient B by 30%: The coefficient β (Figure 2) is increased by 30%. According to Section 8.1 this is necessary for structures with a damping measure $D < 0.05$.

Lehr's damping measure: Damping factor of the structure for calculation of the interaction factor when combining the modal components according to the CQC method.

The button  opens a dialog which shows the related spectral acceleration $\beta(T) \cdot \kappa \cdot \alpha$.

Combination method:

- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

Coefficients as per OENORM B 4015

Type of the subsoil: Define the foundation coefficient and the reaction parameters according to OENORM B 4015, Table 9.

Foundation coefficient $k1$: Value according to Table 9.


Reaction parameters $T1, T2, B0$: Coefficients for determining the building reaction coefficient k_2 .

Lehr's damping measure: Measure ξ for determining the factor to account for building damping η and the interaction factor for the combination of the modal components according to the CQC method.

$$\eta = \sqrt{\frac{10}{5 + 100 \cdot \xi}}$$

Building ductility $k5$: Ratio of the limit deformation and the elastic deformation.

Factor for regard of the building type: The factor k_4 should generally be set to 1.

The button  opens a dialog which shows the related spectral acceleration $k_1 \cdot k_2 \cdot k_4$.

Combination method:

- *Standard:* The program automatically selects one of the two following combination methods (see *Basics > Response spectrum OENORM B 4015*).
- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

Coefficients as per SIA 261


Subsoil class: Define the reaction parameters according to SIA 261, Table 24.

Reaction parameters S , TB , TC , TD : Coefficients for determining the design spectrum.

Lehr's damping measure: Measure ξ for determining the factor that accounts for building damping η . For the design spectrum according to Section 16.2.4.1 5% viscous damping is assumed.

Importance factor γ_I : Factor according to Table 25.

Behavior coefficient (horiz.) q : The capacity of a structure to resist the earthquake action in the nonlinear zone with post-limit stiffness can be accounted for with the behavior coefficient q . For reinforced concrete the value can be taken from Table 14, SIA 262. It is used for the horizontal earthquake action. For vertical earthquake actions q is internally set to 1.5 by the program.

The button  opens a dialog which shows the related spectral acceleration $S_d(T) \cdot g / a_{gd}$.

Combination method:

SRSS method: Square root of the sum of the squares.

Coefficients as per EN 1998-1:2013 (Horizontal Elastic Response Spectrum)


Ground type: Define the parameter values according to the basic standard EN 1998-1, Table 3.2 or 3.3.

Parameters S , TB , TC , TD : Coefficients for determining the horizontal elastic response spectrum for the earthquake action.

The parameters are suggested in accordance with the ground type and the type of the response spectrum.

Lehr's damping measure: Measure ξ for determining the factor that accounts for building damping η .

Importance factor γ_I : Factor according to Section 4.2.5.

The button  opens a dialog which shows the related spectral acceleration $S_e(T) / a_g$ (with $a_g = \gamma_I \cdot a_{gR}$).

Coefficients as per DIN EN 1998-1:2021 (Horizontal Elastic Response Spectrum)

Subsoil circumstances 475 years: Define the parameter values according to DIN EN 1998-1, Table NA.1 or NA.2.

Parameters S , TB , TC , TD : Coefficients for determining the horizontal elastic response spectrum for the earthquake action.

The parameters are suggested in accordance with the subsoil circumstances and the spectral acceleration $S_{aP,R}$.

Lehr's damping measure: Measure ξ for determining the factor that accounts for building damping η .

Importance factor γ_I : Factor according to Table NA.5.

The button  opens a dialog which shows the related spectral acceleration $S_e(T) / a_g$ (with $a_g = \gamma_I \cdot a_{gR}$).

Alternative Response Spectrum

Combination method:

- *SRSS method:* Square root of the sum of the squares.
- *CQC method:* Complete quadratic combination.

Lehr's damping measure: Damping factor of the structure for calculation of the interaction factor when combining the modal components according to the CQC method.

Standard: Function of the standard response spectrum.

Vertical: Function of the vertical response spectrum. The standard response spectrum will be used for the vertical direction, if no function is defined.

Store Static Pseudo Loads

The static pseudo loads of the modal components that are calculated for an earthquake analysis with response spectra are saved and can be subsequently imported into a load case. If necessary, this can be done separately for the accelerations directions.

Point Masses

This function of the *structure* menu is used to define the point masses for dynamic analysis. They act in addition to the masses from the selected load cases and the mass distribution of the structure, which are automatically determined by the program. The point masses are linked to the element nodes and are represented with a filled circle.

Off

Remove point masses from the graphical view.

Enter

Define point masses at the selected nodes by specifying their components.

Components of a point mass

Mux, Muy, Muz

Translational masses in direction of the global axes [t].

Mphi.x, Mphi.y, Mphi.z

Rotatory masses about the global axes [$t\ m^2$].

Generate Masses From Loads

It allows you to select load cases from which to create additional nodal masses.

This is done by dividing nodal loads in the global z direction that result from the load cases by the gravitational acceleration ($9.81\ m/s^2$) and applying them as additional point masses (translational masses) in the x, y and z directions.

The following load types, which do not represent vertical loads in this sense, are ignored by the program.

- Influence surface, Influence line
- Insert
- Dynamic element collapse
- Dynamic train load
- Fire scenarios
- Buckling eigenvalues
- Creep and shrinkage
- Support displacement
- Superposition
- Theory
- Initial strain, Temperature
- Prestressing
- Load group
- Point moment on beams
- Fixed end forces of beams
- Free line moments on beams and area elements
- Line torsional moment on beams

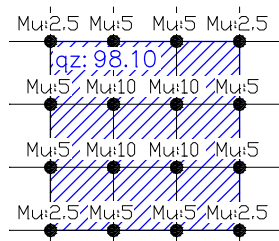
The *Dead load* load type is also not taken into account as the dead load of the system is already automatically included in the mass matrix.

Load cases

Select load cases to add them to the current action.

Load factor

The individual load cases are weighted by a factor.

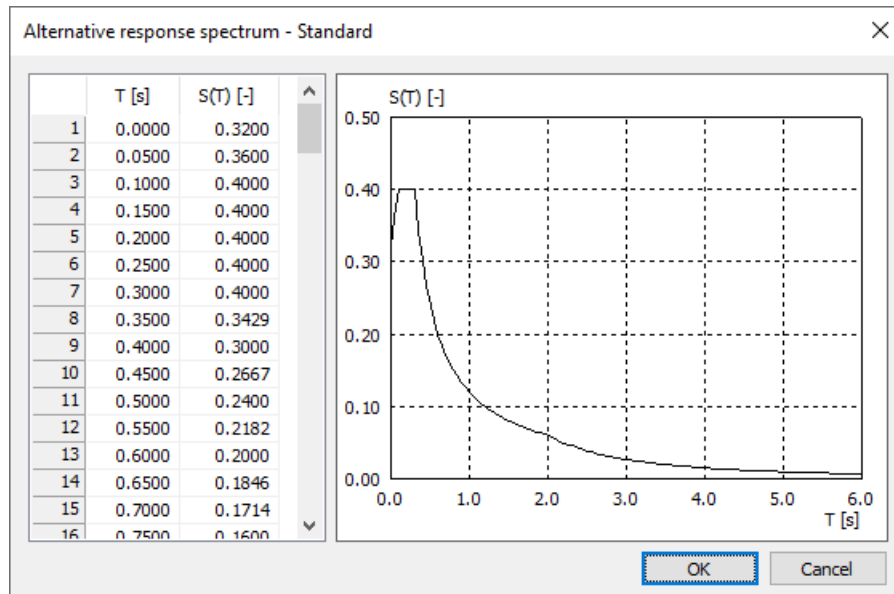


Mass distribution for an element grid of 1 m and a load q_z of 98.1 kN/m²

The point masses of load cases resulting from dynamic analysis can be examined with the database using '*Results, Masses generated from loads*'.

Alternative Response Spectrum

This dialog allows you to specify a user-defined standardized response spectrum. The specifications can be made in the analysis settings - dynamics if the corresponding analysis mode is selected.



Oscillation time T

Vibration period [s].

Coefficient S(T)

Coefficient of the user-defined standardized response spectrum.

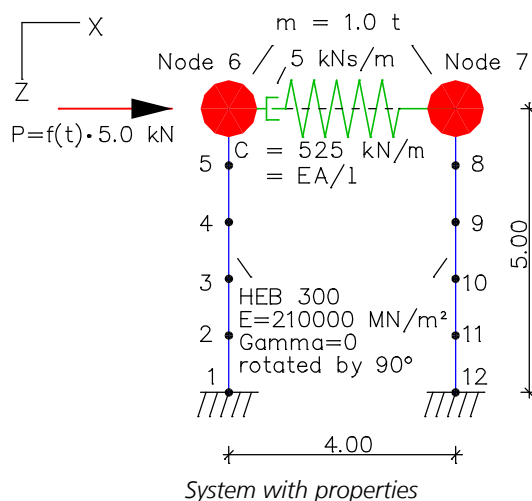
Examples

Linked Two-Mass Oscillator

Essential functions of dynamic structure analysis are illustrated based on the example of a two-mass oscillator. A spring element connects the two point masses. The necessary inputs are shown respectively.

The following dynamic analyses are carried out:

- Eigenvalues and eigenmodes.
- Response spectrum according to EN 1998-1:2013.



Eigenvalues and Eigenmodes

Setting Dynamics

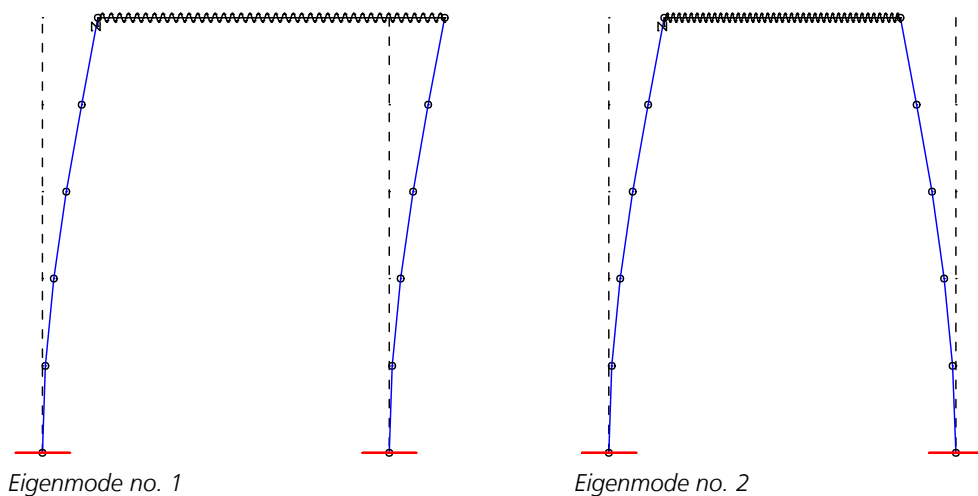
Analysis mode	Eigenvalues
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X

The calculation log includes:

Natural frequencies

No.	Eigenvalue [1/s ²]	rel. tol. [-]	omega [rad/s]	frequency [Hz]	periode [s]
1	4.3142e+02	7.2896e-27	20.771	3.306	0.302502
2	1.4814e+03	1.1543e-28	38.489	6.126	0.163245

The eigenmodes and resonant frequencies that are determined can be graphically displayed with the *Results* function.

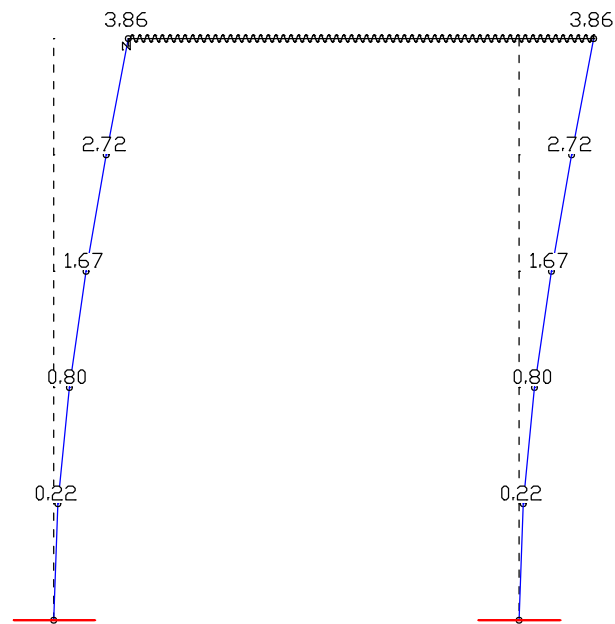


Response Spectrum EN 1998-1:2013

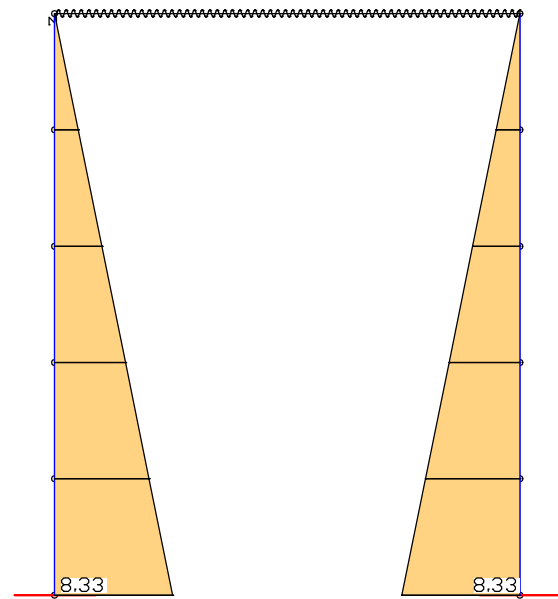
The horizontal static pseudo loads are to be calculated according to the basic standard EN 1998-1 for the eigenmodes under analysis. The results of the evaluation of the response spectrum of the individual eigenmodes are statistically superposed.

Setting Dynamics

Analysis mode	Response spectrum EN 1998-1:2013
Eigenvalues, number	2
Max. vector space	2
Iteration accuracy	1 [%]
Effect of the masses In direction Around axis	X
Soil accelerations Apply components separately Referenz system angle alpha	$a_x = 1 \ a_y = 0 \ a_z = 0 \ [m/s^2]$ No 0 [°]
Coefficients of EN 1998-1:2013 Ground type Response spectrum of type Parameter for response spectrum Lehr's damping measure Importance coeff. gamma I Behavior coefficient q Combination method Store static substitute loads	A - Rock or other rock-like geological formation... 1 S = 1 TB = 0.15 TC = 0.4 TD = 2 0.05 1 1.5 Standard No



Deformations after statistical superposition [mm]

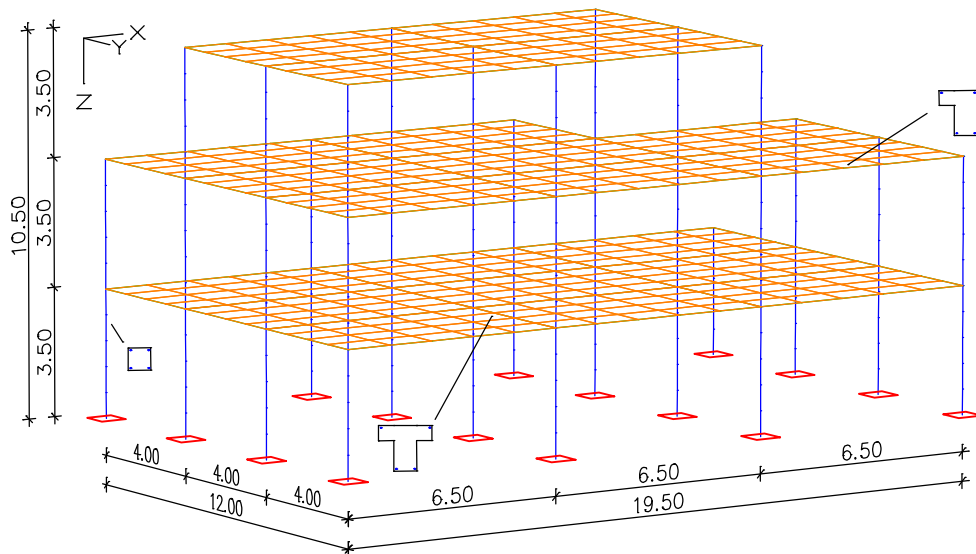


Moments after statistical superposition [kNm]

Earthquake Check According to DIN EN 1998-1:2021

The following example illustrates how to apply the response spectrum method on an irregular 3D structure. The handling of a similar model is also shown in the video (German language)

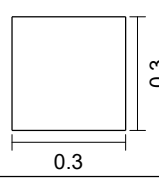
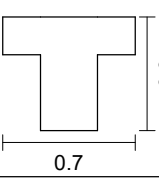
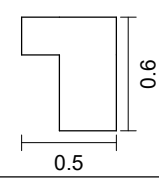
"https://download.infograph.de/video_de/Erdbebennachweis%20nach%20der%20Antwortspektrenmethode.mp4".



Material

Concrete C20/25 according to DIN EN 1992-1-1
 Reinforcing steel 500 M / 500 S

Section properties

1	Polygon 	Columns Centroid [m] $y_s = 0.150$ $z_s = 0.150$ Area [m ²] $A = 9.0000e-02$ Moments of inertia [m ⁴] $I_x = 5.0000e-04$ $I_y = 6.7500e-04$ $I_1 = 6.7500e-04$ $I_z = 6.7500e-04$ $I_2 = 6.7500e-04$ Main axis angle [Grad] $\Phi = 0.000$ $I_{yz} = 0.0000e+00$ Ignore Iyz in member stiffnes.
2	Polygon 	T-beams Centroid [m] $y_s = 0.350$ $z_s = 0.238$ Area [m ²] $A = 2.6000e-01$ Moments of inertia [m ⁴] $I_x = 3.0000e-03$ $I_y = 7.8821e-03$ $I_1 = 7.8821e-03$ $I_z = 6.6167e-03$ $I_2 = 6.6167e-03$ Main axis angle [Grad] $\Phi = -0.000$ $I_{yz} = 0.0000e+00$ Ignore Iyz in member stiffnes.
3	Polygon 	Edge beams Centroid [m] $y_s = 0.305$ $z_s = 0.264$ Area [m ²] $A = 2.2000e-01$ Moments of inertia [m ⁴] $I_x = 3.0000e-03$ $I_y = 6.8424e-03$ $I_1 = 7.5143e-03$ $I_z = 3.5288e-03$ $I_2 = 2.8569e-03$ Main axis angle [Grad] $\Phi = -22.322$ $I_{yz} = 1.6364e-03$ Ignore Iyz in member stiffnes.
4	Area	Slabs Element thickness [m] $d_z = 0.2000$ torsion-proof Orthotropy d_{zy}/d_z = 1 E-Modulus slab/plain = 1

Properties for the earthquake check

$S_{aP,R}$ (475 years, Aachen): 2.557 m/s²

Soil class: A

Importance category III:

Behavior coefficient:

Soil acceleration $a_{gR} = 2.557 / 2.5 = 1.023$ m/s²

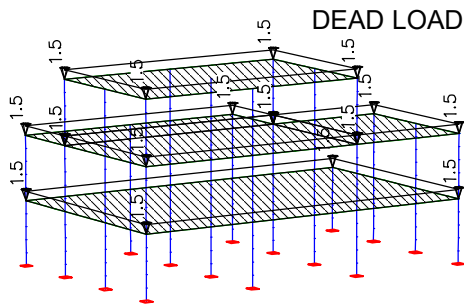
Geological subsoil class: R

$\gamma_1 = 1.2$

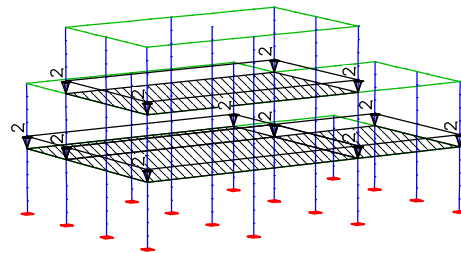
$q = 0.8 \cdot 1.5 = 1.2$

For the displayed framework structure made of reinforced concrete the following calculations are carried out:

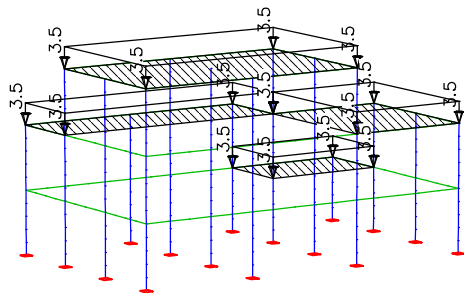
- Determination of the eigenvalues and eigenmodes of the structure model under consideration of the masses according to Section 4.3.3.3.1 of the standard.
- Determination of the horizontal static pseudo loads according to Section 4.3.3.3.1 of the standard.
- Check of the effective modal masses of the structure model.
- Statistical combination of the modal internal forces and displacements according to Section 4.3.3.3.2 of the standard.
- Consideration of random torsion actions according to Section 4.3.3.3.3 of the standard.
- Combination of the stress as a result of the components of the earthquake actions according to Section 4.3.3.5.1 (3) of the standard.
- Combinations of the earthquake action with other actions according to DIN EN 1990.
- Design according to DIN EN 1992-1-1.
- Consideration of influences according to 2nd order theory (P-D effects) according to DIN EN 1998-1, Section 4.4.2.2 (2).



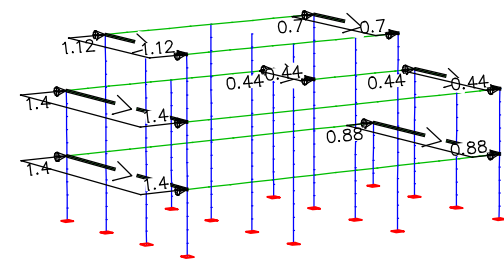
Load case 1: Dead load and expansion



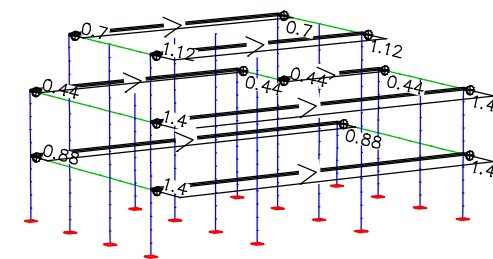
Load case 2: Traffic on inner areas



Load case 3: Traffic on outer areas



Load case 4: Wind in x direction



Load case 5: Wind in y direction

Calculation settings for dynamic

Effect of the masses
 in direction X Y Z
 around axis X Y Z

Eigenvalues
 Number of eigenvalues: 5
 Maximum vectorspace : 10
 Iteration accuracy : 1.00 [%]

Response spectrum DIN EN 1998-1:2021
 Basic values of the soil acceleration
 ax: 1.023 [m/s²]
 ay: 1.023 [m/s²]
 az: 0.000 [m/s²]
 Acceleration components are applied separately
 Angle of the reference system alpha: 0.00 [°]
 Soil behavior : A-R
 Reaction parameter horizontal S: 1.000 TB: 0.100 TC: 0.200 TD: 2.000
 Reaction parameter vertical S: 1.000 TB: 0.050 TC: 0.200 TD: 1.200
 Lehr's damping measure : 0.050
 Importance factor GammaI: 1.200
 Behavior coefficient qh: 1.200 qv: 1.000

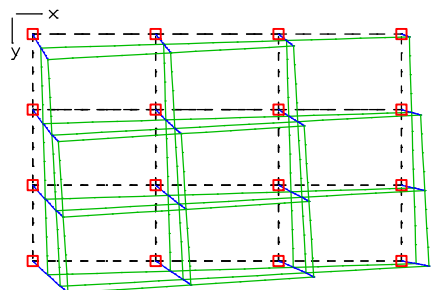
Generate masses from loads

Load case	factor
1	1.00
2	0.21
3	0.30

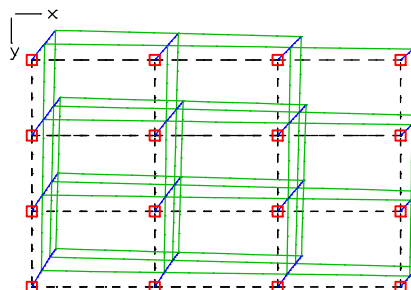
Sum of the installed masses (global)

	Mx [t]	My [t]	Mz [t]
Volume masses :	495.034	495.034	495.034
Point masses :	0.000	0.000	0.000
Masses of LC :	121.890	121.890	121.890
Sum :	616.923	616.923	616.923

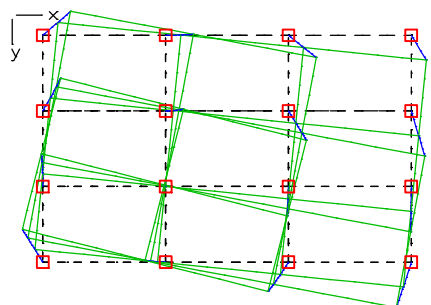
Resonant frequencies and eigenmodes



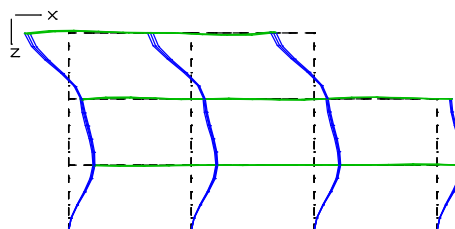
Eigenmode 1: $f = 1.43$ Hz



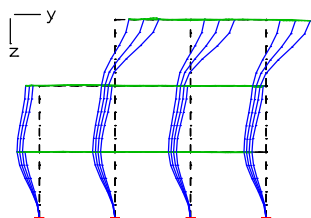
Eigenmode 2: $f = 1.47$ Hz



Eigenmode 3: $f = 2.03$ Hz



Eigenmode 4: $f = 3.54$ Hz



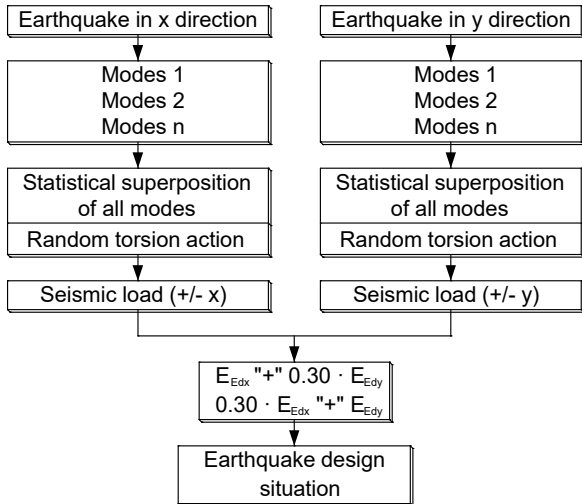
Eigenmode 5: $f = 3.63$ Hz

Effective modal masses (global)

Mode shape	Mx eff [t]	My eff [t]	Mz eff [t]
1	323.206	183.332	0.000
2	209.813	329.520	0.000
3	13.016	37.500	0.000
4	46.327	5.072	0.034
5	3.753	41.475	0.003
Sum:	596.115	596.899	0.037
Part of the total masses:	97 [%]	97 [%]	0 [%]

The effective modal masses that are considered are 97% of the applied masses. This means that according to Section 4.3.3.3.1 (3) of the standard the number of eigenmodes analyzed is sufficient.

Calculation scheme



According to Section 4.3.3.5.1 (3), the check for the horizontal earthquake action is to be carried out in two directions that are perpendicular to one another. The calculation scheme shown above depicts the procedure. The statistical superposition of the modal components is carried out by the program. The random torsion actions according to Section 4.3.3.3 (1) of the standard are considered in the following manner for this structure.

Comparative calculation

Foundation force

$$F_b = \frac{S_d(T)}{a_g} \cdot a_g \cdot m$$

$$= 0.595 \cdot 1.023 \cdot 1.2 \cdot 617$$

$$= 450 \text{ kN for } T = 1/1.43 = 0.7 \text{ s}$$

with

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j}$$

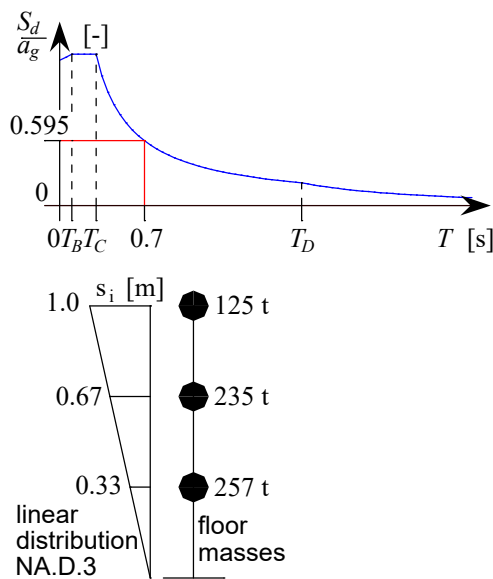
$$M_{1i} = e_{1i} \cdot F_i = h_i \cdot L_i^2 / 6$$

Load ordinate of the trapezoidal load of the story *i*

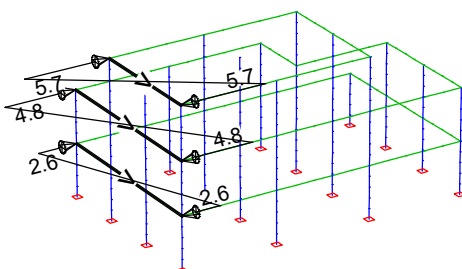
$$h_i = \pm 0.3 \cdot F_i / L_i$$

Eccentricity of F_i

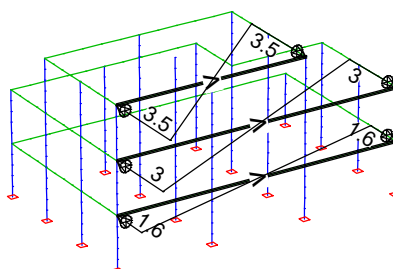
$$e_{ai} = \pm 0.05 \cdot L_i$$



Floor above story	Height z_i [m]	Mass m_i [t]	$s_i \cdot m_i$ [tm]	H-Force F_i [kN]	Width Lx_i [m]	Trap. load hx_i [kN/m]	Width Ly_i [m]	Trap. load hy_i [kN/m]
3	10.5	125	125	153	8	± 5.7	13	± 3.5
2	7	235	157	192	12	± 4.8	19.5	± 3.0
1	3.5	257	86	105	12	± 2.6	19.5	± 1.6
Sum		617	367	450				



Load case 51: Random torsion (x)



Load case 52: Random torsion (y)

Combination of the stress

Combination of the stress as a result of the components of the earthquake actions according to Section 4.3.3.5.1 (3) of the standard.

Load case combination 1, Edx "+" 0.3 Edy

1. Variable exclusive action		Factor
	Response spectrum overlaid (x)	-1.000
	Response spectrum overlaid (x)	1.000
2. Variable exclusive action		Factor
	Response spectrum overlaid (y)	-0.300
	Response spectrum overlaid (y)	0.300
3. Variable exclusive action		Factor
51	Random torsion (x)	-1.000
51	Random torsion (x)	1.000
4. Variable exclusive action		Factor
52	Random torsion (y)	-0.300
52	Random torsion (y)	0.300

Load case combination 2, 0.3 Edx "+" Edy

1. Variable exclusive action		Factor
	Response spectrum overlaid (x)	0.300
	Response spectrum overlaid (x)	-0.300
2. Variable exclusive action		Factor
	Response spectrum overlaid (y)	1.000
	Response spectrum overlaid (y)	-1.000
3. Variable exclusive action		Factor
51	Random torsion (x)	-0.300
51	Random torsion (x)	0.300
4. Variable exclusive action		Factor
52	Random torsion (y)	-1.000
52	Random torsion (y)	1.000

Combinations of the earthquake action with other actions according to DIN EN 1990.

DIN EN 1992-1-1 actions

Standard design group

G - Dead load

$\Gamma_{sup} / \Gamma_{inf} = 1.35 / 1$

Load cases

1 G, Dead load

QN - Imposed load, traffic load

$\Gamma_{sup} / \Gamma_{inf} = 1.5 / 0$

Combination coefficients for: Superstructures
Working load - category A: Residential buildings
 $\Psi_{0.0} / \Psi_{0.1} / \Psi_{0.2} = 0.7 / 0.5 / 0.3$

Load cases 1. Variant, inclusive

2 QN, Indoor ceiling
3 QN, Outdoor ceiling

QW - Wind load

$\Gamma_{sup} / \Gamma_{inf} = 1.5 / 0$

Combination coefficients for: Superstructures
Wind loads
 $\Psi_{0.0} / \Psi_{0.1} / \Psi_{0.2} = 0.6 / 0.2 / 0$

Load cases 1. Variant, exclusive

4 QW, Wind in direction x
5 QW, Wind in direction y

AE - Actions due to earthquake

$\Gamma_{sup} = 1$

Load cases 1. Variant, exclusive

K1 Edx "+" 0.3 Edy
K2 0.3 Edx "+" Edy

1. Permanent and temporary situation

Final state

- G Dead load
- QN Imposed load, traffic load
- QW Wind load

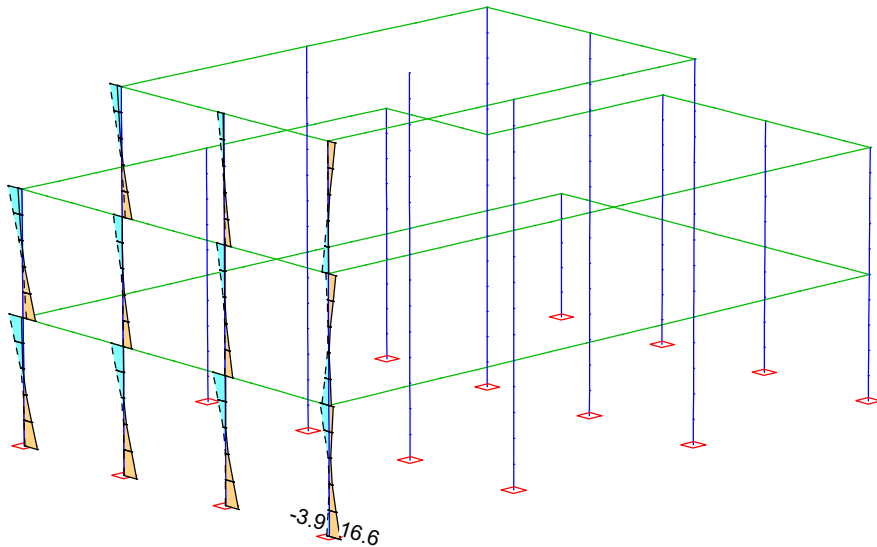
1. Earthquake situation

Final state

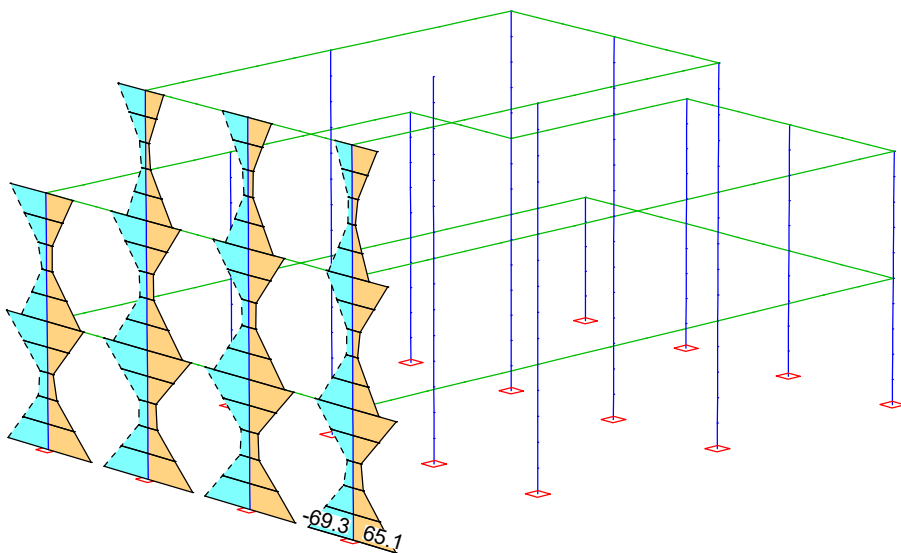
- G Dead load
- QN Imposed load, traffic load
- QW Wind load
- AE Actions due to earthquake

Results

The internal forces $min/max M_z$ displayed below indicate the different influences of wind force (permanent and temporary situation) and earthquake action (earthquake situation) for a selected beam group. It can be seen that the actions from earthquakes are decisive in this case.



Selected internal forces $min, max M_z$ from the permanent and temporary situation [kNm]



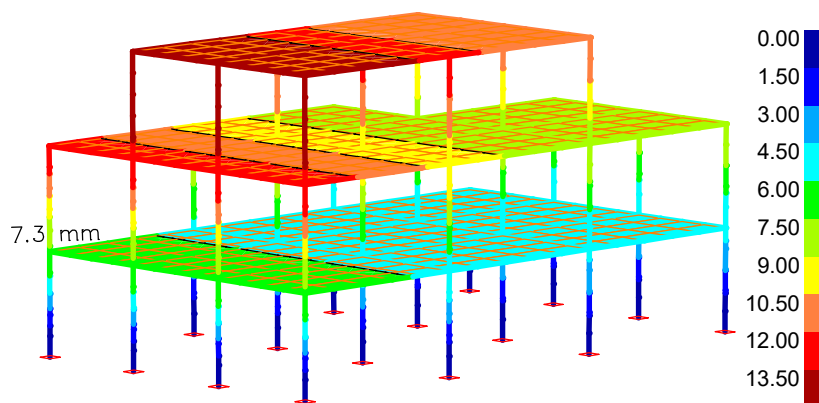
Selected internal forces $min, max M_z$ from the earthquake situation [kNm]

**Combination information - Internal forces Beam 1 - Loc. 1, Node 1 - Mz min, max
DIN1992.E.1 : 1. Earthquake situation, DIN EN 1992-1-1**

Load case	Factor	Nx [kN]	My [kNm]	Mz [kNm]	Qy [kN]	Qz [kN]	Mx [kNm]
1 : G, Dead load	1.00	-199.91	4.43	-2.00	1.65	-3.77	0.00
2 : QN, Indoor ceiling	0.30	-6.17	0.25	-0.10	0.09	-0.22	-0.00
3 : QN, Outdoor ceiling	0.30	-5.17	0.05	-0.05	0.04	-0.03	0.00
51 : Random torsion (x)	-0.30	-0.87	-0.63	-1.05	0.58	0.34	0.02
52 : Random torsion (y)	-1.00	-4.77	-3.42	-5.71	3.14	1.85	0.11
Asp.x : Response spectrum overlaid (x)	-0.30	-10.30	-14.52	-3.89	-2.14	-7.88	-0.07
Asp.y : Response spectrum overlaid (y)	-1.00	-65.86	-11.29	-56.47	-31.11	-6.10	-0.37
Mz min		-293.06	-25.13	-69.26	-27.76	-15.81	-0.30
1 : G, Dead load	1.00	-199.91	4.43	-2.00	1.65	-3.77	0.00
51 : Random torsion (x)	0.30	0.87	0.63	1.05	-0.58	-0.34	-0.02
52 : Random torsion (y)	1.00	4.77	3.42	5.71	-3.14	-1.85	-0.11
Asp.x : Response spectrum overlaid (x)	0.30	10.30	14.52	3.89	2.14	7.88	0.07
Asp.y : Response spectrum overlaid (y)	1.00	65.86	11.29	56.47	31.11	6.10	0.37
Mz max		-118.10	34.29	65.12	31.19	8.01	0.31

Consideration of influences according to 2nd order theory (P-Δ effects)

The example calculation is carried out for the lowest story according to Section 4.4.2.2 (2) of the standard.



Maximum deformation in y-direction due to the earthquake situation

Sum of installed loads and support reactions

LC.	Label	Fx [kN]	Fy [kN]	Fz [kN]
1	G, Dead load	0.000	0.000	5675.250
	Support reactions	-0.000	-0.000	5675.250
2	QN, Indoor ceiling	0.000	0.000	624.000
	Support reactions	-0.000	0.000	624.000
3	QN, Outdoor ceiling	0.000	0.000	819.000
	Support reactions	0.000	0.000	819.000

$$P_{\text{tot}} = \Sigma G_k + \Sigma \psi_2 \cdot Q_k = 5675 + 0.3 \cdot (624 + 819) = 6108 \text{ kN}$$

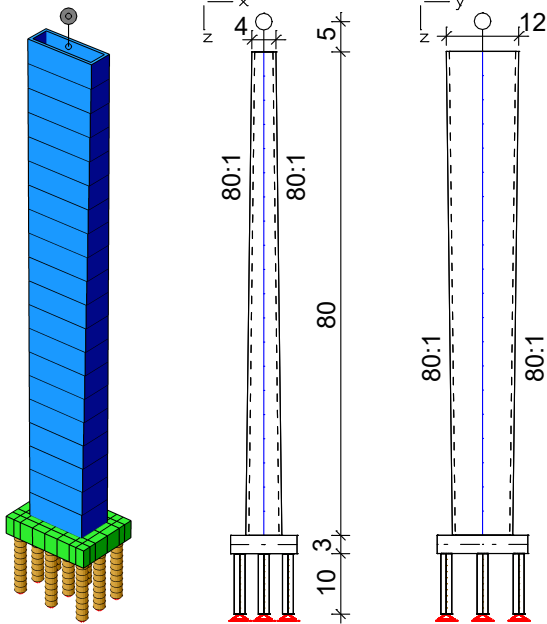
$$d_t = d_s = d_e \cdot q = 7.3 \cdot 1.2 = 8.8 \text{ mm}; \quad h = 3500 \text{ mm}; \quad V_{\text{tot}} = F_b = 450 \text{ kN}$$

$$\theta = \frac{P_{\text{tot}} \cdot d_t}{V_{\text{tot}} \cdot h} = \frac{6108 \cdot 8.8}{450 \cdot 3500} = 0.03 \leq 0.1$$

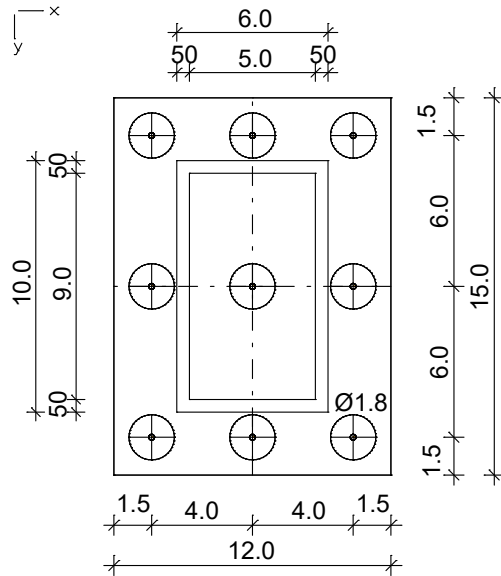
=> **Actions according to the second-order theory do not have to be considered.**

Earthquake Check of a Bridge Pier According to the ZPA Method

The bridge pier shown below illustrates the response spectrum method using the ZPA method. The structure consists of C35/45 according to DIN EN 1992-1-1. The horizontal bedding moduli for the piles were assumed to be 500 MN/m³ in this example. In vertical direction the piles are fixed at their lower end. The mass of the superstructure of the bridge assumed for the example was taken into account via a point mass (5000 t) 5 m above the pier. In the context of the example, only the global x-direction is examined and earthquake zone 3 and the subsoil conditions "A-R" according to DIN 4149 are assumed. For better comparability, the statistical combination method SRSS is used in all cases.



Structure and dimensions [m]



Top view with pier cross-section at the clamping point

Sum of the installed masses (global)

	Mx [t]	My [t]	Mz [t]
Volume masses :	4888.313	0.000	0.000
Point masses :	5000.000	0.000	0.000
Masses of LC :	0.000	0.000	0.000
Sum :	9888.313	0.000	0.000

Effective modal masses (global)

Mode shape	Mx eff [t]	My eff [t]	Mz eff [t]
1	6554.330	0.000	0.000
2	940.564	0.000	0.000
3	521.525	0.000	0.000
4	787.150	0.000	0.000
5	586.590	0.000	0.000
6	0.000	0.000	0.000
7	88.883	0.000	0.000
8	21.056	0.000	0.000
9	342.112	0.000	0.000
Sum:	9842.209	0.000	0.000
Part of the total masses:	100 [%]	0 [%]	0 [%]

As can be seen from the table, with 9 eigenmodes already 100 % of the eigenmasses are excited. This can therefore be regarded as a reference solution. The 1st and 2nd eigenmodes have an effective modal mass of 66.3 % and 9.5 %. The first two eigenmodes thus excite 75.8 % of the masses, which does not meet the standard requirement of 90 % excitation. With the assumed subsoil conditions "A-R", the plateau value of the response spectrum is only reached with the third eigenmode (T = 0.14 sec < TC). The 6th eigenmode is again below the plateau (T = 0.038 sec < TB).

Calculation settings for dynamic

```
Effect of the masses
in direction X
around axis

Eigenvalues
Number of eigenvalues: 9
Maximum vectorspace : 40
Iteration accuracy : 1.00 [%]
Response spectrum DIN 4149 (release 04/2005)
Basic values of the soil acceleration
ax: 0.800 [m/s²]
ay: 0.000 [m/s²]
az: 0.000 [m/s²]
Angle of the reference system alpha: 0.00 [°]
Soil behavior : A-R
Reaction parameter S: 1.000 TB: 0.050 TC: 0.200 TD: 2.000
Lehr's damping measure : 0.050
Importance coeff. Gamma1: 1.400
Behavior coefficient q: 1.500
```

Natural frequencies

No.	Eigenvalue [1/s²]	rel. tol. [-]	omega [rad/s]	frequency [Hz]	periode [s]
1	1.8975e+00	4.9592e-23	1.377	0.219	4.561308
2	2.2315e+02	2.7791e-25	14.938	2.377	0.420613
3	2.0408e+03	1.6256e-20	45.175	7.190	0.139085
4	7.4138e+03	8.9772e-16	86.103	13.704	0.072973
5	1.5208e+04	1.8038e-13	123.322	19.627	0.050950
6	2.7683e+04	4.2009e-11	166.383	26.481	0.037763
7	3.3484e+04	9.6951e-12	182.987	29.123	0.034337
8	7.4012e+04	3.3768e-10	272.052	43.298	0.023096
9	1.4252e+05	6.1725e-06	377.519	60.084	0.016643

An oscillation time of 0.2 sec was assumed for the ZPA value. Thus the spectral value $S_{a,ZPA}$ is assumed to be the same size as the plateau value of the response spectrum and thus fits well for oscillation times below 0.2 sec.

Calculation settings for dynamic

Effect of the masses
in direction X
around axis

Eigenvalues

Number of eigenvalues: 2
Maximum vectorspace : 40
Iteration accuracy : 1.00 [%]
Response spectrum DIN 4149 (release 04/2005)

Basic values of the soil acceleration

ax: 0.800 [m/s²]
ay: 0.000 [m/s²]
az: 0.000 [m/s²]

Angle of the reference system alpha: 0.00 [°]

ZPA-Method (Zero Period Acceleration) activated, oscillation time T_ZPA: 0.200 [s]

Soil behavior : A-R

Reaction parameter S: 1.000 TB: 0.050 TC: 0.200 TD: 2.000

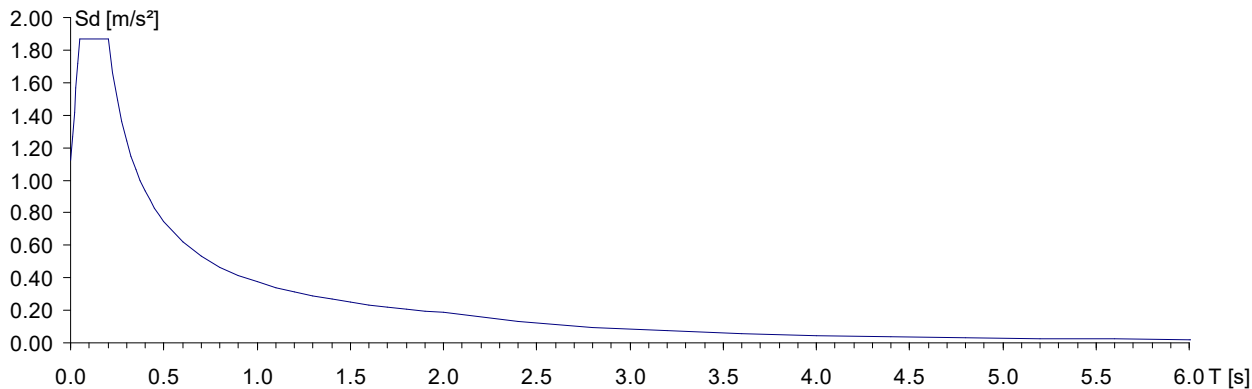
Lehr's damping measure : 0.050

Importance coeff. Gamma: 1.400

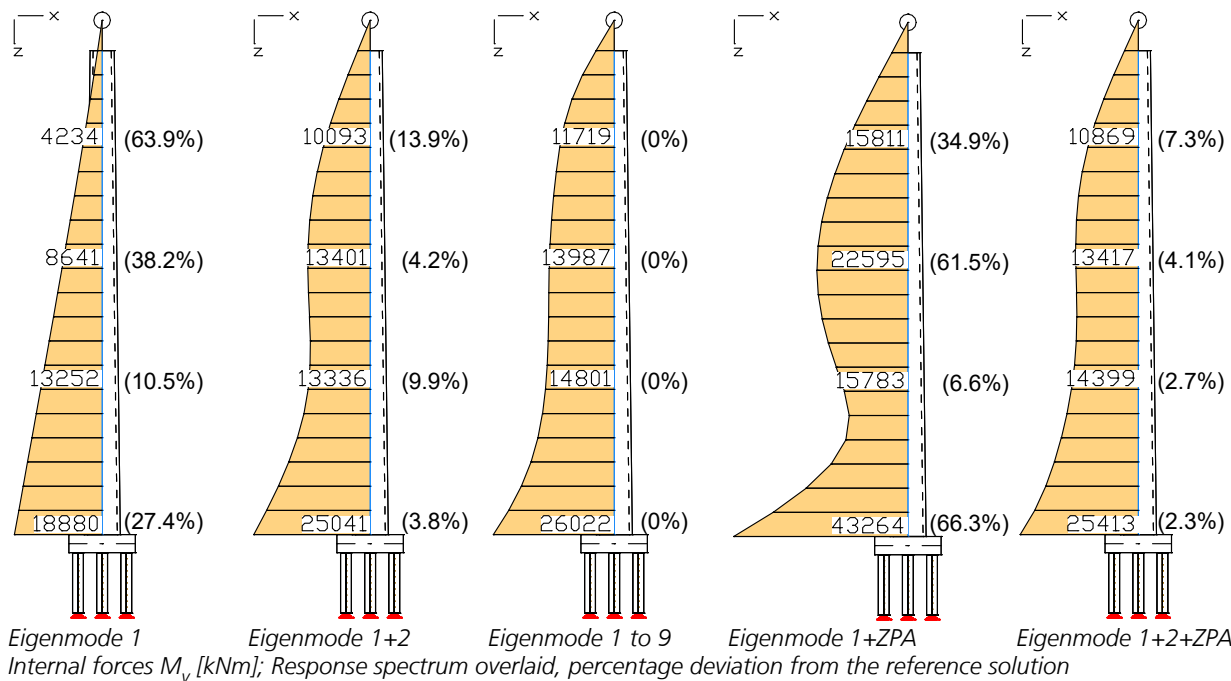
Behavior coefficient q: 1.500

Effective modal masses (global)

Mode shape	Mx eff [t]	My eff [t]	Mz eff [t]
1	6554.330	0.000	0.000
2	940.564	0.000	0.000
3	2393.436	0.000	0.000
Sum:	9888.331	0.000	0.000
Part of the total masses:	100 [%]	0 [%]	0 [%]



Response spectrum



As can be seen from the comparison of the internal forces due to the superimposed earthquake loads, the ZPA method with two eigenmodes provides a result similar to the reference solution with 9 eigenmodes. You can see that the ZPA method only approximates high-frequency eigenmodes well. This is one of the reasons for the remaining deviations.

Pushover Analysis of a Four-storey Reinforced Concrete Frame

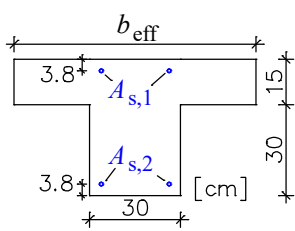
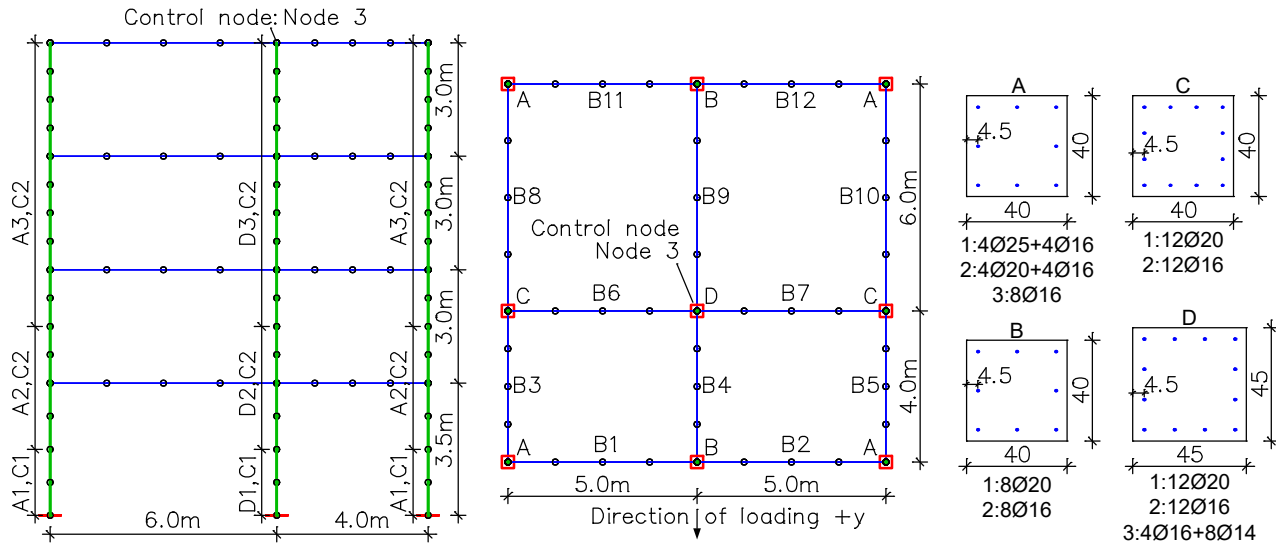
This example shows the procedure for a pushover analysis for a four-storey reinforced concrete frame. The example was selected based on the publication

OS Modeler – Examples of application, Version 1, University of Ljubljana, 2008, Matjaž Dolšek

From the results of the pushover analysis, the target displacement for a defined seismic load is determined. The target displacement is determined by the N2 method.

Structural properties

The dimensions of the calculation model with the existing reinforcement quantities can be seen in the following figures. The material C25/30 according to EN 1992-1-1 (basic standard) was selected. The yield strength f_y of all reinforcement is 500 MN/m². The longitudinal reinforcement is strapped in all cross sections.

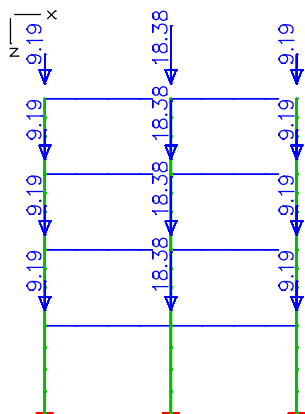


Beam	b_{eff} [cm]	$A_{s,1}$ [cm ²]	$A_{s,2}$ [cm ²]
B3, B5	70	4.618	4.210
B1, B2, B11, B12	80	4.618	4.210
B8, B10	90	4.618	4.210
B4	110	3.848	3.079
B6, B7	130	3.848	3.079
B9	150	3.848	3.079

On the program side, the implemented method is divided into the calculation steps described under "Pushover calculation according to EN 1998-1:2013".

Dynamics

In order to reach the storey masses examined in the example (1st floor 87 t, 2nd and 3rd floor 86 t, 4th floor 83 t), the masses involved in the vibration have been supplemented by the missing shares with *Generate masses from loads*.



Generate masses from loads

Load case	factor
1	1.00

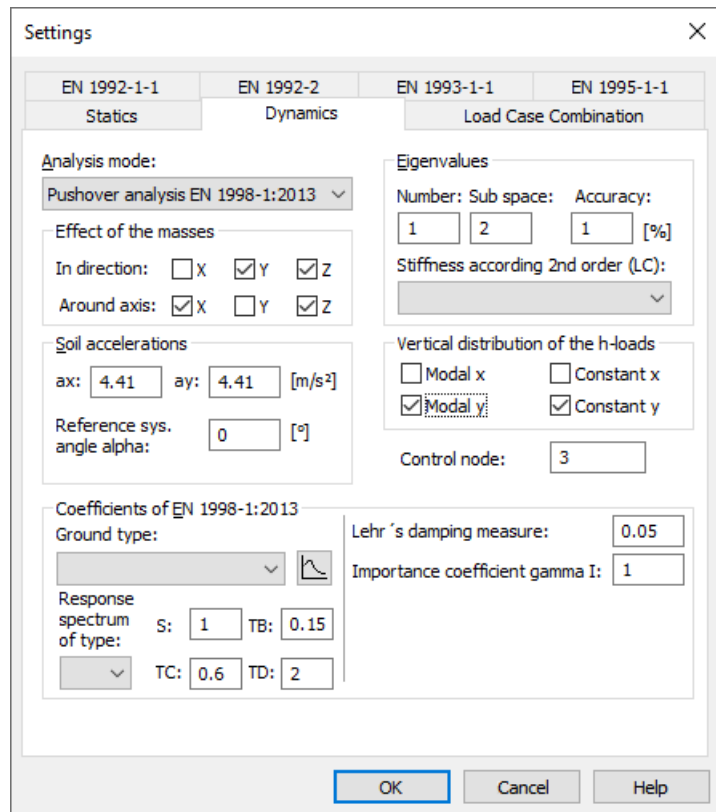
Extract from the listing *Finite elements dynamics* to control the considered masses (87 + 2·86 + 83 = 342 t):

Sum of the installed masses (global)

	Mx [t]	My [t]	Mz [t]
Volume masses :	0.000	192.180	192.180
Point masses :	0.000	0.000	0.000
Masses of LC :	0.000	149.888	149.888
Sum :	0.000	342.068	342.068

Load case 1: G

For the subsequent pushover analysis i.a. horizontal static pseudo loads are required. These are applied in the appropriate direction to the mass points using the mass assignment and the basic modal form according to EN 1998-1, Annex B.



The dynamics setting dialogue requires beside the analysis mode the following information:

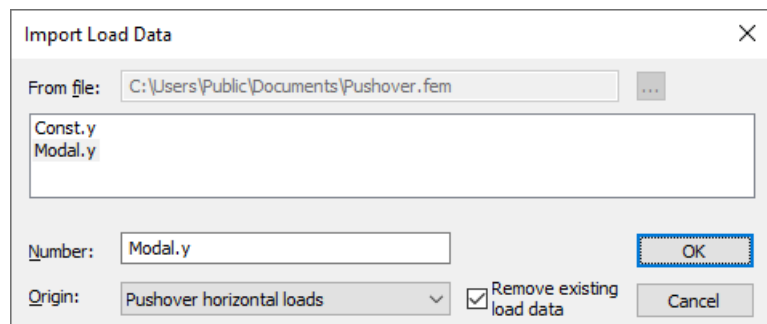
- *Effect of the masses*: In the example the structure should not swing in the x direction or around the y axis.
- *Eigenvalues*: Enough eigenvalues to get the basic model form in the y direction.
- *Soil acceleration*: In the example the masses were accelerated in the y-direction with $0.45 \cdot g = 4.41 \text{ m/s}^2$.
- *Coefficients of EN 1998-1:2013*: The elastic response spectrum used in the example is defined by the shown values (S , T_B , T_C , T_D , *damping measure* and γ_I).
- *Vertical distribution of the h-loads*: In the example a modal distribution of horizontal loads (*modal y*) is assumed. According to the standard, Section 4.3.3.4.2.2, additionally a *constant* distribution of the h-load (uniform acceleration response) is to investigate.
- *Control node*: By the standard the eigenmode at the control node (usually in the roof level) is to scale to 1. According to Section 4.3.3.4.2.3 and Annex B the control node is also used for calculating the equivalent mass m^* of an oscillator with a single degree of freedom, the displacement and capacity curves as well as the transformation coefficient Γ .

Note:

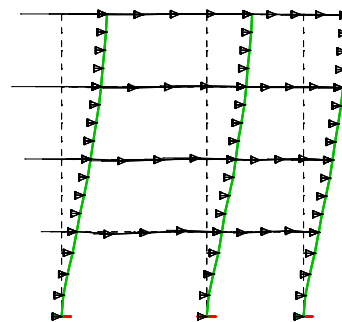
The dynamic calculation can only be performed if on the settings page *Statics* the *Nonlinear analysis* is switched off.

Nonlinear Structural Analysis

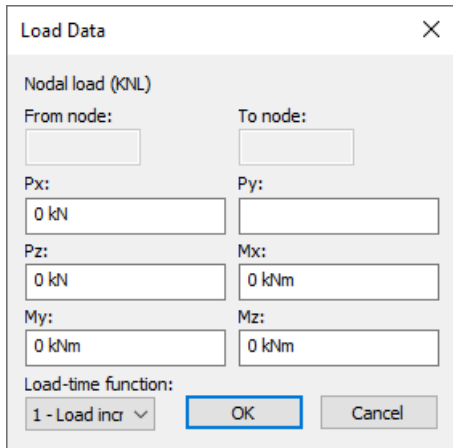
For the nonlinear pushover analysis, load cases are required which include both the static load components (G) and the horizontal load components (earthquakes) to be increased. The horizontal load components to be increased (*Modal.y* or *Const.y*) have already been determined within the dynamic calculation. They have to be assigned to load cases and the degree of load increase has to be defined. In general, the load components are initially defined in separate load cases and then summarized by a load group.



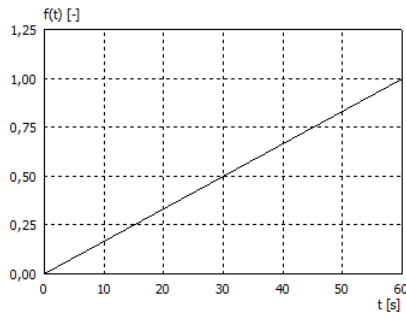
Import of horizontal static pseudo loads in a load case



Eigenmode 1 and imported loads (*Modal.y*)

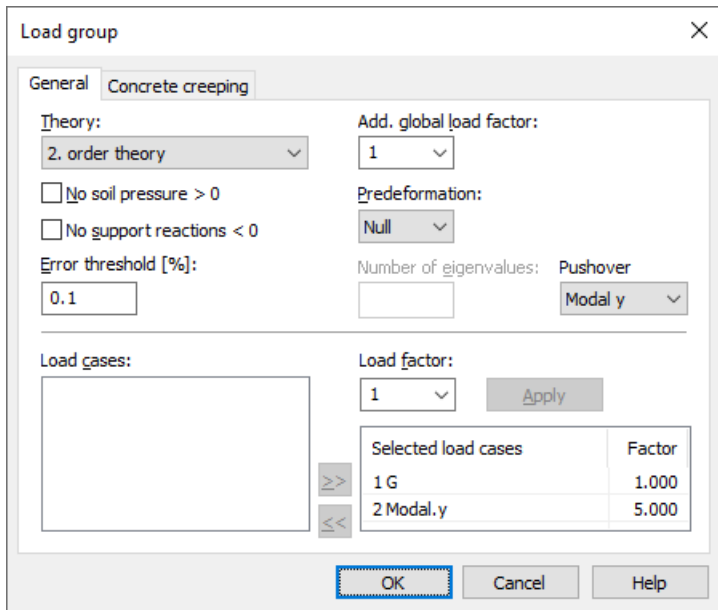


After import into a load case, the node loads still have to be provided with a load-time function, because only the horizontal node loads are to be increased in the context of the nonlinear analysis.



Example of the *load-time function*

The defined increase in loads is not significant for the arc length method. The vertical loads (G) should be constant in all load levels.



Load case 3: *Pushover, Modal.y*

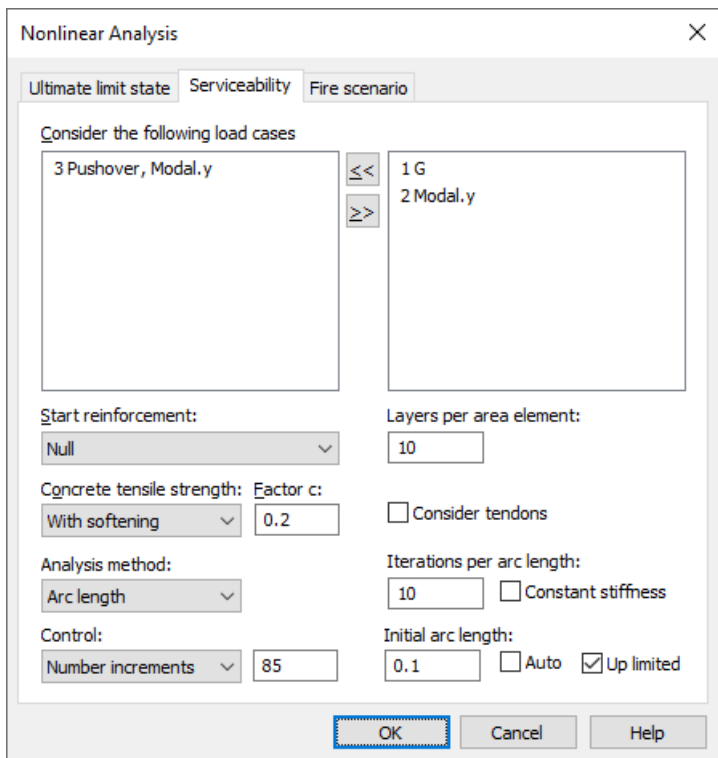
For the nonlinear pushover analysis, a load case 3 is defined by a load group in this example.

As the structure deforms greatly, the second-order theory is selected. To increase accuracy, the error threshold was set to 0.1 %.

In the earthquake situation, a characteristic G-load is generally assumed (load factor 1.0). The horizontal displacement of the control node should reach approximately 150 % of the target displacement according to Section 4.3.3.4.2.3.

In this example, the arc length method is used. The size of the h-loads reached depends heavily on the parameters of the arc length method in addition to the set factor for load case 2 *Modal.y*. The sign of the factor defines the load direction (here +y).

Furthermore, the *pushover* case matching the h-loads is to be selected (here *Modal.y*).

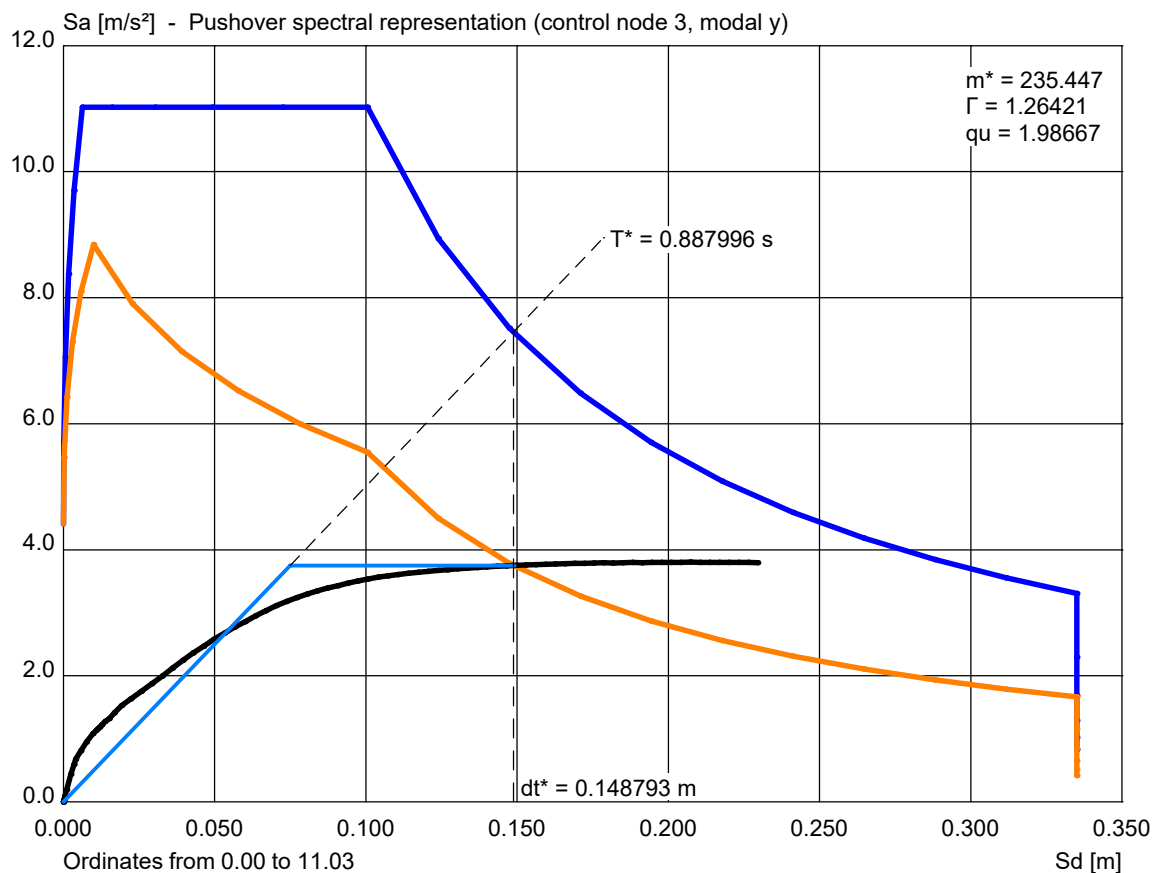
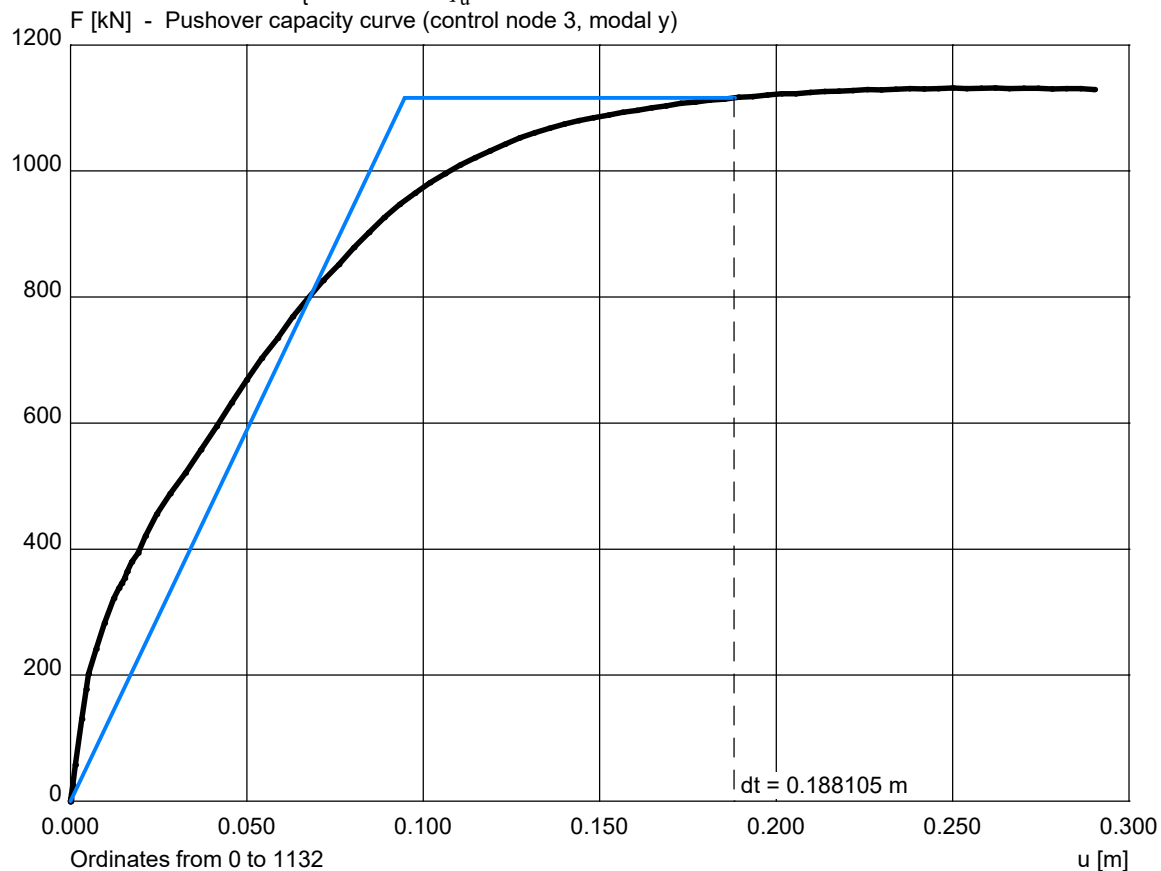


In the settings for *Nonlinear analysis / serviceability* in addition to the selection of the load case and the *arc length* analysis method the following settings have been made:

- *Start reinforcement*: If *Null* is selected, the basic reinforcement of the reinforcing steel layers of the cross sections is used.
- *Concrete tensile strength*: This option defines the behaviour of the concrete in the tensile zone for all concrete cross sections. The comparison calculation assumed a concrete tensile strength to be applied.
- *Control*: The number of increments of 85 selected here must in conjunction with the other parameters of the arc length method be chosen so that a sufficient horizontal displacement of the control node is achieved.
- *Iterations per arc length*: To prevent the arc length becoming very small, the number has been increased to 10.
- *Initial arc length*: This has been limited to the top to get enough intermediate results in the load-displacement curve.

Results

After the two calculation steps, in addition to the nodal deformations and internal forces per load level, the capacity curve in the multi-mass system and spectral representation is available. Their linearization is iterative according to EN 1998-1, Section B.5. The presented results are in very good agreement with the results of Matjaž Dolšek ($m^* = 227.9$ t, $\Gamma = 1.279$, $F = 1102$ kN, $T^* = 0.87$ s, $d_t^* = 0.146$ m, $q_u = 2.01$).



References

- Bathe, K. J.
 Finite Elemente Methoden (Finite Element Methods). Springer Verlag, Berlin 1986.
- DIN 4149:1981
 Buildings in german earthquake areas – Design loads, analysis and structural design of buildings.
 Publisher: DIN Deutsches Institut für Normung e.V., Beuth Verlag, Berlin 1981.
- DIN 4149:2005-04
 Buildings in german earthquake areas – Design loads, analysis and structural design of buildings.
 Publisher: DIN Deutsches Institut für Normung e.V., Beuth Verlag, Berlin 2005.
- DIN EN 1998-1/NA:2021-07
 National Annex – Nationally determined parameters – Eurocode 8: Design of structures for earthquake resistance –
 Part 1: General rules, seismic actions and rules for buildings.
 Publisher: DIN Deutsches Institut für Normung e.V., Beuth Verlag, Berlin 2021.
- Dolšek, M.
 OS Modeler – Examples of application, Version 1, University of Ljubljana, 2008
- EN 1998-1:2004 + AC:2009 + A1:2013
 Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.
 Publisher: CEN Europäisches Komitee für Normung, Brüssel, Beuth Verlag, Berlin 2013.
- Fajfar, P.; Fishing, M.
 N2-a method for non-linear seismic analysis of regular buildings.
 Proc.9th World Conf. on Earthquake Engineering, Vol.V, Tokyo-Kyoto, 2-9 August 1988, pp. 277-287.
 Maruzen, Tokyo, 1989.
- Fajfar, P.
 A nonlinear analysis method for performance-based seismic design.
 Earthquake Spectra, August 2000, Vol. 16, No. 3, pp. 573-592.
 Earthquake Engineering Research Institute (EERI), Oakland 2000.
- Hughes, T. J. R.
 The finite element method, Prentice-Hall Int. Ed., New Jersey 1987.
- Lecture Notes in Engineering
 Finite Element Analysis for Engineering Design. Springer Verlag, Berlin 1988.
- Link, M.
 Finite Elemente in der Statik und Dynamik (Finite Elements in Statics and Dynamics).
 Teubner Verlag, Stuttgart 1984.
- Meskouris, K.
 Baudynamik (Structural Dynamics). Verlag W. Ernst & Sohn, Berlin 1999.
- Müller, F. P.
 Baudynamik (Structural Dynamics), Betonkalender Teil II. Verlag Ernst & Sohn, Berlin 1978.
- OENORM B 4015:2007-02
 Eurocode 8: Design of structures for earthquake resistance –
 Part 1: General rules, seismic actions and rules for buildings –
 National specifications concerning ÖNORM EN 1998-1 and national comments.
 Publisher: Austrian Standards Institute, Vienna.
 Austrian Standards plus GmbH, Vienna 2007.
- Petersen C.
 Schwingungsdämpfer im Ingenieurbau, pp 269-277,
 Publisher: Maurer Söhne GmbH Co. KG, München 2001.
- Schwarz, H. R.
 Methode der finiten Elemente (Methods of Finite Elements). Teubner Studienbücher.
 Teubner Verlag, Stuttgart 1984.
- SIA 261:2020 Bauwesen
 Einwirkungen auf Tragwerke (Actions on structures). Swiss standard SN 505 261,
 Registered standard of the Swiss Association for Standardization SNV.
 Publisher: Schweizerischer Ingenieur- und Architektenverein, Zürich 2020.
- U.S. Nuclear Regulatory Commission (NRC)
 Regulatory Guide 1.92, Revision 3 –
 Combining Modal Responses and Spatial Components in Seismic Response Analysis –
 Office of Nuclear Regulatory Research.
 Publisher: U.S. Nuclear Regulatory Commission (NRC), NRC Washington D.C. 2012
- Zienkiewicz, O. C.
 Methode der finiten Elemente, Studienausgabe (Methods of Finite Elements, Study Edition).
 Hanser Verlag, München 1984.

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