Verband der
Chemischen Industrie e.V.
Wir gestalten Zukunft.

COMMENTARY TO THE VCI-GUIDELINE

THE SEISMIC LOAD CASE IN PLANT ENGINEERING

Design, dimensioning and construction
of structures and components in the chemical industry
based on DIN EN 1998-1

March 2022 – Rev. 03/23

This document is the English translation of the German document

ERLÄUTERUNGEN ZUM VCI-LEITFADEN
DER LASTFALL ERDBEBEN IM ANLAGENBAU

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The VCI represents the economic policy interests of more than 1,700 German chemical and pharmaceutical companies as well as German subsidiaries of foreign corporations towards politics, authorities, other sectors of the economy, science and the media. In 2021, the industry had a turnover of around 220 billion euros and employed over 466,000 people.

Overview of major changes to the commentary document on the VCI-Guideline 2012

1) Incorporation of the innovations from the National Annex to Eurocode 8 Part 1 (DIN EN 1998-1/NA:2021) updated in 2021 with regard to the description of the seismic action; in particular:

Section 5.1:	Zone-free description of the seismic action; maximum value (plateau) of the response spectrum $S_{aP,R}$ as input value for the calculation of the response spectrum (previously: a_{gR})
Section 5.2:	Soil factor S dependent on the ground condition at the site and on the spectral acceleration in the plateau range $S_{aP,R}$
Clause 5.3(1):	Omission of the conversion relationship between importance factor and return period
Clause 5.4(2):	Description of the vertical component of the seismic action

2)	Sections 1(4) & 5.1(2):	Note that even in areas of very low seismicity, proof of seismic safety may be required
3)	Section 3:	Extension of the definitions of terms by the term "components"
4)	Sections 4 and 7.2:	Notes on anchor design
5)	Section 4.c:	Extension of the notes on conceptual and structural design by further pictorial explanations
6)	Section 6.1.b:	Note on damping values and behaviour factors for silos and tank structures
7)	Section 6.2:	More detailed explanations on non-linear static calculation methods
8)	Section 6.4:	More detailed explanations on the simplified design equation for non-structural components
9)	Clause 7.2(1):	Verification of the functionality of safety-relevant elements at the ultimate limit state
10)	Clause 10(5):	Determination of seismic action for plants with limited remaining operating time
11)	Clause 10(6):	Note on possible non-linear static calculation methods for the assessment of existing facilities
12)	Entire document:	Editorial changes and additions

Changes within revision of March 2023:

1)	Section 5.2:	Reference to DIN EN 1998-1/NA/A1:2023
2)	Entire document	Editorial changes and additions

Table of Contents

Preliminary remark:

The present commentary to the VCI-guideline "The seismic load case in plant engineering" contains further explanations and background information and presents individual aspects of the guideline in more detail. The individual sections correspond to the sections of the guideline. For more in-depth study, please refer to the literature listed at the end of this commentary document.

1. Scope

Re: (1), (2) and (3) General information on the scope of application

No further explanations.

Re: (4) Application of the VCI-guideline in areas of very low seismicity

According to DIN EN 1998-1/NA D:2021 [28] NDP re. 3.2.1(5), cases of very low seismicity are those in which the equation $a_g \cdot S$ is not greater than $0.5\ m/s^2$, i.e. $\frac{S_{aP,R}}{2.5} \cdot \gamma_I \cdot S \leq 0.5\ m/s^2$. In plant engineering, this general criterion is not suitable for deciding whether a structure must be verified against seismic effects. This is because high seismic loads may occur even with very low ground accelerations in cases where structures exhibit unfavourable mass distributions, where structures have only a small wind projection area but a large mass, or where structures are not designed for horizontal loads (e.g. components inside buildings).

Therefore, following note 1 to DIN EN 1998-1/NA D:2021 [28] NDP re. 3.2.1(5), it must always be checked whether the stresses due to seismic action become decisive compared to the stresses due to wind action. This can at first be done roughly by comparing the base shear from $F_b = m_{total} \cdot (S_{aP,R} \cdot \gamma_I \cdot S)$ or $F_W = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref}$, respectively [12], [13]. As a result, a stability check may also be required if substances with a high risk potential (i.e. high importance factor according to section 5.3) are handled at a site with very low reference ground acceleration (cf. former seismic zone 0).

If the seismic forces are smaller than the wind forces, it is nevertheless recommended for precautionary reasons to at least apply section 4 of the VCI-guideline (conceptual and structural design).

2. Normative references

The documents explicitly stated in clause 2 (1) of the VCI-guideline are those to which the guideline refers directly or indirectly. In addition, other standards and directives may also contain information on the (seismic) design of facilities and in particular specific components of the chemical industry or related industries (e.g. DIN EN 13480, DIN EN 13445, etc. – see also the reference list at the end of this commentary document in section 11).

3. Terms and definitions

In facilities of the (chemical) industry, there are usually a large number of **components** in the sense of the VCI-guideline. Such components sometimes differ considerably in shape, size, mass, type of support, dynamic behaviour, function, importance, etc. What they have in common, though, is that they only support themselves and their contents. For this reason, they are considered as non-structural with regard to the building structure. Often, their failure in the event of an earthquake can pose a risk to people, the environment or the operational reliability e.g. due to the component's function, the substances handled or their location within the supporting structure.

Due to the diversity of plants, a list of possible **components** cannot be complete, but the following examples are noted for clarification: (pressure) vessels, aggregates, appliances, pumps, motors, (small) heat exchangers, control cabinets, pipes, ventilation ducts, cable ducts, suspended ceiling elements, and many more.

4. Conceptual and structural design

4.a Supporting structures of facilities

Re: (1), (2) and (3) Notes on constructional design of load-bearing structures

When designing facilities in seismic zones, it may be sensible to consider certain design principles already in the basic design of the facilities in order to avoid the typical damages known from past earthquakes. In principle, the entire load-bearing structure should participate evenly in the load transfer. Accordingly, favourable facility designs are as regular and consistent as possible in terms of their mass and stiffness distribution, both in plan and in elevation of the structure.

Regularity in plan

Compact plans are favourable (Figure 4.1). Dynamically independent structural parts and divided plan areas should be separated from each other by joints. Irregular plans with re-entrant corners should be avoided as well as large or unfavourably distributed penetrations in floor slabs, as they strongly impair the load transfer through the stiffening plates. In order to reduce torsional vibrations, the lateral bracing elements should be arranged close to the outer edges of the structure. If the centre of mass (originating from components and distributed loads) and the centre of stiffness are close to each other, the torsional susceptibility of the structure is reduced.

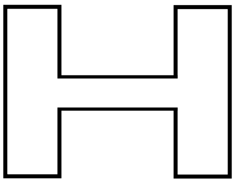
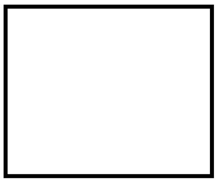
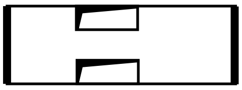
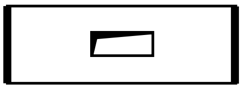
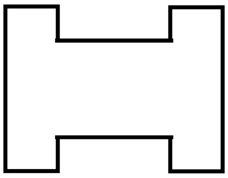
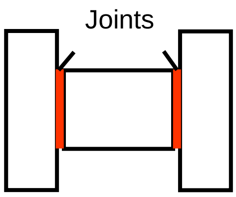
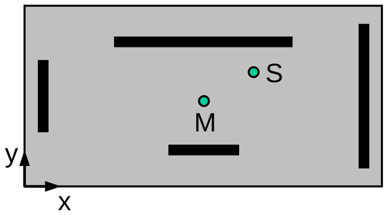
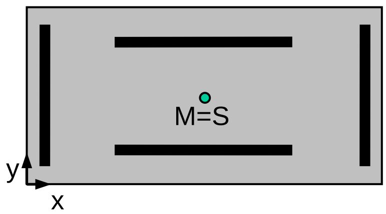
Plan shapes	***Unfavourable***	***Favourable***
Choose compact plans		
Arrange recesses in floor panels sensibly		
Joints Separate structural parts with joints		
Eccentricities Position centre of stiffness and centre of mass close to each other		

Figure 4.1: **Regularity in plan**

In DIN EN 1998-1 [27] clause 4.2.3.2(6), conditions are given for the assessment of the regularity in plan that refer to the distance e_0 between the centre of stiffness and the centre of mass, and to the torsional radius r of each storey, respectively.

These conditions are repeated in equation (4.1) and the context is explained in more detail below.

$$
\begin{array}{llll}
e_{0x} \leq 0.3 \cdot r_x & \text{and} & r_x \geq l_s & \text{or} \\
e_{0y} \leq 0.3 \cdot r_y & \text{and} & r_y \geq l_s & \text{respectively}
\end{array}
\qquad (4.1)
$$

where $e_{0x(y)}$ = Distance between the centre of stiffness and the centre of mass of the considered storey in x- and y-direction, respectively (in each case ⊥ to the considered seismic action direction; cf. Figure 4.2)

$r_{x(y)}$ = Square root of the torsional radius in x- or y- direction (cf. equation (4.2))

l_s = Radius of inertia of the storey mass in plan (cf. Figure 4.3)

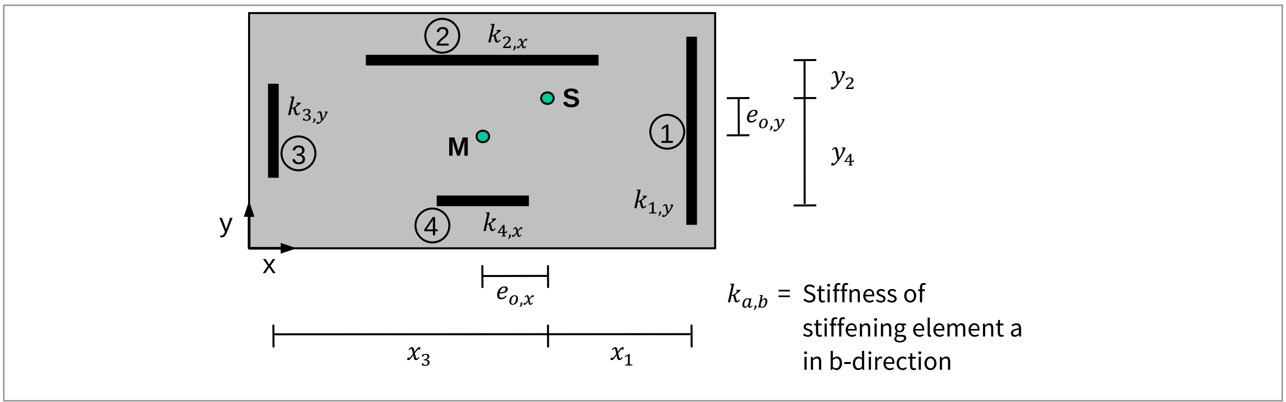


Figure 4.2: Terminology for the assessment of regularity in plan

The torsional radius used in condition 1 of equation (4.1) (r_i is the square root of the actual torsional radius r_i^2) corresponds to the ratio between the torsional stiffness and the lateral stiffness of the floor under consideration. If the centres of stiffness of all floors are approximately on a vertical straight line, the torsional radius can be determined according to equation (4.2).

$$r_y = \sqrt{\frac{k_T}{k_x}} = \sqrt{\frac{\sum_{i=1}^{n} k_{i,x} \cdot y_i^2 + \sum_{j=1}^{m} k_{j,y} \cdot x_j^2}{\sum_{i=1}^{m} k_{i,x}}} \qquad (4.2)$$

where r_y^2 = Torsional radius of the considered floor for earthquake direction x

k_T = Torsional stiffness of the floor

k_x = Translational stiffness of the floor in the direction of seismic action

$k_{i,x(j,y)}$ = Translational stiffness of the stiffening element i positioned in x-direction i.e. parallel to the seismic action (or stiffening element j positioned in y-direction, i.e. perpendicular to the seismic action)*

$n \ \ (m)$ = Number of bracing elements in direction of the seismic action (n) or perpendicular to the direction of seismic action (m)

$y_i \ \ (x_j)$ = Distance between the centre of stiffness and the middle of stiffening element i or j, respectively; in each case measured perpendicular ⊥ to the bracing plane either positive or negative

* For bracing walls that transfer the applied horizontal load via bending, the wall stiffness can be determined from the moment of inertia of the wall cross-section: $k_{i,x} = E_i \cdot I_{i,y}$, with $I_{i,y}$ being the moment of inertia of wall element i about the global y-axis vertical to the seismic direction x under consideration. If shear deformations are taken into account, the wall stiffness must be reduced (e.g. according to [70]).

The radius of inertia l_s is equal to the distance from the axis of rotation of a body at which the mass m must be imagined as concentrated mass circulating without changing the moment of inertia of the body (Figure 4.3). For a rectangular surface of the dimension $L \times B$ with evenly distributed mass, the following expression applies:

$$l_s = \sqrt{\frac{L^2 + B^2}{12}} \tag{4.3}$$

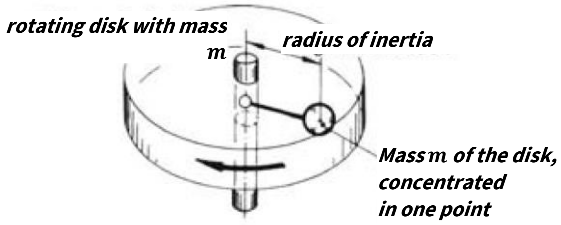
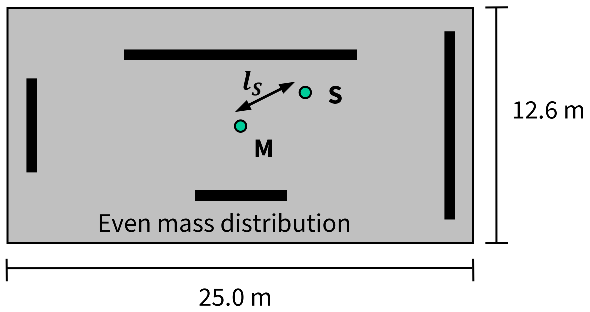


Figure 4.3: Graphical explanation of the radius of inertia [56]



$$l_s = \sqrt{\frac{L^2 + B^2}{12}} = \sqrt{\frac{25.0^2 + 12.6^2}{12}}$$

$$l_s = 25.26\ m$$

Figure 4.4: Example of the radius of inertia for a rectangular plan

Regularity in elevation

Load-bearing structures that are rather compact also with respect to their elevation are preferable to slender structures. The stiffness distribution should be regular in elevation, (sudden) changes in stiffness should be avoided unless they correspond to the distribution of the masses. In particular, soft storeys and short columns should be avoided. A horizontal or vertical offset of columns or floor slabs leads to large additional stresses in the structure and should be avoided (Figure 4.5).

Connections between adjacent load-bearing structures and structural parts divided by joints must be designed to be flexible, as rigid connections can lead to damaging interactions.

When large masses are located on upper floors horizontal loads result in high stresses on the supporting structure and the foundation. It should therefore be the aim of process-engineering design to arrange heavy (individual) components on the lower levels of the supporting structure as far as possible.

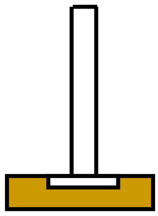
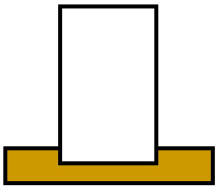
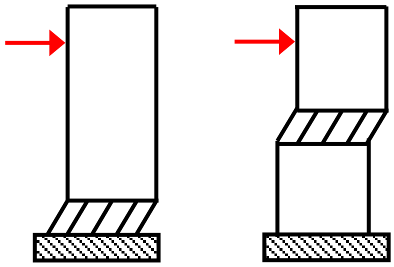
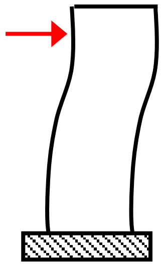
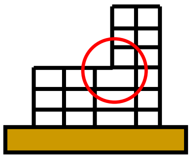
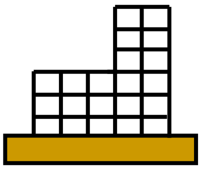
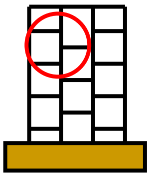
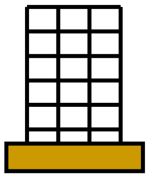
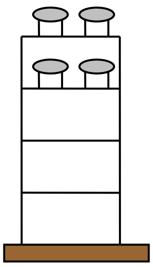
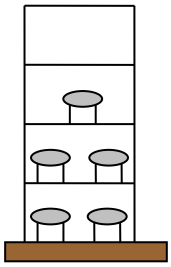
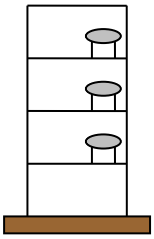
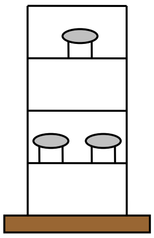
Stiffness distribution	***Unfavourable***	***Favourable***
Prefer compact elevations		
Avoid soft floors		
Stiffness distribution	***Unfavourable***	***Favourable***
Avoid misalignment (offset) of columns		
Avoid offset of floor slabs		
Mass distribution		
Arrange heavy components at low heights, if possible		
Arrange heavy components as centrally or symmetrically as possible		

Figure 4.5: **Regularity in elevation**

Bracing systems

Bracing systems must be arranged and connected by floor slabs in such a way that a direct horizontal load transfer is ensured and torsional vibrations are avoided. The bracing systems of the global structure should run continuously from the top elevation of the structure to the foundation level. It should be noted that columns at building corners must bear increased loads in cases where they are part of two bracing systems in perpendicular building directions. Thus, bracing in two axes at the same corner should be avoided (Figure 4.6).

If requirements of process engineering necessitate the staggered arrangement of bracing elements (e.g. shear walls), it must be ensured by appropriate dimensioning of the intermediate areas that the transfer of the seismically induced (horizontal) loads is guaranteed through all structural parts involved down to the foundation and subsoil. It might be favourable to arrange additional bracing elements to reduce susceptibility to torsion.

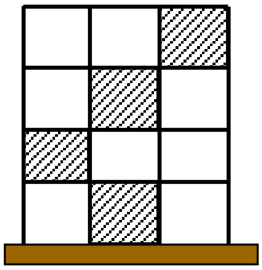
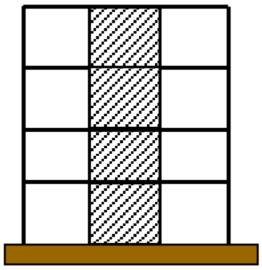
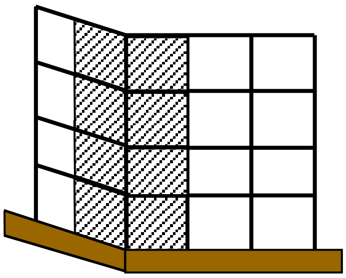
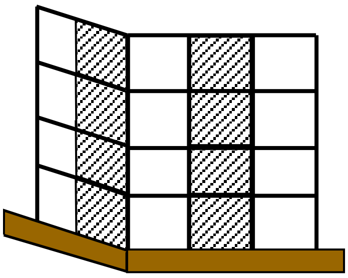
Bracing systems	***Unfavourable***	***Favourable***
Vertically continuous bracing		
Avoid bracing in two axes at building corners		

Figure 4.6: Bracing systems in elevation

Dissipative areas

For an economic design under seismic loads, structures should be designed in such a way that they are able to dissipate part of the induced energy. This can be achieved by capacity design, in which the structural engineer defines dissipative areas at which plastic joints will develop under the design seismic action. The dissipative areas are to be arranged in such a way that kinematic structural failure does not occur after the formation of all anticipated design joints.

In addition, they must have sufficient plastic deformation capacity to ensure the global deformation capability of the structure. The non-dissipative areas of the structure are designed with an increased resistance so that they remain in the elastic range as anticipated even in the case of overstrength in the plastic joints. Thereby, a plastic mechanism of the building is predetermined by the structural engineer allowing the structure to develop large deformations. The additional design and detailing requirements going along with the capacity design for various types of construction are regulated in DIN EN 1998-1 [27] in sections 5 to 9.

Alternatively, seismic safety can also be ensured by maintaining a high load-bearing resistance (Figure 4.7). In this case, the structure remains essentially in the elastic range for the design earthquake, and the components show only limited non-linear behaviour. This "quasi-linear" elastic design makes sense for the seismic action predicted in Germany and is recommended as it is easy to apply. However, for countries with high seismic actions, this approach leads to uneconomical design of structures.

In general, a high degree of static indeterminacy promotes redundant structural behaviour, e.g. the stability of the overall structure is less prone to a local failure.

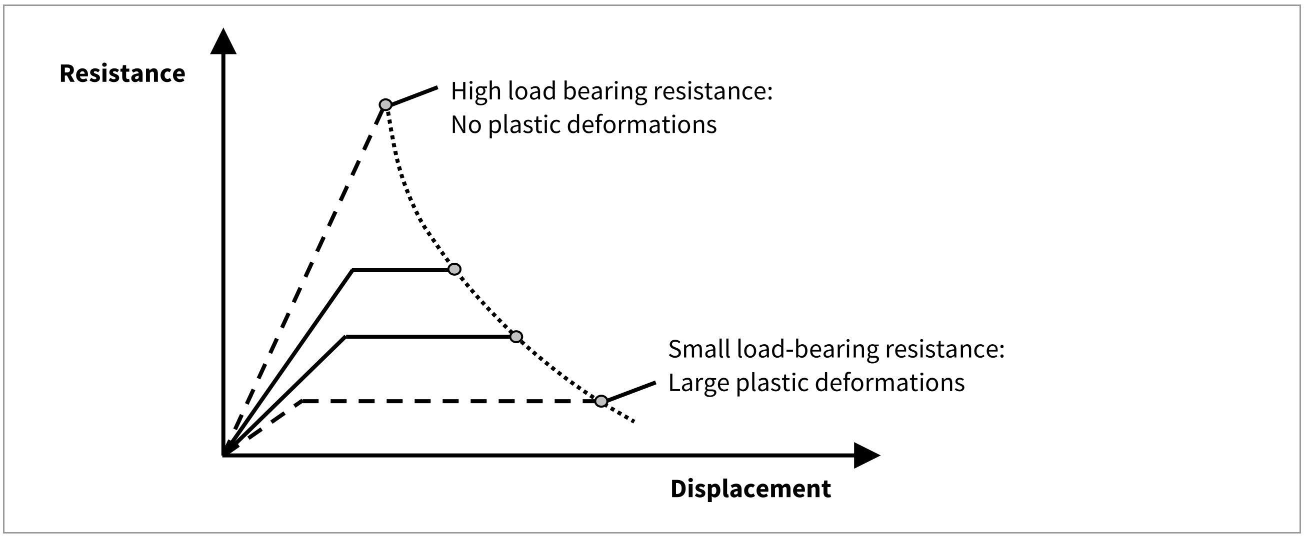


Figure 4.7: Concepts for economic structural design

Expansion joints

When arranging expansion joints, the following design principles must be followed:

- Expansion joints must pass through the entire building structure. This applies both to the load-bearing structures such as floors and walls and to their cladding (plaster layers, floor screed, etc.).
- The pathway of joints should be kept as simple as possible; protrusions and corners should be avoided. Unclear pathway of joints can obscure the joints' purpose and are therefore often unintentionally rendered malfunctioning by the construction workers.
- The joint design should be as simple as possible, and the different joint functions should be assigned to different structure parts (e.g. for force transmission, deformability, sealing, fire protection).

- Ensuring ductile structural behaviour leads to larger deformations in the event of an earthquake; that must be taken into account when planning and designing joints.
- Movements that are permitted by a joint must be pursued consistently. The supporting structure must not be damaged by the movements.
- The detailing of joints must be planned carefully and monitored during execution. Damage or functional flaws that occur during construction can hardly be repaired afterwards.

Re: (4) General notes on the design of foundations

Mixed foundation systems (e.g. pile foundation combined with slab foundation) should be avoided within a structure (cf. DIN EN 1998-5 [34] clause 5.2 (1)P). Mixed foundation systems are, however, tolerable for dynamically independent parts of the structure. Additional rules for the conceptual design of foundations can be found in DIN EN 1998-5 [34] section 5.1 and clause 5.2 (2).

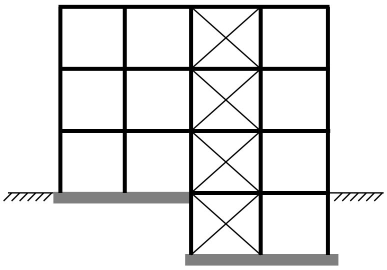
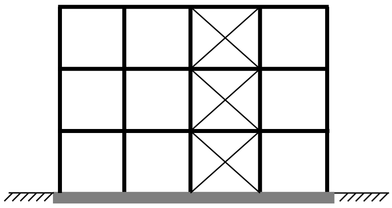
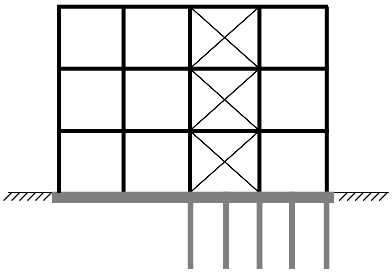
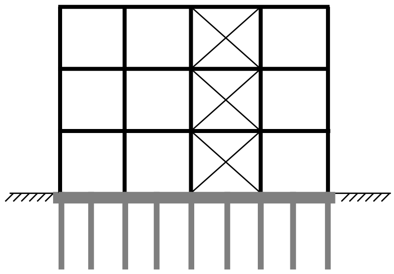
Foundation systems	***Unfavourable***	***Favourable***
Different foundation depths		
Combined pile and slab foundation		

Figure 4.8: Foundation systems

4.b Free-standing vessels, silos, tanks and process columns

Re: (1) Notes on constructional design of tanks, vessels and silos supported at grade

The seismic loading on free-standing tanks, vessels and silos strongly depends on the type of support. In case of a ring support directly mounted to the foundation, high seismically induced stresses occur at the base of the wall shell. The combination of vertical compressive stresses, ring tensile stresses and high shear stresses can lead to elastic-plastic failure in the base area, also known as "elephant foot buckling". This type of failure or combination of actions must be considered when designing the shell. In the case of shells with decreasing wall thickness over height, it may be necessary to carry out such verifications not only in the direct base area, but also at a change of shell thickness. In order to reduce buckling defects, it is recommended to provide even anchoring along the entire shell circumference. In the case of unanchored structures, uplift and/or sliding can cause damage to joints and connected piping.

Re: (2) Notes on constructional design of elevated tanks and silos

In the case of elevated vessels, silos or tanks, the interaction with the substructure must be taken into account, as this has a decisive influence on the dynamic behaviour. In particular, overturning and thus loss of overall stability must be prevented; the risk of overturning is particularly high when a large mass has a high centre of gravity.

For process columns, an even distribution of mass and stiffness along the height should be achieved, as far as feasible by process engineering, in order to avoid an unfavourable effect of higher eigenmodes. If possible, the process components should have low eccentricities in plan in order not to activate additional torsional vibrations due to mass eccentricities.

Re: (3) Design of anchors

Substructures and elevated supports of components are allowed to dissipate part of the energy induced by the earthquake, e.g. by plastic deformations. On the other hand, their connections to the foundation or to the supporting structure and the anchors, respectively, should be designed in such a way that they behave linear-elastically (non-dissipative) in the event of an earthquake. Thereby, possible overstrength in the anchored component must be taken into account (see also DIN EN 1992-4 [16] clause 9.2 (3) b) and clause C.3 (2)). For the mathematical handling of this requirement, see the explanations to clause 7.2.b (4).

Re: (4) Ductility of pipe connections

No further explanations.

4.c Non-structural components and pipes within the supporting structure

From (1) till (5) Notes on constructional design of non-structural components

Non-structural components (mechanical equipment, vessels, pipes, facade elements, etc.) can be exposed to high seismically induced forces due to their large masses and their high position. Their anchoring must be selected and dimensioned in such a way that the seismic loads can be transferred safely into the supporting structure. For most components, the horizontal seismic action is more critical than the vertical seismic action. This is because the components are usually thoroughly designed for (vertical) gravity, but often not for horizontal actions. However, depending on the design and fastening, vertical seismic actions can also have a significant influence (e.g. when the component is fastened to a wall in a cantilevered manner). By following simple design rules for the erection and anchoring of non-structural components, typical earthquake damage can be avoided with little effort.

With increasing height above grade, the accelerations within the structure due to seismic ground motion generally increase. Accordingly, components at upper elevations are generally exposed to larger seismic action than those at lower elevations of the facility. Components that are located strongly off-centre can be subjected to larger seismic action than components located centrally or symmetrically within the structure due to torsional vibrations of the structure.

Components with a large mass compared to the mass of the load-bearing structure can have an unfavourable influence on the global vibration behaviour. Therefore, they should be arranged at a low height and centrally or symmetrically within the structure, as far as feasible with respect to process-engineering requirements.

Aggregates and vessels directly anchored

Often, friction between the component support and the floor slab is assumed to ensure the component's permanent position. In the event of an earthquake, however, this can lead to slipping of the component. Therefore, it is generally recommended to secure the position by constructional means (Figure 4.9).

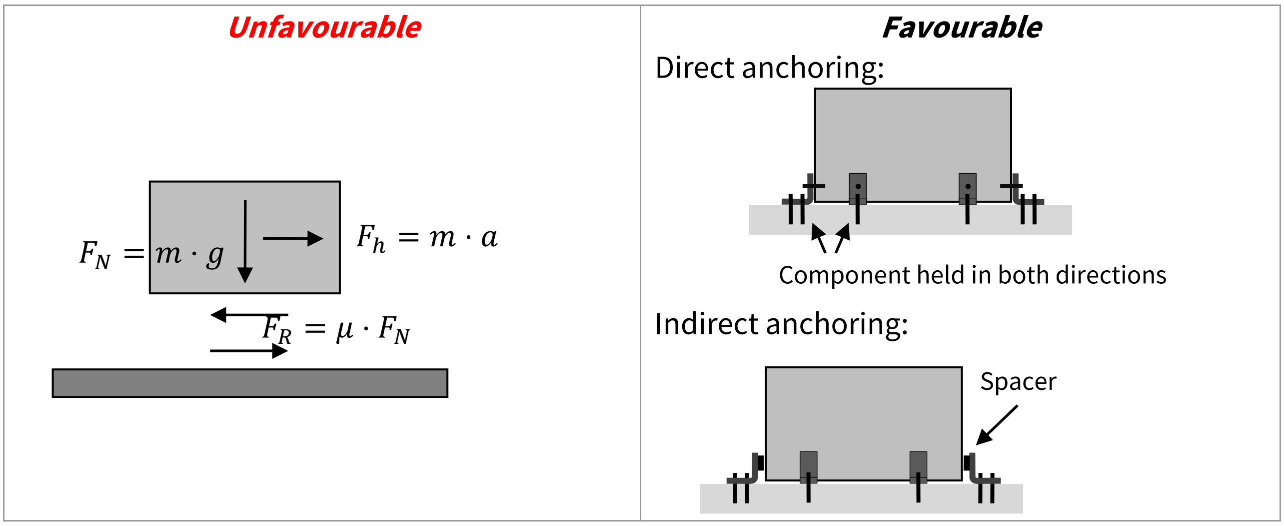
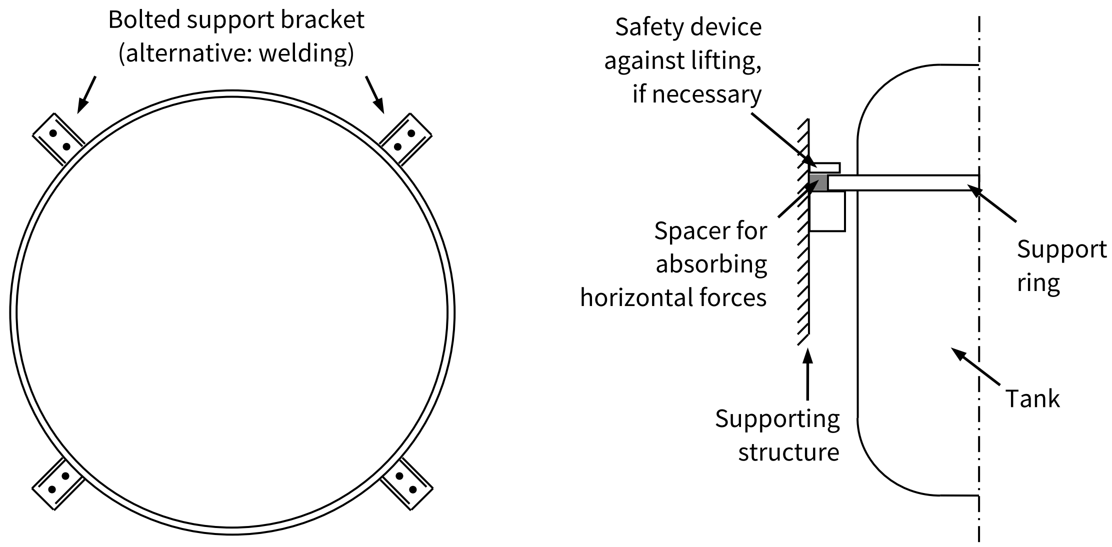


Figure 4.9: Anchoring of components located directly on the floor

Vessels and equipment on brackets or support rings should also be firmly connected to the supporting structure. This can be achieved either by welded or bolted connections or by means of spacers to avoid horizontal movement (Figure 4.10). Apart from that, the professional anchoring is to be carried out according to the relevant guidelines.



a) b)

Figure 4.10: Exemplary storage of suspended vessels;
a) Bracket support; b) Support ring with spacer

Structures with a small contact area and a high centre of mass must be anchored particularly carefully for seismic action (Figure 4.11), so that subsequent damage (e.g. tearing-off of pipes) can be ruled out. The evaluation of damage reports has shown that falling down of insufficiently secured heavy elements is the most frequent and most significant cause of damage [44]. This applies not only to falling equipment or vessels, but also to architectural non-structural components (facade elements, parapets, interior and exterior walls, etc.).

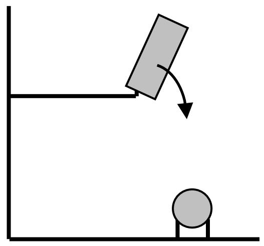


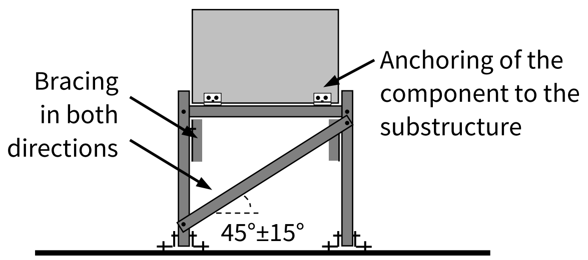
Figure 4.11: Overturning and falling down of components

Tank-type vessels should be firmly anchored to the base along the entire vessel circumference, as even a slight lifting of the vessel can cause the typical damage cases "elephant foot buckling" and "diamond shape buckling" in the vessel shell (cf. section 4.b).

Elevated aggregates and vessels

Components that are elevated must be adequately braced in all directions. A common damage pattern is buckling or bending of the bracing elements of the support. The bracing should be mounted at an angle of 45° ± 15° to the vertical (Figure 4.12 a). Rigid base points of column supports can be reinforced with stiffeners (Figure 4.12 b). It can also be useful to reinforce vessel rings, for example made of GRP, at the connection points of the base supports, as the vessel shell is subject to high local stresses in this area (Figure 4.12 b).

Slender components with a high centre of mass are at risk of overturning in the event of an earthquake, so that special care must be taken regarding their adequate anchoring to the foundation or to the supporting structure, respectively.



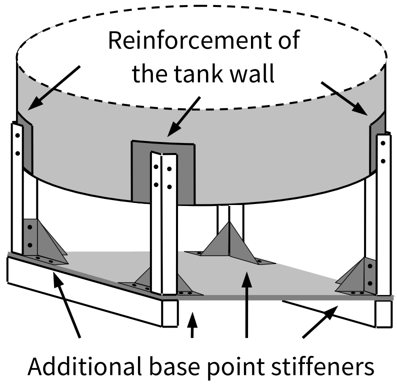


Figure 4.12: **a) Diagonal bracing of elevated components**
b) Reinforcement of column bases of elevated components

In the case of horizontal tanks, attention must be paid to stiffening the substructure both in the longitudinal and transverse direction of the tank. Depending on the shape of the saddle and the scale of the decisive seismic action, additional measures for stiffening may be necessary. Horizontal vessels in saddles with temperature-induced movement must be secured against slipping within the saddle supports (Figure 4.13).

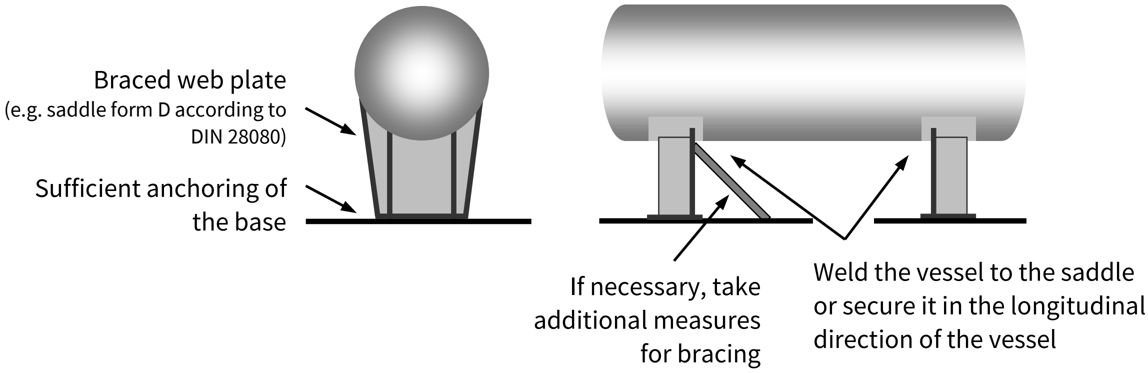


Figure 4.13: **Horizontal vessel; bracing in longitudinal and in transverse direction**

When anchoring tanks designed according to AD 2000 codes of practice ([2]-[7]), it must be ensured that the anchoring complies with the boundary conditions specified in these codes. In the case of vessels on (pipe) feet, for example, rigid clamping to the foundation / supporting structure must be ensured by arranging the anchors accordingly (Figure 4.14).

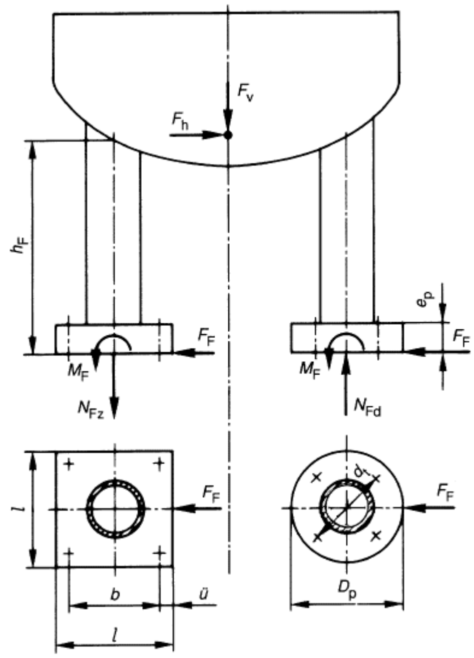


Figure 4.14: Base point design for upright vessel on (pipe) feet according to AD 2000 S3-3

Suspended vessels and aggregates

Components suspended from platforms / floors or other parts of the building structure swing or shake more than components that are mounted upright, and vibrations tend to build up. Therefore, sufficient bracing must be ensured. Furthermore, a sufficient distance to adjacent components must be ensured or the movement must be limited by spacers to prevent collision. Figure 4.15 and Figure 4.16 show favourable designs and arrangements of suspended components.

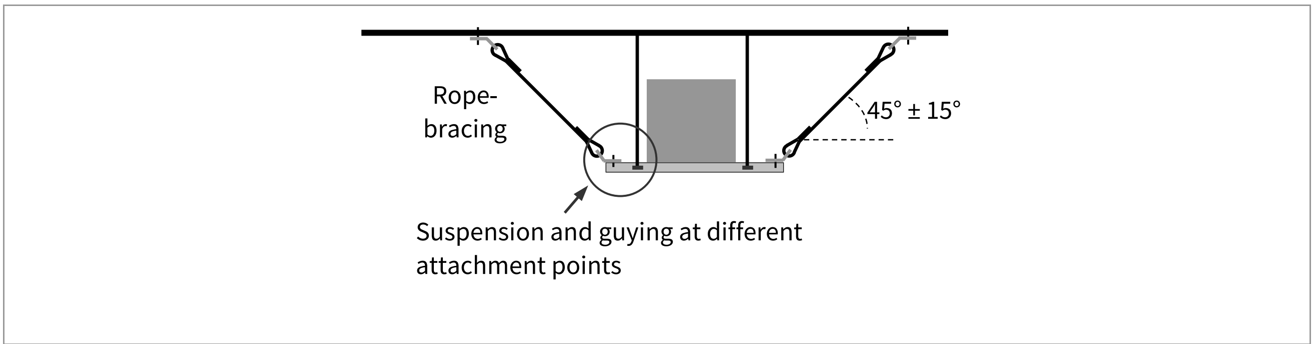


Figure 4.15: Bracing of suspended individual components

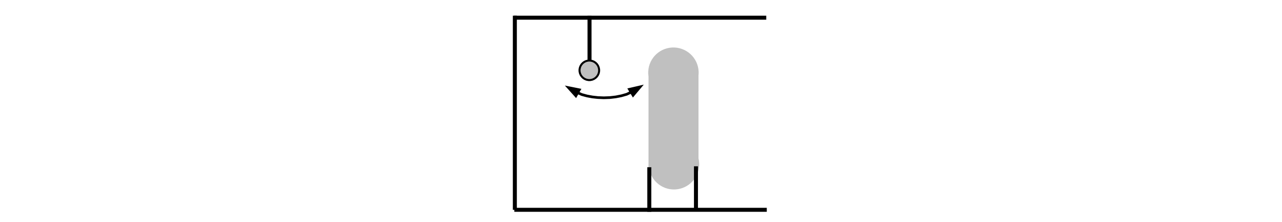


Figure 4.16: **Distance to adjacent components**

Pipes and (cable) ducts

Suspended or elevated pipes and (cable) ducts must be sufficiently braced in the longitudinal and transverse direction. Bracing is particularly necessary where relatively large masses or equipment such as pumps or similar are installed in the course of the pipe.

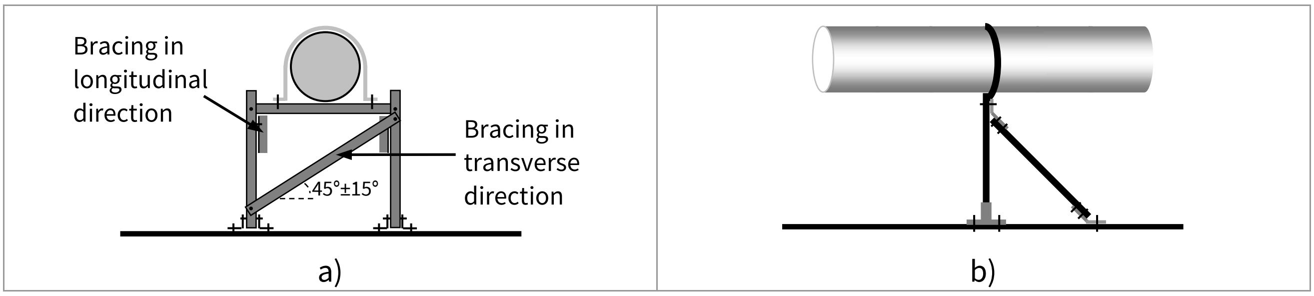


Figure 4.17: **Positioning of bracing a) in transverse direction and b) in longitudinal direction for elevated pipes and (cable) ducts**

The bracing should be positioned at an angle of 45° ± 15° to the vertical [42] (Figure 4.17 a). Steel profiles can be used for this purpose, whereby bracing on one side is sufficient (Figure 4.17 b). The choice of the bracing system is, in principle, up to the design engineer, but it must be made for each pipe section/segment consistently. Figure 4.18 a shows a reasonable bracing system for suspended pipes and Figure 4.18 b shows an example for a duct.

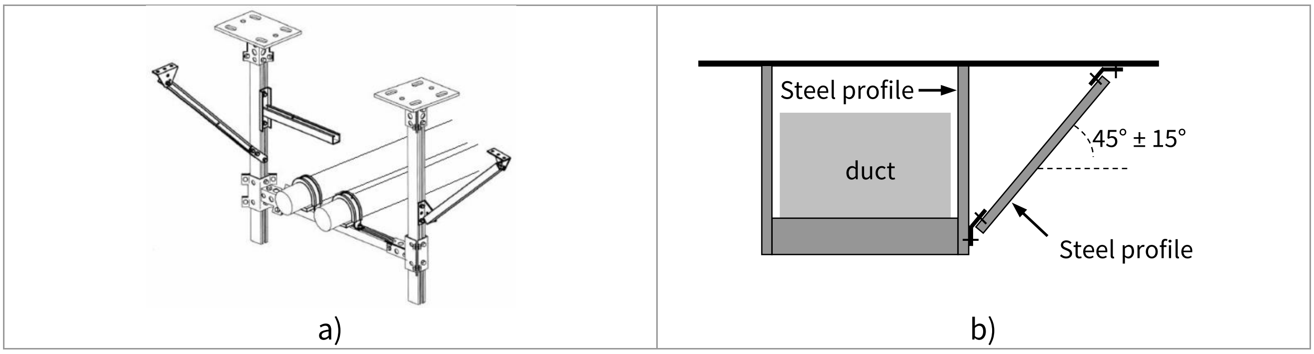


Figure 4.18: **Positioning of bracing for suspended pipes**

Pipes must be able to withstand relative displacements between components that swing differently. This can be achieved by expansion bends, elastic pipe connections and/or a suitable course of pipe routing and suspensions (Figure 4.19). Critical areas are found where expansion joints of the supporting structure are bridged or where large swinging masses determine the deformations. If pipes are connected to flexibly mounted equipment (e.g. rotating machines), care must be taken to ensure sufficient space for movement of the pipe.

Rupture of pipe lines that carry hazardous substances must always be prevented by taking appropriate measures.

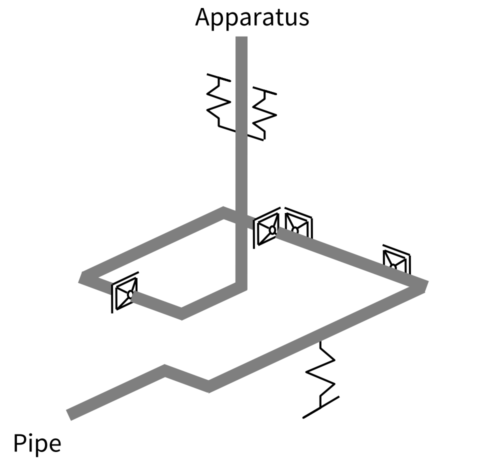


Figure 4.19: Flexible connections of pipes to components

Re: (6) Design of anchors

The load-bearing capacity of anchors in reinforced concrete or masonry is dependent on

- the size and the number of anchors,
- the anchor length,
- the distance between the individual anchors,
- the distance of the anchors to the nearest concrete or masonry edge, and
- the condition of the material (especially in the existing building).

The required minimum dimensions can be found in the certificates of the anchor manufacturers. Mechanical anchors generally require larger anchor spacing and edge distances than chemical-based adhesive anchors. Certain anchor types may be completely excluded from use in seismic areas due to poor dynamic properties. In this regard, the manufacturer's must be taken into account. The design of anchors in concrete structures is regulated by DIN EN 1992-4 [16].

Fasteners to steel elements are usually made with screws or bolts. Just like the fasteners, the base plates must be able to bear the seismically induced loads. The required verifications for connections in steel constructions are given in DIN EN 1993-1-8 [18].

Plastic deformations should only develop in the substructure / elevation frames of the non-structural component, but not in the fasteners (see also explanations to clause 4.b (3)). This means that the decisive actions on the anchors are determined with a behaviour factor $q = 1.0$ for anchor verifications (see also DIN EN 1992-4 [16] Annex C.3).

Clamp-connections intended as fixed supports can loosen under dynamic loads and may become a sliding support. For this reason, a sufficiently large support surface must be provided for clamped connections in order to prevent the support from slipping off; otherwise the clamped connection must be secured against untightening by suitable measures.

In principle, the manufacturer's instructions for anchoring and fastening systems must always be followed.

Re: (7) Infill masonry

As seen in numerous previous earthquakes, masonry infill walls are rather susceptible to damage due to seismic loads,. This is mainly due to the following properties of masonry infill walls during seismic loading (see also [54]):

- While frames form a relatively soft and rather ductile load-bearing structure, masonry walls, in contrast, are very stiff and brittle. At the beginning of an earthquake, the infill masonry takes over the full seismic action. However, they can transfer horizontal forces basically only by forming compression diagonals. Due to the large inclination of the compression diagonal, this quickly leads to sliding in a horizontal bed joint and, thus, to failure of the masonry.
- After the failure of the infill masonry, the frame columns experience additional stresses (in particular due to shear forces) as a result of the reciprocal displacement of two wall parts (the sliding in a horizontal bed joint described above), which can lead to shear failure of the adjacent frame columns.
- Partial infill of frames, such as those created by bricking up window parapets (Figure 4.20), are particularly unfavourable, as the effect of "short columns" is created here.
- In the case of loads perpendicular to the wall axis (out-of-plane loading), masonry infill panels can collapse easily (Figure 4.21). This is because infill walls typically carry little vertical loads that are too small to exceed / superpose the tensional stresses during out-of-plane loading. Additionally, infill walls are often not held laterally by the frame.

Notes on the constructional design and numerical verifications of masonry infill walls can be found in DIN EN 1998-1 [27] sections 4.3.6, 5.9, and 6.10.3.

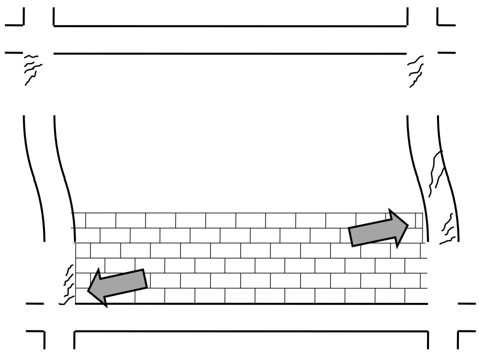


Figure 4.20: Additional load on the frame due to partial infill (according to [54])

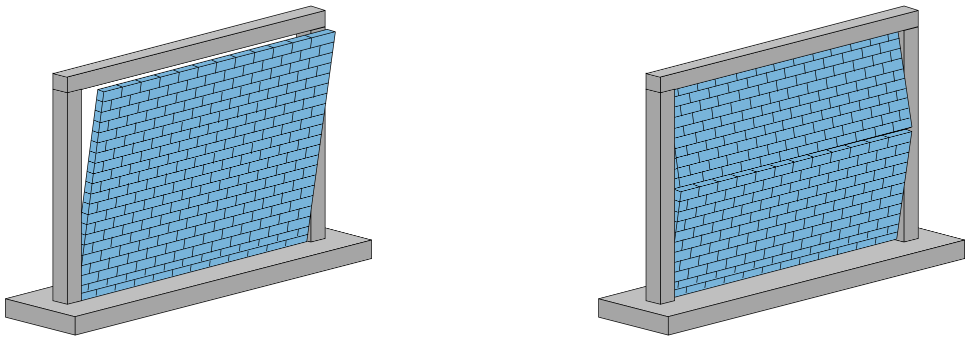


Figure 4.21: Typical failure modes of masonry infill walls due to out-of-plane loading

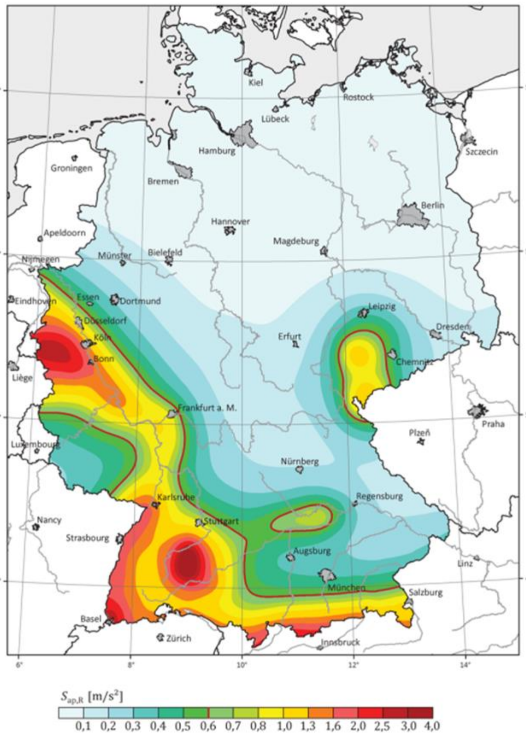
5. Seismic action

5.1 Seismic action at the plant site

Re: (1) Maximum spectral response acceleration as input value for determining the acceleration response spectrum

In DIN EN 1998-1/NA:2021 [28], the decisive seismic action in Germany is no longer specified by assigning a site to a seismic zone, but by site-specific values that are based on probabilistic hazard analyses for the whole of Germany. This eliminates the unrealistic sudden increase in ground acceleration that previously appeared at boundaries of seismic zones. Accordingly, a specification of intensity intervals is no longer given in the national annex.

Figure NA.1 of DIN EN 1998-1/NA:2021 shows the spatial distribution of the relevant hazard parameter for the reference return period $T_{NCR} = 475\ a$ as a schematic sketch (Figure 5.1). However, this graphical representation is not suitable for the definition of the relevant seismic action at a specific site because the map scale is too large. Instead, the maximum spectral response acceleration (plateau value of the acceleration response spectrum) $S_{aP,R}$ is to be taken from the normative digital annex of the standard. This normative digital standard annex contains, within the file "SaPR.csv", the maximum spectral response accelerations for ground condition A-R (rock) and $T_{NCR} = 475\ a$. The values are given for a coordinate grid of 0.1 degrees latitude and longitude distributed uniformly over the map of Germany. In order to determine the seismic action at the plant site or at a certain address, respectively, the gridded spectral values must be interpolated from the nearest reference points of the calculation grid to the plant site. The values of the .csv file mentioned above result from extensive probabilistic hazard analyses for the whole of Germany[1] and are the basis of Figure NA.1 of DIN EN 1998-1/NA:2021 [28]. Details on the scientific background of the data can be found in the accompanying information to the normative digital annex of the standard.



475a (Figure 5.1): **Spatial distribution of the maximum spectral response acceleration $S_{aP,R}$ (plateau value of the acceleration response spectrum) for ground condition A-R and return period $T_{NCR} = 475\ a$ (DIN EN 1998-1/NA:2021 [28] Figure NA.1)**

[1] The plateau values $S_{aP,R}$ in the normative digital annex are each calculated as the mean value from the results of a probabilistic seismic hazard analysis. In an additional informative digital annex, the spectral values are also given as quantile values with 16 %, 50 % (median) and 84 % probability of exceedance.

Another significant change in the determination of seismic action is the specification of the input value for the calculation of the response spectrum: While previous code editions referred to the reference peak ground acceleration on rocky ground condition as input value ($\boldsymbol{a_{gR}}$), the relevant parameter is now the maximum spectral response acceleration (plateau value of the response spectrum) $\boldsymbol{S_{aP,R}}$.

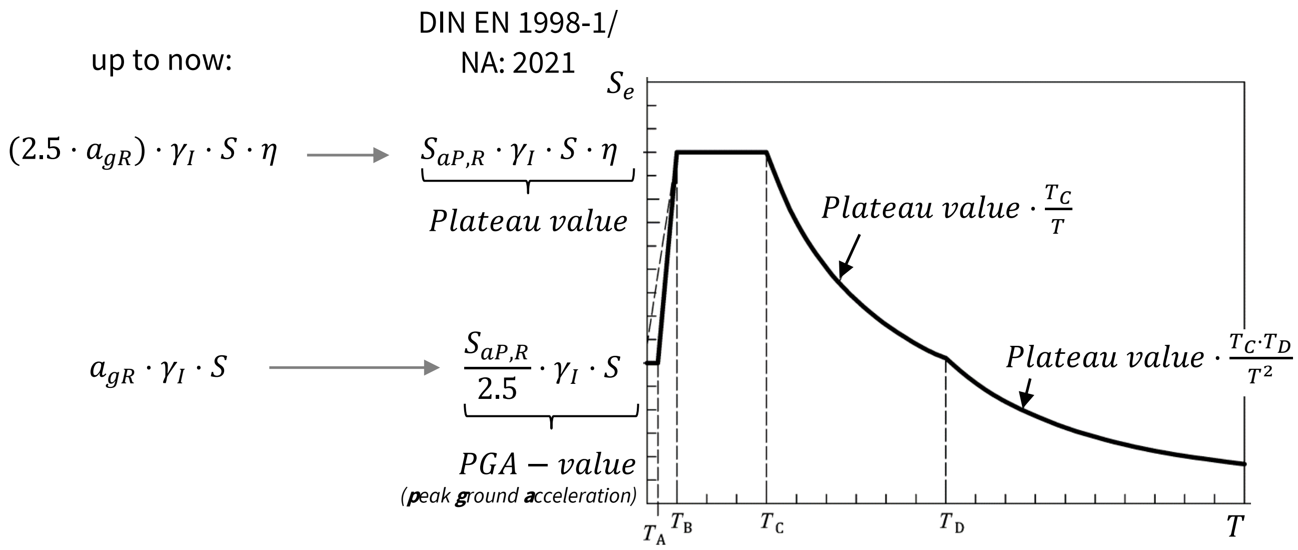


Figure 5.2: **Input values for defining the acceleration response spectrum, shown exemplarily for the elastic response spectrum according to DIN EN 1998-1/NA:2021 [28]**

The spectral response acceleration $S_{aP,R}$ shown in Figure NA.1 of DIN EN 1998-1/NA:2021 [28] is related to a reference return period of $T_{NCR} = 475\ a$. This represents the statistical average return period of the design earthquake event for common buildings and corresponds to a probability of exceedance of 10 % within 50 years, the calculative service life of common buildings.

In case of increased risks for humans and the environment or increased requirements for the safety of the plant, the design is to be based on a stronger or rarer earthquake. The magnitude of the seismic action is scaled according to DIN EN 1998-1 [27] section 4.2.5 or DIN EN 1998-1/NA:2021 [28] NDP re. 4.2.5(5)P, respectively, by means of the importance factor γ_I (see also section 5.3).

From a seismological point of view, this scaling of the seismic action by means of an importance factor is an approximation, since the individual seismological and geological conditions at the site are not taken into account. For example, for sites that experience no seismic action in a 475-year earthquake, there are mathematically no seismic effects even with an arbitrarily higher return period (because $0\ m/s^2 \cdot \gamma_I = 0\ m/s^2$), although the area affected by a stronger earthquake may include the site under consideration.

Nonetheless, the code-given scaling by means of the importance factor also applies to the design and assessment of facilities and components within the scope of the VCI-guideline, namely because of the following reasons: So far, individual seismic hazard maps have only been published for few return periods[2]. Therefore, only a very limited gradation of the decisive seismic action is possible. And, thirdly, no conversion between return periods and importance factors is given for sites in Germany (cf. last clause of section 5.3).

Re: (2) Consideration of design criteria at sites with very low seismicity

According to DIN EN 1998-1/NA D:2021 [28] NDP re. 3.2.1(5), cases of very low seismicity are those where the product $a_g \cdot S$ is not greater than $0.5\ m/s^2$, i.e. $\frac{S_{aP,R}}{2.5} \cdot \gamma_I \cdot S \leq 0.5\ m/s^2$. In plant engineering, this general criterion is not suitable for deciding whether a structure must be designed to withstand seismic effects. This is because high seismic loads can occur even with very low ground accelerations in cases where structures exhibit unfavourable mass distributions, where structures have a small area exposed to wind but a large mass, or where structures are not designed for horizontal loads (e.g. components inside buildings).

Therefore, and following note 1 to DIN EN 1998-1/NA D:2021 [28] NDP re. 3.2.1(5), it must always be checked whether the stresses due to seismic action become decisive compared to the stresses due to wind action. This can at first be done roughly by comparing the base shear from $F_b = m_{total} \cdot (S_{aP,R} \cdot \gamma_I \cdot S)$ or $F_W = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref}$, respectively [12], [13]. As a result, verification of stability may also be required if substances with a high risk potential are handled at a site with very low reference ground acceleration (cf. former seismic zone 0) and a high importance factor must, thus, be applied (cf. section 5.3).

If the seismic forces are smaller than the wind forces, it is nevertheless recommended for precautionary reasons to at least apply section 4 of the VCI-guideline (conceptual and structural design).

[2] For the return periods $T_{NCR} = 975\ a$ and $T_{NCR} = 2475\ a$, the spatial distribution of $S_{aP,R}$ is shown in the informative annex NA.E of DIN EN 1998-1/NA:2021 [28] as schematic sketch. However, no mathematical relationship between the importance factor and the return period of the design earthquake is specified for sites in Germany (cf. last clause of section 5.3); therefore, the safety level of the higher return periods cannot be compared directly to the safety level of the importance factors. If (e.g. for comparative calculations) acceleration values are taken from seismic hazard maps for a return period deviating from 475 years, these acceleration values are not to be scaled additionally with an importance factor.

5.2 Ground conditions, geology and subsoil

Re: (1) to (3) Subsoil conditions and deep geology conditions; soil factor S

In DIN EN 1998-1/NA:2021 [28] NDP re. 3.1.2(1) a distinction is made between the geological conditions, to which soil layers below a depth of about 30 m are assigned [29], and the overlying subsoil. The classification is based on the associated shear wave velocity in the soil. The combination of subsoil conditions and geological conditions results in the designation of a ground condition. The ground condition at site influences the shape of the response spectrum, since both the control periods T_B and T_C (which define the width of the plateau) and the soil factor S (which scales the spectrum as a whole) are assigned depending on the ground condition (see also section 5.4 of these explanations).

The subsoil class is usually determined by a soil survey or is estimated conservatively. The geological soil class at the construction site is given by the geological services of the federal states. Figure NA.G.1 in the informative annex G of DIN EN 1998-1/NA:2021 [28] shows as a schematic sketch the geological soil classes in areas of Germany where $S_{aP,R} \geq 0.6\ m/s^2$. It was prepared on the basis of the map of geological soil classes in DIN 4149:2005-04. In the future [29], the geological soil classes will be accessible as a digital annex to DIN EN 1998-1/NA:2021 [28], similar to the values of spectral response acceleration. It is expected that this digital annex and the corresponding revised map will also cover areas of Germany where $S_{aP,R} \geq 0.4\ m/s^2$. The normative digital annex will presumably provide as .csv-file the decisive geological soil class for squares with lateral length of 1 km together with the geographical coordinates of the centre of the square.

The amplification of the seismic action due to the dynamic behaviour of the soil (described by the soil factor S) depends on the magnitude of the seismic action: at low spectral accelerations, the amplification is relatively lower than at high spectral accelerations. For this reason, the soil factor S according to DIN EN 1998-1/NA:2021 [28] is no longer assigned only in relation to the ground condition, but additionally in relation to the spectral acceleration $S_{aP,R}$ expected at the considered site (Table NA.2 of DIN EN 1998-1/NA:2021 [28]).

The control periods T_B, T_C and T_D are different for different return periods. They are given for $T_{NCR} = 475a$ in Table NA.1 of DIN EN 1998-1/NA:2021 [28].

5.3 Importance factors

Re: (1) Importance factors in plant engineering

The importance factor γ_I (Index *I* = "importance") is used to scale the response spectrum in order to apply stronger seismic action in the design of structures and components with increased risk for (human) health and safety or the environment than it would be required for common buildings according to the standard (cf. section 5.1).

The reference return period for the design of common buildings according to DIN EN 1998-1 [27] is T_{NCR}= 475 years; consequently, an importance factor of $\gamma_I = 1.0$ is assigned. Deviating importance factors are defined in DIN EN 1998-1/NA:2021 Table NA.5 depending on the importance category of the building.

In contrast to common buildings, facilities in the chemical industry and related industries consist of numerous different components that expose different risks in the event of an earthquake. In this context, the term component covers both entire facility sections as well as individual structural parts up to components of process engineering. The decisive level of seismic action for each individual component results from the characteristics and quantity of the substances handled in the component, from the type of processes carried out and from the importance of the component in the operational processes of the overall facility. Accordingly, the integrity of components that store, handle or process toxic material, as well as equipment that is essential for the safe operation of the facility, is of particular importance. The main danger to humans and to the environment is primarily the release of volatile, toxic substances and the risk of explosion.

Deviating from DIN EN 1998-1 [27], the importance factors in the VCI-guideline are, therefore, not linked to building categories, but are assigned depending on the damage potential and the possible damage effects with regard to health and safety (Table 5.1), environmental protection (Table 5.2) and the safety of lifeline facilities (Table 5.3). The importance factor regarding health and safety is determined based on the hazard statement code (H-statement) of the handled substance according to the Regulation on Classification, Labelling and Packaging of Substances and Mixtures [52]. If the H-statement of the handled substance is not listed, the importance factor is to be determined according to the damage potential. If different substances are handled, the largest assigned importance factor is decisive.

The importance factors can be interpreted as follows:

- Common buildings as well as components without increased significance for the overall facility and without special risk for humans and the environment are assigned an importance factor of 1.0. This means that they are designed for earthquakes that have an average return period of 475 years at the plant site.
- If there is an increased risk in the event of damage, a stronger earthquake must be considered in the design. From a statistical point of view, its probability of exceedance is lower and its mean return period is greater. By choosing an importance factor greater than 1.0, the seismic action is increased in the design.

If the importance factor is used for the design of components, it is usually named γ_a (cf. section 6.4). However, this is only a notational distinction; the assignment of the numerical value is carried out in the same way as for γ_I depending on the damage potential and the area possibly affected according to Tables 5.1 to 5.3 of the VCI-guideline.

In DIN EN 1998-1 [27] note to clause 2.1 (4), a mathematical relationship between the importance factor and the mean return period of the assumed design earthquake is given (see also [71]).

$$\gamma_I \sim \frac{1}{(T_{LR}/T_L)^{1/k}}$$

with T_{LR} = mean return period of the reference seismic action (e.g. 475 years for the verification at the ultimate limit state)
T_L = targeted mean return period (e.g. 1300 years)
k = exponent that depends on the local seismicity

In order to employ this relationship, however, knowledge of the exponent k is required. The exponent proposed in DIN 1998-1 [27] is not valid for Germany according to DIN EN 1998-1/NA:2021 NCI re. 2.1(4). Since no alternative value for k is given, the importance factors of the standard as well as the VCI-guideline for sites in Germany cannot be converted across-the-board into mean return periods, but are only used as a linear scaling of the seismic action. The above formula from DIN EN 1998-1 [27] note to clause 2.1 (4) is, therefore, explained here for information purposes only.

Re: (2) Minimum value of the importance factor

An importance factor of less than 1.0 is not permitted by the VCI-guideline. Thus, at least an average return period of 475 years must be considered in the design at the ultimate limit state. An exception to this is the verification of existing facilities with a short remaining operating time (see section 10).

Re: (3) Separate consideration of dynamically independent structural components

No further explanations.

5.4 Basic representation of the seismic action

Re: (1) and (2) General information on the representation of seismic action

The seismic action is usually described by an elastic ground acceleration response spectrum ("elastic response spectrum"). An elastic response spectrum represents graphically the maximum response of linear elastic single-degree-of-freedom-oscillators (SDOF oscillators) of equal damping but different natural frequencies to a given time history of motion. Response spectra in general are set up by subjecting a large number of SDOF oscillators to the same time history of base excitation. The maximum responses of the SDOF oscillators (acceleration S_a, velocity S_v or displacement S_d) are then plotted vs. the natural period of the SDOF oscillators.

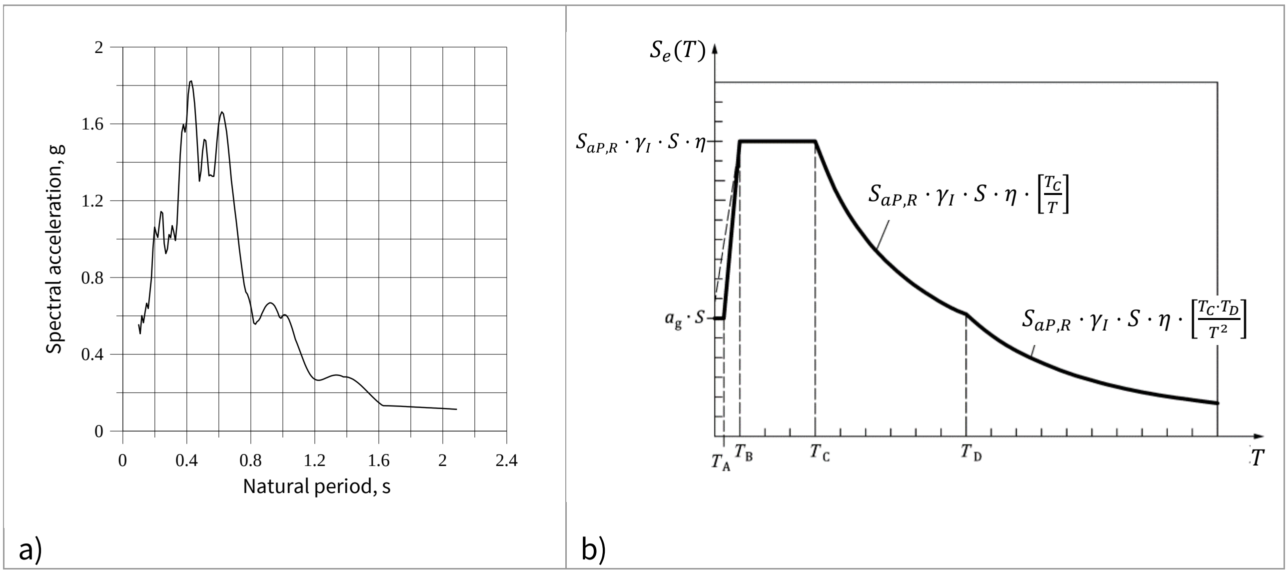


Figure 5.3: a) Acceleration response spectrum of a measured acceleration time history (Petrovac earthquake)
b) Elastic acceleration response spectrum according to DIN EN 1998-1/NA

For measured earthquake time histories, the graphical representation as a response spectrum shows the frequency content of the earthquake (Figure 5.3 a). For design purposes, design standards commonly use smoothed response spectra that represent the envelope over numerous "real" response spectra. These smoothed acceleration response spectra (Figure 5.3 b) indicate the maximum value of ground acceleration at $T = 0$ (free-field acceleration). Above T_A, the ordinate increases linearly to the maximum expected response of the corresponding SDOF oscillator
(at $T = T_B$) and continues after a plateau of constant acceleration response ($T_B < T < T_C$) as a decreasing function ($T > T_C$).

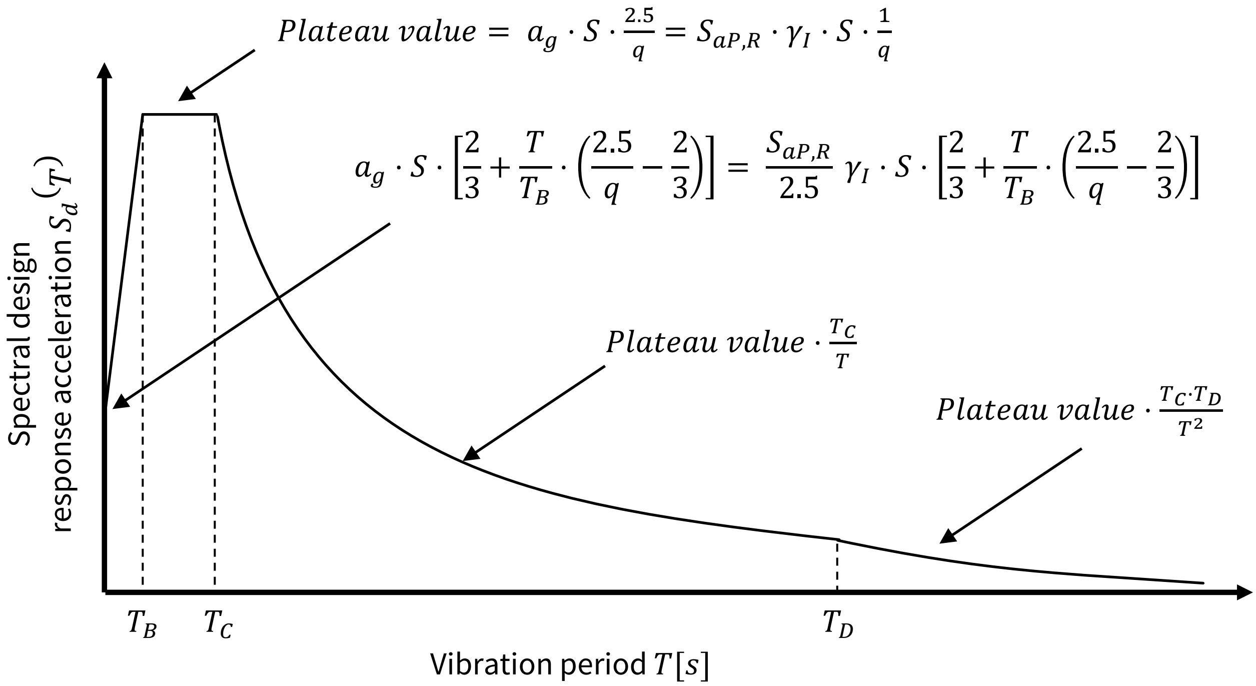
The equations for the determination of the horizontal elastic response spectrum are given in DIN EN 1998-1/NA:2021 [28] NDP re. 3.2.2.2(2)P based on the maximum spectral response acceleration $S_{aP,R}$ and in DIN EN 1998-1 [27] section 3.2.2.2 based on the reference ground acceleration a_{gR}.

The elastic response spectrum is scaled to a reference value for the structural damping of 5 % viscous damping. If the damping of the investigated structure deviates from the reference value of 5 %, the influence of this deviation on the elastic acceleration response is considered by means of the damping correction factor $\eta = \sqrt{10/(5+\xi)} \geq 0.55$ (see also explanations to clause 6.1 a (3)). Lower damping increases the vibration response, higher damping reduces it.

In the vertical elastic response spectrum, the plateau value of the spectrum is three times the vertical free-field acceleration (for comparison: horizontal response spectrum: factor 2.5; vertical design spectrum (see following section): also factor 2.5). The control periods of the vertical response spectrum are independent of the ground condition ($T_B = 0.05\ s$; $T_C = 0.2\ s$; $T_D = 1.2\ s$), the vertical ground acceleration a_{vg} is taken as a proportion of the horizontal ground acceleration ($a_{vg} = 0.7 \cdot a_g$), and an amplification of the vertical seismic action by the soil does not have to be taken into account (soil factor $S = 1.0$) (DIN EN 1998-1 [27] section 3.2.2.3 or DIN EN 1998-1/NA:2021 [28] NDP re. 3.2.2.3(1)P, respectively).

Re: (3) Design spectrum to enable consideration of energy dissipation

Real structures are usually able to dissipate part of the induced energy, for example by plastic deformations (non-linear material behaviour) or due to friction in bolted connections. That is taken into account when designing a structure according to DIN EN 1998: The elastic response spectrum is reduced by application of a behaviour factor q that depends on the material used as well as on the type and detailing of the structure. The resulting response spectrum is called design response spectrum. The behaviour factor not only describes the deformation capacity of the structure, but also includes the influence of a structural damping deviating from 5 % (see also DIN EN 1998-1 [27] clause 3.2.2.5 (3)P).



For a return period of 475 years, the following applies according to DIN EN 1998-1/NA:2021 [28] equation NA.1 and with DIN EN 1998-1 [27] clause 3.2.1(3):

$$a_g = \frac{S_{aP,R}}{2.5} \cdot \gamma_I$$

Figure 5.4: Shape of the design spectrum according to DIN EN 1998-1/NA:2021 [28]

A behaviour factor of $q = 1.5$ essentially includes common damping effects and low values of overstrength, but not non-linear response effects of the structure. A structural design that considers a behaviour factor of $q = 1.5$ thus corresponds to the assumption of a linear-elastic building response. A q-factor >1.5 includes (non-linear) energy dissipation and load redistribution effects.

Table 5.1 provides an overview of the limits of the behaviour factors for different building materials. These values apply to common buildings; for thin-walled shells (e.g. tanks) a lower value for q_{min} may be realistic (cf. section 6.1.b (6)). When utilizing a behaviour factor greater than 1.5, it should be noted that extensive additional verifications are required to ensure the assumed ductility. It is regulated in the corresponding material-specific parts of DIN EN 1998-1 [27] and the associated sections of DIN-EN 1998-1/NA:2021 [28], what additional verifications become necessary (see also general design criteria for dissipative load-bearing structures in DIN EN 1998-1 [27] section 6.5.2).

Table 5.1: Limits for the behaviour factors of the different building materials

Behaviour factor	Reinforced concrete	Steel	Steel-concrete composite	Wood	Masonry
min q	1.5	1.5	1.5	1.5	1.5
max q	6.5	8.0	7.2	4.0	3.0

The shape of the horizontal design spectrum for a linear calculation (DIN EN 1998-1 [27] clause 3.2.2.5(4)) is similar to that of the elastic response spectrum. However, the behaviour factor q replaces the damping correction factor $\eta(\xi)$ as quotient $1/q$, the control period T_A is set to zero, and the free-field acceleration is not fully applied in the design spectrum (initial value of the elastic spectrum: $1.0 \cdot a_{gR} \cdot \gamma_I \cdot S$; initial value of the design spectrum: $\frac{2}{3} \cdot a_{gR} \cdot \gamma_I \cdot S$). This reduction, though, is hardly relevant in practice, as this value is only used for vibration modes with extremely high natural frequencies. Therefore, the plateau value is often applied from $T = 0$, i.e. the plateau range is extended to the y-axis of the diagram. The minimum value of spectral acceleration ($\beta \cdot a_g$) is not used in the design spectrum for German seismic areas (see DIN EN 1998-1/NA:2021 [28] NDP re. 3.2.2.5(4)P).

Re: (4) Vertical component of the seismic action

The parameters a_{vg}, S, T_B, T_C and T_D for the determination of the vertical design spectrum are specified according to DIN EN 1998-1 [27] clause 3.2.2.5(5) and according to DIN EN 1998-1/NA:2021 [28] Table NA.3, respectively. The behaviour factor for the description of the vertical component of the seismic action should not be assumed greater than $q_v \leq 1.5$ following DIN EN 1998-1 [27] 3.2.2.5 (6).

Re: (5) Representation of seismic action using time histories

According to the VCI-guideline's clause 6.2 (2) and in analogy to DIN EN 1998-1 [27] section 4.3.3.4.3, non-linear time history calculations are permitted for structural design, but are explicitly not recommended for plant engineering.

For the sake of completeness, however, it should be noted at this point that the time histories of the ground motion that are to be used for non-linear dynamic analyses of structures must fulfil certain criteria that are specified in DIN EN 1998-1 [27] section 3.2.3.1.

5.5 Combinations of the seismic action with other actions

Re: (1) Combination factors $\psi_{2,i}$

According to DIN EN 1990 [10] section 6.4.3.4, the effect of the combination of all actions in the seismic design situation must be determined employing equation (5.1) given below.

Thereby, the design value of seismic action A_{Ed} in plant engineering is to be determined according to equation (5.2) considering the masses from permanent loads $G_{k,j}$ and, if applicable, reduced live loads and operating loads $Q_{k,i}$ (cf. DIN EN 1998-1 [27] section 3.2.4).

$$E_d = E\left(\sum_{j\geq 1} G_{k,j} \text{"+"} \;\; \mathrm{P} \;\; \text{"+"} \;\; \mathrm{A_{Ed}} \;\; \text{" + "} \;\; \sum_{i\geq 1}(\psi_{2,i} \cdot Q_{k,i})\right) \tag{5.1}$$

where
- E_d = Design value of an action
- $G_{k,j}$ = Characteristic value of a permanent action j
- P = Decisive representative value of prestressing
- A_{Ed} = Design value of seismic action
- $\psi_{2,i}$ = Combination coefficient for quasi-permanent value of a variable action i
- $Q_{k.i}$ = Characteristic value of a variable action i

$$A_{Ed} = A\left(\Sigma G_{k,j} \text{"+"} \Sigma \psi_{E,i} \cdot Q_{k,i}\right) \text{ with: } \psi_{E,i} = \varphi \cdot \psi_{2,i} \qquad \text{and} \qquad \varphi = 1.0 \tag{5.2}$$

where
- $A(\,)$ = „Effect due to "
- $\psi_{E,i}$ = Combination coefficient for a variable action i
- φ = Factor for the calculation of $\psi_{E,i}$

all other parameter definitions: see equation (5.1)

The exceptionality of the earthquake situation is taken into account by including the masses from permanent loads without increasing safety factors and reducing the masses from traffic and operating loads with the combination factor $\psi_{2,i}$. The reduction accounts for the probability that the variable loads are not fully present in the event of an earthquake. (Floors in plant engineering, for example, are designed for high variable area loads to ensure flexibility for mounting situations and reutilization, but during operation, large platform areas are often load-free). Since the combination coefficients according to DIN EN 1990/NA [11] do not take into account either operating loads or the specific features of facilities with regard to live loads, Table 5.4 of the guide has to be used to determine the combination coefficients $\psi_{2,i}$.

Regarding operating loads, it should be noted that permanently existing operating loads (e.g. permanently filled vessels) must be considered as permanent loads. Regarding operating loads and filling loads, it may be necessary to consider different load constellations during operation (see explanations to (3)).

Re: Table 5.4 – Combination factors ψ_{2i}

The combination factors ψ_{2i} take into account the probability that the variable loads are not fully present in the event of an earthquake (see commentary re (1)). This is particularly relevant when facilities / primary structures with numerous variable loads are analysed. Therefore, the variable operating loads may be reduced to up to 60 % ($\psi_2 = 0.6$) for an average consideration of such a facility / primary structure. This is in reference to DIN EN 1990 NA:2010 [11] Table NA.A.1.1 “Traffic areas”. Operating loads are those loads that occur at maximum during the designated production process (e.g. nominal volume of vessels). The reduction then applies equally to the determination of the masses according to equation (5.2) to calculate the natural frequencies of the structure and to the combination of the individual load cases according to equation (5.1).
If operating loads are hardly variable (e.g. stirring vessels that are only emptied for a short time and refilled directly afterwards), these operating loads are to be taken into account without reduction. Depending on the configuration of the facility / primary structure, it may be necessary to consider several different operating load constellations (see commentary re (3)).

If the seismic safety of an individual component is investigated, the nominal operation of the component is decisive for design. In this case, the operating loads are to be considered with combination factor $\psi = 1.0$. The probability that the seismic design event occurs at the same time as a major accident (exceedance of operating load) is regarded as very low. Therefore, exceptional incident loads concerning a major accident or pressure test loads are not considered in the seismic design situation.

Constrained forces that result, for example, from the change in length of a pipe due to changes in operating temperature and pressure (pipe thrust) must be taken into account in analogy to variable operating loads provided that the constrained forces have an unfavourable effect in the event of an earthquake.

A distinction is made regarding temperature effects between those that act present quasi-permanently as operating temperatures ($\psi = 1.0$) and those that arise occasionally from outside, for example due to weather influences ($\psi = 0$).
In the case of friction forces, the deformation due to seismic loading must be considered. Frictional forces that act favourably should not be taken into account.

If a load transfer via friction forces is nevertheless considered, the friction factor should be chosen conservatively at approx. 50 % of the static friction coefficient (e.g. $\mu = 0.15 \ldots 0.20$ for friction of steel on concrete). Additionally, the frictional force should be calculated considering the uplifting vertical seismic action (e.g. $H_{frict} = \mu \cdot m_{component} \cdot \left(9.81\frac{m}{s^2} - (0.7 \cdot S_{apR} \cdot \gamma_I)\right)$; see also guideline clause 5.4(4) and corresponding explanation). The favourable friction force is included in the superposition of the actions with a combination factor of $\psi_2 = 0.9$.

The combination factor for snow loads is in accordance with the regulation of DIN EN 1998-1/NA:2021 [28] NDP re. 4.2.4(2) P. The other listed actions are self-explanatory.

Re: (2) Reduction factor φ

The reduction factor φ is used to take into account unfavourable forms of vibration due to the different use of different storeys in common buildings; it should not be used in the design of facilities within the scope of the VCI-guideline, i.e. it should be set to 1.0. Due to large single masses in plant engineering and the associated complex natural forms of vibration, the guideline instead requires that different filling constellations are considered individually in the design and that the decisive operating load case is determined (see also explanation to (3)).

Re: (3) Decisive constellations of operating load

When determining the dynamic response of a structure, the masses must be considered realistically. This is because they not only influence the inertia force according to " $F = m \cdot a$ " but also the vibration periods of the structure. Those eigenperiods in turn define the relevant response acceleration (see response spectrum in Figure 5.4: shifting the vibration period T on the abscissa leads to smaller or larger spectral accelerations $S_d(T)$ on the ordinate). Thus, it may be necessary to consider different operating load constellations depending on the configuration and the use of the facility.

When determining the decisive operating load case in the event of an earthquake, it is recommended to consider the following constellations:

- Average level of filling of the process components and tanks during typical plant operation,
- Unfavourable level of filling for transversal vibration (maximum realistic filling level in the upper third (elevation) of the plant, minimum realistic filling level in the lower two thirds (elevation) of the plant),

- Other constellations that deviate strongly from the earthquake-compatible design criteria according to section 4 and are rated critical for the vibration behaviour of the facility.

The decisive filling and operating load constellation can be determined by expertise if the design engineer has sufficient experience in seismic design or it can be determined by calculations with different possible constellations.

The considered operating load constellation then applies both for the definition of masses when determining the dynamic response of the structure as well as for the superposition of load cases according to DIN EN 1990 section 6.4.3.4 or guideline clause 5.5(1), respectively.

Example: *Complete filling of all tanks on one floor level*
↳ maximum mass from operating load to determine the natural frequency/period
→ Loading from seismic action as “seismic load case”
↳ Load case "operating loads" (vertical loads) from completely filled tanks
↳ Combination of the stresses from the load cases „dead load", "operating loads", "seismic load case", and other load cases, if applicable (see also explanatory notes to clause 5.5(1))

Re: (4) Constrained forces

No further explanations.

6 Structural analysis

Re: (1) Requirement for safety verifications in case of plant modifications

Section 1.3 of DIN EN 1998-1 [27] points out that the scope of application of the EN 1998 series also refers to building modifications. Accordingly, changes to the supporting structure or changes to the mass distribution require corresponding safety verifications according to the standard or guideline. In contrary to common buildings, structural changes or changes in the equipment arrangement and -position are not unusual in plant engineering (e.g. due to changing requirements of process engineering). Therefore, special attention is drawn here to the obligation to provide safety verifications in case of plant modifications.

6.1 Modelling

6.1.a Load-bearing structures of facilities

Re: (1) and (2) General notes on numerical modelling

The numerical model of the facility must reflect realistically the dynamic structural behaviour under seismic action. For this purpose, all essential mass and stiffness properties must be taken into account in the model. Simplifications are permitted under certain conditions:

Non-structural components and non-structural parts of the main structure can be considered in the model as point masses, if they do not influence significantly the overall vibration behaviour of the load-bearing structure. If significant load increases result for example from the filling of vessels, it is generally sufficiently accurate to consider that by additional masses at the component's centre of mass. The hydrodynamic behaviour of liquids in rather small tanks can be neglected for the design of the overall structure (cf. section 6.4).

If the criteria of regularity are met, the structure may be investigated using planar models. In the case of irregular mass or stiffness properties that cannot be transferred into a planar model via special boundary conditions, a spatial model must be used (see also the above explanation on section 4.a equation (4.1)).

The stiffness-increasing contribution of non-structural parts of the load-bearing structure, such as masonry infill walls, should conservatively be neglected in the numerical model, since the stiffness of these elements is usually greatly reduced during an earthquake because of cracks and further damage.

However, the masses of non-structural parts of the load-bearing structure must be taken into account in the numerical model.

Re: (3) Reference value for structural damping

The reference value of structural damping is in accordance with the value given in DIN EN 1998-1 [27] section 3.2.2.2. For certain constructions and materials, the selection of a damping value deviating from 5 % may be necessary. In this case, the damping should be adjusted accordingly in the numerical model of the load-bearing structure and in the determination of the elastic response spectrum (cf. section 5.4 of this commentary document) (DIN EN 1998-1 [27] clause 3.2.2.2 (3)). Lower damping increases the acceleration response within the load-bearing structure.

When using the design spectrum, damping values deviating from 5 % are already covered by the construction-specific behaviour factor q (see also explanations to 5.4(3) or DIN EN 1998-1 [27] clause 3.2.2.5 (3)P).

Re: (4) Modelling of seismic protection systems

While non-linear time history calculations are generally not recommended for the design of structures on conventional foundation (cf. clause 6.2 (2) of the VCI-guideline), the non-linear response of seismic protection systems can only be represented realistically employing this method of analysis.

Re: (5) Soil-structure interaction

The interaction between the soil and the structure can influence significantly the vibration behaviour of the load-bearing structure: On the one hand, vibration periods, mode shapes and modal participation factors of the flexibly supported structure differ from those of the rigidly supported structure. On the other hand, the movement of the flexibly supported structure may contain a significant tilt component that is not taken into account when assuming a rigid foundation. Furthermore, the radiation damping and the internal damping of the soil influence the overall damping of the flexibly supported structure. Therefore, the soil-structure interaction must be taken into account for the modelling and the computational investigation of the structure in the following cases (DIN EN 1998-5 [33] clause 6 (1)P):

- for structures where effects of II. order theory (P-δ effects) play an important role,
- for structures with massive or deep foundations,
- for slender, high structures,
- for structures on very soft soil with mean shear wave velocity $v_{s,max}$ below 150 m/s, such as soil that cannot be assigned to any of the soil classes specified in DIN EN 1998-1/NA:2021 [28] NDP re. 3.1. 2(1) (soil "worse" than soil class C; in this case, however, separate investigations of the influence of the ground conditions on the seismic action are required according to DIN EN 1998-1/NA:2021 [28], NDP re. 3.1.2(1) (ii).
- In the case of pile foundations, the influence of the soil-structure interaction must always be investigated (DIN EN 1998-5 [33] clause 6 (2)P).

The simplest way to include soil-structure interactions in the numerical model is to use substitute variables for the bedding (translation and rotation) and the damping according to equations (6.1) (see also Figure 6.1 [75]).

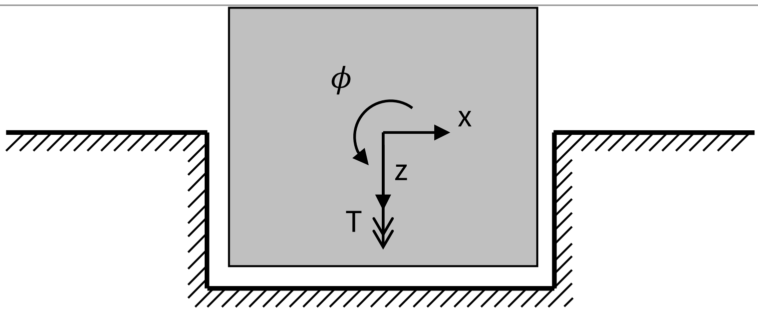


Figure 6.1: Coordinates for substitute variables for the bedding (according to [75])

$$c_z = C \cdot A \qquad c_x = 0.5 \cdot C \cdot A \qquad c_\phi = 2 \cdot C \cdot I_A \qquad c_T = 0.8 \cdot C \cdot I_T \tag{6.1}$$

where c_i = Translational and rotational spring stiffness ($i = z, x, \phi, T$) [MN/m] or [MNm/rad], respectively
A = Foundation area [m²]
C = Dynamic modulus of subgrade reaction [MN/m³] using equation (6.2) or Table 6.1
I_A = Moment of inertia with regard to tilting [m⁴] (to be determined for both horizontal axes)
I_T = Moment of inertia with regard to rotation [m⁴]

The dynamic modulus of subgrade reaction C can be calculated from the dynamic Young's modulus and the foundation area using equation (6.2).

$$C = \frac{E_d}{0.4 \cdot \sqrt{A}} \tag{6.2}$$

where E_d = dynamic constrained modulus of the soil; see [75].
A = Foundation area

Alternatively, approximate values of the dynamic modulus of subgrade reaction are given for various soil types in Table 6.1, which apply to a foundation area of A > 10m². For smaller foundations, the values are to be multiplied by the given factor.

Table 6.1: Dynamic subgrade reaction modulus C for different soil types [76]

Soil type	**σ [MN/m²]**	**C [MN/m³]**
I Soils of low strength (clay and loam in plastic state, sandy loam, medium-density floury sand, and the soils mentioned under II and III if they are interspersed with layers of silt and peat)	up to 0.15	up to 30
II Soils of medium strength (clay and loam at the plastic limit, sand)	0.15 – 0.35	30 - 50
III Solid soils (clay and loam in semi-solid to solid state, gravelly coarse sand, loess and loess-like loam)	0.35 – 0.5	50 - 100
IV Rock	over 0.6	over 100
These values apply to A > 10 m² **For A < 10 m² they are to be multiplied by the factor $\frac{3.2}{\sqrt{A}}$ (A in m²)**		

As an alternative to the method of substitute values described above, the subgrade reaction model according to Winkler may be used, which is explained in more detail in [66]. Half-space solutions for circular and rectangular foundations can be found for example in [75] and [76].

Re: (6) Damping characteristics of the soil

The damping constant k of the soil can be derived for the radiation as follows

$$k = \frac{E_d \cdot A}{v} = E_d \cdot A \cdot \sqrt{\frac{\rho}{E}} \tag{6.3}$$

where k = Damping constant for the radiation damping of the soil
v = Shear wave velocity of the soil
ρ = Density of the soil
E = static constrained modulus of the soil; see [76]
Further parameters: see equation (6.2)

The relationship in equation (6.3) assumes that the dilatation waves in the soil are limited to a truncated cone enveloping the foundation. Further details can be found for example in [75].

Re: (7) Consideration of significant change in characteristics of the soil

Notes on the mathematical consideration of changes in characteristics of the soil resulting from dynamic loading can be found in various places in DIN EN 1998-5 [33]. A simplified verification against soil liquefaction is provided in the normative annex NA.H of the national annex to DIN EN 1998-5:2021 [34].

6.1.b Free-standing vessels, silos, tanks and process columns

Re: (1) Characteristics of the numerical model

No further explanations.

Re: (2) Soil-structure interaction

For the consideration of soil-structure interaction, the explanations in section 6.1.a apply.

Re: (3) Hydrodynamic effects in liquid-filled tank structures

For the computational consideration of hydrodynamic effects in liquid-filled tank structures, see section 6.2 (7) of this commentary document.

Re: (4) Stored granular material

The simplification regarding the consideration of mass of stored granular material is in accordance with DIN EN 1998-4 [31] clause 3.3 (4).

Re: (5) Damping of stored liquids and bulk material

The specification of the damping of liquids is in accordance with DIN EN 1998-4 [31] clause 2.3.3.2 (1). For silos, the mass of stored granular material may be assumed to be rigidly connected to the silo shell (see clause 6.1.b (4)). A damping value of 5 % for the entire system is usually realistic. This assumption is in accordance with the regulations in prEN 1998-4:2018 [32].

Re: (6) Limitation of the behaviour factor q

The behaviour factor q describes in general terms the reduction of seismic response due to, inter alia, load redistribution in redundant systems, overstrength, plastic deformation and energy dissipation. Vessels themselves should always be designed assuming low-dissipative load-bearing behaviour (ductility class DCL). The specification of different maximum q-values for the design of the vessel itself depending on the material ($q \leq 1.2$ for steel vessels, $q \leq 1.5$ for reinforced concrete vessels) is in accordance with the regulations of prEN 1998-4:2018 [32] and is due to the fact that thin-walled shells generally have less potential for the formation of the effects mentioned in the first sentence of this clause.

Depending on the type of construction, substructures of vessels may be designed taking into account dissipative load-bearing behaviour ($q_{Substructure} > 1.5$). In this case, however, the stress on the elevated tank itself should still be determined assuming a non-ductile substructure because the greater deformations of the ductile substructure can possibly lead to a buckling failure of the elevated thin-walled tank. It should also be noted that in the case of a ductile design of the substructure (ductility class DCM or DCH), separate verifications depending on the chosen structural detailing and construction material must be carried out in order to ensure the assumed ductility (cf. clause 5.4 (3)).

Re: (7) Modelling of very slim vertical parts of a facility

No further explanations.

6.1.c Non-structural components and piping

Re: (1) Independent modelling of components and load-bearing structure

Non-structural components in facilities have a wide range of possible static systems. Due to their complexity and the usually high number of non-structural components within facilities, it would require great effort to model and investigate them together with the load-bearing structure. If the components do not influence significantly the dynamic behaviour of the load bearing structure through their own vibration behaviour, the modelling of the components can be carried out separately from the numerical model of the load bearing structure by means of suitable equivalent static systems. The design force F_a that is applied to the substitute system is to be determined according to section 6.4. It is applied to the equivalent static system according to the mass and stiffness distribution of the component.

Re: (2) Modelling of above-ground pipes

No further explanations.

Re: (3) Modelling of buried pipes

Notes on the modelling of buried pipes can also be found in the informative annex B of DIN EN 1998-4 [31] or in the relevant literature (e.g. [45], [46], [64]).

6.2 Methods of analysis

Re: (1) and (2) Permissible methods of analysis

The lateral force method of analysis and the modal response spectrum analysis are explained in more detail in section 4.3.3.2 and section 4.3.3.3 of DIN EN 1998-1 [27]; notes on the application of these methods of analysis can also be found in the relevant literature. In addition, non-linear methods of analysis (DIN EN 1998-1 section 4.3.3.4) may also be used.

It is recommended to use the lateral force method of analysis or the modal response spectrum analysis for the design and planning of new facilities. For the safety verifications of existing facilities, non-linear static methods of analysis may be applied if the structural capacity needs to be utilised more efficiently to provide the necessary verifications.

Non-linear static methods of analysis

Non-linear static methods of analysis (e.g. pushover analyses) are based on the determination of load-deformation curves (pushover curves) computed at the non-linear numerical model of the structure (Figure 6.2). For this purpose, the load sum F_b of the monotonically increasing horizontal loads F_i (while vertical loads are kept constant) is shown in relation to the displacement of a control point (e.g. the roof displacement Δ_{Roof}). The pushover curve can be evaluated using various methods (e.g. the capacity spectrum method, the N2 method, or the direct displacement-based design). The verification principle is briefly illustrated below referring to the capacity spectrum method. For details on the calculation procedure and further background information as well as explanations on other non-linear static calculation methods, see DIN EN 1998-1/NA:2021 [28] NCIs on the subsections of chapter 4.3.3.4 and DIN EN 1998-1 [27] Annex B as well as the relevant literature on the subject (e.g. [56], [66]).

Basically, the following applies: Using the non-linear static calculation methods mentioned above, it is possible to evaluate the global structural behaviour and to estimate the non-linear load-bearing capacity of the structure. Static verifications of individual structural elements and detailed verifications cannot be carried out by applying non-linear static methods of analysis.

Furthermore, the numerical modelling necessary for the determination of the pushover-curve requires extensive knowledge of the actual properties of the supporting elements.

For this reason, non-linear static methods of analysis are typically not used for standard structural design in plant engineering. However, these non-linear static calculation methods may be used to determine a realistic behaviour factor for existing plants.

<u>Brief description of the capacity spectrum method</u>

The non-linear load-bearing behaviour of the structure is initially represented in terms of a pushover curve in the load-displacement diagram, as explained above (Figure 6.2 b). Special attention must be paid to the distribution of the horizontal loads (e.g. distributed linearly along the height, in accordance with the first or with further mode shapes, etc.), but also to the choice of a suitable numerical model of the structure, as well as to the choice of control point and to the influence of torsional vibrations. In order to be able to compare the structure capacity with the seismic demand, the pushover curve is transformed into the capacity curve of an equivalent single-degree-of-freedom oscillator (in terms of a spectral acceleration – spectral displacement-diagram) by means of mathematical relationships (Figure 6.2 c). Similarly, the seismic demand – usually given in terms of a response spectrum as a function of the period of vibration $S_a(T)$ – is transferred into a spectral acceleration – spectral displacement-diagram) (Figure 6.2 d). Since the structure activates greater damping during non-linear response, response spectra with higher damping must usually be set up additionally. In order to evaluate the capacity spectrum method, the capacity curve and the damped response spectra are superposed in the $S_a - S_d$ –diagram (Figure 6.2 e).

The intersection of the two curves with the coordinates $S_{d,p}$ and $S_{a,p}$ represents the so-called performance point, which shows the expected inelastic response of the equivalent single-degree-of-freedom oscillator to the assumed earthquake. If there is no intersection between the capacity curve and the decisive response spectrum, the demand of the seismic event is greater than the capacity of the structure in every state of deformation – the structure will not withstand the earthquake impact.

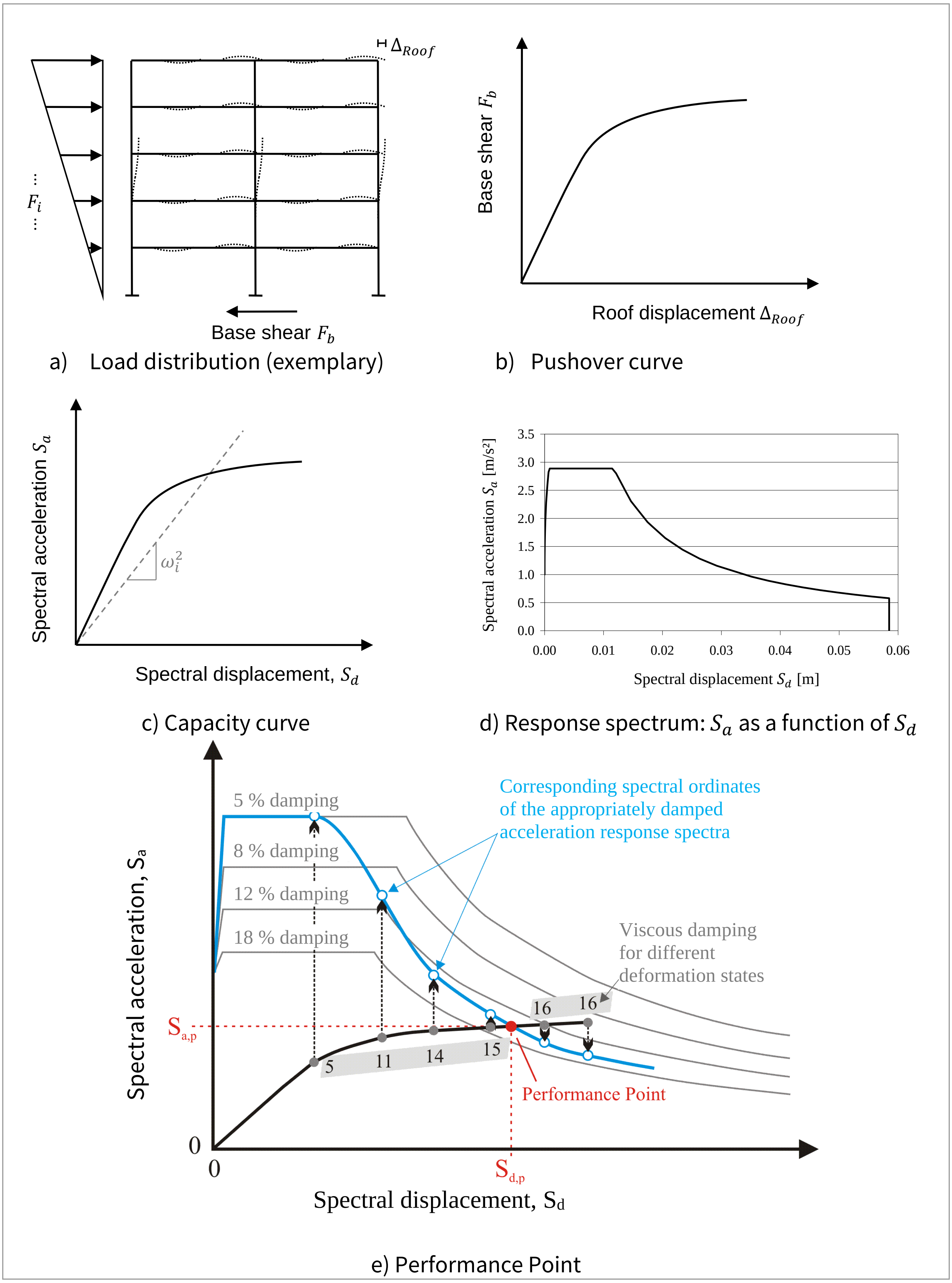


Figure 6.2: Principle of the capacity spectrum method

Nonlinear dynamic methods of analysis

Direct integration methods solve the differential equation of motion of the numerical model time-step by time-step taking into account the currently valid stiffness and damping properties of the system. This is the most comprehensive option for investigating seismically loaded structures, but it also requires by far the greatest computational effort. Non-linear dynamic time history analyses facilitate a very precise estimate of the real building behaviour during an earthquake. However, the accuracy depends on the diligence taken during modelling (regarding the non-linearity of the stiffness and damping as well as the model idealisation), the determination of the numerical (integration) parameters as well as the choice of the applied time histories of the seismic action.

Due to the large computational effort and the high sensitivity of the results with regard to the modelling and the determination of the input values, non-linear dynamic methods of analysis are not recommended for design purposes in plant engineering. An exception is the verification of structures equipped with seismic protection systems (cf. clause 6.1a (4) of the VCI-guideline or the commentary document). In special cases of assessment of existing facilities, time history calculations may also provide information on the detailed dynamic behaviour (e.g. through fluid-structure interaction calculations for liquid-filled tanks).

Re: (3) Application of two horizontal components of the seismic action

The horizontal components of the seismic action are to be assumed as acting simultaneously. The stress quantities (e.g. internal forces, support forces, etc.) can be calculated separately for each direction and then superposed by means of suitable procedures (see DIN EN 1998-1 [27] section 4.3.3.5).

In the case of axially symmetrical structures (e.g. upright cylindrical tanks), only one horizontal direction may be considered for simplification, but its action should be multiplied by a factor of 1.12 to take into account the directional superposition [63].

Re: (4) Application of the vertical component of the seismic action

In accordance with the regulations of DIN EN 1998-1 [27] section 4.3.3.5.2, vertical accelerations must be considered only for the design of the following structural parts and components:

- Load-bearing elements that support columns or large masses (Figure 6.3),
- Horizontal or nearly horizontal load-bearing elements with spans > 20m,
- Horizontal or nearly horizontal cantilevered elements with a length > 5m,
- Horizontal or nearly horizontal prestressed elements.

The above regulation applies regardless of the magnitude of the vertical acceleration; the minimum acceleration specified in DIN EN 1998-1 [27] section 4.3.3.5.2 is not adopted by the VCI-guideline. However, the stresses resulting from the vertical acceleration only have to be taken into account for the design of the loaded elements under consideration and the load-bearing elements or structural areas directly connected to them. The superposition of the stresses from horizontal and vertical seismic action is carried out in accordance with DIN EN 1998-1 clause 4.3.3.5.2 (4).

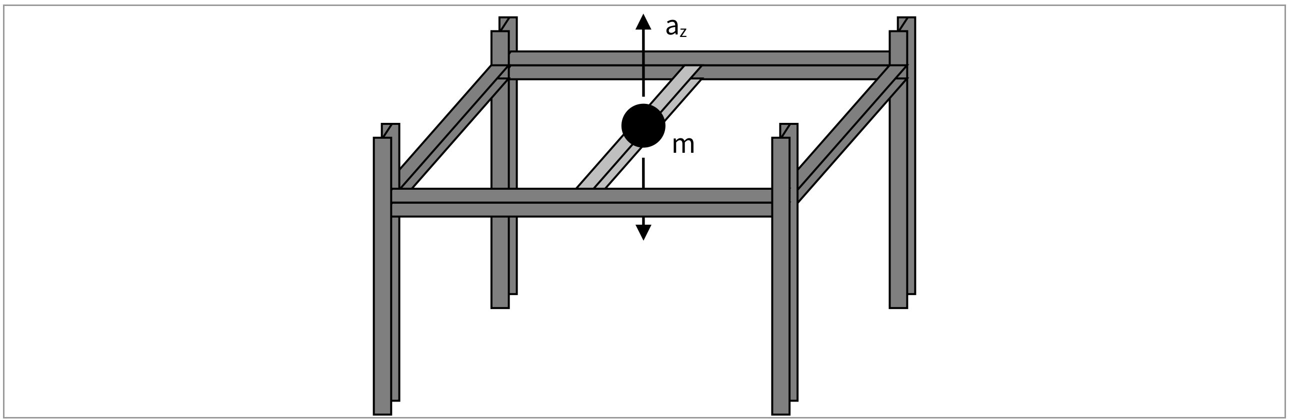


Figure 6.3: Application of vertical accelerations on single beams carrying large masses

Re: (5) Calculation of foundations

In addition to the regulations in the DIN EN 1998-5 [33], a simplified procedure for the verification against bearing failure of shallow foundations is given in the associated national annex DIN EN 1998-5/NA:2021 [34] in its normative annex NA.I. DIN EN 1998-1 section 4.4.2.6 [27] regulates the determination of the stress values for foundation components.

Re: (6) Calculation of silos

Regarding the application of the seismically induced annular pressure on cylindrical silos or silo chambers see also Figure 6.11 on page 61.

Re: (7) Calculation of liquid-filled tank structures

In the case of seismic action, liquid-filled tank structures are stressed not only by the mass inertia of the shell, but also by the inertia of the stored fluid and the interaction between fluid and shell. The overall vibration behaviour of the tank can be described referring to the vibration modes and the associated load components shown schematically in Figure 6.4.

The convective component (sloshing vibration) and the impulsive rigid component (rigid body movement) are independent of the design of the tank shell. The impulsive flexible component (interaction vibration), on the other hand, generates design-relevant pressure components, especially on flexible tank shells (e.g. thin-walled steel tanks); in rigid tanks it is generally negligible for design.

The hydrodynamic effects mentioned above must be considered in the design of tank structures. For rigid tanks (e.g. made of reinforced concrete), the calculation method according to Housner (e.g. [62], [66]) can be used. However, as this method does not include the impulsive flexible vibration component, it is unsuitable for the design of thin-walled tanks, for example made of steel. Furthermore, Housner's method only provides the seismically induced base shear, the overturning moment at the tank base, and the height of the sloshing wave at the liquid surface. Internal forces of the shell and failure mechanisms in the cylinder can only be estimated indirectly and only as a rough approximation.

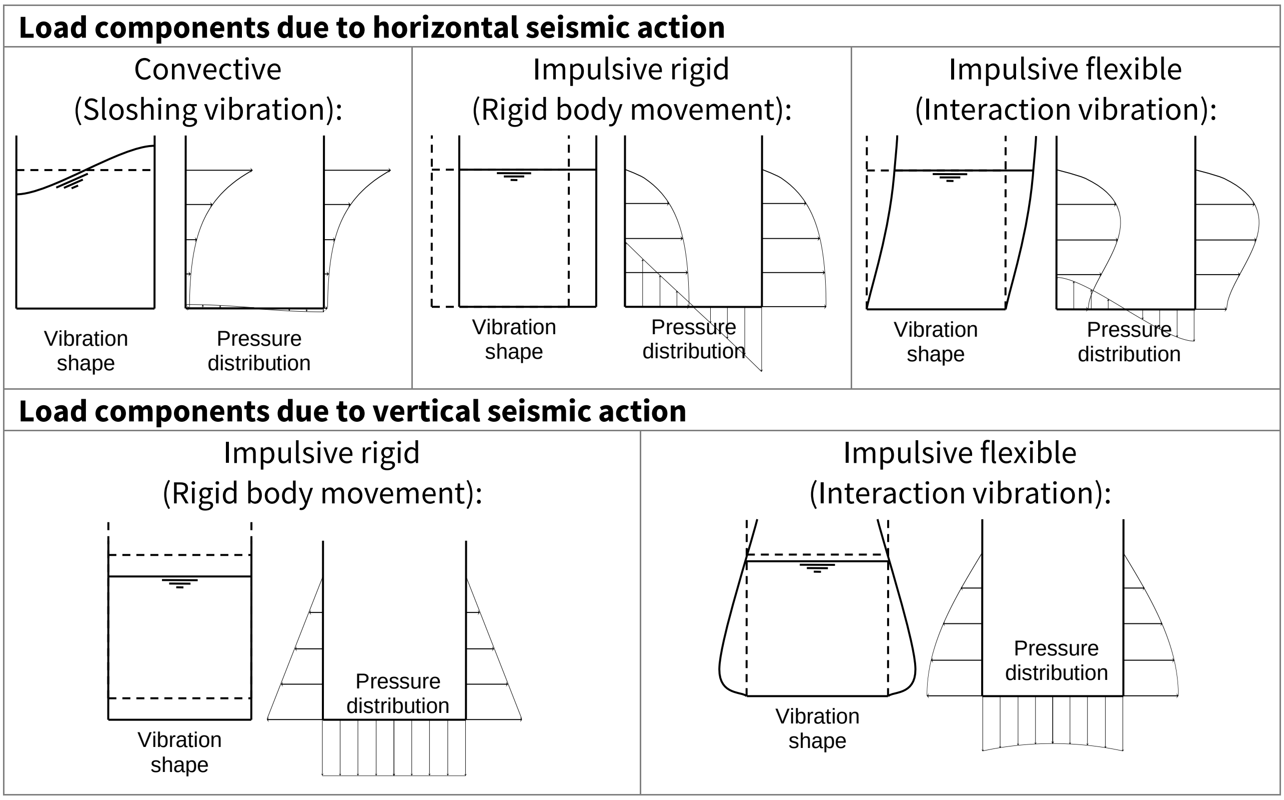


Figure 6.4: Seismically induced vibration modes in liquid-filled tank structures and associated qualitative pressure distribution on the tank shell

A comprehensive calculation concept, which also allows the direct determination of the internal forces of the shell, is based on a formulation of the pressure components as a function of the dimensionless cylindrical coordinates ξ, ζ and θ (equation (6.4)).

These pressure functions result mathematically from the velocity potential of a liquid and various boundary conditions to take into account the respective vibration component. The calculated pressure components can then be applied to a finite element model of the tank and are, thus, incorporated in the overall calculation.

$$p_j(\xi = 1, \zeta, \theta) = R \cdot \rho_L \cdot C_j(\zeta, \gamma) \cdot \cos(\theta) \cdot a_j \cdot \Gamma_\mathrm{j} \qquad (6.4)$$

where p_j = Pressure component j (convective, impulsive rigid (horiz. or vert.) or impulsive flexible (horiz. or vert.))

ξ, ζ, θ = Tank coordinates (dim.-less radius, dim.-less height, circumferential angle)

R = Tank radius (with dimension)

ρ_L = density of the stored liquid

C_j = Calculation coefficient of pressure component j;
as formula: series expansion of $cosh$- or Bessel functions, respectiv. [60][58];
can be tabulated as a function of tank slenderness [67]; [68]

γ = Tank slenderness $\gamma = H/R$

a_j = seismic action on the vibration component j as spectral value

Γ_j = Participation factor of the vibration component j

The complete formulae to determine the pressure components are given in the informative annex A of DIN EN 1998-4 [31]. Alternatively, a detailed presentation of the calculation concept, including background information and all necessary equations, can be found in [67] and [66]. Tables for the determination of the calculation coefficient C_j are also provided there. The handling of the otherwise quite complex mathematical formulation of the coefficient in the context of tank design is therefore redundant.

The pressure functions named above apply to the common case of above-ground, upright, cylindrical, ground-anchored tank structures.

Re: (8) Variability of ground motion for above-ground pipes

No further explanations.

Re: (9) Calculation of buried pipes

Notes on the calculation of buried pipes can also be found in the informative annex B of DIN EN 1998-4 [31].

6.3 Calculation of displacements

Re: (1) Reference to the code text

No further explanations.

6.4 Non-structural components

Non-structural components comprise non-structural building parts (e.g. non-structural interior and exterior walls, parapets, balustrades, wall cladding) as well as technical installations (e.g. apparatuses, vessels, pipes, pumps and other process engineering components; see also explanation of the term "components" in section 3).

Re: (1) Simplified equation for design of components

For the design of non-structural components within buildings, standards often assume that the seismically induced horizontal acceleration increases linearly over the height of the building. Thus, a linearly height-dependent calculation formula for the calculation of the horizontal equivalent static load is provided (design force given as a function of the installation height z over the structure height H). However, since the vibration behaviour of supporting structures in plant engineering sometimes deviates significantly from supporting structures in common buildings, the above assumption can lead to incorrect and unsafe equivalent static loads ([61], [74]).

Therefore, it is recommended at this point to determine the horizontal equivalent static load F_a following guideline 450 of the American Federal Emergency Management Agency [40] according to equation (6.5) and thus to consider the actual vibration behaviour of the load-bearing structure.

Alternatively, the equivalent static load can be calculated according to equation (1) of the guide-line, which specifies the static equivalent load independent of the installation location of the component within the load bearing structure. Using this simplified equation, the acceleration at the building site (ground acceleration) is multiplied by scaling factors to estimate the acceleration at the centre of mass of the component. The equation is equivalent to the upper limit of the simplified design equation of ASCE 7-16 [44] for a structural damping of $\xi = 5\ \%$.

The scaling factor that considers the amplification effects due to the supporting structure depends on the damping properties of the supporting structure: the smaller the structural damping of the supporting structure, the larger the floor acceleration response within the supporting structure (correction factor $\eta = \sqrt{10/(5+\xi)}$ where ξ = structural damping of the supporting structure in %; see also explanations to clause 6.1 a (3)). Typical damping values for elastic load-bearing behaviour are: reinforced concrete structure: $\xi = 3 - 5\ \%$; welded steel structure as well as prestressed concrete: $\xi = 2 - 3\ \%$; bolted steel structure: $\xi = 5 - 7\ \%$ [56]. Structures have higher damping values when loaded up to the yield point.

Amplification effects due to possible dynamic interaction between component and supporting structure are included by the global scaling factor 1.6.

In the following, the individual elements of the simplified design equation for non-structural components are explained in groups:

ground acceleration at the building site	$S \cdot a_{gR} = S \cdot \frac{S_{aP,R}}{2.5}$
consideration of amplification through the supporting structure (if necessary, consider structural damping <5 %; Factor 2.5 corresponds to scaling to the plateau of the elastic response spectrum of the structure)	$2.5 \cdot \eta \cdot 1.0$
➔ „calculative max. floor acceleration“:	➔ $S_{e,max} = 2.5 \cdot S \cdot \eta \cdot a_{gR} \cdot 1.0$
consideration of amplification due to interaction between component and supporting structure	1.6
➔ calculative max. acceleration of the component:	➔ $a_{component} = 1.6 \cdot S_{e,max}$
consideration of the risk potential of the component	γ_a
mass of the component	m_a
➔ equivalent static load „$F = m \cdot a$“	➔ $F_a = 1.6 \cdot S_{e,max} \cdot \gamma_a \cdot m_a$

Often, the substructure of components is significantly stiffer in one horizontal direction than in the orthogonal direction. In this case, it is usually sufficient to verify the substructure applying the simplified equivalent static load in the weaker direction. If a decisive direction cannot be identified clearly, both directions must be checked. The assessment depends on individual local conditions.

Re: (2) Design equation for non-structural components considering the installation height above ground as well as the vibration behaviour of the supporting structure

The more detailed design equation for non-structural components is based on guideline 450 of the American Federal Emergency Management Agency [40]. It doesn't estimate the floor acceleration a_i as a lump sum, but requires its determination by means of a modal analysis of the numerical model of the load-bearing structure[3].

[3] If the (elastic) floor response spectrum for the installation level of the component is known, equation (6.5) can be used analogously. In this case, the dynamic amplification that depends on the vibration period of the component is already included in the floor response spectrum; the spectral acceleration of the floor response spectrum $S_a(T_{component})$ then replaces the product $a_i \cdot A_a$ in equation (6.5).

In addition, the dynamic amplification factor A_a that accounts for possible interaction between the component and the structure is specified with respect to the type of component. Details on the individual factors of the equation are explained below.

Subsequently, the application of the equation is illustrated by an example.

$$F_a = a_i \cdot m_a \cdot \frac{\gamma_a}{q_a} \cdot A_a \cdot A_T \tag{6.5}$$

F_a must not be set smaller than: $0.3 \cdot S_{e,max} \cdot \gamma_a \cdot m_a$
and need not be assumed to be greater than: $1.6 \cdot S_{e,max} \cdot \gamma_a \cdot m_a$.

where F_a = Equivalent static load for a horizontal seismic direction

a_i = Resulting acceleration of the installation level i [m/s²] in the considered seismic direction, determined from a structural analysis with the modal response spectrum analysis employing the <u>elastic</u> response spectrum with $\gamma_I = 1.0$

γ_I = Importance factor of the supporting structure [-]

m_a = Mass of the part / component [t]

γ_a = Importance factor of the part / component according to section 5.3 [-]

q_a = Response modification factor of the component depending on the constructional detailing [-] (Table 6.2 and Table 6.3)

A_a = Dynamic amplification factor of the component [-] (Table 6.2 and Table 6.3, Figure 6.5)

A_T = Factor to account for torsional vibrations of the supporting structure that amplify the components' acceleration [-] (see equation (6.6))

$S_{e,max}$ = Plateau value of the elastic response spectrum [m/s²], determined with the building's importance factor $\gamma_I = 1.0$ and the damping correction factor that is applicable to the supporting structure η; $S_{e,max} = 2.5 \cdot S \cdot \eta \cdot a_{gR} \cdot 1.0$

S = Soil factor

η = Damping correction factor to account for structural damping of the supporting structure; $\eta = \sqrt{\frac{10}{5+\xi}} \geq 0.55$; $\eta(\xi = 5\ \%) = 1.0$; $\eta(\xi = 2\ \%) = 1.2$

ξ = Value of the viscous damping of the supporting structure [%]; usually = 5 %; for certain constructions and materials, however, the choice of a lower damping value may be necessary (see explanatory notes to clause 6.1 a(3))

The equivalent static load according to equation (6.5) must be determined for both horizontal seismic directions, since the structural response determined by modal response spectrum analysis can differ for the different structural (axial) directions. The resulting stresses of the investigated element are then to be superimposed in a suitable manner.

The product of the component mass m_a and the expected maximum floor acceleration a_i is the basis of the equivalent static force calculated according to equation (6.5). Thereby, index i designates the installation level on which the component is mounted.

The floor acceleration a_i can be determined in a simple way by structural analysis using the modal response spectrum analysis (see example below). Thereby, the actual vibration behaviour of the structure is considered in the design of the component since the floor acceleration a_i is calculated on the specific structure instead of a linear scaling (which would approximate the first eigenmode of a common building). In the calculation models of the load-bearing structure used for the determination of a_i, the components can usually be represented as concentrated masses on the floors without modelling their substructures (see explanations to clause 6.1 a (2)). No significant additional computational effort is required for the determination of the floor accelerations.

For the design of non-structural components, the potential of the load bearing structure to dissipate energy by non-linear material behaviour must not be accounted for. That is reasonable because the structure initially dissipates the energy introduced by the earthquake only via viscous damping but not by non-linear material behaviour. Therefore, the floor accelerations are determined using the elastic response spectrum (modal response spectrum analysis, see above). Typically, a value of 5 % viscous damping may be assumed for the structural damping (see guideline clause 6.1 a (3)). However, for certain structures and materials it may be necessary to choose a damping value different from 5 % (steel structures, for example, may have a much lower damping value than the reference damping 5 %; see explanations to clause 6.4(1)). In this case, the damping should be adjusted accordingly when determining the elastic response spectrum (DIN EN 1998-1 [27] clause 3.2.2.2 (3)).

In order to determine the actual horizontal design force F_a that is applied in the centre of mass of the component, the product of mass and acceleration described above is scaled by some factors:

The factor γ_a represents the importance factor of the component, which is assigned to the component according to its damage potential and the possible damage effects according to Tables 5.1 to 5.3 of the VCI-guideline. The distinction between γ_a (for components) and γ_I (for the supporting structure) is merely a notational differentiation of the indices.

The factor q_a is the response modification factor of the component and, in analogy to the behaviour factor q of the supporting structure, takes into account the possibility of energy dissipation within the component structure itself. Substructures on which the component is mounted on the actual supporting structure level may also be taken into account here. For components with a high plastic deformation capacity, a maximum value of $q_a = 2.5$ may be applied in the more detailed design equation. A response modification factor of $q_a > 1.0$ may only be applied if the design of the component and its substructure ensure corresponding energy dissipation. Energy dissipation due to plastic deformations in anchors and other connecting means should not be taken into account in the calculation (cf. explanations to clause 4.c (6)).

The dynamic amplification factor A_a considers the dynamic increase of the acceleration response of the non-structural component / apparatus compared to the floor acceleration. As with q_a, a possible substructure on or within the mounting level of the actual supporting structure is to be taken into account. It depends on the ratio of the eigenperiods of the component and the main supporting structure T_a/T_1. For very stiff components (eigenperiods less than 0.06 s, which corresponds to an eigenfrequency greater than 16.7 Hz), generally no dynamic amplification is expected. In this case the dynamic amplification factor is set to $A_a = 1.0$. Table 6.2 and Table 6.3 suggest reference values for the amplification factor for exemplary types of components.

If the ratio of eigenperiods between the component and the load-bearing structure is known, the dynamic amplification factor A_a can also be determined using the curve shown in Figure 6.5. Here, the theoretically correct lowering of the dynamic amplification to 1.0 for secondary structures that are much stiffer than the supporting structure ($T_a/T_1 \rightarrow 0$; dotted line) is not taken into account in the diagram because of the uncertainties in determining small period ratios. The relatively wide plateau range also takes into account the resonance effects with higher natural frequencies of the structure [60] that are often relevant for soft structures. The qualitative curve in Figure 6.5 is based on investigations by the American National Centre for Earthquake Engineering Research [52].

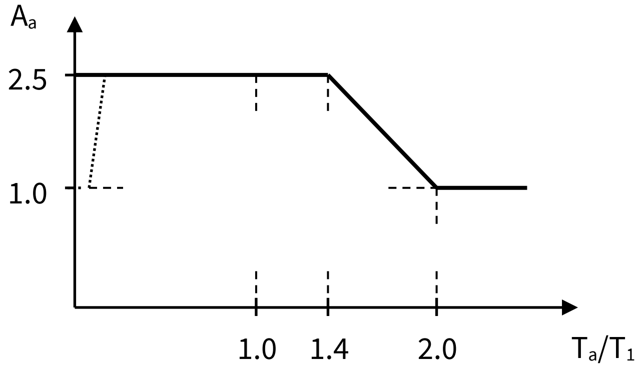


Figure 6.5: Diagram for determining A_a as a function of the period ratio between component and load bearing structure (according to [41])

Table 6.2 and Table 6.3 provide examples for amplification factors A_a and response modification factors q_a based on the ASCE 7-16 standard [44] for various types of components. For components not named, values are to be chosen sensibly. Likewise, the values given in the tables may be deviated from in justified cases, whereby A_a must not be chosen smaller than 1.0 and q_a must not be assumed to be greater than 2.5.

Table 6.2: Parameters A_a and q_a for components specific for plant engineering

Components specific for plant engineering	A_a	q_a
Typical mechanical components		
(Pressure) vessels, pumps, compressors etc. directly anchored	1.0	1.0
(Pressure) vessels, pumps, compressors etc. on substructure	1.5	1.5
Thin-walled small vessels, if calculated with eq. (6.5)	1.5	1.2
Furnaces and boilers	1.0	1.5
Slim, deformable components such as small chimneys	2.5	2.0
Conveyor systems	2.5	2.0
Vibration-isolated components	1.0	2.5
Pipe systems		
Highly deformable (e.g. pipes designed for temperature variation)	1.5	2.5
Partially deformable	1.5	1.5
Hardly deformable (e.g. systems made of brittle material)	1.5	1.0
Truss constructions	1.5	2.0

Notes:

For rigid components (frequency > 16 Hz), A_a can generally be assumed to be 1.0. For flexible components, a value of 2.5 can be assumed for A_a.When estimating the frequency of components, the support within the structure (e.g. on soft intermediate girders of a steel platform) should be taken into account [73]. More precise values for A_a can be determined depending on the ratio between the natural period of the supporting structure and the natural period of the component.

For components with a low plastic deformation capacity, a value of $q_a = 1.0$ is generally assumed. For components with a high plastic deformation capacity, a value of $q_a = 2.5$ can generally be chosen. Intermediate values can be assigned reasonably.

Table 6.3: Parameters A_a and q_a for non-structural building components

Architectural / building component	A_a	q_a
Non-structural masonry walls	1.0	1.5
Non-structural walls made of other materials	1.0	2.0
Parapets and balustrades	2.5	2.5
Facade elements and wall cladding		
Highly deformable (elements and their substructure)	1.0	2.5
Hardly deformable (elements and their substructure)	1.0	1.5
Suspended ceiling panels	1.0	2.5

The notes of Table 6.2 apply analogously.

The torsion factor A_T considers the influence of torsional vibrations of the load bearing structure on the seismic response of the components. If the floor accelerations are determined on a 3D-model, $A_T = 1.0$. If the floor accelerations are determined on two 2D-models of the load-bearing structure, the torsion factor can be estimated via the torsional sensitivity of the floor level. That is described by the condition in DIN EN 1998-1 [27] clause 4.2.3.2 (6) (see also section 4.a above or equation (6.7)): In the case of structures that are not very susceptible to torsion, $A_T = 1.0$ may be applied. If torsional vibrations occur to a certain extent, then components on the periphery of the building will be subjected to greater accelerations than those near the centre of stiffness. Accordingly, the acceleration determined on the 2D-model must, in this case, be multiplied by a torsion factor $A_T > 1$ depending on the torsional susceptibility of the structure and the location of the component. If the criteria regarding regularity according to Eq. (6.7) or Eq. (4.1) are only just met, $A_T = 3$ should be applied. Intermediate values can be estimated with engineering expertise. In the case of highly irregular structures, the floor acceleration should be determined on a 3D-model of the load-bearing structure (cf. DIN EN 1998-1 [27] clause 4.3.1 (5)).

$$1.0 \leq A_T \leq 3.0 \qquad (6.6)$$

Estimate of A_T via engineering assessment of the torsional susceptibility of the structure:

$$e_{0i} \leq 0.3 \cdot r_i \qquad \text{and} \qquad r_i \geq l_s \qquad (6.7)$$

Re: (3) Combination of seismic action with static loads

No further explanations.

Example:

The equivalent static load (verification at the ultimate limit state) for an elevated tank on the mid-level of a five-storey production facility is to be determined. The system's characteristics can be taken from the detailed model in Figure 6.6 a).

Site-specific seismic action:	$S_{aP,R} = 1.563\ m/s^2$ *according to DIN EN 1998-1/NA [28]*
Geological-/soil conditions:	*ground condition B-R* ➔ *Soil factor* $S = 1.2$
Eigenfrequency of the tank:	*2.5 Hz*
Tank content:	*Highly flammable substance (non-volatile) with potential effects only within the facility* ➔ $\gamma_a = 1.2$
Ductility of the tank substructure:	*moderate* ➔ $q_a = 1.5$

Mass and stiffness distribution of the load-bearing structure almost symmetrical in plan

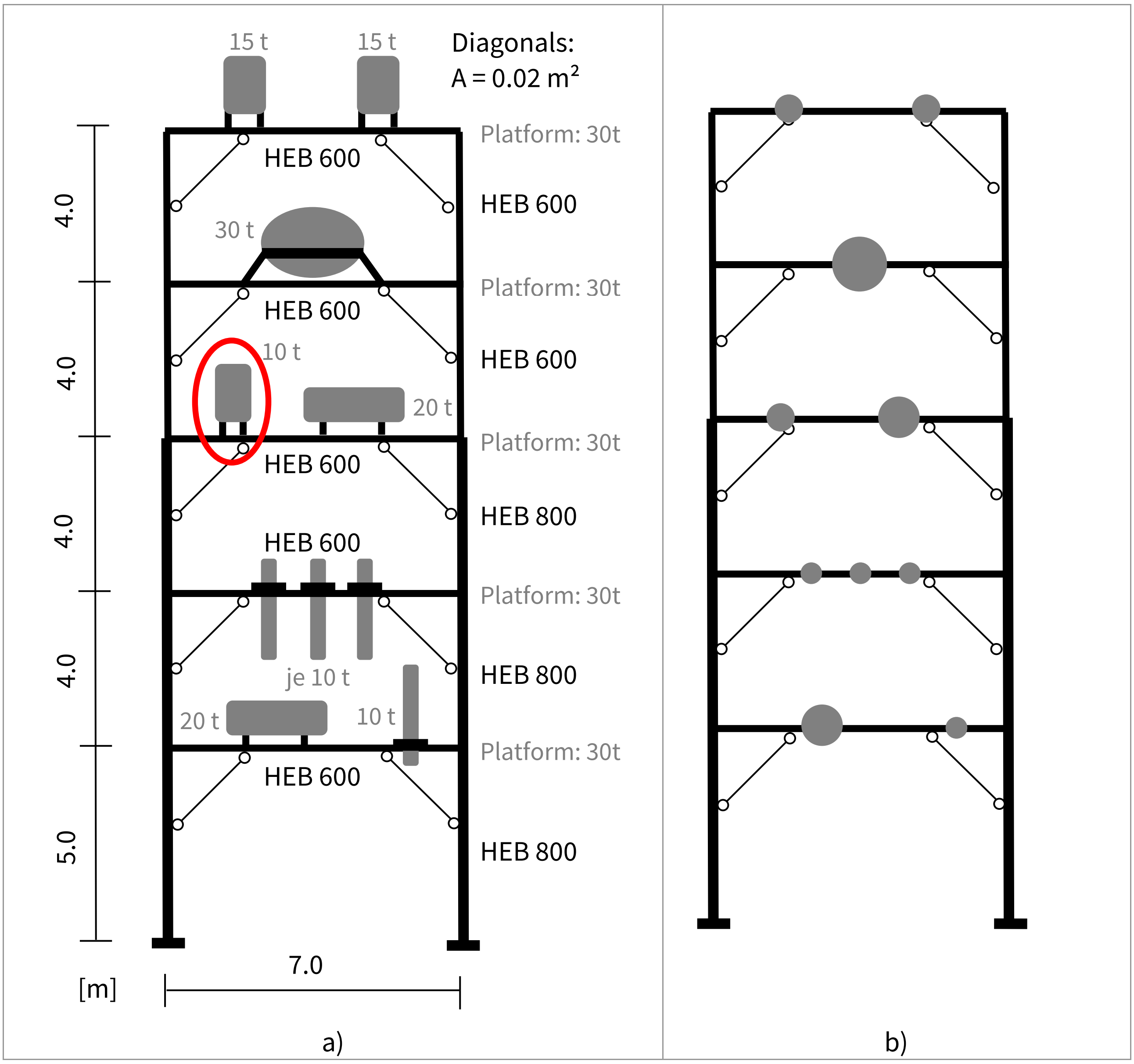


Figure 6.6: System details of the exemplary facility;
a) detailed model; b) simplified model for the calculation

Since the mass and stiffness distribution of the supporting structure is almost symmetrical in plan and the substructures of the aggregates and components do not change significantly the vibration behaviour of the overall supporting structure, the components' masses are considered as point masses on the floors in planar frame models (Figure 6.6 b). The calculation consists of a modal analysis of the structure to determine the eigenmodes and a subsequent spectral analysis to calculate the floor accelerations a_i.

The modal analysis yields the significant eigenperiods of the structure in the main direction as $T_1 = 0.65\ s, T_2 = 0.21\ s, T_3 = 0.11\ s, T_4 = 0.08\ s, T_{13} = 0.06\ s$. *The first three eigenmodes are shown in Figure 6.7.*

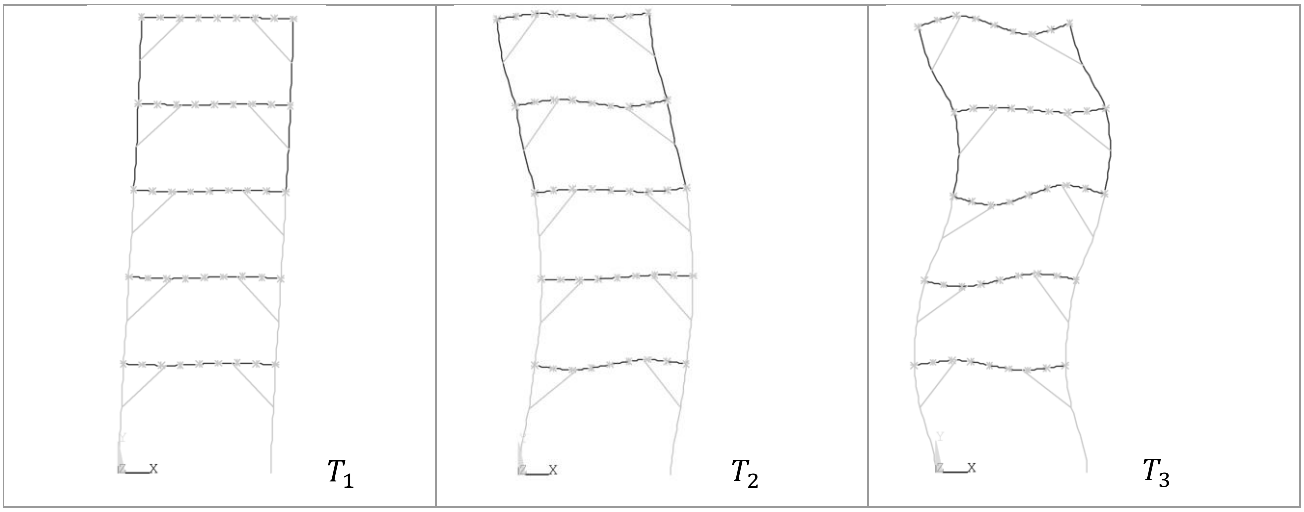


Figure 6.7: The first three eigenforms of the investigated frame

Based on the results of the modal analysis, the maximum accelerations at each level of the structure can be calculated from the elastic response spectrum of the building site (with $\xi = 0.05$ and $\gamma_I = 1.0$ for the load-bearing structure; cf. Figure 6.8) by means of spectral analysis. For each relevant eigenperiod j, the acceleration response $S_{a,j}(T_j, \xi_j)$ is determined from the elastic response spectrum and, based on this, the modal share in the floor acceleration is calculated according to equation (6.8).

$$\underline{a}_j = S_{e,j} \cdot \Gamma_j \cdot \underline{\Phi}_j \tag{6.8}$$

where $\underline{a}_j$ = *Vector of floor accelerations for mode j*

$S_{e,j}$ = $S_{a,j}(T_j, \xi_j)$ = *Ordinate of the elastic acceleration response spectrum according to DIN EN 1998-1/NA [28] for the corresponding eigenperiod and damping of mode shape j and* $\gamma_I = 1.0$ *(the importance factor is included later)*

$\underline{\Phi}_j$ = *Eigenvector of mode j normalised to the mass matrix (* $\underline{\Phi}_j^T \cdot \underline{\underline{M}} \cdot \underline{\Phi}_j = 1$*)*

Γ_j = *Participation factor of mode j with* $\Gamma_j = \underline{\Phi}_j^T \cdot \underline{\underline{M}} \cdot \underline{r}_j = \sqrt{M_{j,eff}}$*)*

$\underline{\underline{M}}$ = *Mass matrix*

$M_{j,eff}$ = *Effective modal mass of mode j*

$\underline{r}_j$ = *Vector that relates the displacements in the essential degrees of freedom with the base displacement in the direction of excitation*

*By means of a suitable superposition rule (e.g. "**S**quare **R**oot of the **S**um of the **S**quares" or "**C**omplete **Q**uadratic **C**ombination"), the decisive floor acceleration $a_{i,decisive}$ is obtained from the n relevant modal components.*

$$\text{e. g. SRSS-superposition: } a_{i,decisive} = \sqrt{\sum_{j=1}^{n} \left(a_{i,j}\right)^2} \tag{6.9}$$

where $a_{i,decisive}$ = *Decisive acceleration of floor i from all modal contributions*

$a_{i,j}$ = *Acceleration response of floor i in eigenmode j*

In many software packages for structural analysis, modules for dynamic analyses are provided, where the modal analysis of a structure, the determination of the effective modal masses in the response spectrum analysis and the superposition of the modal components of the accelerations are implemented. They may be called up automatically, so that the manual implementation of equations (6.8) and (6.9) per matrix algebra is not necessary. For mathematical background information on modal and spectral analysis, refer to the relevant literature on structural dynamics (e.g. [54], [57], [67], [71]).

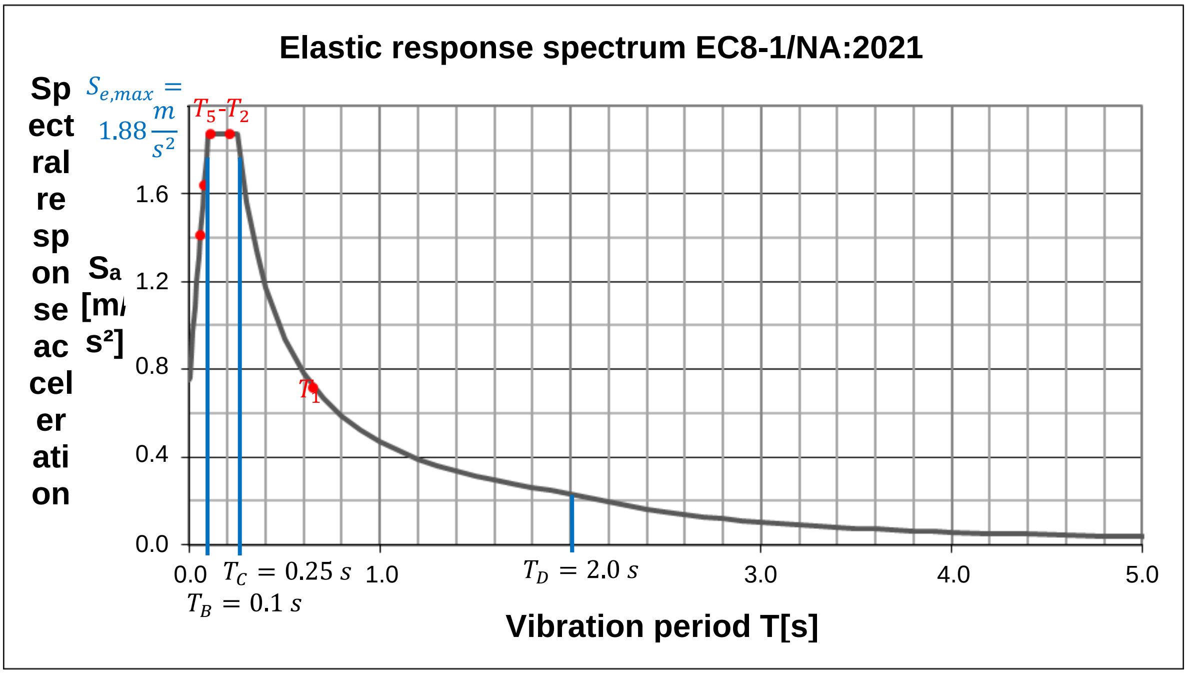
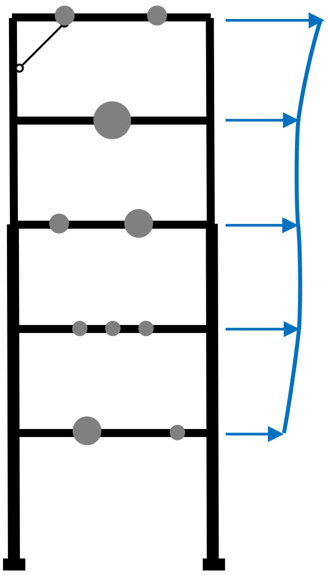


Figure 6.8: ***Elastic response spectrum at the given plant site ($\gamma_I = 1.0, \xi = 5\ \%$)***

The resulting floor accelerations for the above example result in:



$$a_5 = 1.34\ m/s^2$$
$$a_4 = 0.97\ m/s^2$$
$$\boldsymbol{a_3 = 0.96\ m/s^2}$$
$$a_2 = 0.98\ m/s^2$$
$$a_1 = 0.77\ m/s^2$$

The correction factors γ_a, q_a and A_a required in the following result from the properties of the tank and its substructure:

The content of the tank is non-volatile, but highly flammable and will have consequences to humans within the fenced-in plant area in an event of damage. With regard to environmental protection, only minor consequences outside the facility are to be expected in an event of damage; there are no requirements with regard to lifeline facilities. This results in an importance factor of $\gamma_a = 1.2$ according to Table 5.1 to Table 5.3 of the VCI-guideline. For the substructure of the tank a response modification factor of $q_a = 1.5$ is assigned (if a response modification factor greater than 1.5 was used in the verification, additional verifications would have to be carried out to ensure the assumed ductility). The eigenperiod of the tank was given as $T_a = 1/2.5\ Hz = 0.4\ s$ and the basic period of the supporting structure resulted from the modal analysis as $T_1 = 0.65\ s$. With the ratio $T_a/T_1 = 0.62$, the dynamic amplification factor according to Figure 5 results in $A_a = 2.5$. An amplification of the seismic force on the tank due to torsional vibrations of the load-bearing structure is not expected because of the almost symmetrical mass and stiffness distribution in plan. Therefore, the torsion factor is set to $A_T = 1.0$. The mass of the tank is $m_a = 10\ t$, whereby the content of the tank is already included in this value as a mass that vibrates with the tank. This results in the following design force:

$$F_a = a_i \cdot m_a \cdot \frac{\gamma_a}{q_a} \cdot A_a \cdot A_T = 0.96 \cdot 10 \cdot \frac{1.2}{1.5} \cdot 2.5 \cdot 1.0 = 19.2\ kN$$

The minimum design force condition $F_{a,min} = 0.3 \cdot S_{e,max} \cdot \gamma_a \cdot m_a$ must be checked. Here, $S_{e,max}$ may be read as the plateau value of the elastic response spectrum from Figure 6.8 at 1.88 m/s². In addition, the design force must not be chosen greater than $F_{a,max} = 1.6 \cdot S_{e,max} \cdot \gamma_a \cdot m_a$:

$$F_{a,min} = 0.3 \cdot 1.88 \cdot 1.2 \cdot 10 = 6.8 < \boldsymbol{F_a = 19.2} < 36.1 = 1.6 \cdot 1.88 \cdot 1.2 \cdot 10 = F_{a,max}$$

The decisive equivalent static load is $F_{a,x} = 19.2\ kN$, the lower and upper limits do not apply here.

The floor accelerations due to seismic action in the perpendicular direction of the load-bearing structure are determined in an analogous manner. In the present fictitious example, the same structural model is used, but this time with columns rotated by 90° (loading of the weak axis). The eigenperiods and the floor accelerations in this case result in $T_1 = 1.16\ s$, $T_2 = 0.38\ s$, $T_3 = 0.22\ s$, $T_4 = 0.16\ s$ and $T_5 = 0.13\ s$, as well as $a_1 = 0.50\ m/s^2$, $a_2 = 0.51\ m/s^2$, $\boldsymbol{a_3 = 0.51\ m/s^2}$, $a_4 = 0.55\ m/s^2$, and $a_5 = 0.69\ m/s^2$. The dynamic amplification factor of the component remains at the maximum value $A_a = 2.5$ due to the ratio of eigenperiods $T_a/T_1 = 0.4/1.16 = 0.34$. The energy dissipation capacity of the component is assumed to be the same in both directions, so that the design force in the perpendicular structural direction is calculated as $F_{a,y} = 0.51 \cdot 10 \cdot (1.2/1.5) \cdot 2.5 \cdot 1.0 = 10.2\ kN$. Again, the lower and upper limits do not apply either.

For the design of the substructure of the tank and its anchoring, the two horizontal components of the earthquake action must be considered as acting simultaneously, like it is done in the structural analysis of buildings. For this purpose, the resulting stress quantities (e.g. stresses or support reactions) must be combined by means of a suitable method (cf. DIN EN 1998-1 [27] section 4.3.3.5.1(2)).

In order to verify the sufficient anchoring at the ultimate limit state based on linear-elastic behaviour (see explanations to clause 7.c (2) or to clause 4.c (6)), the determined support reactions are to be multiplied by the response modification factor q_a applied before.

The stresses from permanent loads and from unfavourable operating loads are to be superposed to those due to seismic loading.

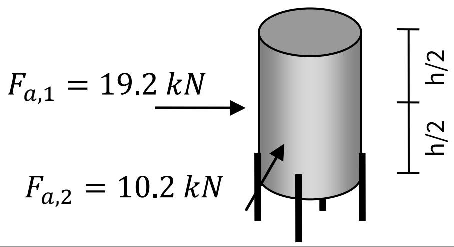


Figure 6.9: Equivalent static loads for dimensioning of the substructure of the tank

For the design of the tank itself, the force F_a is to be distributed to the tank shell according to the mass and stiffness distribution. Hydrodynamic effects due to liquid fillings may be neglected in the present case (cf. section 6.4 (5) of the VCI-guideline and the explanations).

End of the example

Re: (4) Multi-storey components

Provided that multi-storey components are flexible enough not to influence significantly the overall vibration behaviour of the structure, these components and their anchors can be designed using the equivalent static load F_a described above. For this purpose, the horizontal equivalent static load must be calculated at each horizontal support based on the floor acceleration at the supporting level and the mass fraction of the component acting on the corresponding bearing. This load is then applied to the anchoring (Figure 6.10). Alternatively, the equivalent static load can be determined according to equation (1) of the VCI-guideline and applied to the supports. Depending on the individual situation it might be necessary to verify that the multi-storey component can withstand relative displacements of the individual floors that provide the horizontal supports without damage.

Very rigid multi-storey components that have a significant influence on the overall vibration behaviour of the building structure must be incorporated in the numerical calculation model of the building.

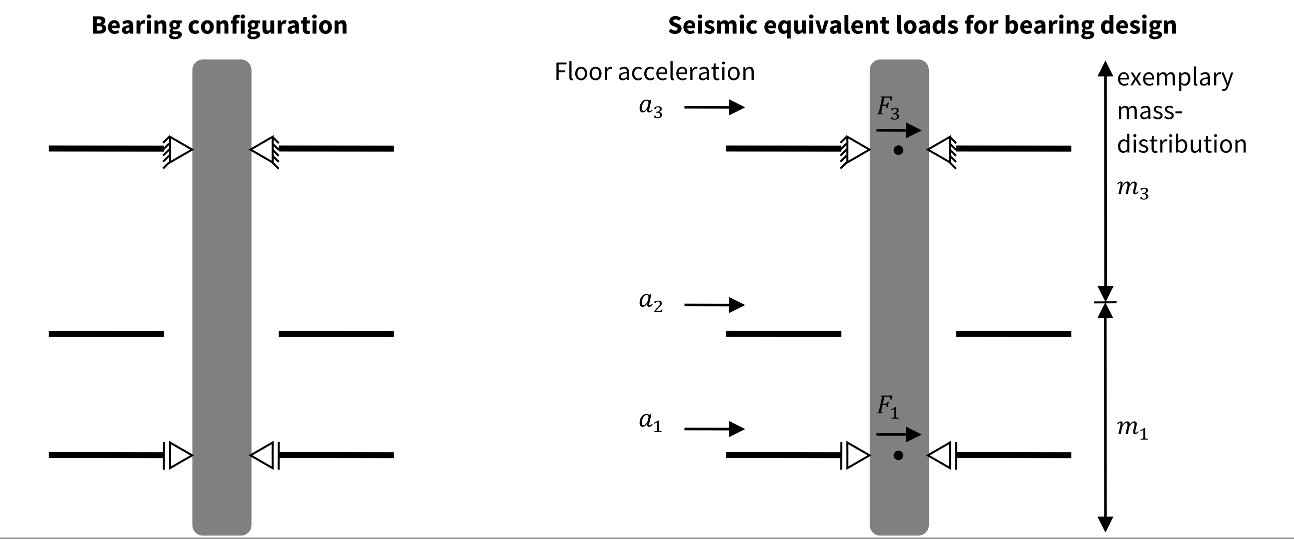
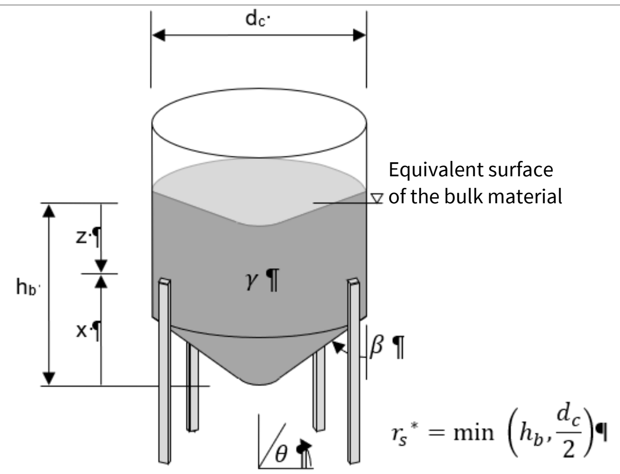


Figure 6.10: **Calculation and application of equivalent loads for multi-storey process columns**

Re: (5) Verification of the tank shell / component housings etc.

For compact secondary structures such as equipment, smaller vessels, pumps, etc., verification of the anchoring is usually sufficient. It might be necessary, however, to comply with operation-related limit values of acceleration or displacement in case of pumps, equipment or systems of measurement and control technology. For larger and more complex secondary structures, where the casing/component itself must be verified, the equivalent static load must be distributed to its static system according to the mass and stiffness distribution of the secondary structure. For silos, DIN EN 1998-4 [32] section 3.3 provides information on the distribution of the seismically induced horizontal and vertical loads (Figure 6.11).



Seismically induced annular pressure on the wall of cylindrical silos according to DIN EN 1998-4 Eq. 3.1

$$\Delta_{ph,s} = \Delta_{ph,s0} \cdot cos\theta$$

where

$$\Delta_{ph,s0} = \alpha(z) \cdot \gamma \cdot \min\,(r_s{}^*; 3x)$$

respectively in the funnel area:

$$\Delta_{ph,s0} = \frac{\alpha(z) \cdot \gamma \cdot \min\,(r_s{}^*; 3x)}{cos\beta}$$

$\alpha(z) = \frac{a(z)}{g}$ with a(z) as response acceleration of the silo

Figure 6.11: **Geometrical characteristics of bulk silos and seismically induced horizontal reference pressure according to DIN EN 1998-4, section 3.3**

Re: (6) Hydrodynamic effects

For the design of smaller vessels and their anchoring, the influence of hydrodynamic effects may be neglected. For large liquid-filled vessels, however, hydrodynamic effects must be taken into account. In section 6.2 (7) of this commentary document, procedures for determining the corresponding convective and impulsive load components are explained.

Re: (7) Simplification for light components and small pipes

In the case of non-structural technical components with a total weight of up to 10 kN, it is assumed that constructional means of connection between component and load-bearing structure are sufficient to safely transfer the seismic inertia forces into the supporting structure. The same applies to supports of individual metallic pipes up to a nominal diameter of DN 100 and for such pipes whose support spans were selected in accordance with Annex 2 to AD 2000 code of practice HP 100 R [1]. Larger valves and installations at pipes must be supported separately.

Re: (8) Mathematical verifications for pipes and their supports

Pipes that do not fulfil the above criteria must be examined by means of pipe static calculations including the consideration of a seismic load case.

Wherever pipe supports are assumed within the pipe static calculation, the corresponding supports must be designed to bear the seismic loads and to transfer them into the main load bearing structure.

Pipe routes usually carry several pipes. A mathematical verification is always required for them, even if only pipes with a small nominal diameter are routed.

Equation (1) of the VCI-guideline can be used to determine the equivalent static load for the mathematical verification of pipe routes. Thereby, all pipes should generally be assumed filled when calculating the component mass. If individual pipes are rarely filled during operation, the load on these pipes can be reduced with the combination factor $\psi_2 = 0.6$ according to Table 5.4 of the VCI-guideline. The factor 1.6 takes into account the dynamic amplification of the action on the supporting frame due to the natural vibration of the pipe; in the case of seismic action in longitudinal direction of the pipe, the dynamic increase due to longitudinal natural vibration of the pipe can be lower and the factor for the calculation of the equivalent load in pipe direction can be reduced accordingly. The horizontal components of the seismic action (in direction of the pipe and perpendicular to it) must be assumed to act simultaneously when verifying the supporting frames of the pipe routing (cf. guideline clause 6.2 (3) and explanations on this).

Re: (9) Stresses from relative displacements of the support points

According to DIN EN 1998-4 [32] section 5.3.3, relative displacements of pipes supported directly at grade are only to be taken into account if there is a risk of ground failure or permanent deformation.

In the case of pipes that are supported on different supporting structures or on parts of supporting structures that vibrate independently, the relative displacement of the support points must be determined by calculation. The displacement of a point of the supporting structure can be calculated for this purpose according to DIN EN 1998-1 [27] section 4.3.4.

7. Safety verifications

7.1 General information

Re: (1) Reference to the text of the standard

No further explanations.

Re: (2) Verification of damage limitation

DIN EN 1998-1 [27] and, correspondingly, other design standards of the Eurocode programme, also set requirements for damage limitation (serviceability limit state). However, the national annex DIN EN 1998-1/NA:2021 [28] NDP re. 2.1(1)P is limited to the verification at the ultimate limit state. This limitation is due to the fact that the seismic action in Germany is rather low compared to other European countries. Accounting for the special conditions in plant engineering, the VCI-guideline offers the verification of damage limitation in order to mitigate possible cost-intensive production interruptions through appropriate design of load-bearing structures and non-structural components.

The seismic action relevant for verifications at the serviceability limit state (requirements for damage limitation) depends on the type of facility and is to be determined by the plant operator (cf. section 7.3).

Re: (3) Combination factors for the safety verifications

The combination rule for the seismic design situation can be taken from DIN EN 1990 [10] section 6.4.3.4. For convenience, it is replicated and explained in section 5.5 above.

7.2 Ultimate limit state

Re: (1) Definition of the ultimate limit state

According to DIN EN 1998-1 [27] section 2.2.2, it must be verified for common buildings that "the structural system has the resistance and energy-dissipation capacity specified in the relevant parts of EN 1998". This sometimes does not meet the requirements for safety-relevant elements of a facility or component, since damage to non-structural components or a failure of process-related components can also pose a danger to human health and safety or the environment. Therefore, the definition of the ultimate limit state according to DIN EN 1998-4 [32] section 2.1.2 is broadened in the VCI-guideline.

In this regard, the verification at the ultimate limit state may include compliance with other limit values than stresses and forces. These limit values may relate to deformations of (non-) structural components and process engineering components or to maximum component accelerations. Such requirements apply if compliance with the limit values is necessary to guarantee the safety-relevant functioning of the component. However, this is not to be mistaken with requirements for damage limitation (required for operational or economic reasons), the verification of which is based on earthquakes with higher probabilities of occurrence (cf. section 7.3).

Re: (2) Reference probability of exceedance

The reference probability of exceedance and the reference return period for the ultimate limit state are specified in DIN EN 1998-1/NA:2021 [28] NDP re. 2.1(1) P. For the design of structures of higher importance or increased risks, a correspondingly increased seismic action (= scaled by means of importance factor) is to be taken as a basis (cf. sections 5.1 and 5.3 above).

7.2.a Supporting structures of facilities

Re: (1) Reference to the text of the standard

No further explanations.

7.2.b Free-standing vessels, silos, tanks and process columns

Re: (1) to (3) References to further standard parts

No further explanations.

Re: (4) Verification of anchors

In principle, anchors should be designed in such a way that plastic deformations only form in the substructure of the component or, if applicable, in the component itself, but not in the anchors.

For the verification of the anchors / fasteners, this means that the stress variables (e.g. support reactions of the component) are determined by applying the design spectrum with $q = 1.0$ (see also DIN EN 1992-4 [16] clause 9.2 (3) a); DIN EN 1998-1 [27] clause 4.4.2.6 (2)P).

Thereby, the stress values from the verification of the actual structure ($q > 1.0$) can be used; in this case, these values must be multiplied, i.e. increased, for the verification of the anchors by the assumed q.

Under certain conditions, ductile behaviour of the anchors (i.e. $q > 1.0$) may be assumed in their design (see DIN EN 1992-4 [16] clause 9.2 (3) b) for anchors in concrete; see DIN EN 1998-1 [27] sections 6.5.2 (3) and 6.5.5 for anchors in steel components). In this case, however, additional verifications are to be carried out to ensure the desired behaviour of the anchors / fasteners (see references above).

When applying the ductile design approach for anchors, the following must be taken into account: If the ductility of the anchors / fasteners is assumed in seismic design, this means that the anchors / fasteners will deform plastically during the design earthquake and may have to be replaced after the earthquake.

7.2.c Non-structural components and piping

Re: (1) General regulations for the verification of non-structural components

The design force F_a can be determined either according to equation (1) of the VCI-guideline or according to equation (6.5) of this commentary document. In both cases, the non-linear energy dissipation <u>of the load-bearing structure</u> must not be applied (= use of the elastic response spectrum) because the structure initially dissipates the seismic energy only via viscous damping (see also explanations on section 6.4). Accordingly, ductility may only be considered for the verification of the non-structural component itself or its substructure ($q_a \geq 1.0$ in equation (6.5)), whereby the connecting means should remain linear-elastic at all times (cf. following clause (2)).

Re: (2) Verification of anchors

In principle, anchors should be designed in such a way that plastic deformations only occur in the substructure of the component or, if applicable, in the component itself, but not in the anchors.

If the internal forces for the verification of the anchoring are determined by applying the equivalent static load F_a determined by equation (6.5), this means that the response modification factor of the component must be set to $q_a = 1.0$. It is also possible to use the stress quantities from the verification of the actual component ($q_a > 1.0$); in this case, these quantities must be multiplied i.e. increased for the verification of the anchoring by the assumed q_a. When using the equivalent static load F_a according to equation (1) of the VCI-guideline, non-linear energy dissipation (q-value) is not taken into account anyway.

Under certain conditions, ductile behaviour of the anchorage (i.e. $q > 1.0$) may be assumed in its design (see DIN EN 1992-4 [16] clause 9.2 (3) b) for anchors in concrete; see DIN EN 1998-1 [27] clause 6.5.2 (5)P for anchors in steel elements). In this case, however, additional verifications are to be carried out in order to ensure the desired behaviour of the anchors / fasteners (see references above).

When applying the ductile design approach for anchors, the following must be taken into account: If the ductility of the anchors / fasteners is assumed in seismic design, this means that the anchors / fasteners will deform plastically during the design earthquake and may have to be replaced after the earthquake.

Re: (3) Regulations for the verification of above-ground pipes

No further explanations.

Re: (4) Regulations for the verification of buried pipes

No further explanations.

7.3 Verification of damage limitation

Re: (1) Requirement of verification

While the verification at the ultimate limit state serves to protect humans and the environment, the verification of damage limitation is intended to ensure that even in the case of minor seismic actions (i.e. those with a higher probability of occurrence) no damage occurs to the facility (structure and technical components), the costs of which would be disproportionately high compared to the construction costs. In plant engineering, the above-mentioned costs include both the costs for repair and the financial consequences of a loss of operation.

The verification of damage limitation is not mandatory, but may be required, for example, by the operator of the facility. In the case of load-bearing structures, it essentially refers to the limitation of deformations. In the case of line-like components (pipes, process columns, etc.), it refers to the limitation of the relative displacement of adjacent supports. In the case of concentrated components (tanks, pumps, etc.), the seismically induced accelerations or the resulting inertia forces can impair the functionality of the component.

Re: (2) Seismic action for the verification of damage limitation

DIN EN 1998-1 [27] clause 2.1(1)P note 3 recommends a reference seismic action ($\gamma_I = 1.0$) with a return period of $T_{DLR} = 95$ years or a probability of exceedance of $P_{DLR} = 10$ % in 10 years for verifications at the serviceability limit state. For sites in Germany, this reduced return period can be taken into account in a simplified way by multiplying the importance factor of the facility used for verifications at the ultimate limit state by the factor 0.5 ($\gamma_{I,verification\ of\ damage\ limitation} = 0.5 \cdot \gamma_{I,verification\ at\ ultimate\ limit\ state}$).

Re: (3) Verification by means of the elastic response spectrum

While the capability of non-linear energy dissipation is taken into account in the design at the ultimate limit state (use of the design spectrum reduced by the behaviour factor q; see also explanation to clause 5.4 (3)), the structure should remain in the linear-elastic range for the verification of damage limitation (use of the elastic response spectrum; see also explanation to clauses 5.4 (1) and 5.4(2)).

7.3.a Supporting structures of facilities

Re: (1) Reference to the text of the standard

No further explanations.

7.3.b Free-standing vessels, silos, tanks and process columns

Re: (1) to (3) References to further standard parts

No further explanations.

7.3.c Non-structural components and piping

Re: (1) to (3) References to further parts of the standard

No further explanations.

8. Specific rules

Re: (1) Specific rules for concrete structures

No further explanations.

Re: (2) Specific rules for steel structures

Further information and design examples of moment-resisting steel frames and connections may be found, for example, in [73] and [59].

Re: (3) Specific rules for composite structures made of steel and concrete

No further explanations.

Re: (4) Specific rules for timber structures

No further explanations.

Re: (5) Specific rules for masonry structures

Non-linear static calculation methods may also be used for masonry structures [69].

Re: (6) Specific rules for foundations and retaining structures

No further explanations.

9. Seismic protection systems

Re: (1) Description of basic principles rather than regulatory requirements

Due to the wide-ranging possible options to reduce seismic effects on load-bearing structures by seismic protection systems, no regulatory requirements are noted regarding the design of these systems.

Re: (2) Categorisation of seismic protection systems

The seismic load case sets extraordinarily high demands on a structure. The energy that is transferred into the structure by the ground motion during an earthquake must be distributed and dissipated in such a way that damage to the entire structure and to components is (largely) avoided. The protective effect must be restored immediately after an earthquake occurred so that possible aftershocks cannot cause any damage. This requires a reliable self-centering capacity of the structural system. Minor readjustments and retrofits are accepted here, provided that the overall load-bearing capacity of the facility and the functionality of the process engineering components remain guaranteed during and after the earthquake.

Basically, four different approaches to achieve vibration reduction in structures can be distinguished [73]:

- Ensuring sufficient strength and stiffness
- Providing dissipative elements so that plastic deformations for energy dissipation occur specifically at certain locations of the structure
- Installation of spring/damper systems with balancing masses to reduce vibration amplitudes (passive and active systems)
- Decoupling of the supporting structure from the seismically excited subsoil by means of base isolation

The individual methods may be combined with each other. It is also possible to install local seismic protection systems within the actual supporting structure in order to secure individual structural or non-structural parts that are particularly susceptible to vibrations or that require particular protection.

Re: (3) Strength and stiffness; dissipative structural design

Especially for weaker earthquakes, the stability and serviceability of a structure may be achieved by choosing sufficiently high strength and stiffness. However, in case of stronger earthquakes, restrictions on the serviceability after the earthquake may occur. Likewise, in the case of stronger earthquakes, damage to the structure can lead to considerable repair costs and loss of operation. With this design concept, the vibrations at the foundation are almost completely transmitted into the structure, so that the technical systems within the building are also subjected to high demands. For the reasons mentioned, this concept leads to uneconomical solutions in the case of expected strong earthquakes and is therefore only recommended in areas with low seismicity.

DIN EN 1998-1 [27] incorporates the consideration of a structure's ability for energy dissipation in the design at the ultimate limit state (DIN EN 1998-1 [27] clauses 2.2.2(1)P and 2.2.2(2)).
The dissipation capacity of the structure can be improved by constructional means depending on the type of construction and the chosen building material. Instructions on ductile design can be found in those sections of DIN EN 1998-1 [27] that refer to specific building materials (sections 5 to 9). When the structural design relies on the formation of plastic areas, those plastic hinges must not form in columns – except for column bases, where plastic joints are acceptable.
A ductile design of the overall structure may prevent failure of the structure in the event of an earthquake. However, since the material deforms plastically when the ductile areas are activated, repair measures are usually necessary after an earthquake to restore the original state and safety level of the structure.

The ductility of a structure can be further increased by the installation of dissipators. An exemplary arrangement of dissipators is shown in Figure 9.1: In structures with diagonal bracings, the relative (storey) displacements occurring in the structure during an earthquake cause changes in the length of the diagonals (Figure 9.1 a). These changes in length are rather small, but can be used systematically to dissipate the induced energy (Figure 9.1 b). For this purpose, dissipating elements are mounted within the bracing diagonals. Those elements allow for controlled deformation and dissipate energy for example as friction-based connections, elastic isolators, steel hysteresis dampers or hydraulic dampers.

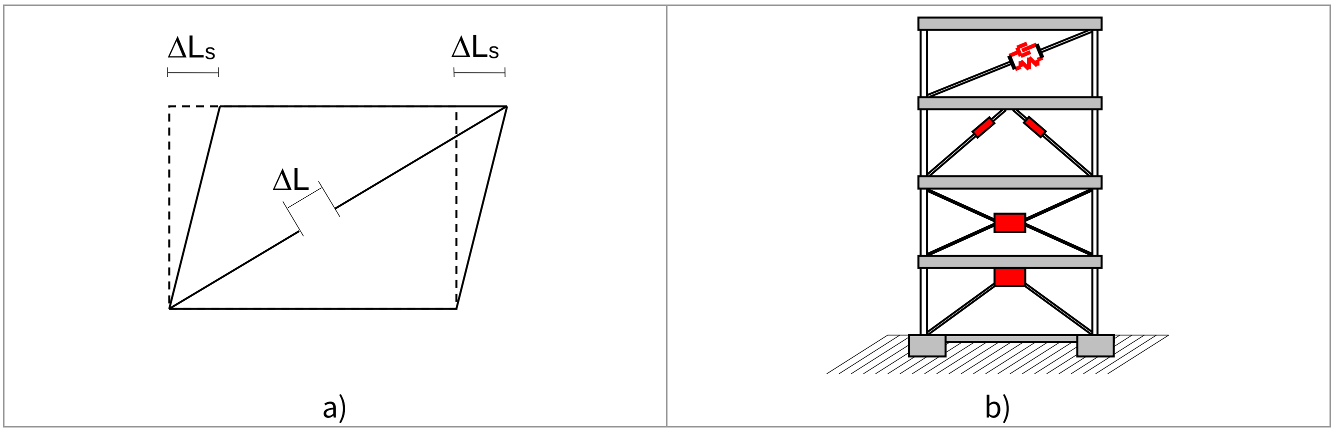


Figure 9.1: **a) Change in length of diagonal bracing**
b) Arrangement of dissipators in diagonal braces

It should be noted that dissipators – apart from hydraulic dampers – have little or no self-centering effect. Thus, the structure is not automatically returned to its original position after an earthquake.

Re: (4) Active and passive spring/damper systems

Another possibility to reduce vibrations is given by spring- / damper systems with balancing masses (Figure 9.2). These systems are often called vibration absorbers, even though they do not eliminate the vibration completely, but reduce it. An additional mass is coupled via springs and dampers to the structure to be damped. This creates a two-mass oscillator (supporting structure + additional mass) from the initially existing idealised SDOF-system (supporting structure). The additional vibrating mass and the coupling to the main system must be designed in such a way that the additional mass and the mass to be damped always vibrate in opposite directions. This method can only achieve reductions in vibration amplitudes in a very narrow frequency range. Typically, it is aimed at reducing the amplitudes of the first eigenfrequency of the structure.

For effective functioning of the absorber, the mass ratio, detuning and damping must be correlated precisely. *Detuning* is a term that describes the ratio between the eigenfrequency of the absorber and the eigenfrequency of the main system.

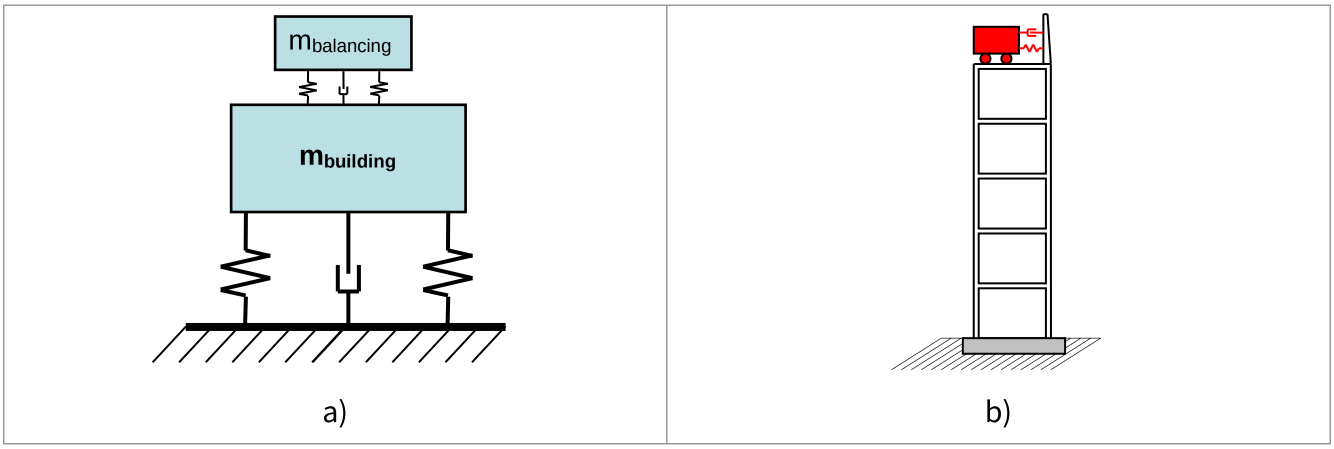


Figure 9.2: **a) System sketch of a vibration absorber**
b) Typical position of a vibration absorber in a multi-storey building

A common application field of vibration dampers (without additional mass) are lightweight bridges. Just like spring- / damper systems with balancing mass, those vibration dampers operate efficiently in a small frequency range. Thus, they are particularly useful if the decisive eigenperiod that is to be damped lies in the plateau of the seismic response spectrum. Note that effective operation of a vibration damper, i.e. effective energy dissipation, generally implies large displacements / relative velocities. Therefore, this type of vibration reduction is less suitable for plant engineering where typically high demands are set on thresholds for tolerance.

Active damping systems use sensors to identify the occurring seismic excitation and react with an electronically controlled additional mass to counteract the deformation of the supporting structure. These systems require complex measurement and control technology and are therefore maintenance-intensive and expensive. In addition, the electronic control requires a continuous power supply [66]. With active systems, a broader frequency spectrum may be attenuated. For plant engineering, however, the use of these systems is problematic, since changes in the mass or stiffness distribution due to structural modifications or changes in production technology require new tuning of the system.

Liquid dampers

In large liquid-filled vessels, the damping of the fluid can be increased for example by suspended grids.

Supporting structures and secondary structures whose overall vibration behaviour is clearly dominated by one vibration mode may also be equipped with tuned liquid dampers to reduce the dynamic reactions.

Re: (5) Base isolation

It is an effective earthquake protection system to decouple (isolate) the structure from the seismic excitation as this combines several positive mechanisms:

- The isolation significantly reduces the natural frequency of the protected structure (i.e. the natural period is increased). As a result, the energy transmitted into the structure is reduced significantly and unfavourable vibrations of the structure / component are minimized. Due to the lower eigenfrequency, the structure may be designed for significantly smaller horizontal accelerations.
- Due to the relative movement between the isolator parts on the building side and on the foundation side, energy may be dissipated by friction / damping in the bearing joint. This further reduces the energy transmitted into the structure and thus reduces the vibration amplitudes. However, this second effect is not provided by all isolation types [66].

With respect to plant engineering, it is a major advantage of this method that both the entire supporting structure as well as individual components (e.g. particularly heavy ones or those requiring specific protection) can be decoupled (Figure 9.3). On the other hand, flexible connections of supply lines (e.g. by means of expansion elements) must be provided when planning base isolation, as the earthquake-induced displacements can increase considerably.

Base isolation in the vertical direction is usually not installed, as vertical seismic loads are typically smaller than the ones in horizontal direction and as the stiffness and strength of the structure are generally sufficient to carry the additional vertical loads (clause 9.3 (3)).

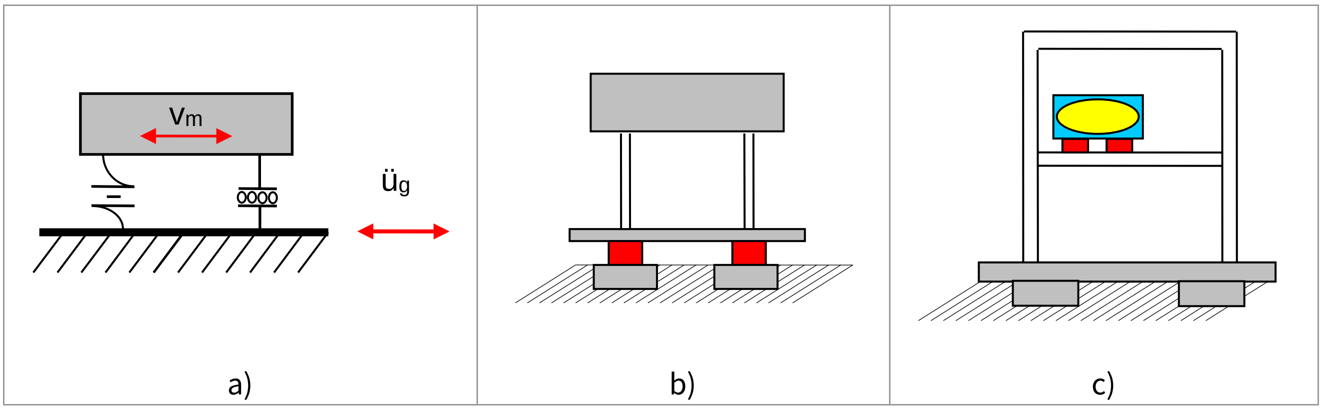


Figure 9.3: a) Schematic sketch of the functionality of base isolation
b) Positioning of isolators as base isolation of a (building) structure
c) Positioning as isolation of a facility part / machine

Various types of bearings are used as base isolation. Combinations of them may also be used (Figure 9.4).

- Elastic isolators (reinforced or unreinforced elastomeric bearings possibly with lead core, springs, steel hysteresis dampers)
- Friction isolators (concave, flat), friction pendulum bearings
- Kinematic isolators (ball/roller bearings)

When selecting the bearing type, the following aspects must be considered:

- The isolated structure must have expansion joints in order to be able to move freely, but this movement must nevertheless be limited by constructional measures. Furthermore, the transfer of regular horizontal loads and displacements (e.g. due to wind, brakeloads of crane hoists, temperature, ...) must be ensured. This can be provided, for example, by independent bracing systems or by additional support connections at the base isolation, which break during the earthquake and subsequently allow free horizontal movement of the main structure.
- Some bearing types for base isolation are also able to transfer vertical loads of the isolated structure (e.g. dead load) to the ground. For others, additional (slide) bearings must be provided for this purpose.
- The damping capacity varies for different types of bearings.
- After an earthquake, the system must return to its initial position by itself (self-centering effect) or with the help of minor readjustments.
- The base isolation must be accessible and maintainable. The durability of the system also plays a major role in cost calculation.
- The bottom parts of the base isolation are subjected to particularly high loads during an earthquake and must be dimensioned and anchored accordingly.

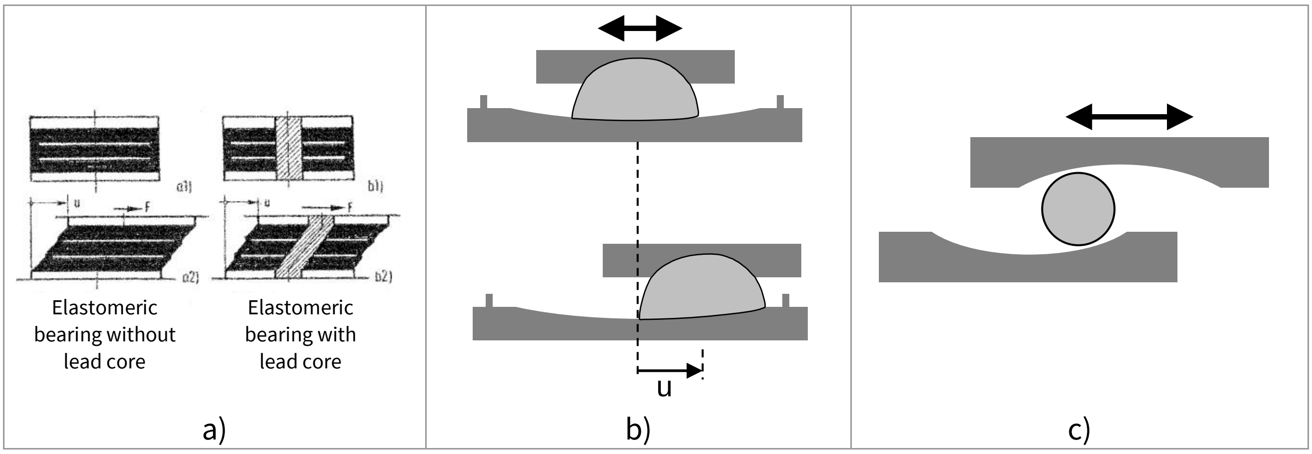


Figure 9.4: **Examples of base isolators [73],**
a) Elastomeric bearing with and without lead core
b) Friction pendulum bearing with sliding shoe
c) Kinematic isolator

Mechanisms of the individual bearing types as well as advantages and disadvantages

Steel hysteresis dampers are used exclusively to reduce horizontal dynamic loads. Thus, additional supports (with gliding capacity in horizontal direction) must be provided to bear vertical loads like dead load of the isolated structure. The damping effect is given by plastic deformation of the steel hysteresis damper, but also by frictional damping at the additional (horizontally gliding) vertical supports. This isolation type does not have any potential to restore the initial position of the structure (no self-centering). In order to prevent fatigue in the damper element, it may also be necessary to replace damper elements after strong earthquakes.

Elastomeric bearings are frequently used in bridge engineering and are most often used as base isolation. Reinforced elastomeric bearings have a high load-bearing capacity in vertical direction and can therefore be employed to transfer dead load. The transfer of regular horizontal loads such as wind can be achieved via different means: additional elements may be provided on the sides of the bearing, which break in case of high loads due to seismic action and establish the freedom of movement of the support. Alternatively, a lead core can be provided in the centre of the bearing, which deforms when subjected to seismic loads. Elastomeric bearings with lead core only have little self-centering potential but dissipate additional energy due to the plastic deformation of the lead core during seismic loading. Elastomeric bearings have particularly good damping behaviour, depending on the material used. However, the material properties are temperature-dependent and elastomeric bearings therefore cannot be used in areas with very low temperatures. Reinforced elastomeric bearings are preferable to unreinforced ones, as they deform less under vertical load and allow for more uniform movement during horizontal dynamic loading.

Friction pendulum bearings have a relatively low overall height and are therefore often used to retrofit structures. Their functionality is based on raising the isolated structure in the event of a horizontal displacement and transforming the kinematic (seismic) energy into potential energy. Gravity thus facilitates the self-centering. Yet, minor readjustments may be necessary after an earthquake event because of friction in the bearing joint. Since the principle of the friction pendulum bearing was only developed in the 1990s, there is still no reliable knowledge about the wear and durability of these bearings. When designing isolators, care must be taken to ensure that only the element parts intended for this purpose undergo plastic deformation. In particular, the connecting elements must remain in the elastic range even in case of strong earthquakes.

Structures equipped with kinematic isolators rest on a large number of balls between dome-shaped troughs. As with friction pendulum bearings, the mechanism is based on a gain of potential energy with horizontal displacement. These bearings have a very good self-centering potential, but hardly any damping effect by the rolling balls. Since they also show very high peak pressure on the balls they are not generally recommended.

10. Assessment of existing facilities

Re: (1) Impacts on the seismic safety of facilities

No further explanations.

Re: (2) Reference to the text of the standard

No further explanations.

Re: (3) to (4) Requirement for regular structural assessment and documentation

Process plants and facilities of the chemical industry or of related industries that are subject to immission control law must be constructed and operated in accordance with the state of the art. If facilities are additionally defined as establishment or part of an establishment according to section 3 clause 5a of the Federal Immission Control Act [50], it is required by section 3 clause 2 no. 2 of the 12th BImSchV (Major Accidents Ordinance, [51]) that "environmental sources of danger such as earthquakes and floods" are explicitly taken into account in the safety considerations.

The regular maintenance intervals for the facilities may be used to assess the seismic safety status of the entire facility. Further investigations are required if critical details are identified during the inspection. Subsequent retrofitting of existing facilities and components must be "proportionate”. This means that the disadvantages of the retrofitting measure (construction costs, business interruption effects during retrofitting) must be balanced against the benefits for (human) health and safety, the environment and the operational reliability of the facility. The benefits for humans and the environment must be quantified in an appropriate manner.

Re: (5) Reduction of the decisive seismic action

If the company plans to decommission the assessed facility in the near future, the probability that the design earthquake will occur within the (remaining) operating time of the facility is reduced. The seismic impact that occurs with equal probability in the shorter time span is smaller than the one that occurs with equal probability in 50 years (50 years = reference service life; cf. section 7.2 of the VCI-guideline). In agreement with the relevant authorities, the remaining operating time can therefore be taken into account in the assessment of the facility, provided that this remaining operating time is less than 15 years. The reduced residual operating time can be taken into account in a simplified manner by multiplying the importance factor of the facility by a factor of 0.75 ($\gamma_{I,existing\ structure} = 0.75 \cdot \gamma_{I\ according\ to\ section\ 5.3}$). The factor 0.75 results from a conservative estimate of the correlation of seismic levels for the reference periods 50a and 25a[4] for sites in Germany.

At the end of the scheduled remaining operating time, the plant must be decommissioned or must be retrofitted to withstand the full seismic loads. A chronological sequence of several verifications with reduced actions in each case is not allowed.

The reduction of the seismic action to account for the remaining operating time is not applicable for verifications of significant modifications of the facility.

Re: (6) Reference to non-linear-static calculation methods

Usually, the lateral force method or the modal response spectrum analysis is used for the verification of existing facilities. Non-linear static calculation methods (see also section 6.2) may be useful to determine a realistic behaviour factor for existing structures and thus to be able to take advantage of load-bearing reserves of the non-linear structural response in their verification.

10.1 Assessment of the current condition

Re: (1) to (2) Basis of the assessment

No further explanations.

Re: (3) Objective of the assessment

Annex A contains an exemplary evaluation form for the initial assessment of the seismic safety of a chemical facility. This evaluation form allows a first rating of the seismic safety.

Re: (4) Consequence if deficiencies have been identified

The regulations of section 10.2 apply to the design of retrofitting measures and to the numerical verifications of the retrofitted structure.

[4] In accordance with AS/NZS 1170.0 [38], a minimum reference period of 25 years should be taken as a basis, even in the case of lower residual operating times.

10.2 Retrofitting

Re: (1) Principle of proportionality

From a legal point of view, it should be noted that the retrofitting of facilities with regard to seismic effects does not serve as averting of danger, but as a precautionary measure. Immission control law therefore does not generally require the adaptation of existing facilities to the current state of the art. Therefore, requirements for new facilities are not automatically to be applied to existing facilities. Whether a retrofitting measure is proportionate may only be assessed on a case-by-case basis i.e. for each individual facility and for each individual requirement.

Re: (2) Constructional measures

Possible structural retrofitting measures may include, among others:

- Improvement, reinforcement or complete replacement of individual elements,
- Modification of the load-bearing system (improvement of the bracing system, elimination of selected structural connections, widening of joints, removal of vulnerable components),
- Optimisation of the mass distribution by changing the position of non-structural components and technical equipment in the structure in accordance with the criteria for the design and dimensioning of new facilities (section 4), as far as possible with respect to process engineering,
- Reduction of permanent loads,
- Installation of new load-bearing elements,
- Functional change of non-structural elements into load-bearing elements,
- Ductility increase as far as technically possible,
- Improvement of damping properties; installation of seismic protection systems (section 9),
- Improvement of the dynamic behaviour of the foundation,
- Improvement of the anchoring of non-structural components and technical installations,
- Replacement of rigid pipe connections with flexible connections; retrofitting of expansion elements.

In accordance with the notes on conceptual and structural design in section 4 of the VCI-guideline and the commentary document, high priority of structural retrofitting measures lies on improving the regularity in plan and/or elevation of the main structure.

If retrofitting measures require an increase in strength, this must not reduce the available global ductility.

Re: (3) Verification of the stability of the reinforced structure

Generally, the facility must be reinforced in such a way that it can bear the seismic actions at site according to the standard (specification of importance factors in accordance with section 5.3 of the VCI-guideline).

However, if it is planned to decommission the facility in the near future and retrofitting measures serve exclusively to eliminate significant deficiencies in the load-bearing capacity or to avoid imminent risks, the remaining operating time of the facility may be taken into account in the numerical verifications. In this case, the regulations in section 10 clause 5 of the VCI-guideline and the corresponding explanations in this commentary document apply.

Re: (4) Confidence factor

No further explanations.

Re: (5) Calibration of the calculation models using eigenfrequency measurements

No further explanations.

Re: (6) Operational measures

Operational measures may include, for example, limiting the permissible variable floor loads (live loads) in supporting structures or reducing the maximum filling level of large vessels, which reduces the inertial forces and thus the stresses. If storage areas are converted into operational areas, a reduced dynamically activated mass in the event of an earthquake may also be assumed (cf. combination factors in section 5.5), which generally reduces the stresses. Possible torsional effects due to irregular mass distribution must be taken into account.

It may also be possible to reduce the risk of damage by changing the usage of critical components / parts of the facility – this may justify the assignment of a smaller importance factor γ_I for numerical verifications of the retrofitting measure.

11. Bibliographic references

For the references on codes and standards given in the VCI-guideline, the current edition of the respective document, including all amendments, applies in each case. The informative references in the commentary document refer to the dated versions of the standards listed below. It cannot be ruled out that references change due to the updating of standards.

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Annex A

Initial assessment of the seismic safety of a chemical facility

The following catalogue of key questions lists essential questions that can be used to assess the seismic safety of a chemical facility during an on-site inspection. These key questions are divided into the categories I. Supporting structure, II. Non-structural components and apparatuses, III. Piping and IV. Storage goods. They refer to the basic design and construction of the facility and individual details as well as to the current condition of the supporting structure, technical components and their fastenings.

During an inspection of the facility, all structural and non-structural components should be assessed with respect to the conceptual ideas of the listed key questions. Hints and clues for the optimum structural design can be found in section 4 of this commentary document. If, for all components, the individual questions are evaluated positively in terms of seismic safety, it can be assumed that the facility provides sufficient resistance against seismic effects. However, if weak points are found, the corresponding details are to be listed in the attached form. There, they can be described in more detail and can be evaluated with regard to the possible extent of damage as well as to its significance for health and safety, the environment and the operational reliability of the entire facility (see the following section "evaluation scheme").

The aim of the evaluation is, on the one hand, to localise critical details of the facility and to prioritise the elimination of the individual weak points on the basis of a “detail index”. On the other hand, a “facility index” can be determined and used to evaluate the seismic safety of the entire facility. This also enables a comparison and prioritising of different facilities.

Note:

The given key questions only consider weak points of typical facilities. Depending on the type and individual design of the facility, further critical details may arise, which must then be considered and listed in an analogous manner.

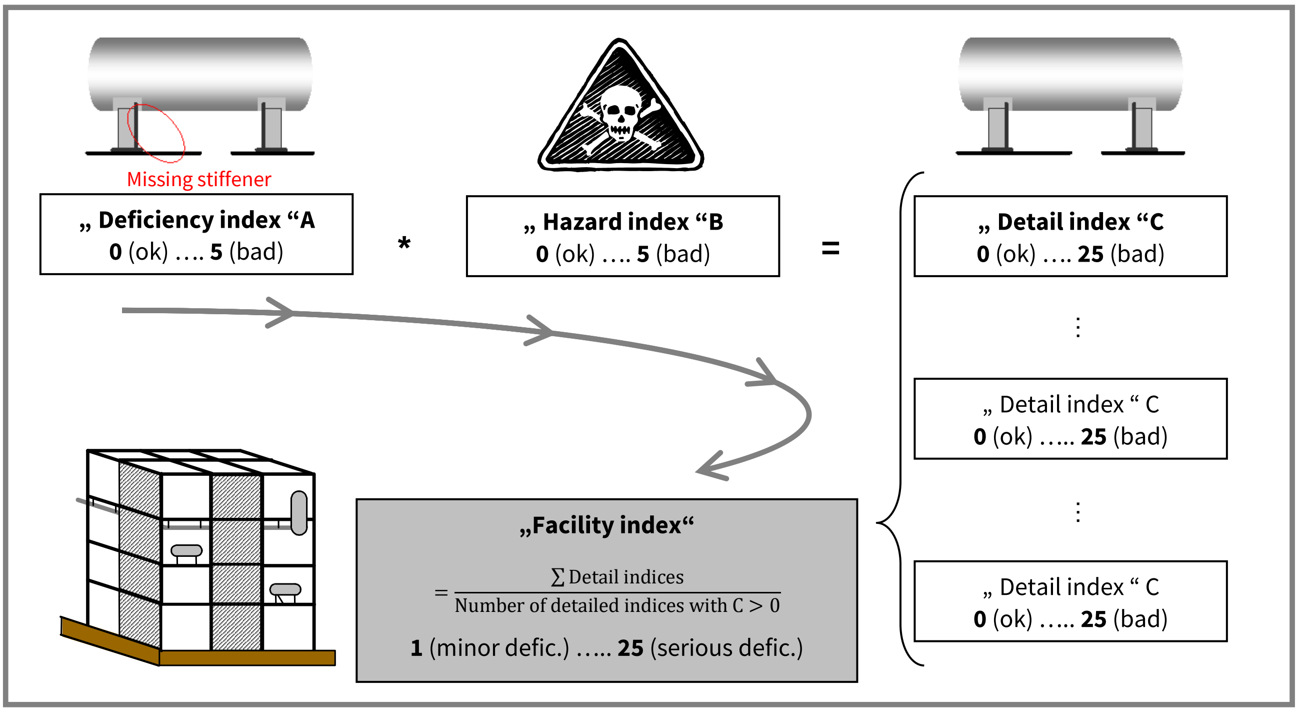
The assessor must therefore have an appropriate expertise in the assessment and seismic evaluation of structures and in the detection of design deficiencies.

The evaluation in terms of numbers (see section "Evaluation scheme") is, by principle, subjective both with regard to the assessment of deficiencies and to their significance. Thus, it makes sense if the same expert carries out the assessment of several facilities of a company in order to ensure comparability of the results.

Assessment scheme:

- The detail-assessment is carried out separately for the severity of the damage / deficiency (deficiency index A) and the hazards that it poses on the overall plant / health and safety / environment (hazard index B). The degree of hazard increases with the risk potential of the handled substance (cf. Table 5.1 of the VCI-guideline), the quantity of the substance and its dispersion risk. Explosion hazard, high pressure, a density comparable to air and lack of enclosure or containment systems favour dispersion.
- Values from 0 (*compliance with the requirements without any problems, no increased importance with respect to health and safety, the environment and the operational reliability of the facility; no measures are required to remedy deficiencies*) to 5 (*component not functional or not present, failure would have serious consequences for health and safety, the environment and / or the operational reliability of the facility; immediate measures are required*) can be assigned. Only integers are permitted as assigned values. A listing of the details in the evaluation form is only necessary if details are rated with a value between 1 and 5. If the rating is 0, there is no deficiency, but this detail can still be listed for information purposes.
- Multiplying the listed indices A (deficiency index) and B (hazard index) results in the detail index C, whose numerical value lies between 0 and 25.
- The facility index results from the average of the detail indices, whereby only those details are considered that show a deficiency (C > 0). Accordingly, the facility index lies between 1 and 25.
- The assessment scheme is graphically illustrated on the following page.
- With regard to the resulting indices, the following priority ranges arise:

Priority range I	**(green):**	**1 - 4**	Problems of minor importance
Priority range II	**(yellow):**	**5 - 12**	Significant deficiencies; remedy is inevitable
Priority range III	**(red):**	**13 - 25**	Serious deficiencies; to be eliminated immediately



Explanation of the assessment scheme

Form: "Detailed listing and assessment":

Column 1:	Consecutive number of the detail listing
Column 2:	Picture of the detail to illustrate the detail description in column 3
Column 3:	Detailed description (location of the detail, exact description of the problem, substances handled if applicable and explanation of the hazard potential)
Column 4:	Importance factor for the described component (if required; may be needed for numerical verification of the component)
Column 5:	Deficiency index A: values from 0 to 5
Column 6:	Hazard index B: values from 0 to 5
Column 7:	Detail index C = A*B; values from 0 to 25

The facility index is calculated on the cover sheet of the evaluation form. In addition, the number of minor, significant and serious deficiencies (priority range 1, 2 or 3) is to be noted here.

COMMENT: An average system with a moderate risk (e.g. facility index 10) may well have individual local serious deficiencies that need to be eliminated as soon as possible! Therefore, the index of the entire facility should be coloured according to the most serious individual deficiency, even if the facility index as a whole is assigned to a lower priority range.

The compilation of the relevant data regarding the facility noted on the cover sheet facilitates a later allocation of the evaluation sheets.

Questionnaire (key questions)

I. Supporting structure

1. Is the stiffness distribution regular in plan and elevation?
2. Is the mass distribution regular in plan and elevation?
3. Are the chosen building material and the chosen type of construction generally appropriate to bear seismic loads?
4. Are the bracings arranged sensibly and effectively?
5. Is it ensured structurally that plastic hinges do not form in columns but in girders?
6. Are expansion joints provided between individual parts of the structure and are they dimensioned adequately?
7. Has the original structure already been changed / retrofitted? If so, do these measures have a negative influence on the load transfer, ductility and seismic safety of the structure?
8. Are there cracks or other damages to the structure?

II. Non-structural components and apparatuses

1. Are the non-structural components (parts of the building or apparatuses / technical components / other heavy objects) adequately connected to the load-bearing structure and are they secured against tipping, slipping or falling?
2. Do vessels and process columns have at least 4 support points that are arranged symmetrically?
3. Are the substructures, bolt- and clamp connections, anchoring and, if applicable, dowelling dimensioned sufficiently?
4. Are the substructures and anchors in flawless condition with regard to damage, corrosion, etc.?
5. Are the anchors designed for the transfer of horizontal loads (e.g. because wind loads have been taken into account in design)?

III. Piping

1. Does the pipe system have sufficient deformability capacity? Is there a sufficient number of expansion elements and deformable pipe connections?
2. Does the pipe system run at a sufficient distance from load-bearing parts or technical components in order to prevent them from colliding with each other?
3. Are the pipe connections dimensioned sufficiently?

4. Do the pipe connections remain tight even under high stress?
5. Are the number and design of the pipe supports dimensioned adequately to bear horizontal loads?
6. Are the pipes protected from external influences (e.g. falling or overturning parts, fire)?
7. Are pipe armatures which are made of fragile material mounted in a stressless state?

IV. Stocked goods

1. Are bottles, barrels, gas cylinders, canisters, crates, (stacked) pallets etc. secured against tipping over or falling down?
2. Are sensitive goods protected from falling parts?

Assessment form for the initial assessment of the seismic safety of a chemical facility

Name of the facility: ______________________

____

Operator: ______________________

Location: ______________________

Seis. action at site: $S_{aP,R}$ = ______________________

Soil- / geol.- condition: ______________________

If applicable, picture / plan of the facility for recognition

Handled substances: ______________________

Facility risk: __________ (Global importance factor in accordance with the VCI guideline "The seismic load case in plant engineering" for the evaluation of the facility's risk; the factor is required for analytical verifications of the entire facility)

Type of facility: ______________________

(e.g. multi-storey steel frame / production facility / tank farm / silo / general cargo warehouse / multi-storey column / pipe bridge / etc.)

Summary of the assessment (*)

Number of all listed details	
Number of listed details with index C > 0	
Sum of the detail-indices (Σ index C)	

▶ Index of the entire facility $= \left(\frac{\sum Detail\ indices}{Number\ of\ detailed\ indices\ with\ C>0}\right)$ (**)		Highlight field in colour according to the most serious individual deficiency	
▶ Number of minor deficiencies (Priority range I)	(1 ≤ index C ≤ 4)		
▶ Number of significant deficiencies (Priority range II)	(5 ≤ index C ≤ 12)		
▶ Number of serious deficiencies (Priority range III) (**)	(13 ≤ index C ≤ 25)		

(*) The assessment scheme is based on Annex A of the commentary document to the VCI guideline "The seismic load case in plant engineering "

(**) serious deficiencies must be eliminated immediately!

Name of the assessor: ______________________

Other participants in the assessment: ______________________

Date of the assessment: ______________________

Con-secut. no.	Image (Photo, sketch, plan detail, or similar)	Detailed description (Location, problem description, substances handled, etc.)	Import. factor (Guidel. 5.3)	Index A (defi-ciency)	Index B (hazard)	Index C (priority C=A*B)
					Carry-over:	
					Sum of the detail indices:	

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Date Signature of assessor