18TH WORLD CONFERENCE ON EARTHQUAKE ENGINEERING
WCEE2024
MILAN

MILAN, ITALY
30TH JUNE - 5TH JULY 2024
www.wcee2024.it

EVOLUTION OF EN 1998-4: SILOS, TANKS AND PIPELINES, TOWERS, MASTS AND CHIMNEYS

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Abstract: *The paper presents prEN 1998-4 (2022) of the new Eurocode generation, which covers seismic design aspects for silos, tanks, above-ground and buried pipelines, ancillary components in industrial plants, and towers, masts and chimneys. This revised document consolidates and summarises the content previously found in EN 1998-4 and EN 1998-6. In particular, the revision now explicitly considers performance levels together with associated return periods. It is consistent with the calculations and design principles derived from the updated general provisions in Parts 1-1, 1-2 and 5, and the design principles presented in Eurocodes 2 and 3. Improvements to the document include the removal of redundancies and inconsistencies and the inclusion of new practical oriented informative annexes. The paper will highlight significant revisions developed under the CEN mandate M515 entrusted to TC250.*

1 Introduction

The new prEN 1998-4 (2022) standard includes seismic design rules for the supply structures silos, tanks and pipelines and provides a new chapter with additional rules for process components and their interactions with the primary structure. Additionally, the regulations from EN 1998-6 (2006) for towers, masts and chimneys have been integrated in a condensed form. The structures covered by prEN 1998-4 (2022) shall be designed to achieve the following objectives: protection of human life and the environment, limitation of damage to preserve full or limited functionality, prevention of induced damage to avoid cascading effects, minimisation of economic and social consequences.

Seismic design can be performed in consequence classes CC1, CC2 and CC3 as defined in prEN 1990 (2022). The choice of the consequence class is based on the severity of collapse or damage, its importance for public safety and civil protection immediately after an earthquake, and its economic, social, and environmental impact. Consequence class CC3 is divided into the two subclasses CC3-a and CC3-b. Class 3-b should be selected if the integrity of the structure, including ancillary elements, is critical for public safety and the environment, or if it could cause damage to connected plant components, nearby buildings, or adjacent facilities. In addition, class CC3-b should be selected for all structures and systems that could endanger the operational civil protection services in the immediate post-earthquake period. In all other cases, CC3 structures may be assigned to subclass CC3-a. The standard does not cover consequence class CC4 for significant risk structures and their associated elements and systems. However, the calculation and design principles given can also be applied to structures classified in consequence class CC4, such as LNG tanks.

The Significant Damage (SD) limit state performance criteria shall be applied. This means that the structure and its associated elements can sustain significant damage while maintaining structural integrity and allowing for controlled leakage of contents. Depending on the characteristics and function of the structure, it may also be necessary to consider the Damage Limitation (DL) or Fully Operational (OP) limit states. The DL limit state is a state in which the system is damaged to a manageable extent and the OP limit state means that the system remains fully functional, with fluid-filled systems retaining their integrity.

The performance requirements are achieved by selecting appropriate return periods of seismic actions in years, $T_{LS,CC}$, depending on the specified limit state and consequence class of the structure under consideration (Table 1). The specified return periods are nationally determined parameters that may be defined differently by the relevant authorities at national level for use in a country. Alternatively, performance factors may be used instead of return periods. The changes in the new standard for different types of structures are summarised in the following chapters.

Table 1: Limit states and consequence classes with corresponding return periods $T_{LS,CC}$

	Consequence class (CC)			
	CC1	CC2	CC3-a	CC3-b
SD	250	475	1300	2500
DL	50	60	150	250

2 Silos

The structural analysis and design of seismically loaded steel, reinforced concrete and pre-stressed reinforced concrete silos is based on static equivalent seismic pressures that must be determined for both the horizontal and vertical components due to seismic excitation. The vertical component is not negligible, as it is often the case in conventional structural design since the acceleration of the highly concentrated silo mass leads to dynamic pressures that are particularly relevant to the design of the ring stiffeners and the hopper outlet. However, the vertical component has not been considered in the current EN 1998-4 (2006) , potentially leading to unsafe silo designs.

2.1 Pressures on circular silo walls and hoppers due to horizontal components of seismic actions

The effect on the shell of circular silos due to the response of the particulate content to the horizontal components of the seismic action of circular silos (or silo compartments) is calculated to:

$$\Delta_{\mathrm{ph,s}} = \Delta_{\mathrm{ph,so}} \cos\theta \tag{1}$$

where θ is the angle ($0^\circ \leq \theta < 360^\circ$) between the radial line to the point of interest on the wall and the direction of the horizontal component of the seismic action, and $\Delta_{\mathrm{ph,so}}$ is the reference pressure normal to the silo wall calculated at points on the silo wall at the vertical distance x from a flat bottom or the apex of a conical or pyramidal hopper:

$$\Delta_{\mathrm{ph,so}} = \alpha(z)\, \gamma_{\mathrm{u}} \min(h_{\mathrm{b}}; d_{\mathrm{c}}/2; 3x) \tag{2}$$

where $\alpha(z)$ is the ratio of the response acceleration at a depth z from the equivalent surface of the stored content, to the acceleration of gravity, γ_{u} is the upper limit of the characteristic bulk unit weight, h_{b} is the overall height of the silo, and d_{c} is the inner dimension of the silo parallel to the horizontal component of the seismic action. The reference pressure normal to the silo wall inside a hopper is calculated with the angle of inclination of the conical hopper wall:

$$\Delta_{\mathrm{ph,so}} = \alpha(z)\, \gamma_{\mathrm{u}} \min(h_{\mathrm{b}}; d_{\mathrm{c}}/2; 3x) \cos\beta \tag{3}$$

The resulting pressure on the silo wall is calculated as the sum of the seismic pressure and the normal hydrostatic pressure p_0. If the sum is negative at any point on the silo wall (implying net suction on the wall), the resulting pressure should be set to zero and the negative pressure should be added to the opposite pressure-loaded side of the silo wall as an additional normal pressure $\Delta_{\mathrm{ph,s,inc}}(\theta)$:

$$\Delta_{\mathrm{ph,s,inc}}(\theta) = \Delta_{\mathrm{ph,s}}(\theta) + \left|p_0 - \Delta_{\mathrm{ph,s}}(\theta)\right| \tag{4}$$

To calculate the seismic pressures, the function of the response acceleration $\alpha(z)$ over the height of the silo is required, which can be determined by response spectrum analysis. As a simplification, the function $\alpha(z)$ can be approximated by linear a linear response acceleration profile, or, in the case of rigid silos, by a constant response acceleration, calculated as the average of the response acceleration at the silo bottom and at the level of the equivalent surface of the stored content.

Figure 3 shows qualitatively the seismic pressures, the hydrostatic pressures due to filling and their superposition for a constant and variable acceleration profile over the silo height. It should be noted that a linear load distribution is assumed in the lower part of the silo to consider that horizontal seismic forces in squat silos do not only act on the silo shell, but also to a large extent also via the friction of the bulk solid in the foundation or substructure.

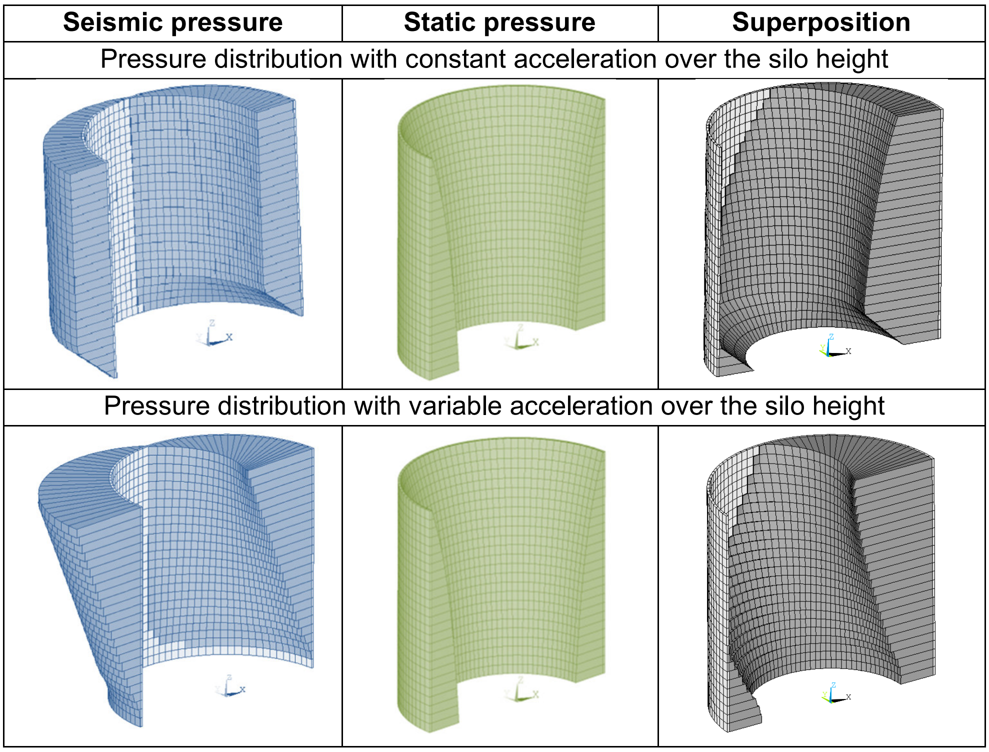


Figure 1. Comparison of qualitatively pressure distributions

2.2 Pressures on silo walls and hoppers due to vertical components of seismic actions

Unlike the current standard EN 1998-4 (2006), the new standard specifies a load approach to consider the vertical seismic action with a scaling factor β_{sc} calculated as follows:

$$\beta_{\mathrm{sc}} = 1 + \frac{S_{\mathrm{rv}}\,(T_{1\mathrm{v}})}{g} \tag{5}$$

where S_{rv} is the ordinate of the reduced response spectrum in the vertical direction at the first natural period $T_{1\mathrm{v}}$. The factor β_{sc} is applied to the frictional pressures p_{wf}, the vertical pressures p_{vf}, and the horizontal pressures p_{hf} for vertical silo walls, silo hoppers and silo bottoms, calculated according to prEN 1991-1-4 (2022).

2.3 Combination of the vertical and horizontal seismic loads

If the force-based approach is used, the structural response of the three components of the seismic action should be evaluated separately and combined. If the silo is not perfectly axisymmetric in plan, the combination should be performed according to prEN 1998-1-1 (2023). For axisymmetric silos in the plan (e.g. vertical cylindrical silos on the ground), a new rule has been introduced. This rule considers only the vertical and a horizontal component of the seismic action to be evaluated. Specifically, the horizontal component is multiplied by a factor of 1.12. The action effect due to the combination of the vertical and horizontal components should be calculated as follows:

$$1{,}12\ E_{\mathrm{Edx}}\ \text{"+"}\ 0{,}30\ E_{\mathrm{Edz}} \tag{6}$$

$$0{,}34\ E_{\mathrm{Edx}}\ \text{"+"}\ 1{,}00\ E_{\mathrm{Edz}} \tag{7}$$

The implementation of the new rule for axially symmetrical silos was necessary because the consideration of only one horizontal component was not entirely correct.

3 Liquid filled tanks

The new standard prEN 1998-4 (2022) provides regulations for steel, reinforced concrete and prestressed precast reinforced concrete liquid storage tanks with both circular and rectangular cross-sections that are subject to seismic actions. The standard includes provisions for tanks that are anchored and unanchored, with either fixed or floating roofs. A distinction is made between above-ground, underground and elevated tanks supported by a substructure. In addition, the informative Annex G provides approaches to consider soil-structure interaction effects specifically for cylindrical tanks. A force-based design approach is proposed for all tank types. The design principles focus primarily on above-ground, vertical, cylindrical and rectangular tanks, as these are most encountered in practice. There are also provisions for both horizontal cylindrical tanks and spherical tanks. The procedure for calculating the seismic loads is explained below using cylindrical tanks as an example. In view of the complex dynamic behaviour of liquid-filled tanks under seismic loading due to the fluid-structure interaction of the tank and its content, the hydrodynamic effects are described in a simplified way by equivalent static pressure distributions applied to the tank wall and bottom. For the horizontal seismic action, the convective pressure due to sloshing vibrations must be considered, as well as the impulsive rigid and flexible pressure components due to the rigid body motion and the interaction vibration mode between the tank structure and its content. In addition, the vertical seismic actions activate impulsive rigid and flexible pressure components. Figure 2 shows the horizontal pressure components, while Figure 3 shows the vertical pressure components.

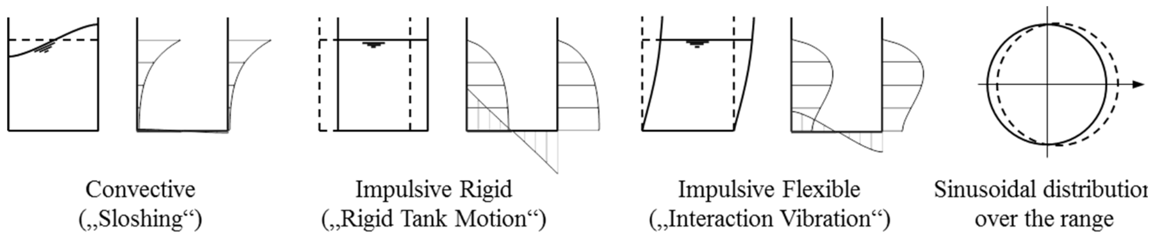


Figure 2. Pressure components due to horizontal seismic actions, Meskouris et al. (2019).

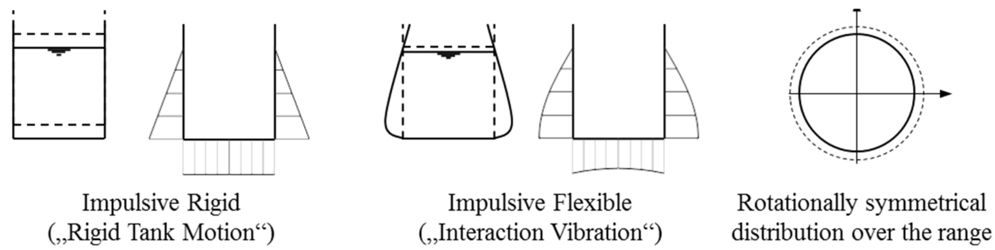
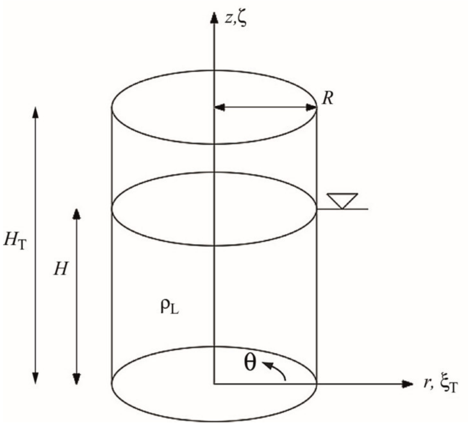


Figure 3. Pressure components due to vertical seismic actions, Meskouris et al. (2019).

The pressure distributions on cylindrical tanks are described using the notations given in Figure 4.



H	filling height
H_{T}	tank height
R	radius
θ	circumferential angle
$\xi_{\mathrm{T}} = r/R$	dimensionless radius
$\zeta = z/H$	dimensionless height
$\gamma = H/R$	ratio filling height/radius
ρ_{L}	liquid density

Figure 4. Notations for vertical cylindrical tanks.

The solution of the governing Laplace differential equation for the boundary conditions of the radial velocity along the tank wall, the axial velocity at the silo bottom, the axial velocity at the liquid surface and the pressure at the free surface of a circular tank with horizontal excitation leads to the following pressure functions of the convective pressure component p_k, the impulsive rigid pressure component $p_{ir,h}$ and the impulsive flexible pressure component $p_{if,h}$:

$$p_c(\xi,\zeta,\theta,t) = \sum_{n=1}^{\infty} \frac{2\cdot R\cdot\rho_L}{(\lambda_n^2-1)} \left[\frac{J_1(\lambda_n\cdot\xi)}{J_1(\lambda_n)}\right]\left[\frac{\cosh(\lambda_n\cdot\gamma\cdot\zeta)}{\cosh(\lambda_n\cdot\gamma)}\right][\cos(\theta)]\,[a_{cn}(t)\cdot\Gamma_{cn}] \quad (8)$$

$$p_{ir,h}(\xi,\zeta,\theta,t) = \sum_{n=0}^{\infty} \frac{2\cdot R\cdot\gamma\cdot\rho_L\cdot(-1)^n}{\nu_n^2}\left[\frac{I_1\left(\frac{\nu_n}{\gamma}\cdot\xi\right)}{I_1'\left(\frac{\nu_n}{\gamma}\right)}\right][\cos(\nu_n\cdot\zeta)]\,[\cos(\theta)]\left[a_{ir,h}(t)\cdot\Gamma_{ir,h}\right] \quad (9)$$

$$p_{if,h}(\xi,\zeta,\theta,t) = \sum_{n=0}^{\infty} 2\cdot R\cdot\rho_L\cdot\left[\frac{I_1\left(\frac{\nu_n}{\gamma}\cdot\xi\right)}{\frac{\nu_n}{\gamma}\cdot I_1'\left(\frac{\nu_n}{\gamma}\right)}\right]\left[\cos(\nu_n\cdot\zeta)\int_0^1 f(\zeta)\cdot\cos(\nu_n\cdot\zeta)\,d\zeta\right][\cos(\theta)]\left[a_{if,h}(t)\cdot\Gamma_{if,h}\right] \quad (10)$$

where n is the summation index, t is the time, J_1 is the first order Bessel function, λ_n are the roots of the derivative of the Bessel functions J_1, I_1 is the modified first order Bessel function, a_{cn}, $a_{ir,h}$, $a_{if,h}$ are the horizontal acceleration time histories, $\Gamma_{cn}, \Gamma_{ir,h}, \Gamma_{if,h}$ are the participation factors and $\nu_n = (n+0.5)\cdot\pi$ is a coefficient. These pressure functions given in EN 1998-4 (2006) are not readily applicable in everyday practice because a coupling of mathematical software with sophisticated FE software is required to manage the complex mathematical functions and to solve the non-linear interaction problem between the tank and the liquid for the impulsive flexible pressure component. The impulsive flexible pressure components must be calculated iteratively as the required deformation shape of the tank is not known in advance. This problem can be solved by applying the so-called "added-mass-model" recommended by Fischer and Rammerstorfer (1991).

A simplified calculation of the horizontal and vertical pressure components is achieved in prEN 1998-4 (2022) by tabulating the series expansions of hyperbolic cosine and Bessel functions in the form of normalised pressure functions C_c, $C_{ir,h}$, $C_{if,h}$, $C_{if,v}$ and corresponding participation factors $\Gamma_c, \Gamma_{ir,h}, \Gamma_{if,h}, \Gamma_{ir,v}, \Gamma_{if,v}$ for the horizontal and vertical pressure components. Figure 5 shows a graphical representation of the normalised impulsive rigid pressure component $C_{ir,h}$ for different ratios of filling heights to radii γ. The pressure components, together with the corresponding periods $T_{con}, T_{ir,h}, T_{if,h}, T_{ir,v}, T_{if,v}$, are defined (11) to (15) and scaled with the spectral accelerations S_e and S_r obtained from elastic or reduced response spectra.

Convective pressure component for horizontal seismic actions:

$$p_c(\xi_T = 1, \zeta, \theta, T_{con}) = C_c(\zeta,\gamma)\ \Gamma_c\ \rho_L\ R\cos(\theta)\ S_e(T_{con}) \quad (11)$$

Impulsive rigid pressure component for horizontal seismic actions:

$$p_{ir,h}(\xi_T = 1, \zeta, \theta, T_{ir,h}) = C_{ir,h}(\zeta,\gamma)\ \Gamma_{ir,h}\ \rho_L\ R\cos(\theta)\ S_r(T_{ir,h}) \quad (12)$$

Impulsive flexible pressure component for horizontal seismic actions:

$$p_{\mathrm{if,h}}(\xi_{\mathrm{T}} = 1, \theta, \zeta, T_{\mathrm{if,h}}) = C_{\mathrm{if,h}}(\zeta, \gamma)\, \Gamma_{\mathrm{if,h}}\, \rho_{\mathrm{L}}\, R \cos(\theta)\, \left(S_{\mathrm{r}}(T_{\mathrm{if,h}}) - S_{\mathrm{r}}(T_{\mathrm{ir,h}})\right) \tag{13}$$

Impulsive rigid pressure component for vertical seismic actions:

$$p_{\mathrm{ir,v}}(\zeta, T_{\mathrm{ir,v}}) = \Gamma_{\mathrm{ir,v}}\, \rho_{\mathrm{L}}\, [H\,(1-\zeta)]\, S_{\mathrm{rv}}(T_{\mathrm{ir,v}}) \tag{14}$$

Impulsive flexible pressure component for vertical seismic actions:

$$p_{\mathrm{if,v}}(\xi_{\mathrm{T}} = 1, \zeta, T_{\mathrm{if,v}}) = C_{\mathrm{if,v}}(\zeta, \gamma)\, \Gamma_{\mathrm{if,v}}\, \rho_{\mathrm{L}}\, R \left(S_{\mathrm{rv}}(T_{\mathrm{if,v}}) - S_{\mathrm{rv}}(T_{\mathrm{ir,v}})\right) \tag{15}$$

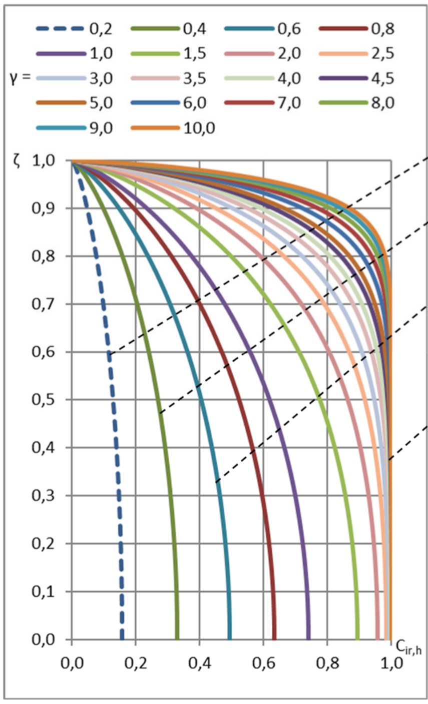
The reduced response spectrum for above-ground tanks should be calculated for both bolted and welded steel tanks in accordance with DC1 using behaviour factors q_R = 1.0, q_D = 1.0, q_S = 1.2. For reinforced concrete or pre-stressed precast reinforced concrete tanks the recommended behaviour factors are q_R = 1.0, q_D = 1.0, q_S = 1.5. Furthermore, a damping ratio of ξ = 0.5 % is recommended for the elastic response spectrum S_e used for the convective pressure component. The superposition of the pressure components should follow the superposition rule A proposed by Fischer and Rammerstorfer (1991) as verified by Chatterji (2022). According to this rule, the resulting pressure for the horizontal pressure components is calculated as follows:

$$p_{\mathrm{h,res}} = \sqrt{(p_{\mathrm{c}})^2 + (p_{\mathrm{ir,h}} + p_{\mathrm{if,h}})^2} \tag{16}$$

The resulting vertical pressure component is calculated using Formula (17)

$$p_{\mathrm{v,res}} = \sqrt{(p_{\mathrm{ir,v}})^2 + (p_{\mathrm{if,v}})^2} \tag{17}$$

These superposition schemes give reasonable results for the typical period range of liquid storage tanks and are consistent with the proposals of Veletsos (1984, 1998), Haroun and Housner (1981), Malhotra (2021), and NZSEE (2009) for ratios of $\gamma \geq 1$. The mass inertia effects of the tank wall, roof and ancillary elements can simply be added to the horizontal and vertical impulsive rigid hydrodynamic pressure components. The resulting horizontal and vertical pressure components $p_{\mathrm{h,res}}$ and $p_{\mathrm{v,res}}$ should be combined according to the rule given in (6) and (7).



$\zeta=\frac{z}{H}$	γ=0.2	γ=0.4	γ=0.6	…	γ=10
1.00	0	0	0		0
0.95	0.0277	0.0569	0.0864		0.6609
0.90	0.0469	0.0965	0.1458		0.8667
0.85	0.0625	0.1289	0.1948		0.9470
0.80	0.0759	0.1569	0.2369		0.9789
0.75	0.0876	0.1813	0.2738		0.9916
0.70	0.0980	0.2030	0.3063		0.9966
0.65	0.1072	0.2222	0.3352		0.9986
0.60	0.1154	0.2394	0.3610		0.9995
0.55	0.1227	0.2547	0.3838		0.9998
0.50	0.1291	0.2683	0.4041		0.9999
0.45	0.1348	0.2803	0.4220		1
0.40	0.1399	0.2909	0.4377		1
0.35	0.1442	0.3000	0.4512		1
0.30	0.1479	0.3078	0.4628		1
0.25	0.1510	0.3143	0.4725		1
0.20	0.1535	0.3196	0.4803		1
0.15	0.1554	0.3237	0.4863		1
0.10	0.1568	0.3266	0.4906		1
0.05	0.1576	0.3283	0.4932		1
0	0.1579	0.3289	0.4940		1

Figure 5. Normalized impulsive rigid pressure component $C_{ir,h}$ for different ratios γ to be multiplied with $\Gamma_{ir,h}\, \rho_L\, R\cos(\theta)\, S_r(T_{ir,h})$, Meskouris et al. (2019).

The tanks should be verified in the seismic design situation at the SD limit state. Steel tanks should be verified in accordance with prEN 1993-4-2 (2021) and reinforced concrete and pre-stressed precast reinforced shells should be verified according to prEN 1992-3 (2021) and FprEN 1992-1-1 (2022). Anchorage systems of tank structures to their foundations should be designed to remain elastic and provide sufficient ductility to avoid brittle failure when seismic design loads are exceeded. Alternatively, above-ground tanks and elevated tanks with substructures can be analysed with non-linear approaches according to prEN 1993-1-6 (2021), using non-linear response history analysis. In this case, damping ratios of ξ = 2 % should be applied for bolted and welded steel tanks, ξ = 5 % for reinforced concrete tanks and ξ = 2 % for pre-stressed precast reinforced concrete tanks.

4 Pipelines

prEN 1998-4 (2022) provides rules for the structural analysis of continuous above-ground and buried pipelines made of steel, unreinforced or reinforced concrete subjected to seismic actions including the effects of transient and permanent ground deformations.

4.1 Above-ground pipeline systems

Above-ground pipeline systems with straight and bended sections and ancillary elements are divided into single lines and redundant networks. A pipeline is treated as a single pipeline if its behaviour during and after a seismic event is independent of other pipelines and the consequences of its failure are solely related to the functions it performs. The seismic analysis of above-ground pipelines on foundations should consider the dynamic structural response under the three components, differential displacements due to the structural response of different substructures along the pipeline of the seismic actions, seismically induced permanent ground deformations, the spatial variability of the ground motions due to wave passage, local site effects and incoherence. However, as pipelines are flexible systems with comparatively small masses, the effects of wave passage and incoherence are less significant than seismically induced permanent ground deformations and local site effects such as geological discontinuities or varying soil conditions. In addition, the design should include ancillary elements such as valves, pumps or instrumentation and their connections to the pipeline as well as the effects of crossings to associated facilities due to connecting pipes and different types of foundations.

The seismic design of above-ground pipelines is limited to ductility classes DC1 and DC2, whereas the substructures of above-ground pipelines can be designed for all ductility classes. The calculations can be performed using response spectrum or time history analysis with uniform excitation of all supports, provided that the spatial variability is taken into account by increasing the design quantities between 10 and 30% with respect to the soil, geological and topographical conditions. The effects of the seismic actions considered for above-ground pipelines are the seismically induced stresses or strains in the pipeline wall which are used for the verification at the SD limit state according to the relevant material codes. In addition, substructures, foundations and anchorages shall be verified.

4.2 Buried pipeline systems

Buried pipelines restrained by the surrounding soil shall be designed for ground motion due to the seismic wave propagation and seismically induced permanent ground deformations due to fault crossings; liquefaction induced phenomena, lateral spreading, landslides, and local soil settlements. The seismic design shall take into account the spatial variability of ground motions due to wave passage, local site effects and incoherence. In addition, the effects of crossings to associated subsystems with connecting pipes and transition areas to above-ground pipelines supported on single foundations and foundation slabs shall be considered. Furthermore, the design should take into account ancillary elements such as valves, tanks, pumps or instrumentation and their connections to the pipeline. Overall, the design rules, calculation models and verifications for underground pipelines are significantly more comprehensive than in the current standard EN 1998-4 (2006).

4.2.1 Modelling and structural analysis for wave propagation

For modelling and calculation purposes, a distinction is made between straight pipelines and pipelines with bends. A pipeline is considered to be straight if the radius of curvature is greater than 20 times the outer diameter of the pipeline as defined in EN 1594 (2013).

For straight pipelines it can be assumed that the pipeline is flexible enough to follow without slippage or interaction the deformation of the soil. This means that the strains in the direction of the straight pipeline due to the wave passage effect are equal to the ground strain, and the maximum axial strain $\varepsilon_a^{\mathrm{p}}$ in the pipeline is calculated as follows:

$$\varepsilon_{\mathrm{a}}^{\mathrm{p}} \leq \frac{PGV}{V_{\mathrm{app}}} \tag{18}$$

where PGV is the horizontal ground velocity at the depth of the pipeline as defined in prEN 1998-5 (2022), 11.2.2. The maximum curvature χ^{p} of the pipeline should be calculated using formula (8.3).

$$\chi^{\mathrm{p}} \leq \frac{PGA}{V_{\mathrm{app}}^2} \tag{19}$$

where PGA is the horizontal ground acceleration at the depth of the pipeline as defined in prEN 1998-5 (2022), 11.2.2. The values for PGV and PGA should be increased by 20 and 30% respectively, depending on the soil and topographical conditions.

Buried pipelines with bends should be analysed by non-linear response history analysis according to prEN 1998-1-1 (2023), 6.5 and 6.6 using a beam model with non-linear spring elements in the axial, transverse and vertical directions to account for the interaction with the surrounding soil (Figure 5). The required spring characteristics with respect to soil conditions are given in Informative Annex D. Special elbow elements can be used in curved sections of pipelines where out-of-roundness and warping dominate the behaviour. If only certain parts of the pipeline need to be modelled in detail, a hybrid modelling approach can be used. Calculations can be carried out using an apparent shear wave velocity $V_{\mathrm{app}} = 1000\ m/s$ in accordance with prEN 1998-5 (2022), 11.2.2 (8) if site-specific investigations are not available. Permanent ground deformations can be taken into account with static analyses by imposing displacements in the non-linear calculation model. Alternatively, a three-dimensional non-linear continuum model can be used to represent the pipeline and the surrounding soil.

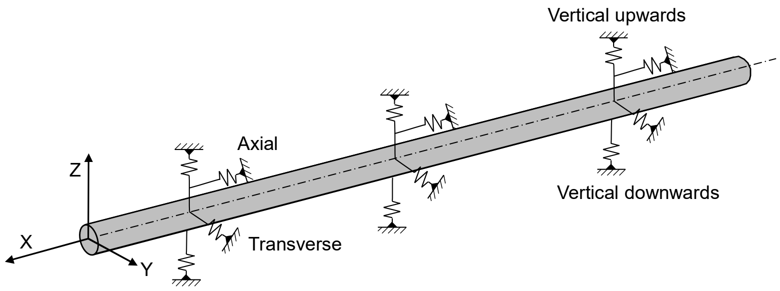


Figure 6. Beam model with non-linear springs representing the surrounding soil.

Seismic wave propagation should be applied by displacement time histories to the pipeline using the apparent shear wave velocity V_{app}. Each set of seismic ground motions should be applied simultaneously in all three directions along the pipeline, with time offsets to account for the effects of wave passage. Since the angle of incidence of the seismic wave influences the seismic action effects on the pipeline cross section, different angles should be investigated to account for directional dependencies, especially in curved areas.

The effect of loss of coherence along the pipeline can be neglected under conditions of uniform soil conditions without geological discontinuities, minimal topographical/basin effects and shear wave velocities differing by no more than 200 m/s. This is because buried pipelines are inherently flexible systems with low mass and insignificant inertial effects. However, where soil and topographical conditions are less favourable, all design parameters must be increased by 20 to 30%.

4.2.2 Modelling and structural analysis for permanent ground motions

For straight pipelines, the effect of fault-pipeline crossing can be easily analysed by an analytical approach using the fault-pipeline crossing angle in the horizontal plane and the fault dip angle. Strike-slip faults are conservatively assumed to slip horizontally only, and dip-slip (i.e. normal or reverse) faults are assumed to slip vertically only. The corresponding fault movements may be estimated from site-specific investigations or approximated using the procedure given in Informative Annex E, which allows the determination of the design differential displacement to be determined using the fault mechanism and geometry, the fault seismicity, the pipeline-fault crossing geometry, and the return period of interest. Buried pipelines with or without bends can also be analysed using the non-linear calculation model as shown in Figure 6 by imposing the fault displacements along the fault plane on the springs connected to the pipeline. It should be noted that the analytical approach is less accurate than using the non-linear calculation model.

In the case of potential liquefaction, the consequences of buoyancy, lateral spreading and local settlements should be evaluated, taking into account prEN 1998-5 (2022), 7.3.5. The effect of buoyancy in liquefied soils can be calculated by applying the buoyancy force per unit length along the liquefied zone to the pipeline. The buoyancy force V_{BU} can be calculated as follows:

$$V_{\mathrm{BU}} = \frac{\pi\, D_{\mathrm{op}}^2}{4} (\gamma_{\mathrm{s}} - \gamma_{\mathrm{c}}) - \pi\, D_{\mathrm{op}}\, t_{\mathrm{p}}\, \gamma_{\mathrm{p}} \tag{20}$$

where D_{op} is the outer diameter of the pipeline, γ_{s} is total unit weight of the soil, γ_{c} unit weight of the pipeline content, γ_{p} is unit weight of the pipeline material and t_{p} is the wall thickness of the pipeline.

In addition, the pipeline may be analysed for permanent transverse and axial ground displacements due to lateral spreading and landslides. Potential landslides and lateral spreading in the area of the pipeline route shall be identified along the entire pipeline based on ground investigations according to prEN 1998-5 (2022), 6.1, and rules for liquefaction assessment according to prEN 1998-5 (2022), 7.3. The amplitudes, lengths and widths of permanent ground motions due to landslides and lateral spreading can be calculated according to prEN 1998-5 (2022), 11.3.3, if no more site-specific investigations are not carried out. The application of transverse and axial permanent ground motions is defined in prEN 1998-4 (2022).

5 Secondary process components

5.1 Design provisions in EN 1998-1 (2004) and EN 1998-4 (2006)

The current version of EN 1998-4 (2006) lacks detailed design provisions specifically tailored to secondary process components, and EN 1998-1 (2004) contains only a highly simplified formula for estimating a static equivalent load for secondary elements. This equivalent load is calculated using an amplified ground acceleration that considers both the installation height and the resonance effects between the component and the first eigenmode of the primary structure. The formulation assumes a linear increase of acceleration with the height, corresponding to a triangular fundamental mode shape. However, given the irregular mass distribution in plan and elevation in industrial facilities, higher modes may become significant, resulting in a non-linear acceleration profile along the height of the building. Another problem is that the equivalent load must be calculated based on the first natural period of the components, which is typically unknown in plant engineering. As a result, the period of the component is conservatively assumed to resonate with the primary mode shape of the structure. The use of the behaviour factors provided in the code is also questionable, as they are defined for standard building construction and cannot be directly applied to plant construction. In summary, the current code does not cover the specific requirements for the modelling, analysis and design of process components in industrial plants. For the reasons mentioned above, a new chapter "Rules for ancillary elements in industrial plants" has been added for the design of plant components, which is described in the following sections.

5.2 Modelling and structural analysis according to prEN 1998-4 (2022)

The new prEN 1998-4 (2023) standard makes a basic distinction between single and multiple supported components. Single supported components are connected to the structure by one or more supports with negligible differential displacements under seismic excitation. They are usually anchored to one or more points of a single rigid diaphragm. Multi-supported components are connected to the structure by multiple supports that may be subject to differential displacements under seismic excitation. Specific rules for modelling and analysis are provided based on the classification of single and multi supported components and the expected dynamic interaction.

Computational models of the support structure in which the components are integrated should be used (i) when single- or multi-support process components affect the overall response due to dynamic interactions between the component and its supporting structure, (ii) when multi-support components affect the deformation of the support structure, and (iii) when multi-support components are sensitive to both the differential deformation of their supports and the vibration of the component itself. If any of these criteria are met, calculations can be performed using modal response spectrum or time history analysis. Modal response spectrum analysis serves as the standard method. However, if the vibration behaviour is dominated by single modes in the horizontal directions, an equivalent static analysis is possible. Multi-support components may require a pure elastic stiffness model to account for the differential deformations across the supports of the structure.

5.3 Non-dissipative and dissipative design approach according to prEN 1998-4 (2022)

The non-dissipative design approach aims to ensure that the anchorage systems for components are strong enough to remain elastic in the seismic design situation. If the horizontal vibration periods of the component and the structure are known, the method given in prEN 1998-1-2 (2023), 7.2 can be applied. In the design phase of industrial plants, there is usually little information available on the installation positions of the components, the dynamic characteristics of the primary structure, the fundamental periods of the primary structure and the overstrength of the anchorages. Therefore, the new prEN 1998-4 (2022) standard provides a simplified alternative calculation with conservative assumptions for which the specific information is not required:

- The maximum value of the amplification factor *AMP* = 7 is proposed, which corresponds to the resonance between the component and the supporting structure.
- It is recommended to use the worst-case period of the structure in the constant acceleration plateau.
- Higher eigenmodes are neglected, as it is assumed that the response of the single column component is determined by individual modes in the horizontal directions.
- Participation factors Γ_1 between 1.5 to 1.8 are recommended for the fundamental mode in the direction under consideration.
- The period dependent behaviour factor q'_{D} is specified to have a maximum value of 1.5 in the extended period range.
- The behaviour factor q'_{an} of the component is limited to 1.5.

Alternatively, a dissipative design approach can be used to protect single supported components by incorporating yielding fuses in the load path connecting the component to the support structure. This ductile mechanism will reduce the seismic accelerations transmitted to the component, provided that there is a limit on the maximum yielding force and a guaranteed minimum ductility capacity in each direction of interest. In addition, overstrength effects in the load path from the component to the supporting structure are limited to 25%. Furthermore, for many process components that are lightweight and of minor importance, verification may be waived under the following conditions: (i) The weight of the component is less than 200 kg, and its centre of gravity is within 1.25 m above or below the connected floor. (ii) The weight of the component is less than 10 kg. (iii) For components with distributed mass, the mass does not exceed 7.5 kg/m. In addition, it is required that the component performance factor γ_{an} does not exceed 1.0 and that flexible connections are provided between the component and associated ductwork, pipes, and conduits.

6 Towers, masts and chimneys

Towers, masts and chimneys as special structures were previously covered in Part 6 of EN 1998-4 (2006). As these structures are essentially related to plant engineering, Part 6 has been integrated into prEN 1998-4 (2022). For this purpose, the content of Part 6 has been revised and shortened and incorporated into chapter 10, together with the two annexes F and G. The chapter covers the design of tall slender structures, gives rules for steel towers, guyed masts and chimneys in addition to prEN 1993-3 (2022) and for reinforced concrete chimneys in addition to FprEN 1992-1-1 (2022). Informative Annex G gives guidance on the seismic design of masonry chimneys and Informative Annex F gives recommendations for the dynamic analysis of towers, masts and chimneys.

7 Summary

The paper presents the new prEN 1998-4 (2022) standard for the seismic design of industrial structures. The standard combines the content of EN 1998-4 (2006) and EN 1998-6 (2006) and follows the seismic design principles of the new generation of Eurocodes. The main improvements include the removal of redundancies, the removal of inconsistencies and the introduction of simplified practical calculation approaches.

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