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Eurocode 8 — Design of structures for earthquake resistance —   
Part 4: Silos, tanks and pipelines, towers, masts and chimneys

Eurocode 8 — Auslegung von Bauwerken gegen Erdbeben — Teil 4: Silos, Tankbauwerke und Rohrleitungen, Türme, Maste und Schornsteine

Eurocode 8 — Calcul des structures pour leur résistance au séismes — Part 4: Silos, réservoirs, tuyauteries, tours, mâts et cheminées

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European foreword

This document (prEN 1998-4:2023) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1998-4:2007 and EN 1998-6:2005.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognise the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

1. Introduction

## Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of parts:

* EN 1990 Eurocode: Basis of structural and geotechnical design
* EN 1991 Eurocode 1: Actions on structures
* EN 1992 Eurocode 2: Design of concrete structures
* EN 1993 Eurocode 3: Design of steel structures
* EN 1994 Eurocode 4: Design of composite steel and concrete structures
* EN 1995 Eurocode 5: Design of timber structures
* EN 1996 Eurocode 6: Design of masonry structures
* EN 1997 Eurocode 7: Geotechnical design
* EN 1998 Eurocode 8: Design of structures for earthquake resistance
* EN 1999 Eurocode 9: Design of aluminium structures
* New parts are under development, e.g. Eurocode for design of structural glass

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

## Introduction to EN 1998 (all parts)

EN 1998 (all parts) defines the rules for the seismic design of new buildings and engineering works and the assessment and retrofit of existing ones, including geotechnical aspects, as well as temporary structures.

NOTE This standard also covers the verification of structures in the seismic situation during construction, when required.

Attention should be paid to the fact that, for the design of structures in seismic regions, the provisions of EN 1998 should be applied in addition to the relevant provisions of EN 1990 to EN 1997 (all parts) and EN 1999 (all parts). In particular, EN 1998 should be applied to structures of consequence classes CC1, CC2 and CC3, as defined in EN 1990:2023, 4.3. Structures of consequence class CC4 are not fully covered by the Eurocodes but may be required to follow EN 1998, or parts of it, by the relevant Authorities.

By nature, perfect protection (a null seismic risk) against earthquakes is not feasible in practice, namely because the knowledge of the hazard itself is characterized by a significant uncertainty. Therefore, in Eurocode 8, the seismic action is represented in a conventional form, proportional in amplitude to earthquakes likely to occur at a given location and representative of their frequency content. This representation is not the prediction of a particular seismic movement, and such a movement could give rise to more severe effects than those of the seismic action considered, inflicting damage greater than the one described by the Limit States contemplated in this standard.

Not only the seismic action cannot be predicted, but in addition, it should be recognised that engineering methods are not perfectly predictive when considering the effects of this specific action, under which structures are assumed to respond in the non-linear regime. Such uncertainties are taken into account according to the general framework of EN 1990, with a residual risk of underestimation of their effects.

EN 1998 is subdivided in various parts

EN 1998-1-1, *Eurocode 8 —* *Design of structures for earthquake resistance – Part 1-1: General rules and seismic action;*

EN 1998-1-2, *Eurocode 8 —Design of structures for earthquake resistance – Part 1-2: Buildings;*

EN 1998-2, *Eurocode 8 —* *Design of structures for earthquake resistance – Part 2: Bridges;*

EN 1998-3, *Eurocode 8 —Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings and bridges;*

EN 1998-4, *Eurocode 8 —* *Design of structures for earthquake resistance – Part 4 Silos, tanks, pipelines, towers, masts and chimneys;*

EN 1998-5, *Eurocode 8 —Design of structures for earthquake resistance – Part 5: Geotechnical aspects, foundations, retaining and underground structures.*

## Introduction to prEN 1998-4

prEN 1998-4 provides specific requirements for earthquake resistant design of new on-ground and elevated silos, on-ground, elevated and underground tanks, above-ground and buried pipeline systems, towers, masts and chimneys and ancillary elements attached to the aforementioned structures or in industrial facilities, which are additional to the ones in other Eurocodes.

prEN 1998-4 is subdivided in ten clauses and includes seven annexes, where Annex A is normative and Annexes B, C, D, E, F, G are informative.

## Verbal forms used in the Eurocodes

The verb “shall” expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may" expresses a course of action permissible within the limits of the Eurocodes.

The verb “can" expresses possibility and capability; it is used for statements of fact and clarification of concepts.

## National annex for prEN 1998-4

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing prEN 1998-4 can have a National Annex containing all national choices to be used for the design of buildings to be constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this document, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by the relevant parties.

National choice is allowed in prEN 1998-4 through notes to the following clauses:

|  |  |  |  |
| --- | --- | --- | --- |
| 4.2(2) | 4.2(3) | 4.3(6) | 4.3(7) |

National choice is also allowed in prEN 1998-4 on the application of the following informative annexes:

|  |  |  |  |
| --- | --- | --- | --- |
| Annex B | Annex C | Annex D | Annex E |
| Annex F | Annex G |  |  |

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

# Scope

## Scope of prEN 1998-4

1. This document is applicable to the seismic design of on-ground and elevated silos, on-ground, elevated and underground tanks, above-ground and buried pipeline systems, towers, masts and chimneys and ancillary elements attached to the aforementioned structures or in industrial facilities.
2. Unless specifically stated, EN 1998‑1‑1:—2, and EN 1998-5:—4 apply.
3. prEN 1998-4 is applicable in complement to the other relevant Eurocodes.

NOTE This document contains only those provisions that, in addition to the provisions of the other relevant Eurocodes, are used for the design of new structures, as listed in (1), in seismic regions. prEN 1998-4 complements in this respect the other Eurocodes.

## Assumptions

1. The assumptions of EN 1998-1-1:—2, 1.2, are assumed to be applied.
2. It is assumed that the changes in a) and b) will not take place during the construction phase or during the subsequent life span for all structures covered by prEN 1998-4, unless proper justification and verification is provided:
3. substantial changes in the structural systems, supporting structures or attached ancillary elements listed in 1.1 (1);
4. substantial changes of masses or mass distribution. This includes, in particular, changes in production, such as specific changes of filling loads, filling states and ancillary elements.

# Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in ‘should’ clauses), permissions (‘may’ clauses), possibilities ('can' clauses), and in notes.

EN 1990:2023, *Eurocode — Basis of structural and geotechnical design*

EN 1991-1-4:—[[1]](#footnote-2), *Eurocode 1 – Actions on structures – Part 1-4: Wind Actions*

EN 1998‑1‑1:—[[2]](#footnote-3), *Eurocode 8 – Design of structures for earthquake resistance – Part 1-1: General rules and seismic action*

EN 1998-1-2:—[[3]](#footnote-4), *Eurocode 8 - Design of structures for earthquake resistance - Part 1-2: Buildings*

EN 1998‑5:—[[4]](#footnote-5), *Eurocode 8 – Design of structures for earthquake resistance – Part 5: Geotechnical aspects, foundations, retaining and underground structures*

EN ISO 80000 (all parts), *Quantities and units*

# Terms, definitions and symbols

## Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, EN 1991-1-4:—1, EN 1998‑1‑1:—2, EN 1998‑5:—4, and the following, apply.

### 

ancillary element

architectural, mechanical or electrical element, system and technical plant component such as container, pipeline, pump, conveyor and many other plant-specific components connected to or supported by the structures of 1.2 (1). An ancillary element is not considered in seismic design as load-carrying element but required for the safe operation of the facility and may be the cause of risk to persons or to the structure in case of earthquake

### 

floating roof

height adjustable roof of tanks. The roof rises and falls with the liquid level in the tank

### 

freeboard

space kept between the top of liquid level and the bottom of the roof slab of the tank or the depth between the overflow pipe and the base of the roof slab

### 

lattice structure

structure in which the joints are not designed to resist the plastic moment of the connected members

### 

masonry chimney

industrial chimney constructed of masonry units and mortar

### 

silo battery

group of silos with individual cells connected to each other or stand-alone and permitting different types of similar solids to be stored separately

### 

tangent tower

transmission tower used where the cable line is straight or has an angle not exceeding 3⁰ in plan. It supports vertical loads, a transverse load from the angular pull of the wires, a longitudinal load due to unequal spans, and forces resulting from the wire-stringing operation, or a broken wire

### 

telescope joint

joint between tubular elements without a flange, the internal diameter of one being equal to the external diameter of the other

### 

transmission tower

tower used to support low or high voltage electrical transmission cables

## Symbols and abbreviations

For the purposes of this document, the symbols and abbreviations given in EN 1990, EN 1998‑1‑1:—2, EN 1998-1-2:—3, EN 1998‑5:—4 apply.

For the symbols related to materials, as well as for symbols not specifically related to the seismic design situation, the provisions of the relevant Eurocodes apply.

In addition, further symbols and abbreviations, used in connection with the seismic design situation, are defined in the present standard where they occur, for ease of use. However, the most frequently occurring symbols used in this document are listed and defined in 3.2.1 and additional abbreviations are given in 3.2.2.

### Symbols

#### Symbols used in Clause 5

*Upper case Latin symbols*

*E* Young’s modulus

*F*b total base shear

*F*bv vertical reaction force at the silo bottom

*H*fmaximum design filling level of the silo content

*M*b overturning moment

*R* radius of circular silo, silo compartment or tank

*Lower case Latin symbols*

*b* horizontal dimension of a rectangular silo parallel to the horizontal seismic action

*d*c inner dimension of a silo parallel to the horizontal seismic action

*m*S total mass of the silo and its content

*p*0 hydrostatic normal pressure

*x*S vertical coordinate of a point on a silo wall from a flat silo bottom or the apex of a conical or pyramidal hopper

*x, y, z* coordinates

*Upper case Greek symbols*

*Δ*ph,s additional normal pressure on the silo wall

*Δ*ph,so reference normal pressure on silo walls

*Δ*ph,s,inc increased additional normal pressure on compression-loaded silo wall

*Lower case Greek symbols*

*α* ratio of the response acceleration of a silo at the level of interest, *z,* to the acceleration of gravity

*β* angle of inclination of the hopper wall in a silo, measured from the vertical, or the steepest angle of inclination to the vertical of the wall in a pyramidal hopper

**sc scaling factor for pressures on vertical silo walls, silo hoppers and silo bottoms

*θ* angle (0⁰ ≤*θ* < 360⁰) for the geometrical description of circular silos and tanks

upper characteristic value of the bulk density of the particulate solid

#### Symbols used in Clause 6

*Upper case Latin symbols*

*B* half width of rectangular tanks

*D* diameter of circular tanks

maximum convective base shear

horizontal base shear due to mass inertia effects of flexible tanks

vertical reaction force due to mass inertia effects of flexible tanks

horizontal base shear due to mass inertia effects of rigid tanks

vertical reaction force due to mass inertia effects of rigid tanks

impulsive flexible horizontal base shear

impulsive flexible vertical reaction force

impulsive rigid horizontal base shear

impulsive rigid vertical reaction force

resulting vertical reaction force

resulting horizontal reaction force

convective base shear for elevated tanks for the 1st eigenmode

convective base shear for elevated tanks for the 2nd eigenmode

impulsive rigid base shear for elevated tanks for the 1st eigenmode

impulsive rigid base shear for elevated tanks for the 2nd eigenmode

correction factor for the impulsive flexible time period in horizontal direction

correction factor for the impulsive flexible time period in vertical direction

*H* filling height of tanks

*H*T total height of a vertical cylindrical tank

increase factor to consider mass inertia effects of flexible tanks in horizontal direction

increase factor to consider mass inertia effects of flexible tanks in vertical direction

*L* half-length of rectangular tank

total convective moment at the base with base pressure

impulsive flexible moment below the base plate

rigid impulsive moment below the base plate

resulting overturning moment below the base plate

overturning moment for unanchored tanks

convective moment just above the base plate

the flexible impulsive moment just above the base plate

impulsive rigid moment just above the base plate

resulting overturning moment above the base plate

velocity response spectrum

*T*con first natural period of the convective mode

*T*if,h impulsive flexible period in horizontal direction

*T*if,vimpulsive flexible vertical period in vertical direction

*T*ir,hfirst natural period of the impulsive response in horizontal direction

*T*ir,vfirst natural period of the impulsive response in vertical direction

*Lower case Latin symbols*

*d*t wall deflection of rectangular tanks

absolute maximum value of the vertical wave height

impulsive rigid lever arms without bottom pressure

impulsive flexible lever arms without bottom pressure

convective lever arms without bottom pressure

impulsive rigid lever arms without bottom pressure

impulsive flexible lever arms without bottom pressure

convective lever arms without bottom pressure

convective spring stiffness

spring stiffness for substructures of elevated tanks

coefficient for unanchored tanks

*k*sp stiffness of substructures of spherical tanks

*l*up uplift length of a rectangular tank

*m*l total liquid mass

*m*ir equivalent impulsive mass

*m*c equivalent convective mass

*m*if flexible impulsive mass

*m*r roof mass

*m*w wall mass

*p*b hydrostatic normal pressure on the uplifted length for unanchored tanks

convective pressure component

impulsive flexible pressure function for horizontal seismic actions

impulsive flexible pressure function for vertical seismic actions

impulsive rigid pressure function for vertical seismic actions

impulsive rigid pressure function on the tank wall for horizontal seismic actions

uniform pressure on the tank wall corresponding to inertia forces

uniformly distributed load for impulsive flexible vibration mode of rectangular tanks

resulting vertical pressure component

resulting horizontal pressure component

dimensionless impulsive rigid pressure function

dimensionless convective pressure function

*s*w uniform thickness of the tank wall or the average in case of stepwise wall thickness

*s*p thickness of the base plate for tanks

maximum displacement of the convective mass for the 1st eigenmode for elevated silos

maximum displacement of the convective mass for the 2nd eigenmode for elevated silos

maximum displacement of the impulsive rigid mass for the 1st eigenmode for elevated silos

maximum displacement of the impulsive rigid mass for the 2nd eigenmode for elevated silos

*Upper case Greek symbols*

participation factor of the impulsive rigid pressure component

participation factor of the impulsive rigid pressure in vertical direction

self-weight of the substructures of elevated tanks

contributing mass of the substructure for spherical tanks

, constants for the calculation of displacements for elevated tanks

*Lower case Greek symbols*

**l specified liquid weight

dimensionless height for vertical cylindrical tanks

Poisson’s ratio of the tank material

angle for unanchored tanks

dimensionless parameter for unanchored tanks

*ξ*T dimensionless radius for vertical cylindrical tanks

liquid density

density of the tank wall

additional membrane stresses in the base plate of unanchored tanks

n-mode shape of an elevated tank

*φ*1*,*an fundamental mode shape value at the height of attachment of the ancillary element to the supporting structure

circular frequency of the impulsive rigid mode for spherical tanks

#### Symbols used in Clause 7

*Lower case Latin symbols*

*r*p pipeline radius

wall thickness of pipelines

#### Symbols used in Clause 8

*Upper case Latin symbols*

greatest outer diameter of the pipeline after its ovaling

smallest outer diameter of the pipeline after its ovaling

mean outer diameter of the pipeline before ovaling

medium diameter of the pipeline

outer diameter of the pipeline

*L*A effective unanchored length between the fault trace and the anchor point

*L*em embedment length of the pipeline

length of the permanent ground motion zone

ovaling value of the pipeline

apparent shear wave celerity of the wave along a pipeline

buoyant force per unit length of the pipeline

width of the permanent ground motion zone

*Lower case Latin symbols*

*d*w wave amplitude

*m*an mass of ancillary element

*u* soil motion

x’ direction of wave propagation

*Upper case Greek symbols*

fault movement along the fault plane in normal direction

fault movement along the fault plane in normal direction

fault movement in axial pipeline direction

fault movement in transverse direction to the pipeline

fault movement in vertical direction to the pipeline

maximum curvature of the pipeline

*Lower case Greek symbols*

fault dip angle

fault-pipeline crossing angle in the horizontal plane

maximum axial strain in the pipeline due to wave propagation

average axial strain in the pipeline due to fault movement

minimum/maximum bending strain in the pipeline

plastic tensile strain of the pipeline

yield strain of the pipeline

unit weight of the pipeline content

unit weight of the pipeline material

total unit weight of soil

maximum amplitude of the permanent ground motion in transverse direction to the pipeline

maximum amplitude of the permanent ground motion in axial direction to the pipeline

apparent wave length of the predominant seismic wave at the ground surface where a pipeline is buried

angle between the direction of wave propagation and the pipeline axis

#### Symbols used in Clause 9

*Upper case Latin symbols*

*AMP* amplification factor for ancillary element acceleration

horizontal seismic force acting on ancillary element

*S*an horizontal floor acceleration spectrum value of ancillary element

elastic response spectra acceleration at the fundamental mode of the ancillary element

supporting structure

fundamental mode period of the ancillary element supporting structure

*Lower case Latin symbols*

behaviour factor of ancillary element

period and ductility dependent behaviour factor of the ancillary element supporting structure

*Lower case Greek symbols*

ancillary element performance factor

*μ*D ductility capacity of the ancillary element anchorage system

ξa damping ratio for ancillary elements

fundamental mode damping of the ancillary element supporting structure

#### Symbols used in Clause 10

*Upper case Latin symbols*

Modulus of elasticity of the cable material

Equivalent modulus of elasticity

*M*o first-order overturning moment at the base level

*P* normal force

*PFA* peak floor acceleration

*Lower case Latin symbols*

*d*r relative deflection between different points of support of the liner

*e*L horizontal eccentricity

*k*r modification factor of the behaviour factor *q* to account for irregularities

horizontal projected cable length

*Upper case Greek symbols*

participation factor of the fundamental mode in the direction under consideration

*Δ* is the displacement corresponding to the moment

*ΔH* vertical distance of adjacent platforms supporting the liner

*Lower case Greek symbols*

wrapping angle of the single chord

unit weight of the cable, including the weight of ice load on the cable

δ*M* additional overturning moment at the base level due to second-order (*P-Δ*) effect

**s slenderness of structural members

tensile stress in the cable

#### Symbols used in Annex A

*Upper case Latin symbols*

dimensionless convective pressure function

convective base shear coefficient

dimensionless impulsive flexible base shear coefficient

impulsive rigid base shear coefficient

dimensionless impulsive flexible pressure function

dimensionless impulsive rigid pressure function

dimensionless impulsive flexible pressure function

convective overturning moment coefficient

impulsive flexible overturning moment coefficient

impulsive rigid overturning moment coefficient

convective overturning moment coefficient

impulsive flexible overturning moment coefficient

impulsive rigid overturning moment coefficient

dimensionless period coefficient in horizontal direction

dimensionless period coefficient in vertical direction

*Upper case Greek symbols*

 participation factor of the convective pressure component

participation factor of the impulsive flexible horizontal mode

participation factor of the impulsive flexible vertical mode

*Lower case Greek symbols*

 correction factor for taking account of the clamping effect

ratio of filling height to tank radius

#### Symbols used in Annex B

*Upper case Latin symbols*

shear modulus of the soil

flexible tank stiffness associated to the impulsive flexible mode

horizontal stiffness of the foundation

rotational stiffness of the foundation

vertical stiffness of the foundation

radius of the foundation

impulsive flexible period of vibration in horizontal direction with soil-structure interaction

impulsive flexible period of vibration in vertical direction with soil-structure interaction

impulsive rigid period of vibration in horizontal direction with soil-structure interaction

impulsive rigid period of vibration in vertical direction with soil-structure interaction

*Lower case Latin symbols*

*m*b mass of the base plate

*a, a2, a3* numerical coefficients for rotational dynamic stiffness modifier

*b1, b2, b3* numerical coefficients for vertical dynamic stiffness modifier

*Lower case Greek symbols*

dimensionless coefficient for the rotational stiffness of rigid circular footings

dimensionless stiffness modifier for the horizontal stiffness

dimensionless stiffness modifier for the rotational stiffness

is the Poisson’s ratio of the soil

is the soil mass density

#### Symbols used in Annex D

*Upper case Latin symbols*

force applied to the pipeline by the spring

ultimate force applied to the pipeline

depth from the soil surface to the centreline of the pipeline

coefficient of earth pressure at rest

dimensionless horizontal bearing capacity factor for fine-grained soil

dimensionless horizontal bearing capacity factor for course-grained soil

dimensionless vertical uplift factor for fine-grained soil

dimensionless vertical uplift factor for coarse-grained soils

dimensionless vertical bearing capacity factor for cohesive soil

dimensionless bearing capacity factor for passive earth pressure due to the self-weight of the soil

dimensionless bearing capacity factor for passive earth pressure due to the downward movement of the pipeline

ultimate transverse force per unit length for the pipeline soil interaction model

ultimate transverse (vertical uplift) force per unit length

ultimate transverse (vertical downward) force per unit length

is the ultimate force per unit length at the soil-pipeline interface

shear wave velocity of the soil

*Lower case Latin symbols*

soil cohesion

ultimate displacement

pipe-coating dependent factor

displacement applied to the soil spring

*Upper case Greek symbols*

axial ultimate relative displacement for pipe-soil interaction model

transverse ultimate relative displacement for pipe-soil interaction model

transverse (vertical uplift) ultimate relative displacement for pipe-soil interaction model

transverse (vertical downward) ultimate relative displacement for pipe-soil interaction model

*Lower case Greek symbols*

soil/pipeline adhesion factor

effective soil unit weight

water unit weight

friction angle between the soil and the pipe

internal friction angle of the soil in degrees (°)

#### Symbols used in Annex E

*Upper case Latin symbols*

confidence factor for the fault recurrence rate

*L*F fault length

*X*Lratio of the distance of the pipeline-fault crossing point to the closest fault end over the fault length

*Lower case Latin symbols*

*a*0,…, *a*8 coefficients that depend on *Δ*F and the recurrence rate

recurrence-rate-independent function of the fault and fault-crossing characteristics

*p*1 to *p*7 coefficients for recurrence rate

*Upper case Greek symbols*

*Δ*F fault differential displacement

*Δ*Fcapa deterministic cap on the fault differential displacement

*Lower case Greek symbols*

recurrence rate approximate estimation

*v*F fault recurrence rate

### Abbreviations

GFRP Glass fibre-reinforced plastic

HDPE High density polyethylene

NDP National determined parameter

PE Polyethylene

G Centre of mass

C Centre of stiffness

RL Rigid link

## S.I. Units

S.I. Units in accordance with EN ISO 80000 (all parts) shall be used.

For calculations, the following units should be used when applicable:

* forces and loads: kN, kN/m, kN/m2
* length, displacement: m, mm
* unit mass: kg/m3, t/m3
* mass: kg, t
* weight density: kN/m3
* stresses and strengths: Pa (= N/m2), kPa (= kN/m2), MPa (= MN/m2)
* moments (bending, etc.): kNm
* acceleration: m/s2

# Basis of design

## Performance requirements

1. Structures addressed by this standard shall be designed to meet the objectives given in a) to f) with an appropriate degree of reliability:
2. protection of human lives and personal injury;
3. lifeline systems important for civil protection to remain operational;
4. protection of environment;
5. prevention of induced damage to connected plant components, nearby buildings and adjacent facilities to avoid cascading effects;
6. limitation of damage to preserve the full or limited functionality;
7. minimisation of economic and social consequences.
8. To meet the performance requirements in (1), it should be verified that structures are designed in such a way that the specified limit states for structures listed in 1.1(1) are not exceeded under prescribed seismic actions, determined with respect to the consequence class of the considered structure.

## Consequence classes

1. The consequence classes CC1, CC2 and CC3 should be applied as defined in EN 1990:2023, 4.3. The choice of the consequence class should depend on the consequences of collapse or damage, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the economic, social and environmental consequences. The specific definitions of the consequence classes given in EN 1990:2023, Annex A, should be considered.
2. Consequence class CC3 should be divided into the two subclasses CC3-a and CC3-b, according to EN 1998‑1‑1:—2, 4.2(3). Class 3-b should be chosen if the integrity of the structure, including ancillary elements, is of vital importance for the public safety and environment or can induce damage to connected plant components, nearby buildings or adjacent facilities. Furthermore, class CC3-b should be selected for all structures and systems which could jeopardise the operational civil protection services in the immediate post-earthquake period. In all other cases, structures of class CC3 may be assigned to subclass CC3-a.

NOTE 1 The definition of the classes CC3-a and CC3-b given in (2) is applied, unless the National Annex or relevant authorities give different definitions.

NOTE 2 Examples for important operational civil protection services in CC3-b are: energy and water supply systems, emergency routes, hospitals, fire stations, telecommunication, safety systems.

NOTE 3 The consequence class CC4 for large risk structures and their ancillary elements and systems is not covered by the present document. The categories of structures of consequence class CC4 where this document or parts of it apply in a country can be found in EN 1990:2023, Annex A.4, the National Annex or can be provided by the relevant authorities. The principles of calculation and design given in this document can also be applied for structures with consequence class CC4 (e.g. LNG tanks).

1. For the application of EN 1998‑1‑1:—2, 4.1(4), ** values should be determined.

NOTE The values of **applicable to the structures addressed in this document and to the associated geotechnical systems are those given in Table 4.1 (NDP), unless the relevant authorities or the National Annex give different values.

Table 4.1 (NDP) — ** values of structures addressed in this standard as listed in 1.1(1)

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | **Consequence class (CC)** | | | |
|  | CC1 | CC2 | CC3-a | CC3-b |
| ****** | 0,60 | 1,0 | 1,4 | 1,8 |

## Limit states and associated seismic actions

1. The performance requirements shall refer to the state of damage in the structures, herein described through the Limit States (LS) according to the definition in EN 1998‑1‑1:—2, 4.3(1).
2. The seismic performance of the structures addressed by in this standard shall correspond to the Significant Damage (SD) limit state, in which the structure and its ancillary elements are significantly damaged, but both retain their structural integrity with controlled leakage of contents.
3. Depending on the characteristics and the function of the structure considered, the LS of Damage Limitation (DL) or Fully Operational LS (OP) may be checked additionally.
4. LS of DL should be defined as the one in which the extent and amount of damage of the considered system, including some of its ancillary elements, is limited, so that after the operations for damage checking and control are carried out, the capacity of the system can be restored up to a predefined level of operation. Liquid-filled systems should remain leak-proof.
5. LS of OP should be defined as the one in which the considered system, including a specified set of ancillary elements integrated with it, remains fully serviceable under the relevant seismic action. Liquid-filled systems should remain leak-proof.
6. Unless (7) is applied, the seismic action should be specified in terms of its return period. The attainment of the performance requirements should be achieved by selecting appropriate return periods, *T*LS,CC, depending on the specified limit states and consequence class of the structure and its ancillary elements under consideration.

NOTE 1 The values of *T*LS,CC according to EN 1998‑1‑1:—2, 4.3(3), for NC, SD and DL limit states, are those given in Table 4.2 (NDP), unless relevant authorities or the National Annex give different values.

NOTE 2 The values of *T*LS,CC at OP limit state applied to specified structures and its ancillary elements can be provided by the relevant authorities or can be found in the National Annex.

NOTE 3 Values are given for NC for completeness in (6) and (7), although this limit state is not covered in the present document. They can be used for existing structures.

NOTE 4 Values of *T*LS,CC for consequence class CC4 are not covered by the present document. They can be found in other European standards (e.g. prEN 14620 for LNG tanks), in the National Annex.

Table 4.2 — (NDP) Return periods *T*LS,CC of seismic action in years

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Limit state** | **Consequence class** | | | |
| CC1 | CC2 | CC3-a | CC3-b |
| NC | 800 | 1600 | 2500 | 5000 |
| SD | 250 | 475 | 1300 | 2500 |
| DL | 50 | 60 | 150 | 250 |

1. Alternatively to (6), performance factors, **LS,CC, may be used alternatively to return periods.

NOTE 1 The values of **LS,CC according to EN 1998‑1‑1:—2, 4.3(4), for NC, SD and DL limit states, are those given in Table 4.3 (NDP), unless the National Annex or relevant authorities give different values.

NOTE 2 Values of **LS,CC for consequence class CC4 are not covered by the present standard. They can be found in other European standards, in the National Annex or can be provided by the relevant authorities.

**Table 4.3** — **(NDP) Performance factors **LS,CC**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Limit state** | **Consequence class** | | | |
| CC1 | CC2 | CC3-a | CC3-b |
| NC | 1,2 | 1,5 | 1,8 | 2,2 |
| SD | 0,8 | 1,0 | 1,4 | 1,8 |
| DL | 0,4 | 0,5 | 0,7 | 0,8 |

NOTE 3 The values of **LS,CC at OP limit state applied to specified structures and its ancillary elements can be provided by the relevant authorities or can be found in the National Annex.

1. The site conditions and seismic action on the ground surface at the location of the structure shall be determined according to EN 1998‑1‑1:—2, Clause 5.
2. The seismic action for geotechnical systems and geotechnical structures shall be determined according to EN 1998‑5:—4, 5.2.

## Modelling and methods of analysis

1. Unless otherwise specified for particular types of structures in the relevant clauses of this standard, the structural modelling and types of analysis (force-based approach, non-linear static analysis and response-history analysis) should be applied according to EN 1998-1-1:—2, 6, and EN 1998-1-2:—3, 5.
2. The value of **introduced in EN 1998-1-1:—2, 6.4.1(5), is equal to 0,08 for structures addressed in this standard as given in 1.1(1).
3. The combination coefficients *ψ*Ei as defined in EN 1998-1-1:—2, 6.2.1(3), for the masses associated to variable actions should be calculated with the minimum values according to EN 1998-1-2:—3, 5.1.2(1), and the combination factors *ψ*2,i as given in EN 1990:2023, Annex A, unless otherwise defined in Clause 5 to 10.
4. Soil-structure interaction effects should be considered using models provided in EN 1998‑5:—4, 8, whenever they may have an adverse influence on the response of the structural system and its ancillary elements. Beneficial soil-structure interaction effects should be modelled in a conservative manner.
5. The specific behaviour factors for structures addressed in this standard as given in 1.1(1) should be applied according 5 to 10.

## Combination of the effects of the components of the seismic action

1. 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 structure is not perfectly axisymmetric in plan, combination should be performed according to EN 1998-1-1:—2, 6.4.4.
2. For axisymmetric structures in plan (e.g. on-ground vertical cylindrical silos and tanks, tubular towers), only the vertical and one horizontal component of seismic action should be evaluated, where the latter should be multiplied by a factor of 1,12. The action effect due to the combination of the vertical and horizontal component should be calculated using Formulas (4.1) and (4.2):

(4.1)

(4.2)

NOTE The factor 0,34 is the result of using the combination according to EN 1998-1-1:—2, 6.4.4.

1. If non-linear response-history analysis is applied, the seismic action should be applied simultaneously in three directions. The input motions for response history analysis and the spatial model of the seismic action should be defined in accordance to EN 1998-1-1:—2, 5.2.3 unless otherwise defined in Clause 5 to 10.
2. If non-linear static analysis is applied, the lateral seismic forces should be applied in the most unfavourable direction with both positive and negative signs. The maximum seismic action effects resulting from both cases should be used. Seismic action effects due to the combination of the horizontal components of the seismic action, the effects of higher modes, torsion and minimum eccentricity should be evaluated according to EN 1998-1-2:—3, 5.3.5.

## Material requirements

### Design to DC1, DC2 and DC3

1. Unless otherwise specified for particular types of structures in the relevant clauses of this document, the material specific rules according to EN 1998-1-2:—3, 10 to 15, should be applied.

### Safety verifications

1. For Significant Damage (SD) limit state verification, partial factors according to EN 1998-1-2:—3, 8 to 15, not lower than 1, should be used, unless otherwise specified for particular types of structures in the relevant clauses of this document.

## Verification to limit states

### General

1. The seismic performance of the structures shall be verified according to EN 1998-1-1:—2, 4.4.
2. In the seismic design situation, it should be verified that the action effects defined in EN 1990:2023, 8.3.4.4, do not exceed the corresponding resistance of structural elements according to EN 1990:2023, 8.3.5, for the specified limit states.
3. The verifications in 4.7.1(2) may be considered as satisfied if the total base shear over the entire specific structures of this part at the base level of the structure (foundation or top of a rigid basement for structures, points of attachment to the structure for ancillary elements) in the seismic design situation is less than the corresponding base shear in other design situations. The total base shear should be calculated with the force-based approach with a behaviour factor of *q* = 1,0 using the elastic design spectrum given in EN 1998‑1‑1:—2.

NOTE 1 For example, this can be the case in regions where the wind action can be higher than the seismic action.

NOTE 2 The definition and verification of the NC limit state is not covered in this document. EN 1998-3:—[[5]](#footnote-6) provides rules for the verification of the NC limit state for existing buildings and structures.

NOTE 3 Due to resonance phenomena in ancillary elements, when clause (3) applies at the level of the supporting structure, it does not necessarily imply that it also applies for the ancillary elements themselves, which can still need to be checked under the seismic design situation.

NOTE 4 The comparison of seismic base shear with that due to other actions can be insufficient for structural members where higher modes of vibration contribute more than the fundamental mode.

### Verification of Significant Damage (SD) limit state

1. The non-exceedance of the SD limit state for the specific structures given in 1.1(1) should be verified according to Clause 5 to 10.

### Verification of Damage Limitation (DL) limit state

1. When appropriate, the DL limit state for the specific structures given in1.1(1) should be verified according to Clause5 to 10.

### Verification of Fully Operational (OP) limit state

1. When appropriate, the OP limit state for the specific structures given in 1.1(1) should be verified according to Clause 5 to 10.

# Rules for silos

## Scope

1. Clause 5 gives rules for the structural analysis and design of steel, reinforced concrete and prestressed precast reinforced concrete silos subjected to seismic actions.

NOTE A distinction is made between:

* on-ground;
* elevated silos, supported on a skirt extending to the ground or by substructures.

## Basis of design

### Design concept

1. The effects of horizontal and vertical components of the seismic action considered on silos should be seismic induced stresses and deformations in the silo wall and seismic forces and moments applied to substructures and foundations.
2. The principles of the seismic analysis procedures may also be applicable for silos made of other materials (e.g. glass fibre-reinforced plastic/polymer (GFRP), high density polyethylene (HDPE) or polyethylene (PE)).

NOTE The definitions of limit states and safety verifications for materials other than those in (2) are not covered by this document.

1. Silos should be designed in DC1.
2. Substructures of elevated silos may be designed according to ductility classes DC1, DC2 or DC3.
3. If seismic protection is provided through base isolation or energy dissipation systems, the design provisions in EN 1998-1-1:—2, 6.8, and in EN 1998-1-2:—3, Clauses 8 and 9, should be applied.

### Safety verification

1. Partial factors *γ*M,i should comply with EN 1998-1-2:—3.
2. Partial factors *γ*M,i given by EN 1993-4-1:—[[6]](#footnote-7), 2.9.2.2, should be used for the limit state verifications of the shell and hopper of steel silos.
3. Overstrength effects that may occur in the substructure of silos designed in DC2 or DC3 should be considered in verifications.

## Modelling and structural analysis

### Modelling

1. The dynamic calculation model of the silo should reproduce accurately the strength, the damping, the geometrical properties and the stiffness and mass distribution of the silo, containment structures, external ancillary elements, connecting pipes, extensions rigidly connected to the silo and the substructure.
2. The dynamic effects of the silo content should be considered by additional structural masses assuming that the particulate content moves together with the silo shell. The distribution of the masses should reproduce the dynamic effects of the silo content. The effects of the content should be considered up to the part of the wall that is in contact with the stored contents, corresponding to the part on which static pressures are exerted according to EN 1991-1-4:—1, 1.2.
3. The silos may be idealised by simple beam models with distributed masses. Elevated silos should be modelled with their substructures. In case the silo and the substructure are modelled separately, their frequencies should differ by more than 20 %. In case of squat or retaining silos according to EN 1991-1-4:—1, 7.1, shell models for the silo wall and rigidly connected volume elements for the content may be used to account for the load transfer by friction and shear at the silo bottom.
4. Steel silos with or without substructures may be analysed assuming linear elastic behaviour according to EN 1993-1-6:—[[7]](#footnote-8), 2.2. If the silo shell is modelled, the modelling rules according to EN 1993-1-6:—7, 5.2, should be applied.
5. Reinforced concrete and prestressed precast reinforced concrete silos with or without substructures may be analysed assuming linear elastic behaviour according to EN 1992-1-1:—[[8]](#footnote-9).
6. Soil-structure interaction effects should be taken into account in accordance with EN 1998‑5:—4, Clause 8, with respect to the type of foundation.
7. The mass of the content should be determined in accordance with EN 1991-1-4:—1, Annex C, using the upper characteristic value of the bulk unit weight **u of the particulate solid. If the specific bulk material is not given in EN 1991-1-4:—1, Annex C, the characteristic value of the bulk unit weight may be determined by material tests.

NOTE The owner or other parties to the project can provide the value of the bulk unit weight.

1. The dynamic effects should be calculated for the maximum filling level determined based on operating conditions. Intermediate filling levels may be taken into account, if the corresponding structural periods for the empty or full silo straddle the upper corner period *T*C of the applied response spectrum, as defined in EN 1998-1-1:—2, 5.2.2.2 (1).
2. In batteries of silos, different likely distributions of full, intermediate filled and empty silos should be considered according to the operation rules of the facility. In each of the silos, only symmetrical filling loads should be considered.
3. In silos internally subdivided in several cells, the most unfavourable distribution of full, intermediate filled and empty cells should be considered to account for unsymmetrically loading conditions under seismic actions.

### Structural analysis

1. Except when (2) or (3) are applied, on-ground or elevated silos should be calculated with the force-based approach using calculation models specified in 5.3.1(3).
2. Silos with or without substructures may be analysed with non-linear approaches according to EN 1993-1-6:—7, 2.2, using non-linear response-history analysis and application rules given in EN 1998-1-1:—2, 6.6.

NOTE Non-linear static analysis of silos with or without substructures are not covered by this document.

1. The substructures of elevated silos may be analysed using non-linear static or non-linear response-history analysis according to EN 1998-1-1:—2, 6.5 and 6.6.
2. If non-linear response-history analysis is performed, damping ratios of 2 % for bolted and welded steel, 5 % for reinforced concrete and 2 % for prestressed precast reinforced concrete silo structures should be used.

### Behaviour factors

#### Behaviour factor for the horizontal components of the seismic action

##### Silos

1. The behaviour factor components for silos in DC1 should be applied as given in a) and b):
2. bolted and welded steel silos: *q*R= 1,0, *q*D= 1,0, *q*S= 1,2;
3. reinforced concrete or prestressed precast reinforced concrete tanks: *q*R= 1,0, *q*D= 1,0, *q*S= 1,5.

##### Substructures of elevated silos

1. The behaviour factors for substructures of elevated silos in DC1, DC2 and DC3 should be applied as given in the relevant parts of EN 1998-1-2:—3.

#### Behaviour factor for the vertical component of the seismic action

1. The behaviour factor *q*v for the silo and any substructure should be applied as min(*q*; 1,5), where *q* is the behaviour factor for the horizontal component of the seismic action applied to the silo per 5.3.3.1.1.

## Seismic loads according to the force-based approach

### Total base shear, overturning moment and vertical reaction force at the silo bottom

1. The total base shear *F*b, overturning moment *M*b and vertical reaction force *F*bv at the silo bottom with or without substructures should be calculated with a response spectrum analysis according to EN 1998-1-1:—2, 6.4.2, by using the calculation models specified in 5.3.1(3) and the combination of the effects of the seismic action as given in 4.5.
2. The lateral forces method of analysis according to EN 1998-1-2:—3, 5.3.3, may be applied for on-ground silos. In this case the base shear, the vertical reaction force and the overturning moment in each horizontal direction may be calculated as given in Formulas (5.1) to (5.3).

(5.1)

(5.2)

(5.3)

where

|  |  |
| --- | --- |
| *T*1h | is the fundamental period of vibration of the silo structure in the horizontal direction under consideration; |
| *T*1v | is the fundamental period of vibration in the vertical direction; |
|  | is the ordinate of the reduced spectrum in horizontal direction at period *T*1h as defined in EN 1998-1-1:—2, 6.4.1(5); |
|  | is the ordinate of the reduced spectrum in vertical direction at period *T*1v; the maximum spectral acceleration in the constant acceleration range may be applied, if the period *T*1v is not explicitly calculated; |
|  | is the total mass of the silo and its content, calculated with the upper characteristic value of the bulk unit weight **u as defined in 5.3.1(7); |
| ** | is the reduction factor used in lateral force method taken between **to |
|  | is the height of the maximum design filling level of the silo content. |

1. The base shear *F*b and overturning moment *M*b in each horizontal direction may be calculated for cylindrical on-ground silos with radius *R* without taking into account grain-wall friction effects, as given by Formulas (5.4) and (5.5).

NOTE These Formulas are more accurate than Formulas (5.2) and (5.3) in this particular case.

(5.4)

(5.5)

where is the density calculated with the upper characteristic value of the bulk unit weight **u as defined in 5.3.1(7).

### Seismic pressures on silo walls and hoppers due to the horizontal seismic actions

#### Reference normal pressure

1. The seismic loads on silo walls should be calculated using an acceleration profile over the silo height, which is described in terms of the function *(z)* corresponding to the ratio of the response acceleration at a depth *z* from the equivalent surface of the stored content over the acceleration of gravity.
2. The function of the response acceleration *(z)* should be calculated with a response spectrum analysis according to EN 1998-1-1:—2, 6.4.3, by using the calculation models specified in 5.3.1(3) and the combination of the effects of the seismic action as described in 4.5. Alternatively, *(z)* may be approximated by linear a linear response acceleration profile between the response accelerations at the silo bottom and the level of the equivalent surface. In case of stiff silos, a constant response acceleration, calculated as the average of the response accelerations at the silo bottom and the level of the equivalent surface of the stored content, may be applied.
3. The effect on the shell of the response of the particulate content to the horizontal components of the seismic action may be represented through an additional normal pressure on the wall, *Δ*ph,s. At points on the silo vertical wall at a vertical distance *x* from a flat bottom or the apex of a conical or pyramidal hopper, the reference pressure normal to the wall *Δ*ph,so may be calculated using Formula (5.6).

(5.6)

where

|  |  |
| --- | --- |
| *α*(*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 theupper characteristic value of the bulk unit weight of the particulate solid as defined in 5.3.1(7); |
| *h*b | is the overall height of the silo, from a flat bottom or the hopper outlet to the equivalent surface of the stored content as defined in EN 1991-1-4:—1, 1.2; |
| *d*c | is the inner dimension of the silo parallel to the horizontal component of the seismic action (inside diameter, *d*c in circular silos or silo compartments, inside horizontal dimension *b* parallel to the horizontal component of the seismic action in rectangular ones) as defined in EN 1991-1-4:—1, 1.2. |

1. Within the height of a hopper, the reference normal pressure may be taken as given by Formula (5.7).

(5.7)

where *β* is the angle of inclination of the conical hopper wall, measured from the vertical, or the steepest angle of inclination to the vertical of the wall in a pyramidal hopper and *x* is the vertical coordinate of a point on a silo wall from a flat silo bottom or the apex of a conical or pyramidal hopper as given in EN 1991-1-4:—1, 8.

#### Loads on walls in circular silos

1. The additional normal pressure (Figure 5.1a) on the wall of circular silos (or silo compartments) may be taken at any level as equal to *Δ*ph,s given by Formula (5.8).

(5.8)

where *θ* is the angle (0° ≤ *θ*  < 360° ) between the radial line to the point of interest on the wall and the direction of the horizontal component of the seismic action.

Figure 5.1 — Additional normal pressure in: (a) circular silos and (b) rectangular silos

#### Loads on walls in rectangular silos or silo compartments

1. The additional normal pressure (Figure 5.1b) on the wall due to a horizontal component of the seismic action parallel or normal to the silo walls may be taken equal to one of the values given in a) to c):
2. on the ‘leeward’ wall which is normal to the horizontal component of the seismic action, by Formula (5.9).

(5.9)

1. on the ‘windward’ wall which is normal to the horizontal component of the seismic action, by Formula (5.10).

(5.10)

1. on the walls parallel to the horizontal component of the seismic action, by Formula (5.11).

(5.11)

#### Loads on silo walls and hoppers due to vertical components of seismic actions

1. The seismic action due to the vertical component should be considered by a scaling factor **sc given in Formula (5.12), applied to the frictional pressures *p*wf, the vertical pressures *p*vfand the horizontal pressures *p*hffor vertical silo walls, silo hoppers and silo bottoms, calculated according to EN 1991-1-4:‑1, Clauses 7, 8 and 9.

(5.12)

where *S*rv is the ordinate of the reduced spectrum at period *T*1v as defined in EN 1998-1-1:—2**,** 6.4.1(6).

#### Superposition with hydrostatic pressure

1. The resulting pressure at any point on the silo wall should be the sum of the hydrostatic normal pressure *p*0 of the particulate material on the wall and the normal pressure of the seismic action effect, *Δ*ph,s. If at any position on the silo wall the sum is negative (implying net suction on the wall), the resulting pressure should be set to zero and the negative pressure should be added to the opposite compression-loaded side of the silo wall. If , the increased additional normal pressure on the compression-loaded side may be calculated as given by Formula (5.13).

(5.13)

## Verification to limit states

### General

1. It should be verified that the action effects in the design seismic situation do not exceed the corresponding resistances of the silo, the substructure and the foundation, including connections, relevant ancillary elements and connecting pipes for the specified limit states. The verification of structural members should comply with EN 1998-1-2:—3, Clause 6.

### Verification of Significant Damage (SD) limit state

1. In application of EN 1998-1-1:—2, 6.2, the SD limit state may be considered as verified if the conditions in 5.5.2.1 to 5.5.2.5 in the seismic design situation are met.

#### Global stability

1. Silos with or without substructures should be verified for overturning and sliding for the total base shear, overturning moment and vertical reaction force calculated according to 5.4.1. Limited sliding of the silo may be accepted, if it is demonstrated that the implications of sliding for the connections between the various parts of the structure and between the structure and any piping are taken into account in the analysis and the verifications.
2. Uplift of on-ground silos should be avoided by sufficient anchorage according to 5.5.2.5.

#### Foundations

1. Silo foundation systems should be verified in accordance with EN 1998‑5:—4, Clause 9 for the supporting loads according to 5.4.1.

#### Silo shell and hopper

1. The seismic action effects in the silo shell and hopper (membrane forces and bending moments, circumferential or meridional, and membrane shear) should be calculated in the seismic design situation.
2. Steel silos should be verified in the seismic design situation according to EN 1993-4-1:—6, Clauses 5 to 9.
3. Reinforced concrete and prestressed precast reinforced shells should be verified according to EN 1992-3 and EN 1992-1-1:—8.

#### Substructures of elevated silos

1. Substructures of elevated silos should be verified in the seismic design situation according to the relevant clauses of EN 1998-1-2:—3.

#### Anchorage systems

1. Anchorage systems of silo structures to their foundation should be designed to remain elastic in the seismic design situation. The anchorage systems should be designed by applying the capacity design principle taking into account all relevant overstrength effects of the silo and its substructure. For DC2 and DC3 the overstrength factors should be applied according to the relevant clauses of EN 1998-1-2:—3. For all ductility classes, a minimum overstrength factor of 1,25 should be used.
2. The anchorage systems of silo structures to their foundation should be provided with sufficient ductility to avoid brittle failures in case the seismic loads are exceeded. Therefore, the strength associated to the steel anchorage plastic mechanism should be smaller than the resistance associated to brittle failure of concrete, also accounting for overstrength of the former. The design principles in accordance with EN 1998-1-1:—2, Annex G, should be applied.
3. Installed anchors should sustain the seismic action effects in tension, shear and combined tension and shear to which they are subjected in the design seismic situation while providing adequate resistance to failure (ultimate limit state) according to EN 1998-1-1:—2, Annex G.

NOTE Fasteners of category C1 provides capacities only in term of resistances at ultimate limit state, while fasteners of category C2 provides capacities in terms of both resistances at ultimate and displacements at damage limitation state and ultimate state. The requirements for category C2 are more stringent compared to those for category C1. The performance category valid for a fastener is given in the corresponding European Technical Product Specification.

1. Pre-installed anchors should be designed to resist the effects of the seismic design loads according to EN 1993-1-8:—10. They should provide design resistance to tension due to uplift forces and bending moments where appropriate. One of the methods in a) to d) should be used to secure anchor bolts into the foundation:
2. a hook;
3. a washer plate;
4. some other appropriate load distributing member embedded in the concrete;
5. some other fixing which has been adequately tested and approved.
6. Structural connections between the silo structure and ancillary elements or connecting pipes should be designed to remain elastic in the seismic design situation. The connections should be capacity designed, taking into account all relevant overstrength effects of the components and connecting pipes.

#### Inlets, outlets, pipes and ancillary elements

1. Inlets, outlets and further pipes should be verified to accommodate stresses and distortions due to relative displacements between silos, between silos and adjacent structures and between silos and the foundation (EN 1998-1-1:—2, 5.2.2.4), without their functions being impaired.
2. If connected pipes induce too high local stresses in the thin walls of steel tanks, flexible connections may be provided by the use of armoured hoses, compensators, swing arm joints or arrangement of flexible bend configurations.
3. In case of silos with base isolation, the deformation compatibility of the structure and the ancillary elements connecting the silo to the ground or to adjacent structures should be verified.
4. Anchorages of ancillary elements (e.g. conveyors, screw feeders) should be verified in the design seismic situation for the SD limit state according to 9.

NOTE The seismic verification of the ancillary elements burdens the producer and is not covered in this document.

### Verification of Damage Limitation (DL) limit state

#### Silo shell and hopper

1. The DL limit state may be considered as verified when the silo shell, the hopper, the substructure, the anchorage, connecting pipes and connections of ancillary elements resist the seismic actions in the elastic range.

NOTE Additional verifications at DL limit state applied to silos can be specified by a relevant authority or can be found in the National Annex or can be elsewhere be provided by the relevant Authorities.

### Verification of Fully Operational (OP) limit state

1. It should be verified that strains (or generalised deformations such as drifts) resulting from the corresponding seismic design situation do not exceed values that are acceptable to maintain the function of the silo and associated equipment.
2. Criteria applicable to the silo and associated equipment, in addition to EN 1998-1-1:—2, 6.7.3(7), should be derived from the analysis of the components the operability of which is required, as well as from the analysis of their supporting systems.

NOTE For a specific project, the relevant parties can specify all components of interest in the verification, together with a description of relevant damage states for each component and the associated requirements.

# Rules for tanks

## Scope

1. Clause 6 gives rules for the structural analysis and design of steel, reinforced concrete and prestressed precast reinforced concrete liquid storage tanks with circular and rectangular cross sections subjected to seismic actions.
2. This clause gives rules for anchored and unanchored tanks with fixed or floating roofs.

NOTE A distinction is made between:

* above-ground;
* underground;
* elevated tanks, supported by substructures.

## Basis of design

### Design concept

1. The effects of horizontal and vertical components of the seismic action considered on tanks should be seismic induced stresses and deformations in the tank wall and seismic forces and moments applied to substructures and foundations.
2. The principles of the seismic analysis procedures may also be applicable for tanks made of other materials (e.g. glass fibre-reinforced plastic/polymer (GFRP), high density polyethylene (HDPE) or polyethylene (PE)).

NOTE The definitions of limit states and safety verifications for materials other than those in (2) are not covered by this document.

1. Tanks should be designed in DC1.
2. Substructures of elevated tanks may be designed according to ductility classes DC1, DC2 or DC3.
3. If seismic protection is provided through base isolation or energy dissipation systems, the design provisions in EN 1998-1-1:—2, 6.8, and in EN 1998-1-2:—3, 8 and 9 should be applied.

### Safety verification

1. Partial factors *γ*M,i should comply with EN 1998-1-2:—3.
2. Partial factors *γ*M,i given by EN 1993-4-2:—21, 2.9.2.2, should be used for the limit state verifications of the shell and of steel tanks.
3. Overstrength effects that may occur in the substructure of tanks designed in DC2 or DC3 should be considered in verifications.

## Modelling and structural analysis

### Modelling

1. Dynamic calculation models of the tank should reproduce accurately the stiffness, the strength, the damping, the mass and the geometrical properties of the tank structure and should account for the hydrodynamic response of the contained liquid.
2. Dynamic calculation models should properly consider effects in a) to e), where relevant:
3. sloshing of the liquid at the free surface (convective component);
4. horizontal and vertical movement of the liquid together with the rigid tank (impulsive component);
5. fluid-structure vibrations of the tank and the liquid in case of flexible tanks (impulsive flexible component);
6. soil-structure-interaction;
7. sliding and uplift in case of unanchored tanks.
8. For the purpose of evaluating the dynamic response under seismic actions, the liquid may be assumed as incompressible.
9. Steel tanks with or without substructures may be analysed assuming linear elastic behaviour according to EN 1993-1-6:—7, 2.2. If the tank shell is modelled, the modelling rules according to EN 1993-1-6:—7, 5.2, should be applied.
10. Reinforced concrete and prestressed precast reinforced concrete tanks with or without substructures may be analysed assuming linear elastic behaviour according to EN 1992-1-1:—8.
11. Soil-structure interaction effects should be taken into account in accordance with EN 1998‑5:—4, Clause 8 with respect to the type of foundation.
12. The mass of the liquid should be determined in accordance with EN 1991-1-1:—20, Annex A, using the specified liquid weights *l*. If the specific liquid is not given in EN 1991-1-1:—20, Annex A, the liquid weight should be taken from reliable sources, or may be determined by material tests.

NOTE The owner or other parties to the project can provide the value of the liquid weight.

1. The dynamic effects should be calculated for the maximum filling level determined based on operating conditions. Intermediate filling levels may be taken into account, if the corresponding vibration periods for the empty or full tank straddle the upper corner period *T*C of the applied response spectrum, as defined in EN 1998-1-1:—2, 5.2.2.2 (1).
2. Tank structures may be analysed with three-dimensional fluid-structure interaction simulation models, which are able to represent the hydrodynamic pressures on the tank shell and bottom under seismic excitation, soil-structure interaction effects as well as sliding and uplift in case of unanchored tanks.
3. The convective, impulsive and impulsive flexible seismic action effects due to horizontal and seismic actions may be calculated separately with dynamic calculation models, in which the hydrodynamic effects of the liquid are considered by additional structural masses on the tank wall.
4. The calculation model for tanks under horizontal seismic actions may be represented by spring-mass models which describe the hydrodynamic response by impulsive rigid, impulsive flexible and convective masses with corresponding lever arms (Figure 6.1).
5. The calculation model for tanks under vertical seismic actions should represent the impulsive rigid response by the total liquid mass rigidly connected to the tank bottom and the impulsive flexible response with the total liquid mass connected by a spring to the tank bottom.
6. Rigid tanks may be analysed without consideration of the flexible impulsive vibration modes in horizontal and vertical directions.
7. Mass inertia effects of the tank wall, roof and ancillary elements should be considered for rigid and flexible tanks.
8. Sliding and uplift of unanchored tanks may be considered in the calculation models by contact formulations or nonlinear springs.

NOTE The seismic loads are defined with the notations and key given in Figure 6.1.

Figure 6.1 — Spring mass model and notations for tanks under horizontal seismic actions

Key

|  |  |
| --- | --- |
| *RL* | rigid link |
| *m*l | total liquid mass |
|  | masses of the tank wall, tank bottom and tank roof |
| *m*ir, *m*if, *m*c | equivalent impulsive rigid, impulsive flexible and convective masses as defined in 6.4.1.4 for vertical cylindrical tanks and in 6.5.1.4 for vertical rectangular tanks |
| *h*ir, *h*if*,h*c | equivalent lever arms without consideration of the bottom pressure as defined in 6.4.1.4 for vertical cylindrical tanks and in 6.5.1.4 for vertical rectangular tanks |
| , , | equivalent lever arms with consideration of the bottom pressure as defined in 6.4.1.4 for vertical cylindrical tanks and in 6.5.1.4 for vertical rectangular tanks |
| *T*ir,h*, T*if,h*,T*con | first natural periods of the impulsive rigid, impulsive flexible and convective vibration modes as defined in 6.4.1.3 for vertical cylindrical tanks and in 6.5.1.3 for vertical rectangular tanks |
| *T*ir,v*, T*if,v | first natural periods of the impulsive rigid and impulsive flexible vibration modes in vertical direction as defined in in 6.4.1.3 for vertical cylindrical tanks and in 6.5.1.3 for vertical rectangular tanks |
|  | ordinate of the reduced spectrum in horizontal direction at period as defined in EN 1998-1-1:—2, 6.4.1(5) using the behaviour factor given in 6.3.3 |
|  | ordinate of the reduced spectrum in vertical direction at period as defined in EN 1998-1-1:‑2, 6.4.1(6) using the behaviour factor given in 6.3.3 |
|  | spectral acceleration obtained from a horizontal elastic response spectrum as defined in EN 1998-1-1:—2, 5.2.2.2(1), corresponding to the convective period and to the damping ratio of the liquid of 0,5% (6.3.2(5)); for convective periods longer than 4,0 s, EN 1998-1-1:—2, 5.2.2.2(13), should be used |
|  | ordinate of the reduced spectrum in horizontal direction at period as defined in EN 1998-1-1:—2, 6.4.1(5) using the behaviour factor given in 6.3.3 |
|  | ordinate of the reduced spectrum in vertical direction at period as defined in EN 1998-1-1:‑2, 6.4.1(6) using the behaviour factor as indicated in 6.3.3 |

NOTE Only the first fundamental periods are considered within the proposed force-based approach in this document.

### Structural analysis

1. Except when (2) or (3) are applied, above-ground and elevated tanks with or without substructures should be analysed with the force-based approach with calculation models specified in 6.3.1. The dynamic effects of the convective and impulsive modes of vibrations should be described by equivalent static pressure distributions applied on the tank wall and bottom.
2. Above-ground tanks and elevated tanks with substructures may be analysed with nonlinear approaches according to EN 1993-1-6:—7, 2.2, using non-linear response-history analysis and application rules given in EN 1998-1-1:—2, 6.6.

NOTE Non-linear static analysis of tanks with or without substructures are not covered by this document.

1. The substructures of elevated tanks may be analysed using non-linear static or non-linear response-history analysis according to EN 1998-1-1:—2, 6.5 and 6.6.
2. If non-linear response-history analysis is performed, the damping ratios in a) and b) may be applied:
3. *ξ*  = 2 % for bolted and welded steel tanks.
4. *ξ*  = 5 % for reinforced concrete tanks.
5. *ξ*  = 2 % prestressed precast reinforced concrete tanks.
6. The value of *ξ* = 0,5% for the damping ratio of water and other liquids, unless otherwise determined, may be adopted where sloshing modes of vibration are concerned.

### Behaviour factors

#### Behaviour factor for the horizontal components of the seismic action

##### Above-ground tanks

1. The behaviour factor components for tanks in DC1 should be applied as given in a) and b):
2. bolted and welded steel tanks: *q*R= 1,0, *q*D= 1,0, *q*S= 1,2;
3. reinforced concrete or prestressed precast reinforced concrete tanks: *q*R= 1,0, *q*D= 1,0, *q*S= 1,5.

##### Substructures of elevated tanks

1. The behaviour factors for DC1, DC2 and DC3 for elevated tanks with their supporting structures should be applied as given in the relevant parts of EN 1998-1-2:—3.

#### Behaviour factor for the vertical component of the seismic action

1. The behaviour factor *q*v for the tank and any substructure should be applied as min(*q*, 1,5), where *q* is the behaviour factor for the horizontal component of the seismic action applied to the tank per 6.3.3.1.1.

## Seismic loads according to the force-based approach for vertical cylindrical tanks

### Above ground anchored tanks

NOTE The seismic loads are defined with the notations given in Figure 6.2.

Key

|  |  |
| --- | --- |
| *H* | filling height |
|  | tank height |
| *R* | radius |
|  | dimensionless radius |
|  | dimensionless height |
|  | circumferential angle |
|  | ratio of filling height to tank radius |

Figure 6.2 — Notations for vertical cylindrical tanks

#### Total base shear, overturning moment and vertical reaction force at tank bottom

##### Impulsive rigid support reactions

1. The maximum impulsive rigid base shear , the impulsive rigid vertical reaction force , the impulsive rigid moment just above the base plate, and the maximum rigid impulsive moment below the base plate, , including the base pressure component may be calculated using Formulas (6.1) to (6.4) respectively.

(6.1)

(6.2)

(6.3)

(6.4)

1. The impulsive rigid support reactions may be calculated as integrals from the pressure functions given in 6.4.1.2 using Formulas (6.5) to (6.8).

NOTE These Formulas are more accurate than Formulas (6.1) to (6.4) in this particular case.

(6.5)

(6.6)

(6.7)

(6.8)

where

|  |  |
| --- | --- |
|  | is the impulsive rigid base shear coefficient as given in Annex A, Table A.7; |
|  | is the impulsive rigid overturning moment coefficient above the base plate as given in Annex A, Table A.7; |
|  | is the impulsive rigid overturning moment coefficient at the base plate as given in Annex A, Table A.7; |
|  | is the participation factor of the impulsive rigid pressure mode as given in Annex A, Table A.7; |
|  | is the participation factor of the impulsive rigid pressure mode in vertical direction: ; |
|  | is the period of the impulsive rigid vibration mode in horizontal direction as given in 6.4.1.3.2; |
|  | is the period of the impulsive rigid vibration mode in vertical direction as given in 6.4.1.3.3. |

##### Convective support reactions

1. The convective base shear , the convective moment just above the base plate and the convective moment at the base including the base pressure component arising from the first sloshing mode may be calculated using Formulas (6.9) to (6.11) respectively.

(6.9)

(6.10)

(6.11)

1. The convective support reactions may be calculated as integrals from the pressure functions given in 6.4.1.2 using Formulas (6.12) to (6.14).

NOTE These Formulas are more accurate than Formulas (6.9) to (6.11) in this particular case.

(6.12)

(6.13)

(6.14)

where

|  |  |
| --- | --- |
|  | is the participation factor of the convective pressure component as given in Annex A, Table A.7; |
|  | is the convective base shear coefficient as given in Annex A, Table A.7; |
|  | is the convective overturning moment coefficient above the base plate as given in Annex A, Table A.7; |
|  | is the convective overturning moment coefficient at the base plate as given in Annex A, Table A.7; |
|  | is the period of the convective vibration mode as defined in 6.4.1.3.1. |

##### Impulsive flexible support reactions

1. The impulsive flexible horizontal base shear , the impulsive flexible vertical reaction force , the impulsive flexible moment just above the base plate and the impulsive flexible moment below the base plate including the base pressure component should be calculated using Formulas (6.15) to (6.18) respectively.

(6.15)

(6.16)

(6.17)

(6.18)

where

|  |  |
| --- | --- |
|  | is the dimensionless impulsive flexible base shear coefficient as given in Annex, Table A.7; |
|  | is the impulsive flexible overturning moment coefficient above the base plate as given in Annex, Table A.7; |
|  | is the impulsive flexible overturning moment coefficient at the base plate as given in Annex A, Table A.7; |
|  | is the correction factor for taking account of the clamping effect as given in Annex A, Table A.5; |
|  | is the participation factor of the impulsive flexible horizontal mode as given in Annex A, Table A.7; |
|  | is the period of the impulsive flexible vibration mode in horizontal direction as given in 6.4.1.3.4; |
|  | is the period of the impulsive flexible vibration mode in vertical direction as given in 6.4.1.3.5. The maximum spectral acceleration in the constant acceleration range may be applied, if the period is not explicitly calculated. |

##### Support reactions for rigid tanks due to mass inertia effects

1. The horizontal base shear , the vertical reaction force and the moment above the base plate due to inertia effects of the tank wall, roof and ancillary elements may be calculated with the heights of the centres of gravity of the tank wall and roof , the period (6.4.1.3.2)and the period (6.4.1.3.3) by using Formulas (6.19) to (6.21).

(6.19)

(6.20)

(6.21)

##### Support reactions for flexible tanks due to mass inertia effects

1. The horizontal base shear , the vertical reaction force and the moment above the base plate due to inertia effects of the tank wall, roof and ancillary elements may be calculated with the height at the centre of gravity of the tank roof , the lever arm calculated as two-third of the tank height , the period (6.4.1.3.4)and the period (6.4.1.3.5) by using Formulas (6.22) to (6.24).

(6.22)

(6.23)

(6.24)

#### Seismic pressures on tank wall and bottom

##### Impulsive rigid pressure component for horizontal seismic actions

1. The impulsive rigid pressure function on the tank wall may be calculated by Formula (6.25).

(6.25)

where

|  |  |
| --- | --- |
|  | is the dimensionless impulsive rigid pressure function according to Annex A, Table A.2; |
|  | is the participation factor of the impulsive rigid pressure component according to Annex A, Table A.7; |
|  | is the period of the impulsive rigid vibration mode in horizontal direction as given in 6.4.1.3.2. |

1. The impulsive rigid pressure function on the tank bottom may be calculated by Formula (6.25) with .

##### Impulsive rigid pressure component for vertical seismic actions

1. The impulsive rigid pressure function on the tank wall may be calculated by Formula (6.26).

(6.26)

where

|  |  |
| --- | --- |
|  | is the participation factor of the impulsive rigid pressure in vertical direction: ; |
|  | is the period of the impulsive rigid vibration mode in vertical direction as given in 6.4.1.3.3. |

1. The impulsive rigid pressure on the tank bottom may be calculated by Formula (6.26) with .

##### Convective pressure component for horizontal seismic actions

1. The convective pressure function on the tank wall may be calculated by Formula (6.27).

(6.27)

where

|  |  |
| --- | --- |
|  | is the dimensionless convective pressure function according to Annex A, Table A.1; |
|  | is the participation factor of the convective pressure component according to Annex A, Table A.7; |
|  | is the period of the convective vibration mode as defined in 6.4.1.3.1. |

1. The convective pressure function on the tank bottom may be calculated by Formula (6.27) with .

##### Impulsive flexible pressure component for horizontal seismic actions

1. The impulsive flexible pressure component on the tank wall may be calculated by Formula (6.28).

(6.28)

where

|  |  |
| --- | --- |
|  | is the dimensionless impulsive flexible pressure function according to Annex A, Table A.3; |
|  | is the participation factor of the impulsive flexible horizontal mode according to Annex A, Table A.7; |
|  | is the period of the impulsive flexible vibration mode in horizontal direction as given in 6.4.1.3.4. |

1. The impulsive flexible pressure function on the tank bottom may be calculated by Formula (6.28) with .

##### Impulsive flexible pressure component for vertical seismic actions

1. The impulsive flexible pressure component on the tank wall may be calculated by Formula (6.29).

(6.29)

where

|  |  |
| --- | --- |
|  | is the dimensionless impulsive flexible pressure function according to Annex A, Table A.4; |
|  | is the participation factor of the impulsive flexible vertical mode according to Annex A, Table A.6; |
|  | is the period of the impulsive flexible vibration mode in vertical direction as given in 6.4.1.3.5. The maximum spectral acceleration in the constant acceleration range may be applied, if the period is not explicitly calculated**.** |

1. The impulsive flexible pressure function on the tank bottom may be calculated by Formula (6.29) with.

##### Mass inertia effects due to the tank mass for flexible tanks

1. The mass inertia effects of the tank wall, roof and ancillary elements may be considered within the calculation of the impulsive flexible hydrodynamic pressure by increasing the horizontal (6.4.1.2.4) and vertical (6.4.1.2.5) impulsive flexible pressure components with the increase factors . The factors may be calculated with the periods (6.4.1.3.4), (6.4.1.3.5) and base shears (Formula (6.15)), (Formula (6.16)) as given by Formulas (6.30) and (6.31).

(6.30)

(6.31)

1. The inertia effects for thin-walled steel tanks may be neglected.

##### Mass inertia effects due to the tank mass for rigid tanks

1. The mass inertia effects of the tank wall, roof and ancillary elements should be added to the horizontal and vertical impulsive rigid hydrodynamic pressures.
2. The vertical and horizontal mass inertia effects may be calculated with the periods (6.4.1.3.2), (6.4.1.3.3)by inducing a uniform pressure to the tank wall and roof in the direction of the seismic action as given by Formulas (6.32) and (6.33).

(6.32)

(6.33)

where

|  |  |
| --- | --- |
|  | is the density of the tank wall or roof; |
|  | is the uniform thickness of the tank wall or the average thickness in case of stepwise wall thickness variation along the tank height. |

#### Fundamental periods of vibrations

##### Convective vibration mode

1. The first natural period *T*con of the convective mode may be calculated using Formula (6.34).

(6.34)

##### Impulsive rigid vibration mode in horizontal direction

1. In case of tanks without consideration of soil–structure interaction, the period of the impulsive rigid vibration mode should be taken equal to zero.
2. The period of the impulsive rigid vibration mode of the tank–foundation system including soil–structure interaction may be calculated as given in Annex B, B.3.

##### Impulsive rigid vibration mode in vertical direction

1. In case of rigid tanks without consideration of soil–structure interaction, the period of the vertical impulsive rigid vibration mode should be taken equal to zero.
2. The period of the vertical impulsive rigid vibration mode of the tank–foundation system including soil–structure interaction may be calculated as given in Annex B, B.4.

##### Impulsive flexible mode in horizontal direction

1. In case of flexible tank neglecting soil–structure interaction, the period of the impulsive flexible vibration mode should be calculated with the Young’s modulus *E* of the tank wall and the correction factor according to Annex A, Table A.8, by Formula (6.35).

(6.35)

1. The period of the impulsive flexible vibration mode of the tank–foundation system including soil–structure interaction may be calculated as given in Annex B, B.5.

##### Impulsive flexible mode in vertical direction

1. In case of flexible tanks without consideration of soil–structure interaction, the period of the impulsive flexible vertical vibration mode should be calculated with Poisson’s ratio of the tank material and the correction factor according to Annex A, Table A.9 using Formula (6.36).

(6.36)

1. The period of the impulsive flexible vibration mode of the tank–foundation system including soil–structure interaction may be calculated as given in Annex B, B.6.

#### Impulsive rigid and convective masses and lever arms

1. The impulsive rigid mass and the corresponding lever arms *,*  for welded and bolted steel tanks may be calculated by either a) or b):
   1. if , using Formulas (6.37) to (6.39).

(6.37)

(6.38)

(6.39)

* 1. if , using Formulas (6.40) to (6.42).

(6.40)

(6.41)

(6.42)

1. The mass and the impulsive rigid lever arms *,*  may be calculated for reinforced concrete or precast reinforced concrete tanks, as given by Formulas (6.43) to (6.47).

(6.43)

(6.44)

(6.45)

(6.46)

(6.47)

1. The convective mass and the convective lever arms *,*  may be calculated as given by Formulas (6.48) to (6.50).

(6.48)

(6.49)

(6.50)

1. The impulsive flexible mass and the corresponding lever arms may be calculated with the impulsive flexible reaction forces in 6.4.1.1.3 by Formulas (6.51) to (6.53).

(6.51)

(6.52)

(6.53)

#### Convective wave height

1. The absolute maximum value of the vertical wave height in cylindrical tanks, , from the at-rest level of the liquid, should be calculated using Formula (6.54) using a damping ratio for sloshing per 6.3.2(5).

(6.54)

NOTE In case of tanks without roofs or with floating roofs, the maximum vertical displacement of the liquid surface is needed to determine the freeboard to prevent over–topping or spillage of the tank contents.

### Above ground unanchored tanks

1. Unanchored vertical cylindrical tanks shall be designed for the consequences of uplift effects.
2. Uplift effects may be analysed by calculating the overturning moment *M*R with an iterative procedure given in a) to d):
3. start from a value of ranging from 1,0, corresponding to no uplift, down to not less 0,3;
4. calculate using Formula (6.58).

(6.55)

1. calculate using Formula (6.59).

(6.56)

1. calculate the overturning moment using Formula (6.60).

(6.57)

The procedure defined in a) to d) should be repeated until the equilibrium with the total overturning moment according to 6.11.2is fulfilled.

1. If uplift occurs with an uplift length *l*up = > 0, the increased axial stresses in the tank wall and the additional radial membrane stresses in the base plate should be considered.
2. The additional radial membrane stresses in the base plate may be calculated using Formula (6.61).

(6.58)

where is the thickness of the base plate and is the hydrostatic pressure on the uplifted length of the base plate.

1. Uplift effects may also be considered in overall calculation models as given in 6.3.1(15).
2. Uplift effects induced by the reduced seismic action should be calculated considering EN 1998-1-2:—3, 6.4.2(2).

NOTE The consideration of uplift effects in overall calculation models is more accurate.

## Seismic loads according to the force-based approach for vertical rectangular tanks

### Above ground anchored tanks

NOTE The seismic loads are defined with the notations given in Figure 6.3.

Figure 6.3 — Notations for vertical rectangular tanks

#### Total base shear, overturning moment and vertical reaction force

##### Impulsive rigid support reactions

1. The maximum impulsive rigid base shear , the impulsive rigid vertical reaction force , the impulsive rigid moment just above the base plate and the maximum rigid impulsive moment below the base plate including the base pressure component may be calculated using Formulas (6.59) to (6.62) respectively.

(6.59)

(6.60)

(6.61)

(6.62)

where

|  |  |
| --- | --- |
|  | is the period of the impulsive rigid vibration mode in horizontal direction as given in 6.5.1.3.2; |
|  | is the period of the impulsive rigid vibration mode in vertical direction as given in 6.5.1.3.2. |

##### Convective support reactions

1. The maximum convective base shear, , the maximum convective moment just above the base plate, and the maximum convective moment at the base, including the base pressure component arising from the first sloshing mode, may be taken as given by Formula (6.63) to (6.65) respectively.

(6.63)

(6.64)

(6.65)

where is the period of the impulsive rigid vibration mode in horizontal direction as given in 6.5.1.3.1.

##### Impulsive flexible support reactions

1. The impulsive flexible support actions may be calculated using the calculation approach for the vertical cylindrical tank according to 6.4.1.1.3 by replacing the circular radius by the half length of the rectangular tank in the direction of the seismic action and the impulsive flexible periods and of the circular tank by the impulsive flexible periods for the rectangular tank given in 6.5.1.3.3 and 6.5.1.3.4.

##### Support reactions for rigid tanks due to mass inertia effects

1. The support reactions may be calculated using the calculation approach for the vertical cylindrical tank according to 6.4.1.1.4, by replacing the impulsive rigid periods and of the circular tank by the impulsive rigid periods for the rectangular tank given in 6.5.1.3.2.

##### Support reactions for flexible tanks due mass inertia effects

1. The support reactions may be calculated using the calculation approach for the vertical cylindrical tank according to 6.4.1.1.5, by replacing the impulsive flexible periods and of the circular tank by the impulsive flexible periods for the rectangular tank given in 6.5.1.3.3 and 6.5.1.3.4.

#### Seismic pressures

##### Impulsive rigid pressure component for horizontal seismic actions

1. The impulsive rigid pressure component on tank walls of length 2*B* (Figure 6.3) perpendicular to the direction of the seismic action may be applied as given by Formula (6.66).

(6.66)

where

|  |  |
| --- | --- |
|  | is the period of the impulsive rigid vibration mode in horizontal direction as given in 6.5.1.3.2; |
| *L* | is the half length of the rectangular tank in the direction of the seismic action; |
| *B* | is the half width of the rectangular tank perpendicular to the direction of the seismic action. |
|  | is the dimensionless impulsive pressure function as given in Formula (6.67). |

(6.67)

1. The impulsive rigid pressure component on the tank bottom may be applied as given by Formula (6.70) as linear function .

NOTE The procedure for the evaluation of the seismic pressures on rigid rectangular tanks of dimension is similar to that described for vertical cylindrical tanks fully anchored at the base.

##### Impulsive rigid pressure component for vertical seismic actions

1. The impulsive rigid pressure function in vertical direction may be calculated by Formula (6.68).

(6.68)

where

|  |  |
| --- | --- |
|  | is the period of the impulsive rigid vibration mode in vertical as given in 6.5.1.3.4; |
|  | The impulsive rigid pressure on the tank bottom may be calculated using Formula (6.68) with . |

1. The impulsive rigid pressure on the tank bottom may be calculated using Formula (6.70) with .

##### Convective pressure component for horizontal seismic actions

1. The convective pressure component on the walls of a rigid rectangular tank may be taken as a function of the vertical coordinate, , and corresponding only to the first sloshing mode, using Formula (6.69).

(6.69)

where

|  |  |
| --- | --- |
|  | is the period of the convective vibration mode as defined in 6.5.1.3.1; |
|  | is the dimensionless pressure function that defines the height wise variations of the first convective components of the wall pressures, given by Formula (6.70). |

(6.70)

##### Impulsive flexible pressure component for horizontal and vertical seismic actions

1. The impulsive flexible pressure component on tank walls perpendicular to the direction of the seismic action, with the dimension 2*B* (Figure 6.3) may be applied as the maximum impulsive flexible pressure distribution from the vertical cylindrical tank according to 6.4.1.2.4 for horizontal seismic actions and according to 6.4.1.2.5 for vertical seismic actions, by replacing the circular radius by the half length of the rectangular tank in the direction of the seismic action and the impulsive flexible period of the circular tank by the impulsive flexible period for the rectangular tank given in 6.5.1.3.3 and 6.5.1.3.4.

NOTE This follows the formulation proposed for cylindrical tanks.

##### Mass inertia effects due to the tank mass for flexible and rigid tanks

1. The mass inertia effects may be considered as proposed for cylindrical vertical tanks according to 6.4.1.2.6 and 6.4.1.2.7 by replacing the impulsive flexible base shears and the periods of the circular tank by the impulsive flexible base shears and periods for the rectangular tanks given in 6.5.1.1.3, 6.5.1.3.2, 6.5.1.3.3 and 6.5.1.3.4.

#### Fundamental periods of vibrations

##### Convective vibration mode

1. The first natural period *T*con of the convective mode may be calculated using Formula (6.71).

(6.71)

##### Impulsive rigid vibration mode in horizontal and vertical direction

1. In case of rigid rectangular tanks without consideration of soil–structure interaction, the periods of the impulsive rigid vibration modes in vertical and horizontal direction should be taken equal to zero.
2. The impulsive rigid periods and and the impulsive flexible period and for rectangular tanks including soil-structure interaction may be calculated using the calculation approaches for vertical cylindrical tanks in 6.4.1.3, replacing the circular radius of a vertical cylindrical tank by the half-length of the rectangular tank in the direction of the seismic action according to 6.3.1.1(2) and Figure 6.3.

##### Impulsive flexible vibration mode in horizontal direction

1. The first impulsive flexible period of vibration may be calculated as given by Formula (6.72).

(6.72)

where is the deflection of the wall on the vertical centreline and at the height of the impulsive mass when the wall is loaded by a load uniformly distributed load in the direction of the seismic action given by Formula (6.73).

(6.73)

1. The impulsive mass and corresponding lever arm may be calculated with an equivalent vertical cylindrical tank according to 6.5.1.4.

##### Impulsive flexible vibration mode in vertical direction

1. The vertical impulsive flexible period of vibration may be calculated as for an equivalent vertical cylindrical tank according to 6.4.1.3.5.

#### Impulsive rigid and convective masses and lever arms

1. The equivalent impulsive and convective masses and corresponding lever arms , , , , , of rigid rectangular steel tanks may be calculated according to Formulas (6.37) to (6.42), replacing the circular diameter by 2*L*, where *L* is the half-length of the rectangular tank in the direction of seismic motion (Figure 6.3).
2. The convective mass and the convective lever arms , may be calculated for reinforced concrete or prestressed precast reinforced concrete tanks using Formulas (6.74) to (6.76).

(6.74)

(6.75)

(6.76)

1. The impulsive rigid mass and the impulsive rigid lever arms , may be calculated for reinforced concrete or precast reinforced concrete tanks using Formulas (6.77) to (6.81).

(6.77)

(6.78)

(6.79)

(6.80)

(6.81)

1. The impulsive flexible mass and corresponding lever arms may be calculated with an equivalent vertical cyclindrical tank according to 6.4.1.4(5).

NOTE This is an only approximation as more accurate analytical solutions are not available.

#### Convective wave height

1. The absolute maximum value of the vertical wave height in cylindrical tanks, , from the at-rest level of the liquid, should be calculated using Formula (6.82) using a damping ratio for sloshing per 6.3.2(5).

(6.82)

NOTE In case of tanks without roofs or with floating roofs, the maximum vertical displacement of the liquid surface is needed to determine the freeboard to prevent over–topping or spillage of the tank contents.

### Above ground unanchored tanks

1. Unanchored tanks shall be designed for the consequences of uplift effects.
2. Uplift effects may be analysed by calculating the overturning moment *M*R with an iterative procedure given in a) to d):
   1. Start with an initial value of the uplifted length *l*up.
   2. Calculate the overturning Moment for the chosen uplifted length *l*up as given in Formula (6.83)

(6.83)

* 1. Check the equilibrium with total overturning moment according to 6.11.2.
  2. Stop iteration or choose new uplifted length *l*up and repeat from step b).

1. If uplift occurs (*l*up > 0), the increased axial stresses in the tank wall and the additional radial membrane stresses in the base plate should be considered.
2. The additional radial membrane stresses in the base plate may be calculated as given in Formula (6.84).

(6.84)

where

|  |  |
| --- | --- |
|  | is the thickness of the base plate; |
|  | is the hydrostatic pressure on the uplifted length of the base plate. |

1. Uplift effects may also be considered in overall calculation models as described in 6.3.1(15).
2. Uplift effects induced by the reduced seismic action should be calculated considering EN 1998-1-2:—3, 6.4.2(2).

NOTE The consideration of uplift effects in overall calculation models is more accurate.

## Seismic loads according to the force-based approach for horizontal cylindrical tanks

### Assumptions

NOTE The seismic loads are defined with the notations given in Figure 6.4.

Key

|  |  |
| --- | --- |
| *H* | filling height |
| *R* | radius |
|  | length in longitudinal direction |
|  | circumferential angle |

Figure 6.4 — Notations for horizontal axis cylindrical tanks

1. Horizontal cylindrical tanks shall be analysed for seismic action along the longitudinal and along the transverse axis (Figure 6.4).
2. A horizontal cylindrical tank, with a liquid level equal to *H*, may be represented by an equivalent rectangular tank with the same depth at the liquid level, the same dimension as the actual one and in the direction of the seismic action, and a third dimension (width) such that the liquid volume is maintained. This approximation is sufficiently accurate for design purposes for *H*/*R* between 0,5 and 1,6. If *H*/*R* exceeds 1,6, the total mass of the tank is moving solidly with the tank.

NOTE This assumption is an approximation as closed analytical solutions are not available.

#### Total base shear, overturning moment and vertical reaction force at tank bottom

1. The support reactions in either the longitudinal or transverse direction may be calculated as for rectangular tanks in 6.5.1.1 using an equivalent rectangular tank as defined in 6.6.1(2). If *H*/*R* exceeds 1,6, the convective support reactions may be neglected, and it may be assumed that the total liquid mass acts as impulsive mass at mid height of the tank.

#### Seismic pressures

1. Approximate values for hydrodynamic pressures induced by seismic action in either the longitudinal or transverse direction may be calculated as for rectangular tanks in 6.5.1.2 considering an equivalent rectangular tank as defined in 6.6.1(2). If *H*/*R* exceeds 1,6, the convective pressure component may be neglected and it may be assumed that the total liquid mass acts as impulsive mass.
2. For a horizontal seismic action in the transverse direction a more accurate solution for the impulsive rigid pressure distribution of a half full tank is given by Formula (6.85).

(6.85)

where

|  |  |
| --- | --- |
|  | is the period of the impulsive rigid vibration mode in horizontal direction as given in 6.6.1.3; |
|  | is the dimensionless pressure function given, for *H* = *R* and, by Formula (6.86). |

(6.86)

#### Fundamental periods of vibrations

1. The vibration modes may be obtained as for rectangular tanks in 6.5.1.3 using equivalent rectangular rigid tanks in either the longitudinal or transverse direction.

#### Impulsive rigid and convective masses and lever arms

1. The equivalent impulsive and convective masses and corresponding lever arms may be approximated for *H*/*R* = 0,5 to 1,6 as for rectangular tanks in 6.5.1.4 with an equivalent rectangular tank defined in 6.6.1(2).
2. If *H*/*R* exceeds 1,6, the tank should be assumed to be completely full, with an impulsive mass equal to the total mass with a corresponding lever arm of *h*ir = *R*.
3. For the filling level *H = R*, the impulsive rigid mass may be taken as and the mass for the first sloshing mode may be taken as .

NOTE The masses *m*ir and mc given in (3) are more accurate than the ones of the approximation in (1).

## Seismic loads according to the force-based approach for elevated tanks

1. Elevated tanks may be analysed using a two-coupled-mass model representing the convective and the impulsive vibration modes, including the mass and flexibility of the substructure (Figure 6.5). The flexibility of the tank shell may be neglected.

Figure 6.5 — Two coupled masses dynamic model for elevated tanks

1. Elevated tanks may be analysed calculating circular frequencies and periods of *n*−mode shapes , and maximum forces and deflections of the elevated tank according to the procedure given in a) to g):
2. evaluation of the impulsive and convective masses , and heights , , *,*  for vertical rectangular (6.5.1.4) or cylindrical (6.4.1.4) tanks; the additional mass should be taken as a maximum of two-thirds of the self-weight of the substructure.
3. calculation of the convective spring stiffness using Formula (6.87)

(6.87)

1. calculation of an equivalent spring stiffness of the substructure.
2. calculation of circular frequencies of the system as a function of the stiffnesses , and masses , , using Formula (6.88).

(6.88)

NOTE The periods of vibration corresponding to the natural circular frequencies are given by . Only the first vibration modes (*n* = 1, 2) are considered in the following ().

1. evaluation of the eigenvector with n = 1, 2 at the level of the mass , which corresponds to the mode shape of the freely vibrating system using Formula (6.89).

(6.89)

1. determination of the maximum displacements , and , corresponding to the masses () and , using Formulas (6.90) and (6.91) for the first mode and Formulas (6.92) and (6.93) for the second mode.

(6.90)

(6.91)

(6.92)

(6.93)

where

|  |  |
| --- | --- |
|  | ; |
|  | ; |
|  | , is the ordinate of the velocity response spectrum corresponding to periods and (*n* = 1, 2) and an appropriate damping ratio for the two modes of vibration conforming to 6.3.2 (5) for the first convective mode and 2 % for the second mode in case of steel substructures and 5 % in case of reinforced concrete substructures. |

1. Calculation of the maximum convective base shears and impulsive rigid base shears for each mode acting on each mass. These base shears may be expressed in terms of the spring constants, and and the corresponding maximum deflections, , and of the convective and rigid mass, as given by Formulas (6.94) and (6.95) for the first mode and Formulas (6.96) and (6.97) for the second mode.

(6.94)

(6.95)

(6.96)

(6.97)

##### Convective support reactions

1. The convective support reactions at the tank bottom may be calculated by using Formulas (6.98) to (6.100).

(6.98)

(6.99)

(6.100)

##### Impulsive rigid support reactions

1. The impulsive rigid support reactions at the tank bottom may be calculated by using Formulas (6.101) to (6.104).

(6.101)

(6.102)

(6.103)

(6.104)

1. If not calculated more precisely, the period should be taken equal to zero.

##### Support reactions due to mass inertia effects

1. The horizontal base shear , the vertical reaction force and the moment at the tank bottom due to mass inertia effects of the tank wall and ancillary elements may be calculated by using Formulas (6.105) to (6.107).

(6.105)

(6.106)

(6.107)

## Seismic loads according to the force-based approach for spherical tanks

### Spherical tanks

NOTE The seismic loads are defined with the notations for spherical tanks as given in Figure 6.6.

Key

|  |  |
| --- | --- |
| *H* | filling height |
|  | dimensionless coordinate |
| *R* | radius |

Figure 6.6 — Notations for spherical tanks

#### Total base shear, overturning moment and vertical reaction force at tank bottom

##### Impulsive rigid support reactions

1. The maximum impulsive rigid base shear , the impulsive rigid vertical reaction force and the impulsive rigid moment at the bottom of the tank may be calculated with the impulsive rigid mass according to Annex A, Table A.11 and the periods *T*ir,h and *T*ir,v using Formulas (6.108) to (6.110).

(6.108)

(6.109)

(6.110)

where

|  |  |
| --- | --- |
|  | is the period of the impulsive rigid vibration mode in horizontal direction as given in 6.8.1.3.2; |
|  | is the period of the impulsive rigid vibration mode in vertical direction. If not calculated more precisely, the period should be taken equal to zero; |
|  | is the equivalent lever arm of the impulsive rigid mass, measured from two-third of the filling height *H* to the bottom of the tank. |

#### Convective support reactions

1. The convective base shear and the convective moment just above the base plate may be calculated with the convective mass according to Annex A, Table A.11 and the period *T*con as given in (6.8.1.3.1) using Formulas (6.111) and (6.112).

(6.111)

(6.112)

where is the equivalent lever arm of the convective mass, measured from the filling height *H* to the bottom of the tank.

##### Support reactions due to mass inertia effects

1. The horizontal base shear , the vertical reaction force and the moment at the bottom of the tank due to mass inertia effects of the tank wall and ancillary elements may be calculated by using Formulas (6.113) to (6.115).

(6.113)

(6.114)

(6.115)

where is the lever arm from the centre of gravity of the tank wall to the tank bottom.

#### Fundamental periods of vibrations

##### Convective vibration mode

1. The first natural period *T*con of the convective mode may be calculated by Formula (6.116).

(6.116)

where is the first circular frequency of the sloshing mode as given in Annex A, Table A.11 for the corresponding filling height.

##### Impulsive rigid vibration mode in horizontal direction

1. The period of the impulsive rigid vibration mode of the spherical tank and its supporting system may be calculated using Formula (6.117).

(6.117)

The circular frequency of the impulsive rigid mode may be calculated with the stiffness of the tank supporting structure, the impulsive rigid mass , the mass of the spherical tank wall including ancillary elements and the contributing mass of the tank supporting system by Formula (6.118).

(6.118)

## Seismic loads on embedded tanks

1. Seismic induced soil pressures on embedded tanks due to soil-structure interaction effects shall be considered.
2. For tanks with rigid walls the seismic induced soil pressures may be calculated according to in EN 1998‑5:—4, 10, 11.
3. Seismic induced soil pressures on embedded tanks with flexible walls should be calculated by appropriate calculations models as described in 6.3.1 taking soil-structure interaction effects into account.

## Superposition of horizontal and vertical seismic pressures

### Superposition of horizontal pressure components due to different modes of response

1. The superposition of the horizontal pressure components should be carried out using Formula (6.119).

(6.119)

### Superposition of horizontal pressure components due to different modes of response

1. The superposition of the two vertical pressure components should be carried using Formula (6.120).

(6.120)

### Superposition of resulting pressures in horizontal and vertical directions

1. The resulting horizontal and vertical pressure components and , calculated according to 6.9.1 and 6.9.2 should be should be combined according to the rules of 4.5.

## Superposition of base shear, overturning moment and vertical reaction force

### Superposition of base shear

1. The superposition of the base shear components should be carried out by applying the SRSS-rule using Formula (6.121).

(6.121)

### Superposition of the overturning moments

1. The superposition of the overturning moments above and below the base plate should be carried out using Formulas (6.122) and (6.123), respectively.

(6.122)

(6.123)

## Verification to limit states

### General

1. It should be verified that the action effects in the design seismic situation do not exceed the corresponding resistances of the tank, the substructure and the foundation, including connections, relevant ancillary elements and connecting pipes for the specified limit states. The verification of structural members should comply with EN 1998-1-2:—3, 6.

### Verification of Significant Damage (SD) limit state

1. In application of EN 1998-1-1:—2, 6.2, the SD limit state may be considered as verified if the conditions in 6.10.2.1 to 6.10.2.5 in the design seismic situation are met.

#### Global verifications and requirements

1. In the seismic design situation at SD limit state, the relevant ultimate limit state verifications required in EN 1992-1-1:—8, Clause 6, EN 1992-3:2006, 6, EN 1993-1-6:—7, 4, EN 1993-1-7:—12, 4, and EN 1993-4-2:—21, 2, for tanks should be made.

#### Global stability

1. Tanks with or without substructure should be verified for overturning, uplifting and sliding for the total base shear, overturning moment and vertical reaction force. Limited sliding of the tank may be acceptable, if it is demonstrated that the implications of sliding for the connections between the various parts of the structure and between the structure and any piping or ancillary elements are considered in the analysis and the verifications.

NOTE The global stability refers to rigid body behaviour and may be impaired by sliding or overturning.

1. Uplift of anchored on-ground tanks should be avoided by sufficient anchorage according to 6.12.2.6.
2. The stability of unanchored on-ground tanks should be verified taking into account the increased seismic action effects as described given in 6.4.2 and 6.5.2.

#### Foundations

1. Tank foundation systems should be verified in accordance with EN 1998‑5:—4, Clause 9, for the supporting loads according to 6.11. The verification should include the verifications of the soil bearing capacity, slope stability, liquefaction and base sliding.

#### Tank shell

1. The seismic action effects in the tank shell (membrane forces and bending moments, circumferential or meridional, and membrane shear) should be calculated in the seismic design situation.
2. Steel tanks should be verified in the seismic design situation according to EN 1993-4-2:—21, 5 to 9.
3. Reinforced concrete and prestressed precast reinforced shells should be verified according to EN 1992-3 and EN 1992-1-1:—8.

#### Substructures of elevated tanks

1. Substructures of elevated tank should be verified in the seismic design situation according to the relevant clauses of EN 1998-1-2:—3.

#### Anchorage systems

1. Anchorage systems of tank structures to their foundation should be designed to remain elastic in the seismic design situation. The anchorage systems should be designed by applying the capacity design principle taking into account all relevant overstrength effects of the tank and its substructure. The overstrength factors in DC2 and DC3 should be applied according to the relevant parts of EN 1998-1-2:‑3. For all ductility classes, a minimum overstrength factor of 1,25 should be used.
2. The anchorage systems of tank structures to their foundation should be provided with sufficient ductility to avoid brittle failures in case the seismic loads are exceeded. Therefore, the strength associated to the steel anchorage plastic mechanism should be smaller than the resistance associated to brittle failure of concrete, also accounting for overstrength of the former. The design principles in accordance with EN 1998-1-1:—2, Annex G, should be applied.
3. Installed anchors should sustain the design loads in tension, shear and combined tension and shear to which they are subjected in the design seismic situation while providing adequate resistance to failure (ultimate limit state) according to EN 1998-1-1:—2, Annex G.

NOTE Fasteners of category C1 provides capacities only in term of resistances at ultimate limit state, while fasteners of category C2 provides capacities in terms of both resistances at ultimate and displacements at damage limitation state and ultimate state. The requirements for category C2 are more stringent compared to those for category C1. The performance category valid for a fastener is given in the corresponding European Technical Product Specification.

1. Pre-installed anchors should be designed to resist the effects of the seismic design loads according to EN 1993-1-8:—10. They should provide design resistance to tension due to uplift forces and bending moments where appropriate. One of the methods in a) to d) should be used to secure anchor bolts into the foundation:
2. a hook;
3. a washer plate;
4. some other appropriate load distributing member embedded in the concrete;
5. some other fixing which has been adequately tested and approved.
6. Structural connections between the tank structure and ancillary elements or connecting pipes should be designed to remain elastic in the seismic design situation. The connections should be capacity designed, taking into account all relevant overstrength effects of the components and connecting pipes.

#### Leak tightness, freeboard and hydraulic systems of the tank

1. In the seismic design situation at SD limit state, the tank shall satisfy the integrity requirement.
2. For the application of (1), a) to c) should be verified:
3. leak tightness or controlled leakage of the tank system;
4. adequate freeboard should be provided in the tank under the maximum vertical displacement of the liquid surface according to 6.4.1.5 and 6.5.1.5, in order to prevent damage to the roof due to the pressure of the sloshing liquid or, if the tank has no rigid roof, to prevent undesirable effects of spilling of the liquid;
5. the hydraulic systems (e.g. piping, pumps) which are part of, or are connected to the tank, should accommodate stresses and distortions due to relative displacements between tanks or between tanks and soil (EN 1998-1-1:—2, 5.2.2.4), without their functions being impaired. The loss of the content in the event of failure of any of its components should be prevented.

#### Inlets, outlets and ancillary elements

1. Inlets, outlets and further pipes should be verified to accommodate stresses and distortions due to relative displacements between tanks, between tanks and adjacent structures and between tanks and the foundation, without their functions being impaired.
2. If connected pipes induce too high local stresses in the thin walls of steel tanks flexible connections may be provided by the use of armoured hoses, compensators, swing arm joints or arrangement of flexible bend configurations.
3. In case of tanks with base isolation, the deformation compatibility of the structure and the ancillary elements connecting the tank to the ground or to adjacent structures should be verified.
4. Anchorages of ancillary elements (e.g. hydraulic systems, pumps) should be verified in the design seismic situation for the SD limit state according to 9.

NOTE The seismic verification of the ancillary elements burdens the producer and is not covered in this document.

### Verification of Damage Limitation (DL) limit state

1. The DL limit state may be considered as verified when the tank shell, the roof, the substructure, the anchorage, connecting pipes and connections of ancillary elements resist the seismic actions in the elastic range.

NOTE Additional verifications at DL limit state applied to tanks can be specified by a relevant authority or can be found in the National Annex.

#### Verification of Fully Operational (OP) limit state

1. It should be verified that strains (or generalised deformations such as drifts) resulting from the corresponding seismic design situation at OP limit state do not exceed values that are acceptable to maintain the function of the tank and associated equipment.
2. Criteria applicable to the tank and associated equipment, in addition to EN 1998-1-1:—2, 6.7.3(7), should be derived from the analysis of the components the operability of which is required as well as from the analysis of their supporting systems.

NOTE For a specific project, the relevant parties can specify all components of interest in the verification, together with a description of relevant damage states for each component and the associated requirements.

# Rules for above-ground pipelines

## Scope

1. Clause 7 gives rules for the structural analysis of continuous above-ground pipelines made of steel, unreinforced or reinforced concrete subjected to seismic actions including the effects of transient and permanent ground deformations.

NOTE Above-ground pipelines can be supported by foundations or elevated by substructures (e.g. pipeline bridges).

## Basis of design

### Design concept

1. For the application of Clause 7, pipeline systems made up of straight and bended sections and ancillary elements (e.g. valves, pumps, instrumentation) should be classified as given in a) or b):
2. single lines,
3. redundant networks.
4. A pipeline should be considered as a single line when its behaviour during and after a seismic event is not influenced by other pipelines, and if the consequences of its failure relate only to the functions demanded from it.
5. The effects of the seismic actions considered on above-ground pipelines should be seismic induced stresses or strains in the pipeline wall.
6. Above-ground pipelines should be designed for the dynamic structural response under the three components of the seismic actions. Differential displacements due to the structural response of different substructures along the pipeline should be considered.
7. The seismic design of above-ground pipelines should consider seismic induced permanent ground deformations given in a) to e):
8. fault crossings;
9. liquefaction induced phenomena;
10. slope stability;
11. landslides;
12. local soil settlements.
13. The design should consider the variability of ground motion due to wave passage, local site effects and incoherence.
14. The seismic design of above-ground pipelines should include ancillary elements such as valves, pumps or instrumentation and their connections to the pipeline.

NOTE The seismic resistances of such electromechanical ancillary elements are not defined in this document and can be provided by the operator of the facility or the manufacturer of the equipment.

1. The seismic design of above-ground pipelines should consider the influences of crossings to associated facilities due to connecting pipes and differing types of foundation.

NOTE Above-ground pipelines are usually connected to other subsystem of the supply infrastructure, such as pumping stations, operation centres, maintenance stations, etc., each of them housing different types of mechanical and electrical equipment. Explicit treatment of these subsystems, however, is not within the scope of this document. The seismic design of mechanical and electrical equipment requires additional specific criteria that are beyond the scope of EN 1998.

1. The principles of the seismic analysis procedures may also be applied to above-ground pipelines made of other materials (e.g. glass fibre-reinforced plastic/polymer (GFRP), high density polyethylene (HDPE) or polyethylene (PE)).

NOTE The definition of limit states and safety verifications for materials other than those in 7.1 (1) are not covered by this document.

1. Above ground pipelines should be designed in DC1 or DC2.
2. Substructures of above-ground pipelines may be designed according to ductility classes DC1, DC2 or DC3.

### Safety verification

1. Partial factors *γ*M,I should comply with EN 1998-1-2:—3.
2. Overstrength effects that may occur in the substructure of above-ground pipelines designed in DC2 or DC3 should be considered in verifications.

## Modelling and structural analysis

### Modelling

1. The dynamic calculation model of the pipeline including the foundation supports and substructures should represent the strength, the damping, the pipeline geometry, the stiffness and mass properties with explicit consideration of the aspects in a) to d), as appropriate:
2. foundation and soil stiffness;
3. mass of the fluid inside the pipeline;
4. dynamic characteristics of the substructures;
5. type of connection between pipeline and substructure.
6. Soil-structure interaction effects should be taken into account using EN 1998‑5:—4, Clause 8. Models for elevated pipelines should consider inertial and kinematic interaction effects. For pipelines with on-ground foundations only the foundation response should be taken into account.
7. Straight pipelines may be idealised by beam models with distributed masses. A pipeline may be considered as a straight pipeline when the radius of curvature is greater than 20 times the outer diameter of the pipelines as in EN 1594:2013, 7.2.1 and 7.2.2.
8. In curved section of pipelines where out of round and warping dominated behaviour can take part, special elbow elements or shell elements may be used to cover these stability problems.

### Structural analysis

#### Force-based approach

1. Above ground pipelines may be calculated with response spectrum analysis according to EN 1998-1-1:—2, 6.4.3 with dynamic calculation models specified in 7.3.1.

#### Displacement-based approach

1. Elevated above ground pipelines with supporting structures may be analysed by means of non-linear static analysis according to EN 1998-1-1:—2, 6.5.
2. A damping ratio *ξ* not greater than 2 % may be used for welded steel pipelines.
3. The analysis of permanent ground deformations may be carried out by static analysis with imposed displacements to the pipeline supports.

## Actions and combination of actions in the seismic design situation

1. The combination of the effects of seismic actions for the dynamic structural response of above-ground pipelines should be made in accordance with 4.5.
2. The design values of seismic actions *A*Ed should be calculated separately for seismic action effects due to dynamic structural response and permanent ground deformations (fault movements, liquefaction phenomena, slope instability, landslides and soil settlements).
3. The design values of the seismic actions *A*Ed for wave propagation and permanent ground deformations should be superimposed with the permanent actions due to self-weight and variable actions due to temperature, filling, operational pressure of the pipeline according to EN 1990:2023, 8.3.4.4. Existing variable actions (e.g. internal operational pressure) should be considered as permanent actions.
4. If the design values of the seismic actions *A*Ed occur simultaneously along of the pipeline, they should be superimposed in the seismic design situation.

## Behaviour factors

### Behaviour factor for the horizontal components of the seismic action

#### Above-ground pipelines

1. Above-ground pipelines made of welded steel should be designed in DC1 or DC2 by using behaviour factors depending on the radius to thickness ratio (*r*p*/t*p) as given in Table 7.1.

Table 7.1 — Behaviour factors for welded steel above-ground pipelines

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| **Structural ductility class** | ***r/t*** |  |  |  |  |
| **DC1** |  | 1,0 | 1,5 | 1,0 | 1,5 |
| **DC2** |  | 1,0 | 1,5 | 2,0 | 3,0 |
| **DC2** |  | 1,0 | 1,5 |  |  |

1. Reinforced concrete above-ground pipelines should be designed in DC1 with *q*R = 1,0, *q*D = 1,0, *q*S = 1,5.
2. Unreinforced concrete above-ground pipelines should be designed with *q*R = 1,0, *q*D = 1,0, *q*S = 1,0.

#### Substructures

1. The behaviour factors for DC1, DC2 and DC3 for supporting structures of above ground pipelines should be applied as given in the relevant parts of EN 1998-1-2:—3.

#### Behaviour factor for the vertical component of the seismic action

1. The behaviour factor *q*v should be applied as min(*q*, 1,5), where *q* is behaviour factor for the horizontal component of the seismic action applied to the pipeline as given in 7.5.1.1.

### Seismic loads

#### Seismic Analysis of above-ground pipelines with multiple supports on foundations

1. The seismic analysis of above ground pipelines on foundations shall take into account the spatial variability of the ground motions due to wave passage, local site effects and incoherence.

NOTE As pipelines are flexible systems with low masses, the effects of wave passage and incoherence are less important than local site effects (e.g. geological discontinuities, heterogenous soil).

1. If response spectrum analysis is applied, the spatial variability may be taken into account according to EN 1998-1-1:—2, 5.2.3.2, by applying local site-specific response spectra at every support along the pipeline.
2. If response-history analysis is applied, the spatial variability may be taken into account according to EN 1998-1-1:—2, 5.2.3.2, by applying input motions reflecting the local soil conditions along the pipeline at every support along the pipeline.
3. Alternatively, a response-spectrum or time history analysis with uniform excitation of all supports may be applied if the design quantities are increased with respect to the soil, geological and topographic conditions as defined in a) to c):
4. All design quantities may be increased by 10% for uniform soil conditions with shear wave velocities varying not more than 200 m/s without geological discontinuities, heterogenous soil conditions and significant topographic/basin effects.
5. All design quantities may be increased by 20% for of soil conditions with shear wave velocities varying more than 200 m/s without significant geological discontinuities, heterogenous soil conditions and topographic/basin effects.
6. All design quantities may be increased by 30% for soil conditions with shear wave velocities varying more than 200 m/s and geological discontinuities, heterogenous soil conditions, non-negligible topographic/basin effects. Additionally, a one-dimensional site response analysis is required.

NOTE The approach described in (4) is an approximation and less accurate than (2) and (3).

#### Seismic analysis of above ground pipelines with substructures

1. The seismic analysis of pipelines on bridge-like substructures (e.g. pipeline bridges) should be carried out using the modelling and analysis approaches for the substructure according to EN 1998-1-2:—3, 5.

#### Permanent ground deformations

##### Fault crossings

1. Above-ground pipelines at fault crossings should be analysed for differential displacements due to fault movements using calculation models given in 7.3.1.
2. The fault movements may be evaluated from site-specific studies.

NOTE Informative Annex E gives an alternative procedure to estimate fault movements.

##### Liquefaction induced phenomena

1. Liquefaction assessment in the area of the pipeline routing shall be carried out along the entire pipeline.
2. In case of liquefaction, the potential consequences given in a) to d) should be calculated according to EN 1998‑5:—4, 7.3.5.
3. exceedance of the load bearing capacity;
4. settlements of foundations;
5. instability of foundations;
6. lateral spreading.

##### Slope stability

1. The slope stability in the area of the pipeline routing should be calculated along the entire pipeline with the displacement-based approach according to EN 1998‑5:—4, 7.2.2.3, to calculate residual displacements for the pipeline design.
2. The residual displacements calculated in (1) should be considered in the structural analysis of the pipeline.

##### Landslides

1. Potential landslides in the area of the pipeline routing should be identified along the entire pipeline based on ground investigations according to EN 1998‑5:—4, 6.1.
2. The amplitudes of the permanent ground motions due to landslides may be calculated according to EN 1998-1-2:—3, 11.3.3, and considered in the structural analysis as imposed displacements to the foundations.

##### Soil settlements

1. Potential soil settlements and soil densification should be calculated under free-field conditions according to EN 1998‑5:—4, 9.4.2.1.4 along the entire pipeline.
2. The settlements calculated in (1) should be considered in the structural analysis of the pipeline as differential displacements to the foundations.

## Verification to limit states

### General

1. It should be verified that the action effects in the design seismic situation do not exceed the corresponding resistances of the pipeline, the substructure and the foundation, including connections and relevant ancillary elements for the specified limit states. The verification of structural members should comply with EN 1998-1-2:—3, Clause 6.

### Verification of Significant Damage (SD) limit state

#### General

1. In application of EN 1998-1-1:—2, 6.2, the SD limit state may be considered as verified if the conditions in 7.6.2.2 to7.6.2.6 are met in the seismic design situation.

#### Global Stability

1. Above-ground pipelines shall be verified for global overturning and sliding of the pipeline on the supports.

#### Pipeline

1. The seismic action effects in the pipeline wall (hoop and axial stresses) should be calculated in the seismic design situation as defined in 7.3.
2. Steel pipelines should be verified in the seismic design situation according to EN 1594:2013.
3. Unreinforced concrete and reinforced concrete pipelines should be verified according to EN 1992-1-1:—8 for the most unfavourable combination of axial strain and curvature in the seismic design situation.

#### Substructures of elevated pipelines

1. Substructures of elevated pipelines should be verified in the seismic design situation according to the relevant parts of EN 1998-1-2:—3.

#### Foundations

1. Shallow foundations of pipelines should be verified in accordance with EN 1998‑5:—4, 9.4.2.
2. Pile foundations of pipelines should be verified in accordance with EN 1998‑5:—4, 9.5.4.

#### Anchorage systems

1. Anchorage systems of above-ground pipelines should be designed to remain elastic in the seismic design situation. The anchorage systems should be designed by applying capacity design principle taking into account all relevant overstrength effects of the pipeline and its substructure. For DC2 and DC3, the overstrength factors should be applied according to the relevant clauses of EN 1998-1-2:—3. For all ductility classes, a minimum overstrength factor of 1,25 should be used.
2. The anchorage systems of above-ground pipelines to their foundations should be provided with sufficient ductility to avoid brittle failures in case the seismic loads are exceeded. Therefore, the strength associated to the steel anchorage plastic mechanism should be smaller than the resistance associated to brittle failure of concrete, also accounting for overstrength of the former. The design principles in accordance with EN 1998-1-1:—2, Annex G should be applied.
3. Structural connections between the pipeline and ancillary elements (e.g. valves, pumps or instrumentation) should be designed to remain elastic in the seismic design situation. The connections should be capacity designed, taking into account all relevant overstrength effects of the ancillary elements.
4. Anchorages of ancillary elements (e.g. valves, pumps or instrumentation) should be verified in the design seismic situation for the SD limit state according to 9.

NOTE The seismic verification of the ancillary elements burdens the producer and is not covered in this document.

1. It should be verified that above-ground pipeline systems satisfy the integrity requirements and maintain their supplying capability as a global servicing system.

### Verification of Damage Limitation (DL) limit state

1. The DL limit state may be considered as verified when the above-ground pipeline, the substructure, the anchorage systems and the connections of ancillary elements resist the seismic actions in the elastic range.

NOTE Additional verifications at DL limit state applied to above-ground pipelines can be specified by a relevant Authority or can be found in the National Annex.

#### Verification of Fully Operational (OP) limit state

1. It should be verified that deformations resulting from the corresponding seismic design situation do not exceed deformations that are acceptable to maintain the function of the pipeline and associated ancillary elements (e.g. valves, pumps or instrumentation).
2. Criteria applicable to the pipeline and associated equipment, in addition to EN 1998-1-1:—2, 6.7.3(3), should be derived from the analysis of the components the operability of which is required as well as from the analysis of their supporting systems.

# Rules for buried pipelines

## Scope

1. Clause 8 gives rules for seismic analysis and design of buried steel, unreinforced concrete, reinforced concrete and prestressed precast reinforced concrete pipeline systems subjected to seismic actions including the effects of transient and permanent ground deformations.

NOTE Buried pipelines are subjected to negligible inertia effects and extend over long distances contrary to building foundations.

1. Although large diameter pipelines are within the scope of this standard, the corresponding design criteria are not for use for apparently similar facilities, like tunnels and large underground cavities.

NOTE EN 1998‑5:—4, Clause 11, contains requirements for the design of underground structures.

## Basis of design

### Design concept

1. The effects of seismic action considered on buried pipelines should be seismic induced stresses or strains in the pipeline wall.

NOTE The greatest risk to buried pipelines is the potential for large permanent ground deformations as a result of fault crossing, liquefaction induced phenomena, landslides and slope instability.

1. Buried pipelines restrained by the surrounding soil should be designed for ground motion due to the seismic wave propagation.
2. The seismic design of buried pipelines should consider seismic induced permanent ground deformations given in a) to e):
3. fault crossings;
4. liquefaction induced phenomena;
5. lateral spreading and landslides;
6. slope stability;
7. local soil settlements.
8. The seismic design should consider the spatial variability of ground motion due to wave passage, local site effects and incoherence.
9. The seismic design of buried pipelines should consider the influences of crossings to associated subsystems (e.g. compressor stations) with connecting pipes, pipeline routing over single foundations and foundation slabs and transition areas to above-ground pipelines.

NOTE Buried pipelines are usually connected to other subsystem of the supply infrastructure, such as compressor stations, operation centres, maintenance stations, etc., each of them housing different types of mechanical and electrical equipment. Explicit treatment of these subsystems, however, is not within the scope of this document. The seismic design of mechanical and electrical equipment requires additional specific criteria that are beyond the scope of EN 1998.

1. The modelling and seismic structural analysis approaches in 8.3 may also be applied to buried pipelines made of other materials (e.g. Glass fibre-reinforced plastic/polymer (GFRP), High density polyethylene (HDPE) or Polyethylene (PE)).

NOTE 1 The definition of limit states and safety verifications for materials other than those in 8.1(1) are not covered by this document.

NOTE 2 The design of pipeline networks can involve additional reliability requirements and design approaches with respect to those provided in the present document.

1. The seismic design of a buried pipeline should include ancillary elements such as valves, tanks, pumps or instrumentation and their connections to the pipeline.

NOTE 1 The seismic resistances of such electromechanical ancillary elements are not defined in this document and can be provided by the operator of the facility or the manufacturer of the equipment.

NOTE 2 Buried pipeline systems and networks are usually cut into homogeneous sections along the pipeline way, because networks are often too extensive and complex to be treated as a whole. The definition of a homogeneous section is defined by the operator of the network. As an example, an urban water distribution system can be separated into a network serving street fire extinguishers and a second one serving private users. The separation would facilitate providing different reliability levels to the two systems. It is to be noted that the separation is related to functions and it is therefore not necessarily physical; two distinct networks can have several elements in common.

NOTE 3 Even though distinction can be made among different pipeline systems, like for instance single lines and redundant systems, for the sake of practicality a pipeline is considered here as a single line if its mechanical behaviour during and after the seismic event is not influenced by that of other pipelines, and if the consequences of its possible failure relate only to the functions demanded from it.

NOTE 4 Informative Annex C provides additional general design considerations for buried pipelines.

1. Separate networks may be identified within the overall network. The identification may result from the separation of the larger-scale part of the system (e.g. regional distribution) from the finer one (e.g. urban distribution), or from the distinction between separate functions accomplished by the same system.

### Safety verification

1. Partial factors *γ*M,i should comply with EN 1998-1-2:—3.

## Modelling and structural analysis

### Modelling

#### No-slippage straight buried pipeline sections

1. A pipeline may be considered as a straight pipeline when the radius of curvature is greater than 20 times the outer diameter of the pipeline as defined in EN 1594:2013, 7.2.1 and 7.2.2.
2. A simple analytical model may be used for a no-slippage straight pipeline to get an upper bound estimate of the strains in the pipeline for wave propagation.

NOTE This model assumes that the pipeline is flexible enough to follow without slippage nor interaction the deformation of the soil. By using this assumption, the strains in the direction of the pipeline due to the wave passage effect conform to the ground strain.

#### Buried pipelines with or without bends

1. Pipeline sections with or without bends may be modelled with appropriate beam elements to calculate axial stresses and strains. The interaction with the surrounding soil may be modelled by non-linear spring elements in the axial, transversal and vertical directions (Figure 8.1). Inertia effects may be neglected.

NOTE 1 Inertia effects are usually negligible for buried pipelines.

NOTE 2 Informative Annex D provides spring element characteristics to model pipe-soil interaction.

1. The non-linear spring elements should represent the changing soil properties along the pipeline routing through geological discontinuities, heterogeneous soil conditions and varying installation depths (e.g. street or railway passing’s) and consider abrupt changes of the pipeline stiffness (e.g. transition to sewer or casing pipes).
2. In curved section of pipelines where out of round and warping dominated behaviour can take part, special elbow elements may be used, which are able to cover these stability problems.

Figure 8.1 — Beam model with non-linear springs representing the surrounding soil

1. If only some parts of the pipeline require detailed modelling to capture out of round and warping effects, a hybrid model approach may by applied.

NOTE The hybrid model consists of beam elements for the straight sections and shell or volume elements for the curved sections of the pipeline. Beams and shells are coupled and connected to non-linear springs in the axial, transversal and vertical directions.

1. Alternatively, the pipeline and the surrounding soil may be modelled by a three-dimensional non-linear continuum model consisting of the pipeline connected by contact elements to a non-linear soil model.

### Structural analysis

1. No-slippage straight buried pipelines subjected to wave propagation may be analysed by strain analysis without considering soil structure interaction effects.
2. Buried pipelines with or without bends may be analysed by non-linear response-history analysis according to EN 1998-1-1:—2, 6.5 and 6.6.
3. Permanent ground deformations may be analysed with static analysis by imposing displacements to the non-linear pipeline model.

NOTE Force-based approaches are not provided for buried pipelines.

### Seismic loads

#### Wave propagation

##### General

1. The strain analysis should be based on the conservative assumption that the pipeline is flexible and forced to deform like the ground deformation without taking into account soil structure interaction effects. The soil motion is idealised as a sinusoidal wave with the wave amplitude *d*w, the apparent shear wave velocity *V*app, which may be assumed equal to 1000 m/s according to EN 1998‑5:—4, 11.2.2 (8), in the absence of site-specific studies, and the wave length **a as given in Formula (8.1).

(8.1)

1. The maximum strains and curvatures for the decisive angles  between the direction of wave propagation x’ and the pipeline axis *x* may be calculated according to EN 1998‑5:—4, Table H.2, if the apparent shear wave velocity is increased by a factor of 2 for the axial strains due to S-waves, and the curvature by a factor of 1,6 due to P-waves (Figure 8.2).

Figure 8.2 — Wave propagation along the pipeline

##### Strain analysis of straight buried pipelines

1. The maximum axial strain in the pipeline should be calculated as given in Formula (8.2).

(8.2)

where is the horizontal ground velocity at the depth of the pipeline as defined in EN 1998‑5:—4, 11.2.2.

1. The maximum curvature of the pipeline should be calculated using formula (8.3).

(8.3)

where is the horizontal ground acceleration at the depth of the pipeline as defined in EN 1998‑5:‑4, 11.2.2.

NOTE The strain and curvature given by (1) and (2) are the conservative estimates of strains in no-slippage straight buried pipelines.

1. The values for *P* and should be increased with respect to the soil and topographic conditions as defined in a) and b):
2. by 20% for soil conditions with shear wave velocities varying more than 200 m/s without significant geological discontinuities, heterogeneous soil conditions and topographic/basin effects;
3. by 30% for soil conditions with shear wave velocities varying more than 200 m/s and geological discontinuities, heterogeneous soil conditions, non-negligible topographic/basin effects.
4. When perfect adherence of the soil and the pipeline does not exist, the pipeline does not perfectly follow the soil movement as the soil slides around the pipeline. In this case, the maximum axial strain should be limited as given by Formula (8.4).

(8.4)

where

|  |  |
| --- | --- |
|  | is the wall thickness of the pipeline; |
|  | is the mean pipeline diameter; |
|  | is the Young’s modulus of the pipeline; |
|  | is the apparent wave length of the predominant seismic wave at the ground surface; |
|  | is the ultimate force per unit length at the soil-pipeline interface. |

NOTE Informative Annex D gives a procedure to calculate *T*u.

1. The apparent wave length may be estimated in the absence of more specific studies by the sum of the lower-corner and upper-corner periods *T*B, *T*C of the constant spectral acceleration range of the reference seismic action defined in EN 1998-1-1:—2, 5.2.2.2(1), using Formula (8.5).

(8.5)

##### Non-linear response-history analysis for buried pipelines with or without bends

1. The seismic ground motion should be represented by displacement time-histories derived by integration from accelerograms fulfilling the criteria of EN 1998-1-1:—2, 5.2.3.1, for the selection, evaluation and choice of the required number of sets of seismic ground motions. The seismic ground motion should always conform to the soil conditions with the lowest shear wave velocity according to EN 1998-1-1:—2, Table 5.1, along the pipeline section under consideration.
2. The seismic wave propagation (Figure 8.2) should be applied as displacement time-histories to the pipeline with the apparent shear wave velocity given in 8.3.3.1.1 (1). Each set of seismic ground motions should be simultaneously applied along the pipeline in all three directions considering the time offset to cover wave passage effects. As the incident angle of the seismic wave influences the seismic action effects in the pipeline section under consideration, different angles should be investigated to cover the directional dependence, especially in the areas of bends.

NOTE Due to the complex seismic behaviour of pipelines layouts with different pipeline bends connected by straight sections, changing soil conditions and installation depths it is usually not possible to calculate the decisive design quantities with only one unfavourable wave propagation direction.

1. The effects of the loss of coherency along the pipeline may be neglected for uniform soil conditions without geological discontinuities, heterogenous soil conditions, negligible topographic/basin effects and shear wave velocities varying not more than 200 m/s, as underground pipelines are flexible systems with relatively low masses and negligible inertia effects. The effects should be considered by increasing all design quantities in case of unfavourable soil and topographic conditions given in a) and b):
2. all design quantities may be increased by 20% for soil conditions with shear wave velocities varying more than 200 m/s without significant geological discontinuities, heterogeneous soil conditions and topographic/basin effects;
3. all design quantities may be increased by 30% for soil conditions with shear wave velocities varying more than 200 m/s and geological discontinuities, heterogeneous soil conditions, non-negligible topographic/basin effects. Additionally, a one-dimensional site response analysis is required.

#### Permanent ground deformations

##### Fault crossing for straight lines of buried pipelines

1. The fault-pipeline crossing may be characterised by the fault-pipeline crossing angle in the horizontal plane, , and the fault dip angle, (Figure 8.3).
2. It may be conservatively assumed that strike-slip faults only slip horizontally and dip-slip (i.e. normal or reverse) faults only slip vertically. The corresponding fault movements and (Figure 8.3) may be estimated by site-specific investigations.

NOTE Informative Annex E gives a procedure to evaluate fault movements if site-specific investigations are not carried out for this purpose.

Key

|  |  |
| --- | --- |
| *P* | pipeline |
| *FM* | fault movement |

Figure 8.3 — Pipeline at different fault movements: (a) plan-view of a strike-slip fault; (b) plan-view and elevation of a normal-slip fault

1. The components of fault movements in axial and transverse direction to the pipeline *∆* and due to the transverse (strike-slip) fault *∆* may be calculated using Formulas (8.6) and (8.7).

*∆* (8.6)

*∆* (8.7)

1. The components of fault movements in axial, transverse and vertical direction to the pipeline *∆*, *∆* and *∆* due to the normal slip *∆* may be calculated using Formulas (8.8) to (8.10).

*∆* (8.8)

*∆* (8.9)

*∆* (8.10)

1. The average axial strain in the pipeline due to the components of fault movements in axial and transverse direction of the pipeline may be calculated using Formula (8.11).

(8.11)

where *L*A is the effective unanchored length, measured as the distance between the fault trace and the anchor point (Figure 8.3).

1. If no bends, tie-ins or other constraints are located near the fault, the length *L*A may be calculated by Formula (8.12).

(8.12)

where

|  |  |
| --- | --- |
|  | is the yield strain; |
|  | is the Young’s modulus; |
|  | is the ultimate force per unit length at the soil-pipeline interface. |

NOTE Informative Annex D gives a procedure to calculate *T*u.

##### Fault crossing for buried pipelines with or without bends

1. Buried pipelines may also by analysed with the non-linear calculation model proposed in 8.3.1.2 by imposing the fault displacements *∆* and *∆* along the fault plane to the springs connected to the pipeline (Figure 8.3).

NOTE The analytical approach in 8.3.3.2.1 is less accurate than the application of the non-linear calculation model.

##### Liquefaction induced phenomena

1. Liquefaction assessment in the area of the pipeline routing shall be carried out along the entire buried pipeline according to EN 1998‑5:—4, 7.3.
2. In case of potential liquefaction, the consequences given in a) to c) should be evaluated with consideration of EN 1998‑5:—4, 7.3.5:
3. buoyancy;
4. lateral spreading;
5. local settlements.
6. The buoyant force per unit length of the pipeline, , in liquified soils may be calculated as given in (8.13).

(8.13)

where

|  |  |
| --- | --- |
|  | is the outer diameter of the pipeline; |
|  | total unit weight of the soil; |
|  | unit weight of the pipeline content; |
|  | unit weight of the pipeline material. |

##### Lateral spreading and landslides

1. Potential landslides and lateral spreading in the area of the pipeline routing shall be identified along the entire pipeline based on ground investigations according to EN 1998‑5:—4, 6.1, and rules for liquefaction assessment given in EN 1998‑5:—4, 7.3.
2. The amplitudes, lengths and widths of permanent ground motions due to landslides and lateral spreading may be calculated according to EN 1998‑5:—4, 11.3.3, if more site-specific investigations are not carried out.
3. The pipeline may be analysed for permanent ground displacements in the transverse and axial directions due to lateral spreading and landslides. Distributions of permanent ground motions in axial and transverse direction may be assumed as shown in Figure 8.4. are the width and length of the permanent ground deformation zones, and the corresponding maximum amplitudes.

Key

|  |  |
| --- | --- |
|  | width and length of the permanent ground deformation zones |
|  | corresponding maximum amplitudes |

Figure 8.4 — Pattern of permanent ground motions in transverse (a) and axial (b) direction

1. The maximum bending strain in pipelines due to permanent ground deformations transverse to the pipeline may be estimated as the minimum/maximum of the two cases given in a) and b):
2. The minimum/maximum bending strain in the pipeline for a large width of the ground deformation zone and a flexible pipeline may be estimated using Formula (8.14):

(8.14)

1. The minimum/maximum bending strain in the pipeline for a narrow width of the ground deformation zone and a stiff pipeline may be estimated using Formula (8.15):

(8.15)

where is the ultimate transverse force per unit length at the soil-pipeline interface.

NOTE may be calculated according to informative Annex D.

1. The maximum axial strain in pipelines due to permanent ground deformations in pipeline direction may be calculated with the block pattern as shown in Figure 8.4b and the embedment length , defined as the length over which the constant force must act to induces the ground strain to the pipeline, according to Formula (8.16) or (8.17).

(8.16)

(8.17)

##### Lateral spreading and landslides for buried pipelines with or without bends

1. Buried pipelines may also be analysed with the non-linear calculation model proposed in 8.3.1.2 by imposing the permanent ground deformation distributions as shown in Figure 8.4 in axial and transverse directions to the springs connected to the pipeline.

##### Slope stability

1. The slope stability in the area of the pipeline routing should be analysed along the entire pipeline with the displacement-based approach according to EN 1998‑5:—4, 7.2.2.3, to evaluate residual displacements for the pipeline design.
2. The residual displacements calculated in (1) should be considered in the structural analysis of the pipeline.

##### Soil settlements

1. Potential soil settlements and soil densification should be evaluated under free-field conditions according to EN 1998‑5:—4, 9.4.2.1.4, along the entire pipeline.
2. The settlements estimated in (1) should be considered in the structural analysis of the pipeline as differential displacements.

## Actions and combination of actions in the seismic design situation

1. The design values of seismic actions, *A*Ed, should be calculated separately for seismic action effects due to wave propagation and permanent ground deformations (soil settlements, fault movements, slope instability, landslides and liquefaction phenomena).
2. The design values of the seismic actions, *A*Ed, for wave propagation and permanent ground deformations should be superimposed with the permanent actions due to self-weight and soil covering and variable actions due to temperature, filling, operational pressure of the pipeline according to EN 1990:2023, 8.3.4.4. Existing variable actions (e.g. internal operational pressure) should be considered as permanent actions.

NOTE For buried steel pipelines, a list of pipeline specific actions is given in EN 1594:2013, 7.3.1.

1. If the design values of the seismic actions, *A*Ed, occur simultaneously along of the pipeline, they should be superimposed in the seismic design situation.

## Verification to limit states

### General

1. It should be verified that the action effects in the regarded seismic design situations do not exceed the corresponding resistances of the pipeline including relevant ancillary elements for the specified limit states.
2. Buried pipelines in stable and homogeneous soil may be checked only for the transient ground motions due to wave propagation.

### Verification of Significant Damage (SD) limit state

#### General

1. In application of EN 1998-1-1:—2, 6.2, the SD limit state may be considered as verified if the conditions in 8.5.2.2 and 8.5.2.3 are met in the seismic design situation.

#### Steel pipelines

1. Buried welded steel pipelines should be verified according for the most unfavourable combination of axial strain and curvature in the seismic design situation.
2. The local and global buckling safety should be verified for pipeline sections in compression. Special attention should be paid to bends and tees.
3. The requirements in (1) and (2) may be considered as satisfied if the following strains in a) to d) in are not exceeded:
4. tensile strain in straight-line pipelines: 3%;
5. tensile strain in bends or tees of pipelines: 1%;
6. compressive strain straight-line pipelines: min {1%; 0,4 *t*p/*D*p};
7. compressive strain in bends or tees of pipelines: 0,35 *t*p/*D*p;

where:

|  |  |
| --- | --- |
| *t*p | is the wall thickness of the pipeline; |
| *D*p | is the mean diameter of the pipeline. |

1. In welded steel pipelines, the ovaling as defined in Formula (8.18) should not be greater than 2,5 %.

(8.18)

where

|  |  |
| --- | --- |
|  | is the out-of-roundness value of the pipeline; |
|  | is the greatest outer diameter of the pipeline after its out-of-roundness; |
|  | is the smallest outer diameter of the pipeline after its out-of-roundness; |
|  | is the mean outer diameter of the pipeline before out-of-roundness. |

1. The welds of buried steel pipelines in areas prone to fault crossings should be controlled according to ISO 13847.

#### Unreinforced, reinforced, and precast prestressed concrete pipelines

1. Unreinforced concrete, reinforced concrete and prestressed precast reinforced concrete pipeline should be verified according to EN 1992-1-1:—8 for the most unfavourable combination of axial strain and curvature in the seismic design situation.

#### Integrity requirement and ancillary elements

1. It should be verified that buried pipeline systems satisfy the integrity requirements and maintain their supplying capability as a global servicing system.
2. Anchorages of ancillary elements (e.g. valves, pumps or instrumentation) should be verified in the design seismic situation for the SD limit state according to 9.

NOTE The seismic verification of the ancillary elements burdens the producer and is not covered in this document.

### Verification of Damage Limitation (DL) limit state

1. The DL limit state may be considered as verified when the buried pipeline resists the seismic design situation in the elastic range.

NOTE Additional verifications at DL limit state applied to buried pipelines can be specified by a relevant Authority or can be found in the National Annex.

#### Verification of Fully Operational (OP) limit state

1. It should be verified that deformations resulting from the corresponding seismic design situation do not exceed deformations that are acceptable to maintain the function of the pipeline and associated ancillary elements (e.g. equipment such as valves, pumps or instrumentation).
2. Criteria applicable to the pipeline and associated equipment, in addition to EN 1998-1-1:—2, 6.7.3(7), should be derived from the analysis of the components the operability of which is required as well as from the analysis of their supporting systems.

NOTE For a specific project, the relevant parties can specify all components of interest in the verification, together with a description of relevant damage states for each component and the associated requirements.

# Rules for ancillary elements in industrial facilities

## Scope

1. Clause 9 gives rules for the seismic design of ancillary elements (non-structural components) attached to structures in industrial facilities.

NOTE 1 Components employing isolators, viscous or friction dampers, or components that can respond by restrained or unrestrained sliding or rocking are not covered. For components that employ dampers for mechanical vibration only in the vertical direction, Clause 9 can be used for design under the horizontal excitations.

NOTE 2 Only components subject to seismic demand due to inertial forces or due to differential displacements between points of attachment are covered. Demands due to interaction with other independently attached components, as well as demands due to impact with the structure or other components are not covered.

NOTE 3 The functioning and the process interdependencies of ancillary elements in industrial facilities are not covered.

## Basis of design

### Design concept

1. The seismic design shall include ancillary elements, connections to the supporting structure and interactions with the supporting structure.
2. Any interactions due to impact among independently attached components as well as between a component and the supporting structure shall be eliminated by providing adequate clearance.

### Safety verification

1. Partial factors *γ*M,i should comply with EN 1998-1-2:—3.

## Modelling and structural analysis

### Modelling

1. The strength, mass and stiffness of the entire system of an ancillary element/component comprising the fasteners, secondary supports, and the component shall be accounted for in modelling.

NOTE 1 Ancillary elements/components are attached to the supporting structure via an anchorage system that can encompass secondary support elements (steel plates, steel angles, legs, suspension rods, cantilever beams, etc.), as well as the fasteners (bolts, screws, anchors, etc.).

NOTE 2 Single-attachment/support components under seismic loading are characterised by one or more supports with negligible differential displacement during seismic excitation, typically due to being anchored to one or more points of a single rigid diaphragm, or a rigid on-ground slab. In many cases, this means that such components are subject to uniform displacement of their supports relative to their resting position, but it does not preclude non-uniform displacement and acceleration of the supports e.g., due to rotation of the supporting slab.

NOTE 3 Multi-attachment/support components are characterised by multiple supports that can be subject to differential displacements during seismic excitation.

1. A model of the supporting structure(s) that incorporates the component should be used for the analysis of both the component and the structure(s) if one or more of clauses (a) to (c) holds:
2. Single/multi-support components that can cause dynamic interaction between the component and the supporting structure(s), influencing its global response, such as components that have non-negligible mass relative to the supporting structure(s);
3. Multi-support components that can influence the deformation of the supporting structure(s), such as components that have non-negligible stiffness relative to the stiffness supplied by the supporting structure(s) between their supports (e.g. between points of attachment at different stories/levels of a supporting frame, between the supporting structure and the ground, or between two or more structurally independent supporting structures);
4. Multi-support components that are sensitive to both the differential deformation of their supports and the vibration of the component.
5. For components for which (2) does not apply, a joint model of the supporting structural member(s) and the component should be used for the analysis of both the structural member(s) and the component if one or more of clauses (a) or (b) holds:
6. Single/multi-support components whose vibration response can locally influence the structural member(s) that they are attached to, such as single-support components having a centre of mass with offset relative to their support that, when vibrating, produce overturning moments to be resisted by the supporting slab or beam;
7. Single/multi-support components whose response can be influenced by the structural member(s) that they are attached to, such as components sensitive to vertical accelerations supported by flexible slabs or beams.
8. Single-support components for which clauses (2) and (3) do not apply may be represented by an appropriate calculation model of the component subject to the floor acceleration spectra produced by the analysis of the supporting structure. Unless better information is available from testing or literature, a component damping of *ξ*a = 2% should be used. Effects due to torsion and overturning should be considered.

NOTE Hydrodynamic forces from sloshing in liquid-storage tank installations can be ignored if their effect on the tank and its supports is insignificant, such as when the convective mode of the tank is well separated from the vibration modes of the supporting structure.

1. Multi-support components for which clauses (2) and (3) do not apply, may be represented by an elastic stiffness-only model that incorporates the stiffness contribution of the component, its internal parts and connections (piping, bends, etc.) and associated secondary support members and fasteners, subject to the differential support deformations produced by the analysis of the supporting structure.
2. Modelling of secondary support members and fasteners should consider EN 1998-1-1:—2,Annex G.

### Structural analysis

1. The analysis of the component shall account for the dynamic interaction of the component and the supporting structure(s).
2. Where 9.3.1(2) applies, modal response spectrum or time-history analysis should be used.
3. Where 9.3.1(3) applies, modal response spectrum or time-history analysis should be used. For single-support components whose vibrational behaviour is dominated by a single mode per each horizontal and the vertical direction, an equivalent static analysis employing spectra derived from the structure for the supports of the supporting member(s), such as the horizontal spectra of 9.3.3, may be applied.
4. Where 9.3.1(4) applies, modal response spectrum analysis of the single-support component may be applied. For single-support components whose vibrational behaviour is dominated by a single mode per each horizontal and the vertical direction, equivalent static analysis of the single-support component may be applied. Both types of analyses may be conducted by employing floor spectra derived from the structure, such as the horizontal spectra of 9.3.3.
5. Where 9.3.1(5) applies, equivalent static analysis of the multi-support component may be applied, employing the support deformations estimated from the analysis of the supporting structure.

### Seismic loads

1. 9.3.3 concerns the estimation of seismic loads only for single-support components to be analysed via modal response spectrum analysis or equivalent static analysis according to 9.3.2(3) and (4).
2. Seismic loads should be assessed for the two horizontal principal axes of the component. When there is ambiguity with respect to the orientation of the component relative to the structure, the component horizontal principal axes may be taken to coincide with the horizontal principal axes of the supporting structure. In that case, if the component has different periods in its two horizontal principal axes, then the most unfavourable orientation with respect to the supporting structure’s principal axes should be used.
3. Seismic loads should be assessed for the vertical direction for components sensitive to vertical accelerations.
4. The effects of the components of seismic action should be combined according to 4.5.

#### Non-dissipative design approach

1. Anchorage systems that do not rely on the formation of a ductile yield mechanism under design loads shall be designed with sufficient strength to remain elastic in the seismic design situation.
2. If the horizontal vibration periods of the component and the structure are known, the procedure of EN 1998-1-2:—3, 7.2, should be applied. In addition, the requirements of (3) should be used when determining , while the considerations of (4) may be used for determining and .
3. If the horizontal vibration period of the component and anchorage system is unknown, but the horizontal vibration period of the structure is known, then the seismic action effects to be applied to the component along each horizontal direction may be determined by applying to them a horizontal force *F*an using Formula (9.1).

(9.1)

where

|  |  |
| --- | --- |
| *F*an | is the horizontal seismic force, acting at the centre of mass of the component; |
| *m*an | is the mass of the component; |
| *S*an | is the value of horizontal floor acceleration spectrum that applies to the component, see (4); |
| *γ*an | is the performance factor of the component as given in EN 1998-1-2:—3, 7.2.2; |
|  | is the behaviour factor of the component equal to 1,35. If is evaluated for use with (2), then it should be taken according to EN 1998-1-2:—3, Annex C, and should not be larger than 1,5 for ancillary elements in industrial facilities. |

1. The value of the horizontal floor acceleration spectrum, *S*an, should be estimated with Formula (9.2).

*S*an = *AMP* × *PFA* (9.2)

where

|  |  |
| --- | --- |
| *AMP* | is an amplification factor equal to 7, unless there is verifiable data that the damping level of the ancillary element justifies a different value. |
| *PFA* | is the peak floor acceleration at the fundamental mode of the structure in the horizontal direction under investigation, calculated by Formula (9.3). |

(9.3)

where

|  |  |
| --- | --- |
|  | is the participation factor of the fundamental mode in the direction under consideration; a value of *Γ*1 = 1,5 may be adopted for most supporting structures, while *Γ*1 = 1,8 may be used if the supporting structures are tanks or silos: |
|  | is the elastic response spectra acceleration according to EN 1998-1-1:—2, 5.2.2.2, at the fundamental period of the supporting structure in the considered direction and the corresponding damping ratio ; when the supporting structure is an on-ground liquid storage tank, the impulsive vibrational period should be considered as the fundamental period; in all cases, the value of employed should not be lower than the elastic response spectra acceleration corresponding to 0.5sec. |
| *φ*1,an | is the fundamental mode shape amplitude at the height *z* corresponding to the component attachment to the supporting structure; a linear distribution over the total height *H* of the supporting structure*,* measured from the ground, may be taken as given in Formula (9.4). |

(9.4)

where

|  |  |
| --- | --- |
|  | is a period dependent behaviour factor derived from the behaviour factor component, , accounting for deformation capacity and energy dissipation capacity (EN 1998-1-1:—2, 6.4.1), which has been used for the design of the supporting structure; should be taken as given in (5); |
|  | is the maximum response spectral acceleration (for 5% damping) corresponding to the constant acceleration range of the elastic response spectrum defined in EN 1998-1-1:—2, 5.2.2.2(1); |
| *F*A | is the ratio of with respect to the zero-period spectral acceleration, as given in EN 1998-1-1:—2, 5.2.2.2(2). |

1. should be taken as given in (a) or (b);
   1. For structures for which it has been verified that their overstrength does not exceed by more than 20% the design overstrength of *q*R*·* *q*S according to EN 1998-1-1:—2, 6.4.1, should be taken as given by Formula (9.5).

(9.5)

where *T*A, *T*C are the corner periods defined in EN 1998-1-1:—2, 5.2.2.2(1).

* 1. For structures for which there is ambiguity about the value of or no verification of the actual overstrength has been undertaken, may be taken as given by Formula (9.6).

(9.6)

1. If the horizontal vibration periods of both the structure, and the component–anchorage system are unknown, (4) may be applied by assuming a period for the supporting structure that maximizes according to EN 1998-1-1:—2, 5.2.2.2.
2. If the displacement-based approach is used for the analysis of the structure, the ratio in Formula (9.3) should be replaced by the acceleration of the inelastic supporting structure determined by Formula (9.7).

(9.7)

where the equivalent force *F*y\* and the equivalent *m*\* of the equivalent Single Degree of Freedom model are given in EN 1998-1-1:—2, 6.5.3. In case of hardening effects after yielding, the force at yield should be used as *F*y\* instead of the force at the target displacement.

1. In case of ancillary elements with uniformly distributed mass, the resultant force *F*an may be distributed proportionally to their mass or their deformed shape, as this is affected by the boundary and connecting conditions to the structure.
2. For equipment sensitive to vertical motion, the supporting structure may be treated as rigid in the vertical direction. Then, the vertical elastic spectrum of EN 1998-1-1:—2, 5.2.2.3, may be used as the spectrum of motion at the point(s) of component attachment. If the component is mounted on vertically flexible supporting members, the flexibility should be considered within the calculation.

#### Dissipative design approach

1. 9.3.3.2 should be used for the design of anchorage systems that protect single-support components by undertaking inelastic deformations to reduce the forces and accelerations transmitted. This should be achieved by ensuring that the anchorage system has a guaranteed maximum yielding force and a guaranteed minimum ductility capacity in each direction of interest, without any additional unknown overstrength.
2. Yielding should be confined to a ductile part of the anchorage system acting as a fuse. For reasons of replaceability a secondary support member, rather than a fastener, may be used as the fuse.
3. The fuse strength (including strain hardening) should be low enough to ensure that the forces and accelerations transmitted to the component do not cause damage
4. The fuse should have a cyclic ductility capacity , verified by testing, where is at least 1,5 and *γ*an is the performance factor of the component as given in EN 1998-1-2:—3, 7.2.2.
5. All other members involved in the load path from the component to the supporting structure should have at least a 25% overstrength with respect to the fuse strength.
6. Given that (2) to (5) hold,the required strength of the fuse, as well as the force applied to the component and its anchorage system, in the direction of interest may be determined via 9.3.3.1(3) to (8), after replacing Formula (9.1) by Formula (9.8) and employing the amplification factor AMP of Formula (9.9).

(9.8)

(9.9)

NOTE The fuse does not require verification for forces, only for deformations regarding the exceedance of its ductility capacity. (6) is only meant to define a sufficient level of strength to ensure that this ductility capacity is not exceeded. Increasing the performance of the fuse is better served by increasing the ductility capacity by using (4) and estimating the required strength via Formula (9.8), and not by providing increased strength, as would imply in Formula (9.1). Increased strength would also mean higher forces transmitted to the component, the structure, and other elements of the anchorage system, which is not necessarily desirable per (1).

## Verification to limit states

### General

1. Ancillary components attached to containing structures, such as the shells of silos, tanks, and pipes, shall not cause damage to the supporting structure; damage shall be limited to the component and/or the anchorage system.
2. For components that are deemed to be critical for the safe operation and/or safe shutdown of the structure, damage shall be restricted to the anchorage system and shall be of such magnitude as to avoid impeding the operability of the component. Its operability shall also not be impeded by damage to other adjacent structural members or non-structural components.

NOTE The above requirements can be deemed to be satisfied if the performance factor of Table 4.1 is applied and adequate clearance is provided from all adjacent structural and non-structural components.

1. No verification is required for components where the supporting structures are subjected to where is the value for low seismicity class as given in EN 1998-1-1:—2, 4.1, in each of the two horizontal directions, and the components are sufficiently connected to the structure to resist such accelerations, while they also satisfy at least one of requirements (a) to (c) and both requirements (d) and (e):
2. the component mass is 200 kg or less with a centre of mass 1,25 m or less above or below the floor that it is connected to;
3. or the component mass is 10 kg or less;
4. or the component mass is 7,5 kg/m or less in the case of distributed mass components;
5. the component performance factor *γ*an is not larger than 1,0;
6. flexible connections are provided between the component and associated ductwork, piping, and conduits.

### Verification of Significant Damage (SD) limit state

1. The SD limit state should be verified for the component and its anchorage systems by applying the seismic actions according to 9.3.2. For fastener verification, EN 1998-1-1:—2, Annex G, should be used. Verification of secondary support members and the component itself should follow the relevant EN standards.

### Verification of Damage Limitation (DL) limit state

1. Where specified, the DL limit state should be verified for the component and its anchorage systems by applying the seismic actions according to 9.3.2. For fastener verification, EN 1998-1-1:—2, Annex G, should be used. Verification of secondary support members and the component itself should follow the relevant EN standards.

### Verification of Fully Operational (OP) limit state

1. Criteria applicable to the components, in addition to EN 1998-1-1:—2, 6.7.3 (7), should be derived from the analysis and/or testing of the components the operability of which is required.

NOTE For a specific project, the relevant parties can specify all components of interest in the verification, together with a description of relevant damage states for each component and the associated requirements.

# Rules for towers, masts and chimneys

## Scope

1. Clause 10 gives rules for the design of tall slender structures.
2. Clause 10 gives rules for steel towers, guyed masts and chimneys in addition to EN 1993-3:—[[9]](#footnote-10).
3. Clause 10 gives rules for reinforced concrete chimneys in addition to EN 1992-1-1:—8.

NOTE 1 Informative Annex G gives information and guidance for the seismic design of Masonry chimneys.

NOTE 2 The design of structures for electrical power transmission and distribution is typically controlled by wind loads, often combined with ice loads or by unbalanced longitudinal wire loads. The seismic design situation generally does not control their design, except when it includes high ice loads or heavy equipment. Earthquake damage is often due to large displacements of the foundations due to landslides, ground failure or liquefaction. The fundamental periods of electrical transmission towers typically range from 0,17 s to 2 s. Following frequency ranges can be used to determine whether earthquake loading is likely to control the structural design of the tower: single-pole types have fundamental mode periods in the 0,67 – 2,0 s range, H-frame structures in the 0,3 - 1 s ranges, four-legged lattice structures in the 0,17 – 0,5 s range, where lattice tangent structures typically have higher periods in this range and angle and dead-end structures have lower periods in the range. The present document does not cover this type of structures.

## Basis of Design

1. Clause 10 should not be applied to cooling towers and offshore structures.
2. For towers supporting tanks, 6.7 should be applied.
3. Reinforced concrete chimneys, steel chimneys and steel towers may be designed in DC1, DC2 or DC3 as described 10.4, 10.5 and 10.6.

## Modelling and structural analysis

#### Modelling

1. The model for analysis should consider a) to e):
2. the rotational and translational stiffness of the foundation;
3. the stiffness of cables and guys;
4. sufficient degrees of freedom (and the associated masses) to determine the response of any significant structural member, equipment or appendage;
5. relative displacements of the supports of equipment or machinery (for example, the interaction between an insulating layer and the exterior tube in a chimney);
6. piping interactions, externally applied structural restraints, hydrodynamic loads (both mass and stiffness effects, as appropriate).
7. Models of electric transmission lines should be representative of the entire line. As a minimum, at least three consecutive towers should be included in the model, so that the cable mass and stiffness is representative of the conditions for the central tower.
8. For the "rigid diaphragm" assumption to be applicable to steel towers, a horizontal bracing system should be provided, with adequate in-plane stiffness and spacing between bracing levels.
9. For the "rigid diaphragm" assumption to be applicable to steel chimneys, horizontal stiffening rings should be provided with adequate in-plane stiffness and spacing between stiffening rings.

NOTE Where increasing the in-plane stiffness of bracing planes or stiffening rings has a negligible influence on main member forces, the stiffness can be considered adequate. Where reducing the spacing between successive bracing planes or stiffening rings has a negligible influence on main member forces, the spacing can be considered adequate.

1. If the conditions for the applicability of the "rigid diaphragm" assumption are not met, a three-dimensional dynamic analysis should be performed, capable of capturing the distortion of the structure within horizontal planes.
2. Numerical modelling should take into account the mass and stiffness of the structural shell and of the liner.

NOTE When the liner (consisting of brick, steel, or other materials) is laterally supported by the chimney structural shell at closely spaced points such that the movement of the liner relative to the shell is considered negligible, the mass of the liner can be incorporated into that of the structural shell, without including separate degrees of freedom for the liner. When the supports of the chimney liner at the top of the chimney and possibly at intermediate points permit movement of the liner relative to the structural shell, the liner can be included in the dynamic analysis model separately (mass and stiffness) from the concrete structural shell.

#### Masses

1. The discretisation of masses in the model shall be representative of the distribution of inertial effects of the seismic action.
2. Where a lumped mass distribution of translational masses is used, rotational inertias should be assigned to the corresponding rotational degrees of freedom.
3. The masses should include all permanent parts, cables, guys, fittings, flues, insulation, any dust or ash adhering to the surface, present and future coatings, liners (including any relevant short- or long-term effects of liquids or moisture on the density of liners) and equipment.
4. The masses associated to variable actions imposed on the platforms and ice or snow loads should be taken into account by combination coefficients Ei according to EN 1998-1-1:—2, 6.2.1(3).
5. If the mass of the cable or guy is significant in relation to that of the tower or mast, the distribution of the mass along the length of the cable or guy should be modelled.

NOTE Considering the total number of cables or guys, their mass can be considered significant above 10% of the mass of the whole structure above foundations.

#### Stiffness

1. In concrete members, the stiffness properties should be calculated taking into account the effect of cracking in accordance with EN 1998-1-1:—2, 6.2.2(1) and (2). If design is based on the elastic response spectrum or a corresponding time-history representation of the ground motion, the stiffness of concrete members should be calculated from the cracked cross-section properties that are consistent with the level of stress under the seismic action.
2. The effect of the elevated temperature on the stiffness of the steel or of reinforced concrete, in steel or concrete chimneys, respectively, should be taken into account.
3. If a cable is modelled as a single spring for the entire cable, the stiffness of the single spring should account for the sag of the cable using the equivalent modulus of elasticity given by Formula (10.1).

(10.1)

where

|  |  |
| --- | --- |
|  | is the equivalent modulus of elasticity; |
|  | is the unit weight of the cable, including the weight of any ice load on the cable in the seismic design situation; |
|  | is the tensile stress in the cable; |
|  | is the horizontal projected cable length; |
|  | is the modulus of elasticity of the cable material. |

1. For application of (3) to strands consisting of wrapped ropes or wires, a value of lower than the modulus of elasticity *E* in a single chord should be considered. In the absence of specific data, the reduction given by Formula (10.2) may be taken.

(10.2)

where is the wrapping angle of the single chord.

NOTE This information can be provided by the manufacturer.

#### Damping

1. If the analysis is performed with the elastic response spectrum, damping ratios lower than 5% may be used, applying the correction defined in EN 1998-1-1:—2, 5.2.2.2(12).

### Structural analysis

1. The seismic action effects should be evaluated using the types of analysis given in 4.4(1).

NOTE Informative Annex F provides supplementary information and guidance on the number of degrees of freedom and the number of modes of vibration to be taken into account in the analysis.

### Behaviour factors

1. The maximum value for the behaviour factor *q* should be calculated according to EN 1998-1-1:—2, 6.4.1(1), with *q*R = 1,0, *q*S = 1,5 and *q*D defined in 10.5, 10.6 and 10.7, respectively for reinforced concrete chimneys, steel chimneys and steel towers.
2. The behaviour factor *q* should be reduced by the modification factor *k*r, reflecting irregular distribution of mass, stiffness or strength (Table 10.1). When more than one of the irregularities are present, *k*r should be assumed to be equal to the lowest values of *k*r multiplied by 0,9.

Table 10.1 – *k*r factors according to existing irregularities

|  |  |
| --- | --- |
| **Description** | ***k*r** |
| Horizontal eccentricity (*e*L) of the centre of mass (*G*) at a horizontal level with respect to the centroid of the stiffness (*C*) of the (vertical) members at that level, exceeding 5 % of the parallel horizontal dimension (*L*) of the structure at same level | 0,8 |
| Openings in the shaft or structural shell causing a 30 % or larger reduction of the moment of inertia of the cross-section | 0,8 |
| Concentrated mass within the top third of the height of the structure, contributing by 50 % or more to the overturning moment at the base | 0,7 |

### Behaviour factors for systems with base isolation or energy dissipation systems

1. If seismic protection is provided through energy dissipation systems, EN 1998-1-1:—2, 6.8, should be applied.
2. If seismic protection is provided through energy dissipation systems, specific or additional rules for the design provided in EN 1998-1-2:—3, Clauses 8 and 9, should be applied.

## Verification to limit states

### Verification of Significant Damage (SD) limit state

#### General

1. The verification of structural members and the structure as a whole should comply with EN 1998-1-1:—2, 6.7.2, the seismic action defined in 4.3(6) and with the conditions in 10.4.1.2 to 10.4.1.7.
2. Structural members and the structure as a whole should possess a capacity larger than the demand obtained from the analysis according to 10.2.2.
3. (2) may be considered satisfied through one of the design approaches in a) or b):
4. design the structure for dissipative behaviour using a value of the behaviour factor *q* greater to 1,5 and applying 10.5, 10.6 or 10.7 for energy dissipation capacity of the different types of structures;
5. design the structure for low-dissipative behaviour in DC1 using a value of the behaviour factor *q* equal to 1,5.
6. Guyed masts should be designed using approach b) in (3).

#### Second-order effects

1. Second-order theory should be considered in the analysis, unless the condition in Formula (10.3) is fulfilled.

δ*M*/*M*o < 0,10 (10.3)

where

|  |  |
| --- | --- |
| δ*M* | is the additional overturning moment at the base level due to second-order (*P-Δ*) effect, where *P* is the vertical load in the seismic design situation and *Δ* is the displacement corresponding to the moment *M*0 calculated with the first-order theory (δ*M = P*Δ); |
| *M*o | is the first-order overturning moment at the base level. |

NOTE For steel structures see also EN 1993-3:—9.

#### Steel Connections

1. For welded or bolted non-dissipative connections, the resistance should be determined according to EN 1993-1-8:—[[10]](#footnote-11) and EN 1993-3:—9.
2. The resistance to be provided for welded or bolted dissipative connections should be greater than the plastic resistance of the connected dissipative member based on the design yield stress of the material as defined in EN 1993-1-1, taking into account the randomness material factor according to EN 1998-1-2:—3, 11.2.2.
3. EN 1993-1-8:—10 should be applied for bolts and welding consumables.
4. Non-dissipative connections of dissipative members made by means of full penetration butt welds may be considered to satisfy (2).

#### Stability

1. The global stability of the structure in the seismic design situation should be verified, taking into account the effect of piping interaction and of hydrodynamic loads, where relevant for the seismic design situation.
2. The stability of structural members may be considered to be verified if the rules relevant to stability verification in EN 1992-1-1:—8, EN 1993-1-1, EN 1993-1-5:—[[11]](#footnote-12), EN 1993-1-6:—7, EN 1993-1-7:—[[12]](#footnote-13) and EN 1993-3:—9 are fulfilled.
3. In structural steel members, class 4 sections may be used, provided that all conditions in a) to c) are met:
4. the specific rules concerning classification of cross-sections in EN 1993-1-1 are fulfilled;
5. the value of the behaviour factor, *q*, is limited to 1,5;
6. the slenderness **s**is not greater than:

* 120 in leg members;
* 180 in seismic primary bracing members;
* 250 in seismic secondary bracing members.

#### Foundations

1. Foundation design should conform to EN 1998‑5:—4.

#### Guys and fittings

1. For ropes, strands, wires and fittings, EN 1993-1-11[[13]](#footnote-14) should be applied.

#### Thermal effects

1. In the seismic design situation, thermal effects of structural member temperatures less than 100ºC may be neglected.
2. Except when (1) is applied, the thermal effects of the normal operating temperature on the mechanical properties of the structural members, such as the elastic modulus and the yield stress, should be taken into account according to EN 1992-1-2:—[[14]](#footnote-15), EN 1993-1-2:—[[15]](#footnote-16) and EN 1994-1-2:—[[16]](#footnote-17).

NOTE For free-standing steel chimneys, see EN 13084-7.

### Verification of Damage Limitation (DL) limit state

#### General

1. Damage considered unacceptable to the structure itself and to ancillary elements should be prevented and calculated in accordance with the seismic action defined in 4.3(6).

NOTE Specific objectives and rules can be agreed for a specific project by the relevant parties. In particular, if the operation of the structure is considered to be sensitive to deformations (for example in telecommunication towers, where deformation could lead to permanent damage of equipment or loss of the signal), reduced limits to displacements can be defined.

1. When using a force-based approach to determine the seismic action effects, displacements for the damage limitation requirement should be obtained in accordance with EN 1998-1-1:—2, 6.4.2(2).

#### Damage limitation criteria for chimneys

1. Waste gas flues in chimneys should be verified for imposed deformations between support points and clearances between internal elements, so that gas tightness is not lost and sufficient reserve is maintained against collapse of the flue gas tube, under the displacements calculated in accordance with 10.4.2.2(3).
2. The requirement for damage limitation in (1) may be considered satisfied if the lateral displacement of the top of the structure, calculated in accordance with 10.4.2.2(3), does not exceed 0,5% of the height of the structure.
3. The relative deflection between different points of support of the liner *d*r, calculated according to 10.4.2.1(2), should be limited for damage limitation of the liner. The relative lateral displacements of adjacent points of support of the liner should be limited as given in a) or b):
4. if provisions are taken to allow relative movement between separate parts of the liner (e.g. by constructing the liner of tubes to be independent from each other, with suitable clearance), the limit should be taken as given by Formula (10.4).

*d*r ≤ 0,020 Δ*H* (10.4)

where Δ*H* is the vertical distance of adjacent platforms supporting the liner.

1. in all other cases, the limit should be taken as given by Formula (10.5).

*d*r ≤ 0,012 Δ*H* (10.5)

NOTE Stricter limits can be agreed for a specific project by the relevant parties.

#### Damage limitation criteria for towers and masts

1. The requirement for damage limitation may be considered satisfied if the lateral displacement of the top of the structure, calculated according to 10.4.2.1 (2), does not exceed 0,5% of the height of the structure.

NOTE 1 Stricter limit can be agreed for a specific project by the relevant parties.

NOTE 2 For masts, a limit on the relative displacements between horizontal stiffening elements can be agreed for a specific project by the relevant parties, depending on the mast function.

### Verification of Fully Operational (OP) limit state

1. Criteria applicable to the structure and associated equipment, in addition to EN 1998-1-1:—2, 6.7.3(7), should be derived from the analysis of the components the operability of which is required as well as from the analysis of their supporting systems.

NOTE For a specific project, the relevant parties can specify all components of interest in the verification, together with a description of relevant damage states for each component and the associated requirements.

## Specific rules for reinforced concrete chimneys

### General

1. 10.5 should be applied to concrete chimneys of annular (hollow circular) cross-sectionas a complement to EN 1992-1-1:—8 and EN 1992-1-2:—14.
2. For free-standing concrete chimneys, EN 13084-2 should be applied, unless otherwise specified in the present standard.
3. Concrete class should be of a class not lower than C20/25, as defined in EN 1992-1-1:—8.

NOTE EN 1992-1-1:—8 can impose higher minimum class according to the exposure class.

### Design for dissipative behaviour

1. Concrete chimneys should be designed with a value of the behaviour factor component not higher than *q*D = 1,3, by applying 10.5.2 within the critical region defined in (2).
2. The critical region should be taken in the cases a) to c), where *D* is the outer diameter of the chimney at the middle of the critical region:
3. from the base of the chimney to a height *D* above the base;
4. from an abrupt change of section to a height *D* above the abrupt change of section;
5. a height *D* above and below sections of chimney where more than one opening exists, where limits are defined by the openings’ edges.
6. A minimum value of the confining reinforcement should be provided, in accordance with the provisions defined for DC3 in EN 1998-1-2:—3, 10.6.3.2 (7) to (9).
7. To avoid implosive spalling of the concrete at the inner surface the value of the ratio of the outer diameter, as defined in (2), to the thickness of the section wall, should not exceed 20.
8. Horizontal construction joints within the critical regions should not be used.
9. Provisions for lap and lap-splicing given in EN 1998-1-2:—3, 10.11.3 (1) to (3), should be applied within the critical regions.

### Minimum reinforcement (vertical and horizontal)

1. In chimneys with an outer diameter, *D*, of 4 m or more, the vertical and the horizontal reinforcement should be placed in two layers (curtains) each: one layer per direction near the inner and the other layer near the outer surface, with not less than half of the total vertical reinforcement placed in the layer near the outer face.
2. In chimneys with an outer diameter of 4 m or more, the minimum ratio of the horizontal reinforcement to the cross-sectional area should be not less than 0,0025. The inner layer should contain not less than one third of the total horizontal reinforcement.
3. In chimneys with an outer diameter of less than 4 m, the entire vertical or horizontal reinforcement may be placed in a single layer (curtain) per direction, near the outer surface. In that case, the ratio of the reinforcement in the outer layer to the cross-sectional area should not be less than 0,003 per direction.
4. Along one third of the height starting from the top of the chimney, where stresses due to the permanent loads are low, the minimum vertical reinforcement ratio may be taken equal to that of the horizontal reinforcement.
5. The spacing of vertical bars should be not more than 250 mm and that of horizontal bars should not be more than 200 mm.
6. The horizontal reinforcement bars should be placed between the vertical bars and the concrete surface. Cross-ties between the outer and the inner layer of reinforcement should be provided at a horizontal and vertical spacing of not more than 600 mm.

### Minimum reinforcement around openings

1. Around the perimeter and the corners of openings, reinforcement should be placed additional to that provided away from the openings. The additional reinforcement should be placed as near to the outside surface of the opening as normal constructional considerations and geometry of the openings permit. The bars should extend past the opening perimeter for a full anchorage length.
2. The area of the additional horizontal and vertical reinforcement in each direction should not be less than that of the bars which are discontinued due to the presence of the opening. Over a horizontal distance from either vertical side of the opening of half the opening width, the vertical reinforcement ratio should not be less than 0,0075.

## Specific rules for steel chimneys

### General

1. 10.6 should be applied to steel chimneys of annular (hollow circular) cross-section, as a complement to EN 1993-3:—9.
2. In the verification of a chimney in the seismic design situation, a corrosion allowance on thickness should be taken into account in accordance with EN 1993-3:—9, 6.3.
3. Weakening of cross-section by cut-outs or openings (manholes, flue inlet) should be compensated for by local reinforcement of the structural shell, taking into account local stability considerations according to EN 1993-3:—9, 6.2.1.

NOTE This reinforcement can be made through stiffeners around the edges of the openings.

1. In the design of details, such as flanged connections according to EN 1993-1-8:—10, the plastic stress distribution should be taken into account.

### Design for dissipative behaviour

1. Steel frame or truss structures which provide lateral support to flue gas ducts of chimneys may be designed for dissipative behaviour using behaviour factors according to EN 1998-1-2:—3, 11.4.2.

NOTE 1 Guyed steel chimneys are generally lightweight. As such, their design for lateral actions is usually governed by wind, unless they have large flares or other masses near the top.

NOTE 2 For typical steel chimneys, seismic loads are usually not governing the design. However, combined cycle power plants chimneys with large diameters (e.g. 8 m) and sizeable openings for inflowing gasses (up to 25 m high) in moderate to high seismicity class, seismic loads can govern the design. In this case, no plastic hinge can safely develop at the base of such chimneys without local buckling occurring first.

### Materials

1. Structural steel should conform to EN 1993-1-1:2022, 5.2, and EN 1993-3:—9, 3.1.
2. The thickness of steel members should conform to EN 1993-1-10:—[[17]](#footnote-18), Table 4.1, depending on the Charpy V-Notch (CVN) energy and other relevant parameters, and to EN 1993-3:—9.
3. The mechanical properties of structural carbon steels S 235, S 275, S 355, S 420, S 460 should be taken from EN 1993-1-1 and, for properties at higher temperatures, from EN 13084-7.
4. Mechanical properties related to stainless steels should be taken from EN 1993-1-4:—[[18]](#footnote-19) for temperature up to 400ºC and from EN 13084-7 at higher temperatures.

### Connections

1. For connection materials, welding consumables reference should be made to EN 1993-1-8:—10 and the relevant product standards specified therein.

## Specific rules for steel towers

### General

1. 10.7 should be applied to steel towers designed for dissipative behaviour, in complement to the relevant parts of EN 1993, including EN 1993-1-1 and EN 1993-3:—9.

### Materials

1. 10.6.3(1) and (2) should be applied.
2. EN 1998-1-2:—3, 11.3, should be applied.
3. The thickness of cold-formed members for towers should not be lower than 3 mm.

### Design for dissipative behaviour

1. Design of steel towers using moment resisting frames or eccentric braced frames should be designed in accordance with EN 1998-1-2:—3, 11.
2. The design of frames with concentric bracings should be in accordance with EN 1998-1-2:—3, 11, and applying 10.6.3.1.

NOTE Figure 10.1 shows typical configurations of steel towers with concentric bracings.

1. If trussed tubes are used in the major diagonals of the tower, the value of the component behaviour factor *q*D should not be greater than 1,3.

#### Design of towers with concentric bracings

1. In the frames in Figure 10.1(a) to (e) and (h), both the tension and compression diagonals should be taken into account in an elastic analysis of the structure for the seismic action.
2. The frames in Figure 10.1(a) to (c) belong to K types of bracings and should be designed in DC1 according to 10.4.1.1(3)b).
3. The frames in Figure 10.1(d) and (h) may be considered similar to V-braced frames with diagonals intersecting on a continuous horizontal member. Design for dissipative behaviour should be in accordance with EN 1998-1-2:—3, 11.10, pertaining to frames with V bracings.
4. For the frame in Figure 10.1(e), design for dissipative behaviour should be in accordance with EN 1998-1-2:—3, 11.10, pertaining to frames with diagonal bracings in which the diagonals are not positioned as X diagonal bracings.
5. The X-braced frames in Figure 10.1(f) and (g) may be considered as frames with X diagonal bracings. In design for dissipative behaviour only, the tension diagonals should be taken into account in linear analysis of the structure for the seismic action. Such design should be in accordance with EN 1998-1-2:‑3, 11.10, pertaining to frames with X diagonal bracings.
6. A horizontal bracing system capable of assuring the required rigid diaphragm action according to 10.3.1.1(3) should be provided for systems designed with *q* greater or equal to 3,5.

NOTE Fully triangulated horizontal bracing systems are exemplified in Figure 10.2.

Figure 10.1 — Configurations of steel frames with concentric bracings

Figure 10.2 — Fully triangulated horizontal bracings, used in towers designed in DC2, DC3

#### Specific rules for the design of electrical transmission towers

1. The design should take into account the adverse effects on the tower of the cables between adjacent towers.
2. The requirement in (1) is considered satisfied if the seismic action effects in the tower structure are calculated by a simple addition of effects in a) and b) (SRSS or similar combination rules should not be used):
3. The seismic action effects due to the forces exerted on the tower by the cables, assuming that the tower moves statically with respect to the adjacent ones in the most adverse direction. The assumed relative displacement should be equal to twice the design ground displacement specified in EN 1998-1-1:—2, 5.2.2.4.(2). A set of all physically possible relative displacements between towers should be analysed, under the assumption that towers are fixed at their base;
4. The seismic action effects due to the inertia loads from a dynamic analysis according to 10.3.1. In the three towers model, a limiting assumption may be made for the two adjacent towers, if these are tangent towers; in this case, inertia loads may be calculated assuming the adjacent tower is elastically supported at the cable level along the direction of the cables.

### Other design rules

1. "Telescope joints" may only be used in tubular steel towers (poles), if they are experimentally qualified.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

1. Joints in towers should be designed and detailed according to EN 1998-1-2:—3, 11, for joints in structural systems of similar type and configuration, designed for the same value of the behaviour factor, *q*, as the tower.
2. (normative)  
     
   Tables for the seismic design of tanks
   1. Use of this normative annex
3. This normative annex contains tables of dimensionless pressure functions and coefficients for the application of Clause 6.
4. The tables should be used with the following variables:

|  |  |
| --- | --- |
| *H* | is the filling height; |
| *R* | is the radius; |
|  | is the dimensionless height; |
|  | is the ratio of filling height to tank radius; |
| *s*w | uniform thickness of the tank wall or the average in case of stepwise wall thickness. |

1. Intermediate parameter values of the tables may be derived by linear interpolation.
   1. Tables of parameter values

Table A.1 — Dimensionless function for the convective pressure component considering the fundamental natural first mode for sloshing

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
| 1,00 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 | 0,8371 |
| 0,95 | 0,8318 | 0,8183 | 0,8012 | 0,7838 | 0,7672 | 0,7300 | 0,6965 | 0,6650 | 0,6351 | 0,6065 | 0,5792 | 0,5283 | 0,4818 | 0,4395 | 0,4008 | 0,3656 | 0,3334 |
| 0,90 | 0,8268 | 0,8007 | 0,7679 | 0,7348 | 0,7039 | 0,6369 | 0,5796 | 0,5284 | 0,4819 | 0,4395 | 0,4008 | 0,3334 | 0,2774 | 0,2307 | 0,1919 | 0,1597 | 0,1328 |
| 0,85 | 0,8220 | 0,7841 | 0,7368 | 0,6898 | 0,6466 | 0,5560 | 0,4825 | 0,4198 | 0,3656 | 0,3184 | 0,2774 | 0,2104 | 0,1597 | 0,1211 | 0,0919 | 0,0697 | 0,0529 |
| 0,80 | 0,8176 | 0,7686 | 0,7080 | 0,6485 | 0,5947 | 0,4857 | 0,4017 | 0,3336 | 0,2774 | 0,2307 | 0,1919 | 0,1328 | 0,0919 | 0,0636 | 0,0440 | 0,0305 | 0,0211 |
| 0,75 | 0,8134 | 0,7541 | 0,6814 | 0,6107 | 0,5479 | 0,4247 | 0,3345 | 0,2651 | 0,2105 | 0,1672 | 0,1328 | 0,0838 | 0,0529 | 0,0334 | 0,0211 | 0,0133 | 0,0084 |
| 0,70 | 0,8095 | 0,7407 | 0,6569 | 0,5763 | 0,5057 | 0,3717 | 0,2788 | 0,2107 | 0,1597 | 0,1211 | 0,0919 | 0,0529 | 0,0305 | 0,0175 | 0,0101 | 0,0058 | 0,0033 |
| 0,65 | 0,8059 | 0,7283 | 0,6343 | 0,5449 | 0,4678 | 0,3259 | 0,2325 | 0,1676 | 0,1212 | 0,0878 | 0,0636 | 0,0334 | 0,0175 | 0,0092 | 0,0048 | 0,0025 | 0,0013 |
| 0,60 | 0,8026 | 0,7168 | 0,6137 | 0,5166 | 0,4339 | 0,2863 | 0,1941 | 0,1333 | 0,0920 | 0,0636 | 0,0440 | 0,0211 | 0,0101 | 0,0048 | 0,0023 | 0,0011 | 0,0005 |
| 0,55 | 0,7995 | 0,7063 | 0,5950 | 0,4910 | 0,4037 | 0,2522 | 0,1623 | 0,1062 | 0,0699 | 0,0461 | 0,0305 | 0,0133 | 0,0058 | 0,0025 | 0,0011 | 0,0005 | 0,0002 |
| 0,50 | 0,7967 | 0,6968 | 0,5780 | 0,4681 | 0,3768 | 0,2228 | 0,1361 | 0,0847 | 0,0531 | 0,0334 | 0,0211 | 0,0084 | 0,0033 | 0,0013 | 0,0005 | 0,0002 | 0,0001 |
| 0,45 | 0,7941 | 0,6883 | 0,5629 | 0,4477 | 0,3532 | 0,1978 | 0,1144 | 0,0676 | 0,0404 | 0,0243 | 0,0146 | 0,0053 | 0,0019 | 0,0007 | 0,0003 | 0,0001 | 0 |
| 0,40 | 0,7919 | 0,6806 | 0,5494 | 0,4298 | 0,3326 | 0,1765 | 0,0967 | 0,0542 | 0,0308 | 0,0176 | 0,0101 | 0,0033 | 0,0011 | 0,0004 | 0,0001 | 0 | 0 |
| 0,35 | 0,7899 | 0,6739 | 0,5377 | 0,4142 | 0,3148 | 0,1586 | 0,0822 | 0,0437 | 0,0236 | 0,0128 | 0,0070 | 0,0021 | 0,0006 | 0,0002 | 0,0001 | 0 | 0 |
| 0,30 | 0,7882 | 0,6681 | 0,5275 | 0,4009 | 0,2996 | 0,1437 | 0,0705 | 0,0355 | 0,0182 | 0,0094 | 0,0049 | 0,0013 | 0,0004 | 0,0001 | 0 | 0 | 0 |
| 0,25 | 0,7867 | 0,6632 | 0,5190 | 0,3897 | 0,2870 | 0,1315 | 0,0613 | 0,0292 | 0,0141 | 0,0069 | 0,0034 | 0,0008 | 0,0002 | 0,0001 | 0 | 0 | 0 |
| 0,20 | 0,7855 | 0,6592 | 0,5121 | 0,3806 | 0,2769 | 0,1219 | 0,0541 | 0,0244 | 0,0112 | 0,0052 | 0,0024 | 0,0005 | 0,0001 | 0 | 0 | 0 | 0 |
| 0,15 | 0,7846 | 0,6561 | 0,5067 | 0,3736 | 0,2690 | 0,1146 | 0,0487 | 0,0209 | 0,0091 | 0,0040 | 0,0018 | 0,0004 | 0,0001 | 0 | 0 | 0 | 0 |
| 0,10 | 0,7839 | 0,6539 | 0,5029 | 0,3686 | 0,2635 | 0,1094 | 0,0450 | 0,0186 | 0,0077 | 0,0032 | 0,0014 | 0,0002 | 0 | 0 | 0 | 0 | 0 |
| 0,05 | 0,7835 | 0,6526 | 0,5006 | 0,3657 | 0,2602 | 0,1064 | 0,0428 | 0,0172 | 0,0069 | 0,0028 | 0,0011 | 0,0002 | 0 | 0 | 0 | 0 | 0 |
| 0 | 0,7834 | 0,6521 | 0,4998 | 0,3647 | 0,2591 | 0,1054 | 0,0421 | 0,0168 | 0,0067 | 0,0027 | 0,0011 | 0,0002 | 0 | 0 | 0 | 0 | 0 |

Table A.2 — Dimensionless function for the impulsive rigid pressure component due to horizontal seismic excitation

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
| 1,00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0,95 | 0,0277 | 0,0569 | 0,0864 | 0,1138 | 0,1390 | 0,1934 | 0,2401 | 0,2819 | 0,3202 | 0,3556 | 0,3885 | 0,4481 | 0,5007 | 0,5475 | 0,5893 | 0,6270 | 0,6609 |
| 0,90 | 0,0469 | 0,0965 | 0,1458 | 0,1916 | 0,2327 | 0,3184 | 0,3882 | 0,4481 | 0,5008 | 0,5476 | 0,5895 | 0,6611 | 0,7195 | 0,7674 | 0,8070 | 0,8397 | 0,8667 |
| 0,85 | 0,0625 | 0,1289 | 0,1948 | 0,2553 | 0,3088 | 0,4166 | 0,5002 | 0,5691 | 0,6272 | 0,6769 | 0,7196 | 0,7882 | 0,8397 | 0,8786 | 0,9079 | 0,9302 | 0,9470 |
| 0,80 | 0,0759 | 0,1569 | 0,2369 | 0,3098 | 0,3732 | 0,4970 | 0,5887 | 0,6610 | 0,7195 | 0,7675 | 0,8071 | 0,8669 | 0,9080 | 0,9364 | 0,9560 | 0,9695 | 0,9789 |
| 0,75 | 0,0876 | 0,1813 | 0,2738 | 0,3571 | 0,4286 | 0,5642 | 0,6601 | 0,7322 | 0,7882 | 0,8322 | 0,8669 | 0,9161 | 0,9471 | 0,9666 | 0,9789 | 0,9867 | 0,9916 |
| 0,70 | 0,0980 | 0,2030 | 0,3063 | 0,3987 | 0,4770 | 0,6211 | 0,7182 | 0,7879 | 0,8397 | 0,8786 | 0,9080 | 0,9471 | 0,9695 | 0,9825 | 0,9899 | 0,9942 | 0,9966 |
| 0,65 | 0,1072 | 0,2222 | 0,3352 | 0,4355 | 0,5193 | 0,6694 | 0,7658 | 0,8318 | 0,8786 | 0,9121 | 0,9364 | 0,9666 | 0,9825 | 0,9908 | 0,9952 | 0,9974 | 0,9986 |
| 0,60 | 0,1154 | 0,2394 | 0,3610 | 0,4680 | 0,5565 | 0,7107 | 0,8049 | 0,8664 | 0,9079 | 0,9364 | 0,9560 | 0,9789 | 0,9899 | 0,9952 | 0,9977 | 0,9989 | 0,9995 |
| 0,55 | 0,1227 | 0,2547 | 0,3838 | 0,4968 | 0,5892 | 0,7459 | 0,8371 | 0,8937 | 0,9301 | 0,9539 | 0,9696 | 0,9867 | 0,9942 | 0,9975 | 0,9989 | 0,9995 | 0,9998 |
| 0,50 | 0,1291 | 0,2683 | 0,4041 | 0,5222 | 0,6178 | 0,7759 | 0,8636 | 0,9153 | 0,9469 | 0,9666 | 0,9789 | 0,9916 | 0,9967 | 0,9987 | 0,9995 | 0,9998 | 0,9999 |
| 0,45 | 0,1348 | 0,2803 | 0,4220 | 0,5445 | 0,6427 | 0,8014 | 0,8854 | 0,9324 | 0,9596 | 0,9758 | 0,9854 | 0,9947 | 0,9981 | 0,9993 | 0,9998 | 0,9999 | 1 |
| 0,40 | 0,1399 | 0,2909 | 0,4377 | 0,5640 | 0,6644 | 0,8230 | 0,9033 | 0,9458 | 0,9692 | 0,9824 | 0,9899 | 0,9967 | 0,9989 | 0,9996 | 0,9999 | 1 | 1 |
| 0,35 | 0,1442 | 0,3000 | 0,4512 | 0,5808 | 0,6829 | 0,8411 | 0,9178 | 0,9563 | 0,9764 | 0,9872 | 0,9930 | 0,9979 | 0,9994 | 0,9998 | 1 | 1 | 1 |
| 0,30 | 0,1479 | 0,3078 | 0,4628 | 0,5951 | 0,6986 | 0,8561 | 0,9295 | 0,9645 | 0,9819 | 0,9906 | 0,9951 | 0,9987 | 0,9996 | 0,9999 | 1 | 1 | 1 |
| 0,25 | 0,1510 | 0,3143 | 0,4725 | 0,6070 | 0,7116 | 0,8684 | 0,9388 | 0,9708 | 0,9859 | 0,9931 | 0,9966 | 0,9992 | 0,9998 | 1 | 1 | 1 | 1 |
| 0,20 | 0,1535 | 0,3196 | 0,4803 | 0,6166 | 0,7221 | 0,8781 | 0,9460 | 0,9756 | 0,9888 | 0,9948 | 0,9976 | 0,9995 | 0,9999 | 1 | 1 | 1 | 1 |
| 0,15 | 0,1554 | 0,3237 | 0,4863 | 0,6240 | 0,7301 | 0,8854 | 0,9513 | 0,9791 | 0,9909 | 0,9960 | 0,9982 | 0,9997 | 0,9999 | 1 | 1 | 1 | 1 |
| 0,10 | 0,1568 | 0,3266 | 0,4906 | 0,6292 | 0,7358 | 0,8906 | 0,9550 | 0,9814 | 0,9923 | 0,9968 | 0,9986 | 0,9998 | 1 | 1 | 1 | 1 | 1 |
| 0,05 | 0,1576 | 0,3283 | 0,4932 | 0,6323 | 0,7392 | 0,8936 | 0,9572 | 0,9828 | 0,9931 | 0,9972 | 0,9989 | 0,9998 | 1 | 1 | 1 | 1 | 1 |
| 0 | 0,1579 | 0,3289 | 0,4940 | 0,6334 | 0,7403 | 0,8946 | 0,9579 | 0,9832 | 0,9933 | 0,9973 | 0,9990 | 0,9998 | 1 | 1 | 1 | 1 | 1 |

Table A.3 — Dimensionless function *C*if,h for the impulsive flexible pressure component due to horizontal seismic excitation

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
| 1,00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0,95 | 0,0232 | 0,0472 | 0,0715 | 0,0951 | 0,1176 | 0,1701 | 0,2180 | 0,2621 | 0,2710 | 0,3063 | 0,3392 | 0,3989 | 0,4516 | 0,4986 | 0,5406 | 0,5505 | 0,5845 |
| 0,90 | 0,0373 | 0,0763 | 0,1156 | 0,1534 | 0,1892 | 0,2704 | 0,3422 | 0,4061 | 0,4014 | 0,4478 | 0,4895 | 0,5609 | 0,6192 | 0,6671 | 0,7066 | 0,6840 | 0,7108 |
| 0,85 | 0,0479 | 0,0983 | 0,1490 | 0,1974 | 0,2426 | 0,3429 | 0,4290 | 0,5032 | 0,4785 | 0,5275 | 0,5699 | 0,6385 | 0,6900 | 0,7289 | 0,7583 | 0,6989 | 0,7155 |
| 0,80 | 0,0561 | 0,1154 | 0,1748 | 0,2311 | 0,2831 | 0,3959 | 0,4896 | 0,5681 | 0,5211 | 0,5681 | 0,6073 | 0,6668 | 0,7079 | 0,7362 | 0,7558 | 0,6626 | 0,6715 |
| 0,75 | 0,0625 | 0,1287 | 0,1949 | 0,2572 | 0,3139 | 0,4341 | 0,5309 | 0,6094 | 0,5405 | 0,5832 | 0,6174 | 0,6663 | 0,6972 | 0,7168 | 0,7291 | 0,6071 | 0,6115 |
| 0,70 | 0,0673 | 0,1388 | 0,2102 | 0,2767 | 0,3364 | 0,4599 | 0,5560 | 0,6316 | 0,5428 | 0,5800 | 0,6086 | 0,6472 | 0,6695 | 0,6824 | 0,6898 | 0,5435 | 0,5453 |
| 0,65 | 0,0708 | 0,1463 | 0,2214 | 0,2907 | 0,3521 | 0,4756 | 0,5683 | 0,6388 | 0,5327 | 0,5641 | 0,5874 | 0,6169 | 0,6326 | 0,6409 | 0,6452 | 0,4790 | 0,4795 |
| 0,60 | 0,0732 | 0,1515 | 0,2290 | 0,2998 | 0,3617 | 0,4825 | 0,5695 | 0,6334 | 0,5133 | 0,5391 | 0,5573 | 0,5793 | 0,5900 | 0,5952 | 0,5977 | 0,4155 | 0,4152 |
| 0,55 | 0,0747 | 0,1546 | 0,2335 | 0,3049 | 0,3661 | 0,4819 | 0,5615 | 0,6179 | 0,4873 | 0,5076 | 0,5215 | 0,5373 | 0,5444 | 0,5475 | 0,5489 | 0,3550 | 0,3543 |
| 0,50 | 0,0752 | 0,1559 | 0,2353 | 0,3063 | 0,3661 | 0,4751 | 0,5460 | 0,5940 | 0,4564 | 0,4717 | 0,4818 | 0,4925 | 0,4970 | 0,4988 | 0,4995 | 0,2980 | 0,2971 |
| 0,45 | 0,0750 | 0,1557 | 0,2348 | 0,3046 | 0,3623 | 0,4630 | 0,5242 | 0,5634 | 0,4222 | 0,4329 | 0,4395 | 0,4461 | 0,4486 | 0,4495 | 0,4498 | 0,2452 | 0,2442 |
| 0,40 | 0,0742 | 0,1542 | 0,2323 | 0,3003 | 0,3553 | 0,4467 | 0,4976 | 0,5277 | 0,3858 | 0,3923 | 0,3959 | 0,3990 | 0,3998 | 0,4000 | 0,4001 | 0,1971 | 0,1960 |
| 0,35 | 0,0729 | 0,1517 | 0,2282 | 0,2938 | 0,3458 | 0,4271 | 0,4673 | 0,4881 | 0,3483 | 0,3508 | 0,3516 | 0,3515 | 0,3509 | 0,3505 | 0,3502 | 0,1537 | 0,1525 |
| 0,30 | 0,0712 | 0,1482 | 0,2228 | 0,2858 | 0,3344 | 0,4053 | 0,4348 | 0,4464 | 0,3108 | 0,3094 | 0,3076 | 0,3044 | 0,3024 | 0,3013 | 0,3007 | 0,1158 | 0,1145 |
| 0,25 | 0,0692 | 0,1442 | 0,2165 | 0,2767 | 0,3218 | 0,3824 | 0,4013 | 0,4038 | 0,2740 | 0,2690 | 0,2646 | 0,2582 | 0,2545 | 0,2525 | 0,2514 | 0,0830 | 0,0817 |
| 0,20 | 0,0670 | 0,1399 | 0,2098 | 0,2671 | 0,3088 | 0,3594 | 0,3684 | 0,3623 | 0,2392 | 0,2307 | 0,2237 | 0,2139 | 0,2082 | 0,2050 | 0,2030 | 0,0562 | 0,0549 |
| 0,15 | 0,0649 | 0,1356 | 0,2030 | 0,2576 | 0,2961 | 0,3376 | 0,3375 | 0,3234 | 0,2075 | 0,1956 | 0,1860 | 0,1725 | 0,1643 | 0,1593 | 0,1562 | 0,0349 | 0,0335 |
| 0,10 | 0,0629 | 0,1316 | 0,1969 | 0,2490 | 0,2848 | 0,3186 | 0,3107 | 0,2899 | 0,1805 | 0,1656 | 0,1535 | 0,1362 | 0,1252 | 0,1180 | 0,1131 | 0,0199 | 0,0185 |
| 0,05 | 0,0613 | 0,1284 | 0,1920 | 0,2422 | 0,2759 | 0,3041 | 0,2906 | 0,2646 | 0,1604 | 0,1431 | 0,1288 | 0,1079 | 0,0940 | 0,0843 | 0,0773 | 0,0105 | 0,0091 |
| 0 | 0,0605 | 0,1269 | 0,1897 | 0,2390 | 0,2718 | 0,2975 | 0,2815 | 0,2532 | 0,1515 | 0,1330 | 0,1176 | 0,0948 | 0,0791 | 0,0679 | 0,0594 | 0,0076 | 0,0062 |

Table A.4 — Dimensionless function *C*if,v for the impulsive flexible pressure component due to vertical seismic excitation

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
| 1,00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0,95 | 0,0107 | 0,0232 | 0,0386 | 0,0587 | 0,0878 | 0,1925 | 0,3468 | 0,5539 | 0,8161 | 1,1349 | 1,5118 | 2,4447 | 3,6227 | 5,0523 | 6,7390 | 8,6873 | 10,9016 |
| 0,90 | 0,0213 | 0,0463 | 0,0769 | 0,1171 | 0,1751 | 0,3838 | 0,6915 | 1,1044 | 1,6271 | 2,2628 | 3,0143 | 4,8744 | 7,2231 | 10,0735 | 13,4364 | 17,3211 | 21,7359 |
| 0,85 | 0,0318 | 0,0691 | 0,1147 | 0,1748 | 0,2613 | 0,5727 | 1,0318 | 1,6481 | 2,4281 | 3,3767 | 4,4983 | 7,2740 | 10,7790 | 15,0326 | 20,0510 | 25,8481 | 32,4362 |
| 0,80 | 0,0421 | 0,0914 | 0,1519 | 0,2314 | 0,3459 | 0,7581 | 1,3659 | 2,1817 | 3,2141 | 4,4698 | 5,9545 | 9,6287 | 14,2684 | 19,8990 | 26,5420 | 34,2157 | 42,9366 |
| 0,75 | 0,0522 | 0,1132 | 0,1881 | 0,2865 | 0,4283 | 0,9388 | 1,6915 | 2,7018 | 3,9803 | 5,5354 | 7,3740 | 11,9241 | 17,6699 | 24,6428 | 32,8693 | 42,3724 | 53,1722 |
| 0,70 | 0,0619 | 0,1343 | 0,2231 | 0,3399 | 0,5081 | 1,1137 | 2,0067 | 3,2052 | 4,7220 | 6,5668 | 8,7480 | 14,1460 | 20,9624 | 29,2345 | 38,9940 | 50,2678 | 63,0800 |
| 0,65 | 0,0712 | 0,1546 | 0,2568 | 0,3912 | 0,5848 | 1,2818 | 2,3095 | 3,6889 | 5,4345 | 7,5578 | 10,0681 | 16,2807 | 24,1257 | 33,6461 | 44,8782 | 57,8533 | 72,5989 |
| 0,60 | 0,0801 | 0,1739 | 0,2889 | 0,4401 | 0,6579 | 1,4420 | 2,5981 | 4,1498 | 6,1136 | 8,5021 | 11,3261 | 18,3150 | 27,1402 | 37,8502 | 50,4858 | 65,0821 | 81,6702 |
| 0,55 | 0,0885 | 0,1922 | 0,3192 | 0,4863 | 0,7269 | 1,5932 | 2,8706 | 4,5851 | 6,7549 | 9,3941 | 12,5143 | 20,2363 | 29,9874 | 41,8210 | 55,7821 | 71,9097 | 90,2380 |
| 0,50 | 0,0964 | 0,2092 | 0,3475 | 0,5294 | 0,7914 | 1,7347 | 3,1255 | 4,9922 | 7,3546 | 10,2281 | 13,6253 | 22,0329 | 32,6497 | 45,5339 | 60,7345 | 78,2940 | 98,2494 |
| 0,45 | 0,1037 | 0,2250 | 0,3737 | 0,5693 | 0,8511 | 1,8654 | 3,3611 | 5,3685 | 7,9090 | 10,9990 | 14,6523 | 23,6937 | 35,1107 | 48,9661 | 65,3125 | 84,1955 | 105,6551 |
| 0,40 | 0,1103 | 0,2394 | 0,3976 | 0,6057 | 0,9055 | 1,9847 | 3,5759 | 5,7117 | 8,4146 | 11,7022 | 15,5890 | 25,2084 | 37,3553 | 52,0963 | 69,4877 | 89,5779 | 112,4094 |
| 0,35 | 0,1162 | 0,2523 | 0,4190 | 0,6384 | 0,9543 | 2,0917 | 3,7687 | 6,0197 | 8,8683 | 12,3332 | 16,4296 | 26,5677 | 39,3695 | 54,9054 | 73,2346 | 94,4080 | 118,4707 |
| 0,30 | 0,1215 | 0,2636 | 0,4379 | 0,6671 | 0,9973 | 2,1858 | 3,9383 | 6,2905 | 9,2674 | 12,8881 | 17,1689 | 27,7631 | 41,1410 | 57,3760 | 76,5300 | 98,6561 | 123,8015 |
| 0,25 | 0,1259 | 0,2734 | 0,4540 | 0,6917 | 1,0341 | 2,2665 | 4,0836 | 6,5226 | 9,6093 | 13,3636 | 17,8023 | 28,7874 | 42,6589 | 59,4929 | 79,3535 | 102,2960 | 128,3691 |
| 0,20 | 0,1297 | 0,2814 | 0,4674 | 0,7121 | 1,0645 | 2,3331 | 4,2038 | 6,7145 | 9,8919 | 13,7568 | 18,3260 | 29,6342 | 43,9137 | 61,2429 | 81,6877 | 105,3051 | 132,1452 |
| 0,15 | 0,1326 | 0,2877 | 0,4779 | 0,7280 | 1,0883 | 2,3854 | 4,2980 | 6,8650 | 10,1136 | 14,0650 | 18,7367 | 30,2984 | 44,8979 | 62,6154 | 83,5184 | 107,6650 | 135,1066 |
| 0,10 | 0,1346 | 0,2922 | 0,4854 | 0,7395 | 1,1055 | 2,4230 | 4,3657 | 6,9731 | 10,2730 | 14,2866 | 19,0319 | 30,7757 | 45,6052 | 63,6018 | 84,8341 | 109,3612 | 137,2350 |
| 0,05 | 0,1359 | 0,2950 | 0,4899 | 0,7464 | 1,1158 | 2,4456 | 4,4065 | 7,0383 | 10,3689 | 14,4201 | 19,2097 | 31,0632 | 46,0313 | 64,1961 | 85,6268 | 110,3830 | 138,5174 |
| 0 | 0,1363 | 0,2959 | 0,4914 | 0,7487 | 1,1193 | 2,4532 | 4,4201 | 7,0600 | 10,4010 | 14,4647 | 19,2691 | 31,1593 | 46,1736 | 64,3946 | 85,8916 | 110,7244 | 138,9457 |

Table A.5 — Correction factor *βc* to consider the clamping degree at the tank bottom

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
| 1 | 1 | 1 | 1 | 1,0780 | 1,1891 | 1,2679 | 1,3291 | 1,3790 | 1,4213 | 1,4578 | 1,5190 | 1,5689 | 1,6112 | 1,6478 | 1,6800 | 1,7089 |

Table A.6 — Participation factor *Γ*if,v for the impulsive flexible component due to vertical seismic excitation

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **Γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
|  | 1,1892 | 1,0958 | 0,9896 | 0,8807 | 0,7807 | 0,5893 | 0,4650 | 0,3815 | 0,3224 | 0,2788 | 0,2453 | 0,1976 | 0,1653 | 0,1420 | 0,1244 | 0,1107 | 0,0997 |

Table A.7 — Coefficients *C*F,j*, C*MW,j*, C*M,j and participation factors *Γ*j for the convective (*j = c*), impulsive rigid (*i = ir, h*) and impulsive flexible pressure components (*i = if, h*)

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
|  | **Convective pressure component** | | | | | | | | | | | | | | | | |
| ***C*F,c** | 0,8704 | 0,7541 | 0,6360 | 0,5328 | 0,4493 | 0,3120 | 0,2355 | 0,1886 | 0,1572 | 0,1348 | 0,1179 | 0,0943 | 0,0786 | 0,0674 | 0,0590 | 0,0524 | 0,0472 |
| ***C*MW,c** | 0,4434 | 0,3985 | 0,3520 | 0,3105 | 0,2758 | 0,2147 | 0,1762 | 0,1494 | 0,1297 | 0,1144 | 0,1023 | 0,0843 | 0,0717 | 0,0623 | 0,0551 | 0,0493 | 0,0447 |
| ***C*M,c** | 0,2488 | 0,2561 | 0,2742 | 0,3063 | 0,3523 | 0,5143 | 0,7170 | 0,9388 | 1,1689 | 1,4023 | 1,6372 | 2,1084 | 2,5801 | 3,0520 | 3,5240 | 3,9963 | 4,4689 |
| ***Γ*c** | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 |
|  | **Impulsive rigid pressure component** | | | | | | | | | | | | | | | | |
| ***C*F,ir,h** | 0,1148 | 0,2386 | 0,3591 | 0,4636 | 0,5478 | 0,6861 | 0,7630 | 0,8102 | 0,8418 | 0,8644 | 0,8813 | 0,9051 | 0,9209 | 0,9321 | 0,9406 | 0,9472 | 0,9524 |
| ***C*MW,ir,h** | 0,0459 | 0,0952 | 0,1435 | 0,1861 | 0,2214 | 0,2834 | 0,3224 | 0,3494 | 0,3694 | 0,3847 | 0,3969 | 0,4150 | 0,4278 | 0,4372 | 0,4445 | 0,4503 | 0,4549 |
| ***C*M,ir,h** | 0,0191 | 0,0723 | 0,1541 | 0,2615 | 0,3950 | 0,8565 | 1,5273 | 2,4290 | 3,5724 | 4,9623 | 6,6008 | 10,6261 | 15,6508 | 21,6749 | 28,6986 | 36,7218 | 45,7444 |
| ***Γ*ir,h** | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 | 1,0000 |
|  | **Impulsive flexible pressure component** | | | | | | | | | | | | | | | | |
| ***C*F,if,h** | 0,0620 | 0,1286 | 0,1937 | 0,2510 | 0,2982 | 0,3801 | 0,4306 | 0,4647 | 0,3693 | 0,3846 | 0,3968 | 0,4149 | 0,4276 | 0,4371 | 0,4443 | 0,3151 | 0,3194 |
| ***C*MW,if,h** | 0,0283 | 0,0586 | 0,0885 | 0,1157 | 0,1393 | 0,1854 | 0,2190 | 0,2448 | 0,2074 | 0,2211 | 0,2322 | 0,2493 | 0,2616 | 0,2708 | 0,2779 | 0,2204 | 0,2247 |
| ***C*M,if,h** | 0,0090 | 0,0352 | 0,0772 | 0,1355 | 0,2118 | 0,4994 | 0,9547 | 1,6012 | 1,9094 | 2,7456 | 3,7493 | 6,2598 | 9,4396 | 13,2871 | 17,8012 | 17,8567 | 22,4761 |
| ***Γ*if,h** | 1,6529 | 1,6581 | 1,6545 | 1,6417 | 1,6226 | 1,5646 | 1,5099 | 1,4656 | 1,7807 | 1,7401 | 1,7087 | 1,6642 | 1,6348 | 1,6141 | 1,5989 | 1,7553 | 1,7393 |

Table A.8 — Correction factor Fh(γ) for the evaluation of the fundamental natural period in horizontal direction *T*if,hwithout consideration of soil–structure interaction effects

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
| 1,6963 | 1,9151 | 2,1465 | 2,3905 | 2,6470 | 3,3433 | 4,1180 | 4,9713 | 5,0903 | 6,9133 | 8,0020 | 10,4150 | 13,1420 | 16,1830 | 19,5380 | 23,2070 | 27,1900 |

Table A.9 — Correction factor Fv(γ) for the evaluation of the fundamental natural period in vertical direction *T*if,vwithout consideration of soil–structure interaction effects

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **γ=0,2** | **γ=0,4** | **γ=0,6** | **γ=0,8** | **γ=1,0** | **γ=1,5** | **γ=2,0** | **γ=2,5** | **γ=3,0** | **γ=3,5** | **γ=4,0** | **γ=5,0** | **γ=6,0** | **γ=7,0** | **γ=8,0** | **γ=9,0** | **γ=10** |
| 2,5937 | 2,7019 | 2,8432 | 3,0139 | 3,2011 | 3,6844 | 4,1477 | 4,5794 | 4,9813 | 5,3572 | 5,7108 | 6,3633 | 6,9577 | 7,5068 | 8,0192 | 8,5014 | 8,9580 |

Table A.10 — First convective circular frequency , convective mass and impulsive mass with respect to the dimensionless filling height

|  |  |  |  |
| --- | --- | --- | --- |
|  |  |  |  |
| -1,0 | 1,0 | 1,0 | 0,0 |
| 0,95 | 1,0170 | 0,98315 | 0,01674 |
| -0,9 | 1,0347 | 0,96594 | 0,03366 |
| -0,8 | 1,0723 | 0,93038 | 0,06816 |
| -0,6 | 1,1583 | 0,85437 | 0,14053 |
| -0,4 | 1,2625 | 0,77117 | 0,21864 |
| -0,2 | 1,3924 | 0,67990 | 0,30381 |
| 0,0 | 1,5602 | 0,57969 | 0,39406 |
| 0,2 | 1,7882 | 0,46981 | 0,50156 |
| 0,4 | 2,1232 | 0,35009 | 0,61560 |
| 0,6 | 2,6864 | 0,22222 | 0,73838 |
| 0,8 | 3,9595 | 0,09363 | 0,87392 |
| 0,9 | 5,7615 | 0,03655 | 0,94414 |
| 0,95 | 8,3121 | 0,01364 | 0,98190 |
| 1,0 | - | - | 1,0 |

1. (informative)  
     
   Soil-structure interaction effects of tanks
   1. Use of this annex
2. This informative annex provides supplementary guidance to Clause 6.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

1. Annex B gives rules to consider soil-structure interaction effects in the calculation of cylindrical tanks.
   1. Impulsive rigid vibration mode in horizontal direction
2. The period  of the impulsive rigid vibration mode of the tank–foundation system including soil–structure interaction may be calculated with the rigid mass given in 6.4.1.4, the horizontal and the rocking stiffness of the foundation and using Formula (B.1).

(B.1)

1. In case of a circular rigid foundation supported at the surface of a homogeneous soil deposit, the horizontal stiffness and rotational stiffness at the level of the tank base may be calculated as given in (B.2) and (B.3) respectively:

(B.2)

(B.3)

where

|  |  |
| --- | --- |
|  | is the radius of circular foundations or the equivalent radius in case of rectangular foundations; |
|  | is the shear modulus of the soil; |
|  | is the Poisson’s ratio of the soil; |
|  | is the horizontal dynamic stiffness modifier (= 1,0); |
|  | is the rotational dynamic stiffness modifier. |

1. The rotational dynamic stiffness modifier in Formula (B.3) may be calculated, using Formula (B.4) as a function of the coefficients and the dimensionless frequency parameter , as given by Formula (B.5).

(B.4)

(B.5)

where

|  |  |
| --- | --- |
|  | is the shear wave velocity of the soil; |
|  | are coefficients depending on Poisson’s ratio of the soil given in Table B.1. |

* 1. Impulsive rigid vibration mode in vertical direction

1. The period of the vertical impulsive rigid vibration mode of the tank–foundation system including soil–structure interaction may be calculated by Formula (B.6).

(B.6)

where *K*V is the vertical stiffness of the foundation, which should be calculated by Formula (B.7).

(B.7)

1. The vertical dynamic stiffness modifier in Formula (B.7) may be calculated using Formula (B.8) as a function of the coefficients and the dimensionless frequency parameter using Formula (B.5) by replacing the horizontal period with vertical period .

(B.8)

where are the coefficients depending on Poisson’s ratio of the soil given in Table B.1.

Table B.1 — Numerical coefficients for rotational and vertical dynamic stiffness modifier of rigid circular footing on homogenous half space

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Coefficient** |  |  |  |  |
|  | 0,8 | 0,8 | 0,8 | 0,8 |
|  | 0,525 | 0,5 | 0,45 | 0,4 |
|  | 0 | 0 | 0,023 | 0,027 |
|  | 0,25 | 0,35 | - | 0 |
|  | 1 | 0,8 | - | 0 |
|  | 0,85 | 0,75 | - | 0,85 |

* 1. Impulsive flexible vibration mode in horizontal direction

1. The period of the impulsive flexible vibration mode of the tank–foundation system including soil–structure interaction for horizontal mode may be calculated using Formula (B.9).

(B.9)

where

|  |  |
| --- | --- |
| , | are the horizontal and rocking stiffness of the foundation according to Formulas (B.2) and (B.3); |
|  | Is the flexible tank stiffness associated to the impulsive flexible mode:  ; |
|  | is the flexible impulsive mass according to 6.4.1.4 (5); |
|  | is the lever arm of from the base according to 6.4.1.4 (5). |

* 1. Impulsive flexible vibration mode in vertical direction

1. The impulsive flexible period of vibration of the tank–foundation system including soil–structure interaction for vertical mode should be calculated using Formula (B.10).

(B.10)

where is the vertical stiffness of the foundation according to Formula (B.7).

1. (informative)  
     
   General design considerations for buried pipelines
   1. Use of this annex
2. This informative annex provides supplementary guidance to Clause 8.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

1. Annex C gives rules for buried pipelines to improve their seismic resistance.
   1. General design consideration for buried pipelines
2. Pipelines should be laid on soils which are checked to remain stable under the design seismic action. When the condition above cannot be satisfied, the nature and the extent of the adverse phenomena should be explicitly assessed, and appropriate design counter measures applied.

NOTE Two extreme cases: Soil liquefaction and fault movements are worth being mentioned, since they require in general design solutions specific to each particular case.

1. Soil liquefaction, whenever it did occur, has been a major contributor to pipelines distress in past earthquakes.
2. Depending on the circumstances, a) or b) should be considered:
3. increasing the burial depth, possibly also encasing the pipes in larger stiff conduits;
4. placing the pipeline above-ground, supporting it at rather large distances on well-founded piers. In this case, flexible joints should also be considered to allow for relative displacements between supports.
5. Design for fault movements requires estimating, sometimes postulating, a number of parameters including: location, size of the area affected, type and measure of the fault displacement.

NOTE Given these parameters, the simplest way of modelling the phenomenon is to consider a rigid displacement between the soil masses interfacing at the fault.

1. The general criterion for minimizing the effect of an imposed displacement is that of introducing the maximum flexibility into the system which is subjected to it.
2. In the case under consideration a) to c) may be done:
3. decreasing the burial depth so as to reduce the soil restraint;
4. providing a large ditch for the pipes, to be filled with soft material;
5. putting the pipeline above ground and introducing flexible and extensible piping members.
6. A pipeline should be continued without bends, if any fault system as those shown Figure C.1 is crossed.

Key

|  |  |
| --- | --- |
| *A* | reverse-slip compression |
| *B* | strike-slip |
| *C* | normal-slip tension |

Figure C.1 — Fault crossing mechanism

1. If a pipeline crosses a fault, the pipeline should be oriented in a way that mainly tension rather than compression stresses are induced.

NOTE An angle of the pipeline to the fault line as close as possible to 90 degrees is considered to minimise compression stresses.

1. Pipelines crossing a fault should be continued without bends at least 50 m before and after the fault.
2. In fault zones, the depth at which the pipeline is buried should be minimised in order to minimise the soil restraint on the pipeline during a fault movement.
3. Within 50 m on each side of the fault, relatively thick-walled pipeline and a coating allowing the lowest possible friction with the surrounding soil should be used. An exchange of the surrounding soil by a special low friction and high compressibility backfill may also be considered as illustrated in Figure C.2.

Key

|  |  |
| --- | --- |
| *A* | pipeline |
| *B* | trench |
| *C* | fault |

Figure C.2 — Trench design at fault crossing

1. (informative)  
     
   Modelling of soil-structure interaction of buried pipelines
   1. Use of this annex
2. This informative annex provides supplementary guidance to 8.3.3.1.1.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

1. Annex D gives rules to define the characteristics of spring elements representing the soil-pipeline interaction in simplified models of analysis.
   1. Characteristics of spring elements
2. A simplified formulation for the soil-pipeline interaction may as given in Formula (D.1) and plotted in the graphs of Figure D.1 for the different directions along the pipeline.

(D.1)

where

|  |  |
| --- | --- |
|  | is the displacement applied to the spring; |
|  | is the ultimate displacement; |
|  | is the force applied to the pipeline by the spring; |
|  | is the ultimate force applied to the pipeline. |

NOTE 1 Figure D.1 shows the curves “force” in function of “displacement” of the bilinear models to account for pipe/soil interaction. The relation drawn in Figure D.1a) is used to model the axial behaviour between the pipeline and the soil. The relation drawn in Figure D.1b) is used to model the transverse behaviour in the horizontal direction. The relation drawn in Figure D.1c) is used to model the transverse behaviour in the vertical direction. The variables and are described in D.4.2; and are described in D.4.3; , , and are described in D.4.4.

NOTE 2 When the pipeline is buried close to the surface (depth lower than ten times the outer diameter), the vertical behaviour is represented by the curve in Figure D.1c); when the pipeline is deeply buried (depth greater than ten times the outer diameter), the vertical behaviour is identical to the horizontal behaviour and is represented by the curve in Figure D.1b).

Figure D.1 — Spring models: (a) axial; (b) transverse horizontal; (c) transverse vertical

* 1. Analytical relations of the spring model
     1. General

1. Each of the Formulas in D.4.2 to D4.3 may be used for coarse-grained soil and fine-grained soils.
   * 1. **Axial spring model**
2. The ultimate axial force per unit length may be calculated with Formula (D.2).

(D.2)

where

|  |  |
| --- | --- |
|  | is the soil cohesion; |
|  | is the outer diameter of the pipeline; |
|  | depth from the soil surface to centreline of the pipeline; |
|  | is the total soil unit weight; |
|  | is the water unit weight; |
|  | is the effective soil unit weight, defined as ; |
|  | is the coefficient of soil pressure at rest; |
|  | is the friction angle between the soil and the pipeline, defined as ; |
|  | is the coating dependent factor; |
|  | is the internal friction angle of the soil in degrees (°); |
|  | is the soil-pipeline adhesion factor defined in Formula (D.3) where *c*’ is expressed in kPa. |

(D.3)

NOTE 1 Representative values for are given in Table D.1.

NOTE 2 The values given in Table D.1 for do not take into account the dilatancy (volume change observed in granular materials subjected to shear deformations) i.e. these values do not account for the stress increment developed at the soil-pipeline interface: due to confined shear conditions, the soil may not expand freely, and therefore, an extra stress develops in the direction normal to the soil-pipeline interface.

Table D.1 — Representative values for the coating dependent factor

|  |  |
| --- | --- |
| **Pipeline coating** |  |
| Concrete | 1,0 |
| Coal tar | 0,9 |
| Rough steel | 0,8 |
| Smooth steel | 0,7 |
| Fusion bonded epoxy | 0,6 |
| Polyethylene | 0,6 |

1. The ultimate relative displacement may be taken according to Table D.2.

Table D.2 — Representative values for the ultimate relative displacement

|  |  |
| --- | --- |
| **Type of soil** | **(mm)** |
| dense coarse-grained soil | 3 |
| loose coarse-grained soil | 5 |
| stiff fine-grained soil | 8 |
| soft fine-grained soil | 10 |

1. The dilatancy should be taken into account to define the design axial resistance for coarse-grained soils.
   * 1. Transverse spring model in horizontal direction
2. The ultimate transverse force per unit length may be evaluated with Formula (D.4).

(D.4)

where

|  |  |
| --- | --- |
|  | is the dimensionless horizontal bearing capacity factor for fine-grained soil, with = 0 for = 0; |
|  | is the dimensionless horizontal bearing capacity factor for coarse-grained soil, with = 0 for = 0. |

1. The dimensionless horizontal bearing capacities may be determined with Formulas (D.5) and (D.6) by using the constants in Table D.3.

(D.5)

(D.6)

Table D.3 — Dimensionless horizontal bearing capacities

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| **Capacity** |  | **a** | **b** | **c** | **d** | **e** |
| ***N*ch** | 0° | 6,752 | 0,065 | -11,063 | 7,119 | - |
| ***N*qh** | 20° | 2,399 | 0,439 | -0,030 | 1,06e-03 | -1,75e-05 |
| ***N*qh** | 25° | 3,332 | 0,839 | -0,090 | 5,60e-03 | -1,32e-04 |
| ***N*qh** | 30° | 4,565 | 1,234 | -0,089 | 4,28e-03 | -9,16e-05 |
| ***N*qh** | 35° | 6,816 | 2,019 | -0,146 | 7,65e-03 | -1,68e-04 |
| ***N*qh** | 40° | 10,959 | 1,783 | 0,045 | -5,43e-03 | 1,15e-04 |
| ***N*qh** | 45° | 17,658 | 3,309 | 0,048 | -6,44e-03 | 1,30e-04 |

1. The ultimate relative displacement may be calculated by Formula (D.7).

(D.7)

* + 1. Transverse spring model in vertical direction

1. The ultimate vertical uplift force per unit length may be evaluated with Formula (D.8).

(D.8)

where is the dimensionless vertical uplift factor for fine-grained soils ( for ), given by Formula (D.9).

for (D.9)

where is the dimensionless vertical uplift factor for coarse-grained soils ( for ), given by Formula (D.10).

(D.10)

1. The relative displacement for the ultimate vertical spring force may be assumed equal to 0,01  to 0,02  for loose to dense coarse-grained soils and stays below 0,1 . For stiff to soft fine-grained soil may be assumed equal to 0,1 to 0,2  and stays below 0,2 . should be calculated according to Formula (D.13).
2. The ultimate vertical downward force (vertical bearing capacity) per unit length may be calculated with Formula (D.11).

(D.11)

where

|  |  |
| --- | --- |
|  | is the dimensionless vertical bearing capacity factor for cohesive soil according to Formula (D.12). |

(D.12)

|  |  |
| --- | --- |
|  | is the dimensionless bearing capacity factor for passive earth pressure due to the self-weight of the soil as given in Formula (D.13). |

(D.13)

|  |  |
| --- | --- |
|  | is the dimensionless bearing capacity factor for passive earth pressure due to the downward movement of the pipeline as given in Formula (D.14). |

(D.14)

1. The relative displacement for the ultimate vertical spring force may be assumed equal to for coarse-grained soils and 0,2 for fine-grained soils.
2. (informative)  
     
   Design differential surface displacement at pipeline – fault crossing
   1. Use of this annex
3. This informative annex provides supplementary guidance to Clauses 8 and 9.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

1. Annex E contains additional provisions to 7.5.2.3.1 and 8.3.3.2.1 for defining the design differential displacement that can be used for designing pipelines, or other elongated structures, crossing a fault at a specified location.

NOTE Informative Annex E provides an approximation of fault surface displacement for given return periods based on gross fault characteristics. Site-specific studies offer more reliable results and can be preferred whenever large displacements are expected, or the consequences of failure are high.

* 1. Differential surface displacements at pipeline – fault crossings

1. The design differential displacement may be characterised by the fault mechanism and geometry, the fault seismicity, the pipeline – fault crossing geometry, and the return period of interest.

NOTE 1 The design differential displacement is measured within the fault plane and in the orientation of the fault rupture.

NOTE 2 The design differential displacement estimated via Annex E only concerns principal faulting. It does not incorporate any additional differential displacement appearing at sites away from the fault trace due to distributed/secondary faulting.

1. The pipeline – fault crossing geometry should be accounted for when applying the design differential displacement in the pipeline analysis. Specifically, the fault dip angle, the pipeline – fault crossing angle, as well as the rupture direction within the fault plane, along which the design differential displacement is applied, should be incorporated in the analysis. Where there is uncertainty regarding the rupture direction, the worst case should be used from the range of potential rupture directions that comply with the seismological characterisation of the fault mechanism, e.g., normal, reverse, strike-slip (Figure E.1).

Figure E.1 — Fault mechanisms: (a) normal fault – pipeline is in tension and bending; (b) reverse fault – pipeline is in compression and bending; (c) strike-slip fault – pipeline is mainly in bending

Key

|  |  |
| --- | --- |
| *A* | crossing point |
| *B* | distance to closest fault-end |
| *C* | fault trace |
| *D* | pipeline |

Figure E.2 — Idealised pipeline – fault geometry in plan

1. The fault and fault-crossing characteristics in (1) may be represented by parameters in a) to d):
2. The fault mechanism is the seismological characterisation of the fault according to the fault plane orientation and the direction of slip. For the purposes of Annex E, three characterisations are used per Figure E.1: Normal, reverse, and strike-slip;
3. *L*F is the fault length (km). If *L*F < 10 km, a minimum length of 10 km should be assumed. If *L*F > 300 km, then Annex E should not be applied;
4. *X*L= *B*/*L*Fis the ratio of the distance *B*, (along the fault trace) of the pipeline – fault crossing point to the closest fault-end over the fault length, 0 < *X*L ≤ 0,50 (Figure E.2);
5. *v*F is the recurrence rate of the fault, i.e. the mean number of events above moment magnitude 5,5 per year. It can be derived from a seismological study, or an available fault source model. Alternatively, the recurrence rate can be calculated as given by Formula (E.1).

(E.1)

where

|  |  |
| --- | --- |
| *p*1 to *p*7 | are coefficients given in Table E.1 when using the mean *S*β,475, and in Table E.2 when using the median *S*β,475; |
| *S*β,475 | is the reference maximum spectral acceleration at *T*β= 1 s corresponding to a return period of *T*ref = 475 years for the pipeline – fault crossing site, as defined in EN 1998-1-1:—2, 5.2.1; |
| *g* | is the acceleration of gravity. |

Table E.1 — Coefficients for recurrence rate (*v*F) approximate estimation using the mean *S*β,475

|  |  |
| --- | --- |
| **Coefficient** | **Value** |
| *p*1 | -10,1539 |
| *p*2 | 16,7322 |
| *p*3 | -76,0447 |
| *p*4 | 5,4398 |
| *p*5 | 0,1262 |
| *p*6 | 74,1251 |
| *p*7 | -0,5065 |

Table E.2 — Coefficients for recurrence rate (*vF*) approximate estimation using the median *S*β,475

|  |  |
| --- | --- |
| **Coefficient** | **Value** |
| *p*1 | -10,2940 |
| *p*2 | 23,6696 |
| *p*3 | -120,9933 |
| *p*4 | 5,0275 |
| *p*5 | 0,1280 |
| *p*6 | 162,7411 |
| *p*7 | -0,4092 |

1. The return period of exceeding a given fault displacement *Δ*F at the pipeline – fault crossing site should be estimated using Formula (E.2).

(E.2)

where

|  |  |
| --- | --- |
| *C*F | is the confidence factor for the estimation of the recurrence rate. *C*F = 1,0 when the recurrence rate is directly determined from a seismological study, or an available seismic source model. If, instead, the approximate Formula (E.1) has been used, then *C*F should be calculated by Formula (E.3). |

(E.3)

|  |  |
| --- | --- |
|  | for recurrence rate approximate estimation using the mean *S*β,475, and for the estimation using the median *S*β,475. |
|  | is a recurrence-rate-independent function of the fault and fault-crossing characteristics calculated using Formula (E.4). |

(E.4)

|  |  |
| --- | --- |
| *a*0,…, *a*8 | are coefficients that depend on *Δ*F and the recurrence rate class per Table E.3, and are provided in Table E.4 for normal faults, Table E.5 for reverse faults, and Table E.6 for strike-slip faults. |

Table E.3 — Recurrence rate (*v*F) classification

|  |  |
| --- | --- |
| **Class** | **Range** |
| Low | *v*F < 0,10 |
| High | *v*F > 0,10 |

Table E.4 — Coefficients of Formula (E.4) for normal fault mechanism

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| **Differential displacement** | **Coeffi-cients** | **Recurrence rate class** | | **Differential displacement** | **Coeffi-cients** | **Recurrence rate class** | |
| Low | High | Low | High |
| *ΔF* = 0,25 m | *a*0 | -3,6511 | -0,9265 | *ΔF* = 1,75 m | *a*0 | -12,5074 | -10,9173 |
| *a*1 | 0,6921 | -1,1847 | *a*1 | 4,0081 | 3,1454 |
| *a*2 | -0,7924 | -8,7234 | *a*2 | -1,7911 | -7,6197 |
| *a*3 | -0,2225 | 0,1750 | *a*3 | -0,7666 | -0,6528 |
| *a*4 | 2,8574 | 6,4744 | *a*4 | 3,3464 | 5,8859 |
| *a*5 | -4,8378 | 1,8547 | *a*5 | -2,5522 | 2,8103 |
| *a*6 | 0,0139 | -0,0140 | *a*6 | 0,0457 | 0,0427 |
| *a*7 | -0,3513 | -0,6960 | *a*7 | -0,3293 | -0,5536 |
| *a*8 | -0,0829 | -2,1222 | *a*8 | -0,9582 | -2,5759 |
| *ΔF* = 0,50 m | *a*0 | -5,4586 | -3,0263 | *ΔF* = 2,00 m | *a*0 | -13,7728 | -12,4003 |
| *a*1 | 1,3003 | -0,3635 | *a*1 | 4,5498 | 3,8982 |
| *a*2 | -0,6753 | -7,9911 | *a*2 | -2,0188 | -7,7081 |
| *a*3 | -0,3152 | 0,0316 | *a*3 | -0,8619 | -0,8109 |
| *a*4 | 2,8847 | 6,1622 | *a*4 | 3,4242 | 5,9055 |
| *a*5 | -4,5365 | 1,8317 | *a*5 | -2,2497 | 2,9971 |
| *a*6 | 0,0189 | -0,0053 | *a*6 | 0,0516 | 0,0545 |
| *a*7 | -0,3391 | -0,6424 | *a*7 | -0,3295 | -0,5490 |
| *a*8 | -0,2635 | -2,1945 | *a*8 | -1,0488 | -2,6330 |
| *ΔF* = 0,75 m | *a*0 | -7,0224 | -4,7633 | *ΔF* = 2,50 m | *a*0 | -16,0707 | -15,1360 |
| *a*1 | 1,8573 | 0,3394 | *a*1 | 5,5441 | 5,3084 |
| *a*2 | -0,8239 | -7,7106 | *a*2 | -2,4315 | -7,9561 |
| *a*3 | -0,4041 | -0,0962 | *a*3 | -1,0375 | -1,1088 |
| *a*4 | 2,9716 | 6,0194 | *a*4 | 3,5581 | 5,9812 |
| *a*5 | -4,1048 | 2,0057 | *a*5 | -1,7353 | 3,3418 |
| *a*6 | 0,0239 | 0,0030 | *a*6 | 0,0626 | 0,0769 |
| *a*7 | -0,3338 | -0,6104 | *a*7 | -0,3304 | -0,5465 |
| *a*8 | -0,4390 | -2,2858 | *a*8 | -1,1959 | -2,7326 |
| *ΔF* = 1,00 m | *a*0 | -8,4603 | -6,3450 | *ΔF* = 3,00 m | *a*0 | -18,4061 | -17,9528 |
| *a*1 | 2,3952 | 1,0138 | *a*1 | 6,6007 | 6,8172 |
| *a*2 | -1,0443 | -7,5878 | *a*2 | -2,8321 | -8,2978 |
| *a*3 | -0,4926 | -0,2241 | *a*3 | -1,2284 | -1,4322 |
| *a*4 | 3,0686 | 5,9413 | *a*4 | 3,6840 | 6,1005 |
| *a*5 | -3,6788 | 2,2074 | *a*5 | -1,2756 | 3,6963 |
| *a*6 | 0,0291 | 0,0117 | *a*6 | 0,0748 | 0,1014 |
| *a*7 | -0,3312 | -0,5885 | *a*7 | -0,3322 | -0,5506 |
| *a*8 | -0,5931 | -2,3689 | *a*8 | -1,3206 | -2,8295 |
| *ΔF* = 1,25 m | *a*0 | -10,0535 | -8,1157 | *ΔF* = 3,50 m | *a*0 | -20,7246 | -20,7531 |
| *a*1 | 3,0145 | 1,8095 | *a*1 | 7,6813 | 8,3474 |
| *a*2 | -1,3295 | -7,5455 | *a*2 | -3,2224 | -8,7129 |
| *a*3 | -0,5966 | -0,3807 | *a*3 | -1,4266 | -1,7618 |
| *a*4 | 3,1807 | 5,8946 | *a*4 | 3,8044 | 6,2535 |
| *a*5 | -3,2148 | 2,4433 | *a*5 | -0,8632 | 4,0569 |
| *a*6 | 0,0353 | 0,0228 | *a*6 | 0,0875 | 0,1264 |
| *a*7 | -0,3299 | -0,5707 | *a*7 | -0,3350 | -0,5596 |
| *a*8 | -0,7496 | -2,4551 | *a*8 | -1,4273 | -2,9241 |
| *ΔF* = 1,50 m | *a*0 | -11,2729 | -9,4956 | *ΔF* = 4,00 m | *a*0 | -23,0154 | -23,5015 |
| *a*1 | 3,4990 | 2,4528 | *a*1 | 8,7735 | 9,8655 |
| *a*2 | -1,5614 | -7,5652 | *a*2 | -3,6064 | -9,1820 |
| *a*3 | -0,6787 | -0,5102 | *a*3 | -1,6291 | -2,0889 |
| *a*4 | 3,2655 | 5,8820 | *a*4 | 3,9223 | 6,4306 |
| *a*5 | -2,8729 | 2,6267 | *a*5 | -0,4880 | 4,4234 |
| *a*6 | 0,0403 | 0,0322 | *a*6 | 0,1007 | 0,1512 |
| *a*7 | -0,3294 | -0,5608 | *a*7 | -0,3387 | -0,5721 |
| *a*8 | -0,8590 | -2,5171 | *a*8 | -1,5204 | -3,0177 |

Table E.5 — Coefficients of Formula (E.4) for reverse fault mechanism

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| **Differential displacement** | **Coeffi-cients** | **Recurrence rate class** | | **Differential displacement** | **Coeffi-cients** | **Recurrence rate class** | |
|  |  | Low | High |  |  | Low | High |
| *ΔF* = 0,25 m | *a*0 | -3,5809 | -0,7625 | *ΔF* = 1,75 m | *a*0 | -16,8189 | -17,4403 |
| *a*1 | 0,2308 | -1,7508 | *a*1 | 5,7183 | 7,0093 |
| *a*2 | -0,1928 | -8,1037 | *a*2 | -2,1580 | -10,4494 |
| *a*3 | -0,1184 | 0,3233 | *a*3 | -1,0185 | -1,4862 |
| *a*4 | 2,5577 | 6,1146 | *a*4 | 1,7454 | 3,8369 |
| *a*5 | -5,1595 | 1,8046 | *a*5 | 12,4648 | 27,4079 |
| *a*6 | 0,0063 | -0,0267 | *a*6 | 0,0552 | 0,0979 |
| *a*7 | -0,3181 | -0,6493 | *a*7 | 0,0269 | 0,1038 |
| *a*8 | -0,0133 | -2,1196 | *a*8 | -4,1008 | -8,4000 |
| *ΔF* = 0,50 m | *a*0 | -5,8650 | -3,5924 | *ΔF* = 2,00 m | *a*0 | -19,3740 | -20,8612 |
| *a*1 | 0,9791 | -0,5353 | *a*1 | 7,0405 | 9,1662 |
| *a*2 | 0,0958 | -7,4030 | *a*2 | -2,7887 | -11,6557 |
| *a*3 | -0,2168 | 0,1087 | *a*3 | -1,2687 | -1,9762 |
| *a*4 | 2,3057 | 5,3286 | *a*4 | 1,6295 | 3,6812 |
| *a*5 | -2,9934 | 5,2232 | *a*5 | 15,9571 | 32,8782 |
| *a*6 | 0,0101 | -0,0147 | *a*6 | 0,0710 | 0,1347 |
| *a*7 | -0,2507 | -0,4754 | *a*7 | 0,0842 | 0,2146 |
| *a*8 | -0,6062 | -3,0354 | *a*8 | -4,8456 | -9,7000 |
| *ΔF* = 0,75 m | *a*0 | -7,9772 | -6,1239 | *ΔF* = 2,50 m | *a*0 | -24,3737 | -27,5073 |
| *a*1 | 1,7541 | 0,6466 | *a*1 | 9,7104 | 13,4480 |
| *a*2 | -0,1424 | -7,5533 | *a*2 | -4,2428 | -14,6526 |
| *a*3 | -0,3311 | -0,1170 | *a*3 | -1,7842 | -2,9611 |
| *a*4 | 2,1602 | 4,8702 | *a*4 | 1,4712 | 3,6458 |
| *a*5 | -0,1287 | 9,2241 | *a*5 | 22,8852 | 43,9777 |
| *a*6 | 0,0154 | -0,0005 | *a*6 | 0,1040 | 0,2095 |
| *a*7 | -0,1904 | -0,3408 | *a*7 | 0,1886 | 0,4016 |
| *a*8 | -1,2983 | -4,0361 | *a*8 | -6,3050 | -12,3316 |
| *ΔF* = 1,00 m | *a*0 | -10,0475 | -8,6611 | *ΔF* = 3,00 m | *a*0 | -30,1500 | -35,2076 |
| *a*1 | 2,5888 | 1,9420 | *a*1 | 12,9626 | 18,6243 |
| *a*2 | -0,5416 | -7,9898 | *a*2 | -6,1739 | -18,9437 |
| *a*3 | -0,4646 | -0,3807 | *a*3 | -2,4301 | -4,1774 |
| *a*4 | 2,0486 | 4,5334 | *a*4 | 1,3568 | 3,9245 |
| *a*5 | 2,8557 | 13,3393 | *a*5 | 31,0118 | 57,4639 |
| *a*6 | 0,0225 | 0,0175 | *a*6 | 0,1464 | 0,3034 |
| *a*7 | -0,1351 | -0,2240 | *a*7 | 0,3020 | 0,5877 |
| *a*8 | -1,9875 | -5,0396 | *a*8 | -7,9999 | -15,5374 |
| *ΔF* = 1,25 m | *a*0 | -12,5237 | -11,8077 | *ΔF* = 3,50 m | *a*0 | -36,4058 | -43,5066 |
| *a*1 | 3,6702 | 3,6747 | *a*1 | 16,5995 | 24,3589 |
| *a*2 | -1,0989 | -8,7274 | *a*2 | -8,6270 | -24,5889 |
| *a*3 | -0,6486 | -0,7496 | *a*3 | -3,1650 | -5,5465 |
| *a*4 | 1,9307 | 4,2143 | *a*4 | 1,3425 | 4,6262 |
| *a*5 | 6,4181 | 18,3916 | *a*5 | 40,0859 | 72,8599 |
| *a*6 | 0,0329 | 0,0437 | *a*6 | 0,1952 | 0,4106 |
| *a*7 | -0,0730 | -0,0958 | *a*7 | 0,4155 | 0,7509 |
| *a*8 | -2,7857 | -6,2540 | *a*8 | -9,8821 | -19,2053 |
| *ΔF* = 1,50 m | *a*0 | -14,5634 | -14,4616 | *ΔF* = 4,00 m | *a*0 | -42,9989 | -52,2117 |
| *a*1 | 4,6127 | 5,2084 | *a*1 | 20,5191 | 30,5038 |
| *a*2 | -1,6038 | -9,4959 | *a*2 | -11,5853 | -31,5342 |
| *a*3 | -0,8156 | -1,0847 | *a*3 | -3,9668 | -7,0337 |
| *a*4 | 1,8448 | 4,0155 | *a*4 | 1,4446 | 5,7725 |
| *a*5 | 9,3248 | 22,6500 | *a*5 | 49,9537 | 89,8675 |
| *a*6 | 0,0428 | 0,0682 | *a*6 | 0,2490 | 0,5283 |
| *a*7 | -0,0248 | 0,0016 | *a*7 | 0,5256 | 0,8834 |
| *a*8 | -3,4229 | -7,2690 | *a*8 | -11,9228 | -23,2660 |

Table E.6 — Coefficients of Formula (E.4) for strike-slip fault mechanism

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| **Differential displacement** | **Coeffi-cients** | **Recurrence rate class** | | **Differential displacement** | **Coeffi-cients** | **Recurrence rate class** | |
| Low | High | Low | High |
| *ΔF* = 0,25 m | *a*0 | -5,1391 | -9,3774 | *ΔF* = 1,75 m | *a*0 | -20,6864 | -14,1208 |
| *a*1 | 2,2983 | 3,9922 | *a*1 | 10,6812 | 4,7091 |
| *a*2 | -0,9885 | 11,1942 | *a*2 | -1,5496 | 8,7087 |
| *a*3 | -0,6845 | -0,8118 | *a*3 | -2,5042 | -0,8686 |
| *a*4 | 2,4665 | -1,9394 | *a*4 | 2,6156 | -0,8941 |
| *a*5 | -2,4378 | -9,3626 | *a*5 | 0,6696 | -7,0357 |
| *a*6 | 0,0536 | 0,0495 | *a*6 | 0,1879 | 0,0522 |
| *a*7 | -0,2615 | 0,1078 | *a*7 | -0,1959 | 0,0722 |
| *a*8 | -0,5319 | 1,0160 | *a*8 | -1,5466 | 0,1559 |
| *ΔF* = 0,50 m | *a*0 | -8,2578 | -10,1806 | *ΔF* = 2,00 m | *a*0 | -22,9875 | -14,8385 |
| *a*1 | 3,9467 | 3,9855 | *a*1 | 11,9600 | 4,8859 |
| *a*2 | -0,5955 | 10,7400 | *a*2 | -1,8706 | 8,4761 |
| *a*3 | -1,0469 | -0,7921 | *a*3 | -2,7764 | -0,8908 |
| *a*4 | 2,3360 | -1,6912 | *a*4 | 2,7207 | -0,8113 |
| *a*5 | -1,8781 | -8,9335 | *a*5 | 1,0450 | -6,8059 |
| *a*6 | 0,0806 | 0,0479 | *a*6 | 0,2076 | 0,0534 |
| *a*7 | -0,2296 | 0,0996 | *a*7 | -0,1988 | 0,0699 |
| *a*8 | -0,7664 | 0,7988 | *a*8 | -1,6473 | 0,0862 |
| *ΔF* = 0,75 m | *a*0 | -10,9863 | -11,0185 | *ΔF* = 2,50 m | *a*0 | -27,1565 | -16,1709 |
| *a*1 | 5,4089 | 4,0921 | *a*1 | 14,2669 | 5,2306 |
| *a*2 | -0,6038 | 10,1893 | *a*2 | -2,5219 | 8,1105 |
| *a*3 | -1,3679 | -0,7991 | *a*3 | -3,2639 | -0,9353 |
| *a*4 | 2,3303 | -1,4567 | *a*4 | 2,9392 | -0,6866 |
| *a*5 | -1,2750 | -8,4390 | *a*5 | 1,6667 | -6,4369 |
| *a*6 | 0,1046 | 0,0483 | *a*6 | 0,2427 | 0,0558 |
| *a*7 | -0,2126 | 0,0907 | *a*7 | -0,2084 | 0,0670 |
| *a*8 | -0,9722 | 0,6136 | *a*8 | -1,8084 | -0,0217 |
| *ΔF* = 1,00 m | *a*0 | -13,5015 | -11,8186 | *ΔF* = 3,00 m | *a*0 | -31,4180 | -17,5211 |
| *a*1 | 6,7661 | 4,2274 | *a*1 | 16,6515 | 5,6120 |
| *a*2 | -0,7515 | 9,7195 | *a*2 | -3,2521 | 7,8030 |
| *a*3 | -1,6635 | -0,8127 | *a*3 | -3,7658 | -0,9866 |
| *a*4 | 2,3699 | -1,2698 | *a*4 | 3,1914 | -0,5850 |
| *a*5 | -0,7217 | -8,0084 | *a*5 | 2,2288 | -6,1299 |
| *a*6 | 0,1265 | 0,0491 | *a*6 | 0,2786 | 0,0585 |
| *a*7 | -0,2027 | 0,0839 | *a*7 | -0,2235 | 0,0646 |
| *a*8 | -1,1461 | 0,4657 | *a*8 | -1,9478 | -0,1081 |
| *ΔF* = 1,25 m | *a*0 | -16,3084 | -12,7178 | *ΔF* = 3,50 m | *a*0 | -35,6177 | -18,8653 |
| *a*1 | 8,2895 | 4,4017 | *a*1 | 19,0106 | 6,0152 |
| *a*2 | -1,0127 | 9,2691 | *a*2 | -4,0269 | 7,5352 |
| *a*3 | -1,9925 | -0,8321 | *a*3 | -4,2601 | -1,0425 |
| *a*4 | 2,4470 | -1,0983 | *a*4 | 3,4636 | -0,4996 |
| *a*5 | -0,1404 | -7,5833 | *a*5 | 2,7332 | -5,8693 |
| *a*6 | 0,1506 | 0,0502 | *a*6 | 0,3138 | 0,0616 |
| *a*7 | -0,1968 | 0,0782 | *a*7 | -0,2421 | 0,0626 |
| *a*8 | -1,3189 | 0,3268 | *a*8 | -2,0682 | -0,1786 |
| *ΔF* = 1,50 m | *a*0 | -18,4723 | -13,4162 | *ΔF* = 4,00 m | *a*0 | -39,7137 | -20,1969 |
| *a*1 | 9,4655 | 4,5483 | *a*1 | 21,3144 | 6,4324 |
| *a*2 | -1,2646 | 8,9707 | *a*2 | -4,8251 | 7,2923 |
| *a*3 | -2,2447 | -0,8491 | *a*3 | -4,7406 | -1,1015 |
| *a*4 | 2,5248 | -0,9887 | *a*4 | 3,7467 | -0,4245 |
| *a*5 | 0,2766 | -7,2929 | *a*5 | 3,1922 | -5,6418 |
| *a*6 | 0,1691 | 0,0511 | *a*6 | 0,3479 | 0,0648 |
| *a*7 | -0,1952 | 0,0749 | *a*7 | -0,2631 | 0,0606 |
| *a*8 | -1,4379 | 0,2354 | *a*8 | -2,1743 | -0,2376 |

1. If the crossing point is not known, then the worst-case value of *X*L = 0,50 should be considered in Formula (E.4).
2. Since Formula (E.2) can only provide *T*R(*Δ*F) given *Δ*F, while in practice the opposite is needed, to determine the design fault displacement *Δ*F corresponding to a given return period *T*LS,CC per the pipeline’s Consequence Class and the Limit State to be verified (4.3), linear interpolation in [*Δ*F, ln*T*R*(Δ*F)] space may be used among the values estimated via Formula (E.2). If it is found that *Δ*F values lower than 0,25 m or higher than 4,00 m (i.e., outside the displacement range appearing in Tables E.4 through E.6) are required to achieve the needed return period, linear extrapolation in [*Δ*F, 1/ln*T*R*(Δ*F)] space may be used, as a conservative option.

NOTE 1 The relationships are shown in Figure E.3 and E.4.

NOTE 2 When extrapolating to very low return periods, i.e., *Δ*F lower than 0,25 m, negative estimates of *Δ*F can be obtained. In such cases, a value of 0,10 m can be used instead, as per (8).

NOTE 3 When extrapolating to very high return periods, i.e., *Δ*F higher than 4,00 m, very large estimates of *Δ*F can be obtained, especially for normal or reverse faults. In such cases, commissioning a more detailed seismological study is recommended.

Figure E.3 — Determination of the *ln* *T*R(*Δ*F) relationship   
via linear interpolation

Figure E.4 — Linear extrapolation of the 1/*ln* *T*R(*Δ*F) relationship   
to values lower than *Δ*F = 0,25 m and higher than *Δ*F = 4,00 m

1. If *v*F has been approximated via Formula (E.1) then the smallest of two values may be used for design: (i) *Δ*F found via clause (6) and (ii) a deterministic cap *Δ*Fcap estimated from Table E.7.
2. If a design value of *Δ*F lower than 0,10 m is found via clauses (6) or (7), then a minimum value of 0,10 m should be used instead.

Table E.7 — Estimation of the fault displacement deterministic cap via the fault length *L*F

|  |  |
| --- | --- |
| **Fault mechanism** | **Fault displacement cap**, ***ΔFcap* (m)** |
| normal or reverse |  |
| strike-slip |  |
| *L*F is given in km, while *Δ*Fcap is in units of m. | |

1. (informative)  
     
   Number of degrees of freedom and of modes of vibration for dynamic analysis of towers, masts and chimneys
   1. Use of this annex
2. This informative annex provides supplementary guidance to Clause 10.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

1. Annex F gives rules for the design of concrete or steel towers, masts and chimneys.
   1. Modelling and analysis
2. A dynamic analysis (e.g. response spectrum or response-history method) should be used when the use of the lateral force method is not considered justified.
3. The analysis should consider a) to e):
4. include a sufficient number of modes to ensure participation of all significant modes;
5. include a sufficient number of masses and degrees of freedom, to determine the response of any structural member and plant equipment;
6. provide the maximum relative displacement between supports of equipment or machinery (for a chimney, the interaction between internal and external tubes);
7. take into account significant effects, such as piping interactions, externally applied structural restraints, hydrodynamic loads (both mass and stiffness effects) and possible non-linear behaviour;
8. provide "floor response spectra", when the structure supports important light equipment or appendices.

NOTE 1 9.3.3 gives rules for floor spectra.

NOTE 2 The criterion to activate at least 90% of the total mass does not ensure the adequacy of the mass discretisation if light equipment or a structural appendix is concerned. In that case, the first condition above might be fulfilled, but the mathematical model of the structure can be inadequate to describe the response of the equipment or appendix.

1. When the analysis of the equipment or appendix is necessary, a "floor response spectrum", applicable for the floor elevation where the equipment/appendix is supported, should be developed.

NOTE This approach is also recommended when a portion of the structure needs to be analysed independently, for instance, an internal masonry flue of a chimney, supported on individual brackets of the structural shell.

1. (informative)  
     
   Masonry chimneys
   1. Use of this annex
2. This informative annex provides supplementary guidance to Clause 10.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

1. Annex G gives rules for the design of masonry chimneys.
   1. Modelling and analysis
2. The behaviour factor *q* should not be higher than 1,5.
3. Chimneys should not support vertical loads in addition to their own weight unless they are designed for them. Masonry chimneys may be constructed as part of the masonry walls or concrete walls of the building.
   1. Design detailing
      1. Footings and foundations
4. Foundations for masonry chimneys should be constructed of concrete according to EN 1997-3:—[[19]](#footnote-20) and EN 1998‑5:—4. They should be at least 300 mm thick and should extend at least 150 mm beyond the face of the chimney or support wall on all sides. Footings should be founded on natural undisturbed ground or engineered fill below frost depth. In areas not subjected to freezing, footings should be at least 300 mm below the ground surface.
   * 1. Minimum vertical reinforcement
5. For chimneys with a horizontal dimension up to 1 m, a total of four 12 mm diameter continuous vertical bars anchored in the foundation should be placed in concrete between leaves of solid masonry or placed and grouted within the cells of hollow masonry units. Grout should be prevented from bonding with the flue liner, to avoid restricting its thermal expansion. For chimneys with a horizontal dimension greater than 1 m, two additional 12 mm diameter continuous vertical bars should be provided for each additional metre in horizontal dimension or fraction thereof.
   * 1. Minimum horizontal reinforcement
6. Vertical reinforcement should be enclosed within 6 mm diameter ties, or other reinforcement of equivalent cross-sectional area, at a spacing of not more than 400 mm.
   * 1. Minimum seismic anchorage
7. A masonry chimney passing through the floors and roof of a building should be anchored at each level of floor or roof at a height more than 2 m above the ground, when constructed completely within the exterior walls. Two 5 mm by 25 mm steel straps should be embedded into the chimney over a minimum length of 300 mm. Straps should be anchored by hooks around the outer bars and should extend by 150 mm beyond the bent at the hook. Each strap should be fastened to a minimum of four floor joists with two 12 mm bolts.
   * 1. Cantilevering
8. A masonry chimney should not project as a corbel from a wall or foundation by more than half of the chimney wall thickness. A masonry chimney should not project as a corbel from a wall or foundation that is less than 300 mm in thickness unless it projects equally on each side of the wall. As an exception, at the second storey of two-storey buildings, corbelling of chimneys outside the exterior walls may be equal to the wall thickness. The projection of a single course should not exceed one-half of the height of the masonry unit, or one-third of its bed depth, whichever is less.
   * 1. Changes in dimension
9. The chimney wall or chimney flue liner should not change in size or shape within 150 mm above or below the level where the chimney passes through a floor or a roof, or their components.
   * 1. Offsets
10. Where a masonry chimney is constructed with a fireclay flue liner surrounded by one leaf of masonry, the maximum offset should be such that the centreline of the flue above the offset does not extend beyond the centre of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in a manner for which the chimney has been designed, the maximum offset limitations may be neglected.
    * 1. Wall thickness
11. Masonry chimney walls should be constructed of solid masonry units, or hollow masonry units grouted solid with not less than 100 mm nominal thickness.

Bibliography

**References contained in recommendations (i.e. “should” clauses)**

The following documents are referred to in the text in such a way that some or all of their content constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative documents could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1594:2013, *Gas supply systems for operating pressure over 16 bar, functional requirements*

EN 1991-1-1:—[[20]](#footnote-21), *Eurocode 1 – Actions on structures – Part 1-1: Specific weight of materials, self-weight of construction works and imposed loads on buildings*

EN 1992-1-1:—8, *Eurocode 2 – Design of concrete structures – Part 1-1: General rules and rules for buildings*

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EN 1992-3, *Eurocode 2 – Design of concrete structures – Part 3: Liquid retaining and containing structures*

EN 1993-1-1:2022, *Eurocode 3 — Design of steel structures – Part 1-1: General rules and rules for buildings*

EN 1993-1-2:—15, *Eurocode 3 – Design of steel structures – Part 1-2: Structural fire design*

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EN 1993-1-8:—10, *Eurocode 3 – Design of steel structures – Part 1-8: Joints*

EN 1993-1-10:—17, *Eurocode 3 — Design of steel structures – Part 1-10: Material toughness and through-thickness properties*

EN 1993-1-11:—13, *Eurocode 3 – Design of steel structures – Part 1-11: Tension components*

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EN 13084-2, *Free-standing chimneys – Concrete chimneys*

EN 13084-7, *Free-standing chimneys – Product specification of cylindrical steel fabrications for use in single-wall steel chimneys and steel liners*

ISO 13847, *Welding steel pipelines*

**References contained in permissions (i.e. “may” clauses)**

The following documents are referred to in the text in such a way that some or all of their content expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1993-1-5:—11, *Eurocode 3 – Design of steel structures – Part 1-5: Plated structural elements*

**References contained in possibilities (i.e. "can" clauses) and notes**

The following documents are cited informatively in the document, for example in "can" clauses and in notes.

1. Under development. [↑](#footnote-ref-2)
2. . Under preparation. Stage at the time of publication: prEN 1998-1-1:2022. [↑](#footnote-ref-3)
3. Under preparation. Stage at the time of publication: prEN 1998-1-2:2023. [↑](#footnote-ref-4)
4. Under preparation. Stage at the time of publication: prEN 1998-5:2022. [↑](#footnote-ref-5)
5. Under preparation. Stage at the time of publication: prEN 1998-3:2023. [↑](#footnote-ref-6)
6. In development. [↑](#footnote-ref-7)
7. Under preparation. Stage at the time of publication: prEN 1993-1-6:2023. [↑](#footnote-ref-8)
8. Under preparation. Stage at the time of publication: FprEN 1992-1-1:2023. [↑](#footnote-ref-9)
9. Under development. [↑](#footnote-ref-10)
10. Under preparation. Stage at the time of publication: prEN 1998-1-8:2023. [↑](#footnote-ref-11)
11. Under preparation. Stage at the time of publication: prEN 1993-1-5:2022. [↑](#footnote-ref-12)
12. Under preparation. Stage at the time of publication: prEN 1993-1-7:2023 [↑](#footnote-ref-13)
13. Under development. [↑](#footnote-ref-14)
14. Under preparation. Stage at the time of publication: FprEN 1992-1-2:2023. [↑](#footnote-ref-15)
15. Under preparation. Stage at the time of publication: prEN 1993-1-2:2022. [↑](#footnote-ref-16)
16. Under development. [↑](#footnote-ref-17)
17. Under preparation. Stage at the time of publication: prEN 1993-1-10:2023. [↑](#footnote-ref-18)
18. Under preparation. Stage at the time of publication: prEN 1993-1-4:2023. [↑](#footnote-ref-19)
19. Under preparation. Stage at the time of publication: prEN 1997-3:2022. [↑](#footnote-ref-20)
20. Under preparation. Stage at the time of publication: prEN 1991-1-1:2023. [↑](#footnote-ref-21)
21. Under development. [↑](#footnote-ref-22)