

The European Union

EDICT OF GOVERNMENT

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EN 1998-4 (2006) (English): Eurocode 8: Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]



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English Version

**Eurocode 8 - Design of structures for earthquake resistance -
Part 4: Silos, tanks and pipelines**

Eurocode 8 - Calcul des structures pour leur résistance aux
séismes - Partie 4: Silos, réservoirs et canalisations

Eurocode 8 - Auslegung von Bauwerken gegen Erdbeben -
Teil 4: Silos, Tankbauwerke und Rohrleitungen

This European Standard was approved by CEN on 15 May 2006.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

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Foreword

This European Standard EN 1998-4, Eurocode 8: Design of structures for earthquake resistance: Silos, tanks and pipelines, 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.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by January 2007, and conflicting national standards shall be withdrawn at latest by March 2010.

This document supersedes ENV 1998-4: 1997.

According to the CEN-CENELEC Internal Regulations, the National Standard Organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonization of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonized technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

¹ Agreement between the Commission of the European Communities and the European Committee for Standardization (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

- EN 1990 Eurocode: Basis of structural 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

Eurocode standards recognize the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognize that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 - Mechanical resistance and stability - and Essential Requirement N°2 - Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonized technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonized product standards³. Therefore, technical

² According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for hENs and ETAGs/ETAs.

³ According to Art. 12 of the CPD the interpretative documents shall :

- a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
- b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etB. ;
- c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving a full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex (informative).

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e. :

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), *e.g.* snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain

- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonized technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonized technical specifications for construction products and the technical rules for works⁴. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1998-4

The scope of EN 1998 is defined in **1.1.1** of EN 1998-1: 2004. The scope of this Part of EN 1998 is defined in **1.1**. Additional Parts of Eurocode 8 are listed in EN 1998-1: 2004, **1.1.3**.

⁴ See Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

EN 1998-4:2006 is intended for use by:

- clients (e.g. for the formulation of their specific requirements on reliability levels and durability) ;
- designers and constructors ;
- relevant authorities.

For the design of structures in seismic regions the provisions of this European Standard are to be applied in addition to the provisions of the other relevant parts of Eurocode 8 and the other relevant Eurocodes. In particular, the provisions of this European Standard complement those of EN 1991-4, EN 1992-3, EN 1993-4-1, EN 1993-4-2 and EN 1993-4-3, which do not cover the special requirements of seismic design.

National annex for EN 1998-4

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1998-4 should have a National Annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1998-4:2006 through clauses:

Reference	Item
1.1(4)	Additional requirements for facilities associated with large risks to the population or the environment.
2.1.2(4)P	Reference return period T_{NCR} of seismic action for the ultimate limit state (or, equivalently, reference probability of exceedance in 50 years, P_{NCR}).
2.1.3(5)P	Reference return period T_{DLR} of seismic action for the damage limitation state (or, equivalently, reference probability of exceedance in 10 years, P_{DLR}).
2.1.4(8)	Importance factors for silos, tanks and pipelines
2.2(3)	Reduction factor ν for the effects of the seismic action relevant to the damage limitation state
2.3.3.3(2)P	Maximum value of radiation damping for soil structure interaction analysis, ξ_{max}
2.5.2(3)P	Values of ϕ for silos, tanks and pipelines
3.1(2)P	Unit weight of the particulate solid in silos, γ , in the seismic design situation
4.5.1.3(3)	Amplification factor on forces transmitted by the piping to region of attachment on the tank wall, for the design of the region to remain elastic in the damage limitation state
4.5.2.3(2)P	Overstrength factor on design resistance of piping in the verification that the connection of the piping to the tank will not yield prior to the piping in the ultimate limit state

1 GENERAL

1.1 Scope

(1) The scope of Eurocode 8 is defined in EN 1998-1: 2004, **1.1.1** and the scope of this Standard is defined in this clause. Additional parts of Eurocode 8 are indicated in EN 1998-1: 2004, **1.1.3**.

(2) This standard specifies principles and application rules for the seismic design of the structural aspects of facilities composed of above-ground and buried pipeline systems and of storage tanks of different types and uses, as well as for independent items, such as for example single water towers serving a specific purpose or groups of silos enclosing granular materials, etc.

(3) This standard includes the additional criteria and rules required for the seismic design of these structures without restrictions on their size, structural types and other functional characteristics. For some types of tanks and silos, it also provides detailed methods of assessment and verification rules.

(4) This standard may not be complete for those facilities associated with large risks to the population or the environment, for which additional requirements are the responsibility of the competent authorities. This standard is also not complete for those construction works which have uncommon structural elements and which require special measures to be taken and special studies to be performed to ensure earthquake protection. In those two cases the present standard gives general principles but not detailed application rules.

NOTE The National Annex may specify additional requirements for facilities associated with large risks to the population or the environment.

(5) Although large diameter pipelines are within the scope of this standard, the corresponding design criteria do not apply for apparently similar facilities, like tunnels and large underground cavities.

(6) The nature of lifeline systems which often characterizes the facilities covered by this standard requires concepts, models and methods that may differ substantially from those in current use for more common structural types. Furthermore, the response and the stability of silos and tanks subjected to strong seismic actions may involve rather complex interaction phenomena between soil-structure and stored material (either fluid or granular), not easily amenable to simplified design procedures. Equally challenging may prove to be the design of a pipeline system through areas with poor and possibly unstable soils. For the reasons given above, the organization of this standard is to some extent different from that of other Parts of EN 1998. This standard is, in general, restricted to basic principles and methodological approaches.

NOTE Detailed analysis procedures going beyond basic principles and methodological approaches are given in Annexes A and B for a number of typical situations.

(7) In the formulation and implementation of the general requirements, a distinction has been made between independent structures and redundant systems, via the choice of importance factors and/or through the definition of specific verification criteria.

(8) If seismic protection of above-ground pipelines is provided through seismic isolation devices between the pipeline and its supports (notably on piles), EN 1998-2:2005 applies, as relevant. For the design of tanks, silos, or individual facilities or components of pipeline systems with seismic isolation, the relevant provisions of EN 1998-1:2004 apply.

1.2 Normative references

(1)P This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

1.2.1 General reference standards

- EN 1990: 2002 *Eurocode - Basis of structural design.*
- EN 1991-4: 2006 *Eurocode 1 - Actions on structures – Part 4: Silos and tanks.*
- EN 1992-1-1: 2004 *Eurocode 2 - Design of concrete structures – Part 1-1: General rules and rules for buildings.*
- EN 1992-3: 2006 *Eurocode 2 - Design of concrete structures – Part 3: Liquid retaining and containing structures.*
- EN 1993-1-1: 2004 *Eurocode 3 - Design of steel structures – Part 1-1: General rules and rules for buildings.*
- EN 1993-1-5: 2006 *Eurocode 3 - Design of steel structures – Part 1-5: Plated structural elements.*
- EN 1993-1-6: 2006 *Eurocode 3 - Design of steel structures – Part 1-6: Strength and stability of shell structures.*
- EN 1993-1-7: 2006 *Eurocode 3 - Design of steel structures – Part 1-7: Strength and stability of planar plated structures transversely loaded.*
- EN 1993-4-1: 2006 *Eurocode 3 - Design of steel structures – Part 4-1: Silos.*
- EN 1993-4-2: 2006 *Eurocode 3 - Design of steel structures – Part 4-2: Tanks.*
- EN 1993-4-3: 2006 *Eurocode 3 - Design of steel structures – Part 4-3: Pipelines.*
- EN 1997-1 : 2004 *Eurocode 7 - Geotechnical design – Part 1: General rules.*
- EN 1998-1 : 2004 *Eurocode 8 - Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.*
- EN 1998-2 : 2005 *Eurocode 8 - Design of structures for earthquake resistance – Part 2: Bridges.*

EN 1998-5 : 2004 *Eurocode 8 - Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects.*

EN 1998-6 : 2005 *Eurocode 8 - Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys.*

1.3 Assumptions

(1)P The general assumptions shall be in accordance with EN 1990: 2002, **1.3**.

1.4 Distinction between principles and applications rules

(1)P The distinction between principles and applications rules shall be in accordance with EN 1990: 2002, **1.4**.

1.5 Terms and Definitions

1.5.1 General

(1) For the purposes of this standard the following definitions apply.

1.5.2 Terms common to all Eurocodes

(1)P The terms and definitions given in EN 1990: 2002, **1.5** apply.

(2)P EN 1998-1: 2004, **1.5.1** applies for terms common to all Eurocodes.

1.5.3 Further terms used in EN 1998

(1) For the purposes of this European Standard the terms given in EN 1998-1: 2004, **1.5.1** and **1.5.2** apply.

1.5.4 Further terms used in EN 1998-4

Independent structure:

a structure whose structural and functional behaviour during and after a seismic event are not influenced by that of other structures, and whose consequences of failure relate only to the functions demanded from it.

1.6 Symbols

(1) For the purposes of this European Standard the following symbols apply:

A_{Ed} design value of seismic action (= $\gamma_l A_{Ek}$)

A_{Ek} characteristic value of the seismic action for the reference return period

b horizontal dimension of silo parallel to the horizontal component of the seismic action

d_c inside diameter of a circular silo

d_g design ground displacement, as given in EN 1998-1:2004, **3.2.2.4(1)**, used in expression (4.1)

g	acceleration of gravity
h_b	overall height of the silo, from a flat bottom or the hopper outlet to the equivalent surface of the stored contents
q	behaviour factor
r	radius of circular silo, silo compartment, tank or pipe
r_s^*	geometric quantity defined in silos through expression (3.5) as $r_s^* = \min(H, Br_s/2)$
t	thickness
x	vertical distance of a point on a silo wall from a flat silo bottom or the apex of a conical or pyramidal hopper
x	distance between the anchoring point of piping and the point of connection with the tank
z	vertical downward co-ordinate in a silo, measured from the equivalent surface of the stored contents
$\alpha(z)$	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
γ	bulk unit weight of particulate material in silo, taken equal to the upper characteristic value given in EN 1991-4:2006, Table E1.
γ_I	importance factor
γ_p	amplification factor on forces transmitted by the piping to region of attachment on tank wall, for the region to be designed to remain elastic, see 4.5.1.3(3)
Δ	minimum value of imposed relative displacement between the first anchoring point of piping and the tank to be taken from given by expression (4.1)
$\Delta_{ph,s}$	additional normal pressure on the silo wall due to the response of the particulate solid to the horizontal component of the seismic action
$\Delta_{ph,so}$	reference pressure on silo walls given in 3.3(8) , expression (3.6)
θ	angle ($0^\circ \leq \theta < 360^\circ$) between the radial line to the point of interest on the wall of a circular silo and the direction of the horizontal component of the seismic action.
λ	the correction factor on base shear from the lateral force method of analysis, in EN 1998-1: 2004, 4.3.3.2.2(1) .
ν	reduction factor for the effects of the seismic action relevant to the damage limitation state
ξ	viscous damping ratio (in percent)
$\psi_{2,i}$	combination coefficient for the quasi-permanent value of a variable action i
$\psi_{E,i}$	combination coefficient for a variable action i , to be used when determining the effects of the design seismic action

1.7 S.I. Units

(1)P S.I. Units shall be used in accordance with ISO 1000.

(2) In addition, the units recommended in EN 1998-1:2004, **1.7** apply.

2 GENERAL PRINCIPLES AND APPLICATION RULES

2.1 Safety requirements

2.1.1 General

(1)P This standard deals with structures which may differ widely in such basic features as:

- the nature and amount of the contents and associated potential danger
- the functional requirements during and after the seismic event
- the environmental conditions.

(2) Depending on the specific combination of the indicated features, different formulations of the general requirements are appropriate. For the sake of consistency with the general framework of the Eurocodes, the two-limit-states format is retained, with a suitably adjusted definition.

2.1.2 Ultimate limit state

(1)P The ultimate limit state for which a system shall be checked is defined as that corresponding to structural failure. In some circumstances, partial recovery of the operational capacity of the system lost by exceedance of the ultimate limit state may be possible, after an acceptable amount of repairs.

NOTE 1: The circumstances are those defined by the responsible authority or the client.

(2)P For particular elements of the network, as well as for independent structures whose complete collapse would entail severe consequences, the ultimate limit state is defined as that of a state prior to structural collapse that, although possibly severe, would exclude brittle failures and would allow for a controlled release of the contents. When the failure of the aforementioned elements does not entail severe consequences, the ultimate limit state may be defined as corresponding to total structural collapse.

(3)P The design seismic action for which the ultimate limit state may not be exceeded shall be established based on the direct and indirect consequences of structural failure.

(4)P The design seismic action, A_{Ed} , shall be expressed in terms of: a) the reference seismic action, A_{Ek} , associated with a reference probability of exceedance, P_{NCR} , in 50 years or a reference return period, T_{NCR} , (see EN 1998-1:2004, **2.1(1)P** and **3.2.1(3)**) and b) the importance factor γ_I (see EN 1990:2002 and EN 1998-1:2004, **2.1(2)P**, **2.1(3)P** and **(4)**) to take into account reliability differentiation:

$$A_{Ed} = \gamma_I A_{Ek} \quad (2.1)$$

NOTE: The value to be ascribed to the reference return period, T_{NCR} , associated with the reference seismic action for use in a country, may be found in its National Annex. The recommended value is: $T_{NCR} = 475$ years.

(5) The capacity of structural systems to resist seismic actions at the ultimate limit state in

the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

(6) To avoid explicit inelastic analysis in design, the capacity of the structural systems to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, may be taken into account by performing a linear-elastic analysis based on a response spectrum reduced with respect to the elastic one, called "design spectrum". This reduction is accomplished by introducing the behaviour factor q , which is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional linear-elastic analysis model, still ensuring a satisfactory performance of the structural system at the ultimate limit state.

(7) The values of the behaviour factor q , which also account for the influence of the viscous damping being different from 5%, are given for the various types of constructions covered by EN 1998-4 in the relevant Sections of this Eurocode.

2.1.3 Damage limitation state

(1)P Depending on the characteristics and the purposes of the structure considered, a damage limitation state that meets one or both of the two following performance levels may need to be satisfied:

- ‘integrity’;
- ‘minimum operating level’.

(2)P In order to satisfy the ‘integrity’ requirement, the considered system, including a specified set of accessory elements integrated with it, shall remain fully serviceable and leak proof under the relevant seismic action.

(3)P To satisfy the ‘minimum operating level’ requirement, the extent and amount of damage of the considered system, including some of its components, shall be 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.

(4)P The seismic action for which this limit state may not be exceeded shall have an annual probability of exceedance whose value is to be established based on the following:

- the consequences of loss of function and/or of leakage of the content, and
- the losses related to the reduced capacity of the system and to the necessary repairs.

(5)P The seismic action for which the ‘damage limitation’ state may not be exceeded shall have a probability of exceedance, P_{DLR} , in 10 years and a return period, T_{DLR} . In the absence of more precise information, the reduction factor applied on the design seismic action in accordance with 2.2(3) may be used to obtain the seismic action for the verification of the damage limitation state.

NOTE: The values to be ascribed to P_{DLR} or to T_{DLR} for use in a country may be found in its National Annex of this document. The recommended values are $P_{DLR} = 10\%$ and $T_{DLR} = 95$ years.

2.1.4 Reliability differentiation

(1)P Pipeline networks and independent structures, either tanks or silos, shall be provided with a level of protection proportioned to the number of people at risk and to the economic losses associated with their performance level being not achieved.

(2)P Reliability differentiation shall be achieved by appropriately adjusting the value of the annual probability of exceedance of the design seismic action.

(3) This adjustment should be implemented by classifying structures into different importance classes and applying to the reference seismic action an importance factor γ_I , defined in **2.1.2(4)P** and in EN 1998-1: 2004, **2.1(3)P**, the value of which depends on the importance class. Specific values of the factor γ_I , necessary to modify the action so as to correspond to a seismic event of selected return period, depend on the seismicity of each region. The value of the importance factor $\gamma_I = 1,0$ is associated to the seismic action with the reference return period indicated in **2.1.2(4)P**.

NOTE For the dependence of the value of γ_I see Note to EN1998-1:2004, **2.1(4)**

(4) For the structures within the scope of this standard it is appropriate to consider three different importance classes, depending on the potential loss of life due to the failure of the particular structure and on the economic and social consequences of failure. Further classification may be made within each Importance Class, depending on the use and contents of the facility and the implications for public safety.

NOTE Importance classes I, II and III/IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, Annex B.

(5) Class I refers to situations where the risk to life is low and the economic and social consequences of failure are small or negligible.

(6) Situations with medium risk to life and local economic or social consequences of failure belong to Class II.

(7) Class III refers to situations with a high risk to life and large economic and social consequences of failure.

(8) Class IV refers to situations with exceptional risk to life and extreme economic and social consequences of failure.

NOTE The values to be ascribed to γ_I for use in a country may be found in its National Annex. The values of γ_I may be different for the various seismic zones of the country, depending on the seismic hazard conditions (see Note to EN 1998-1: 2004, **2.1(4)**) and on the public safety considerations detailed in **2.1.4**. The value of γ_I for importance class II is, by definition, equal to 1,0. For the other classes the recommended values of γ_I are $\gamma_I = 0,8$ for Importance Class I, $\gamma_I = 1,2$ for importance class III and $\gamma_I = 1,6$ for importance class IV,

(9)P A pipeline system traversing a large geographical region normally encounters a wide variety of seismic hazards and soil conditions. In addition, a number of subsystems may be located along a pipeline transmission system, which may be either associated facilities (tanks, storage reservoirs etc.), or pipeline facilities (valves, pumps, etc.). Under such circumstances, critical stretches of the pipeline (for instance, less redundant parts of the system) and critical

components (pumps, compressors, control equipment, etc.) shall be designed to provide larger reliability with regard to seismic events. Other components, that are less essential and for which some damage is acceptable, need not be designed to such stringent criteria.

2.1.5 System versus element reliability

(1)P The reliability requirements specified in **2.1.4** shall apply to the whole system under consideration, be it constituted by a single component or by a set of components variously connected to perform the functions required from it.

(2) Although a formal approach to system reliability analysis is outside the scope of this standard, the designer should give explicit consideration to the role played by the various elements in ensuring the continued operation of the system, especially when it is not redundant. In the case of very complex systems the design should be based on sensitivity analyses.

(3)P Elements of the network, or of a structure in the network, which are shown to be critical, with respect to the failure of the system, shall be provided with an additional margin of protection, commensurate with the consequences of the failure. When there is no previous experience, those critical elements shall be experimentally investigated to verify the acceptability of the design assumptions.

(4) If more rigorous analyses are not undertaken, the additional margin of protection for critical elements may be achieved by assigning these elements to a class of reliability (expressed in terms of Importance Class) one level higher than that appropriate to the system as a whole. Alternatively the Capacity Design rules may be used for the design of critical elements of a structure in the network, taking into account the actual resistance of elements not considered as critical.

2.1.6 Conceptual design

(1)P Even when the overall seismic response is specified to be elastic, structural elements shall be designed and detailed for local ductility and constructed from ductile materials.

(2)P The design of a network or of an independent structure shall take into consideration the following general aspects for mitigation of earthquake effects:

- functional redundancy of the systems;
- absence of interaction of the mechanical and electrical components with the structural elements;
- easy access for inspection, maintenance and repair of damages;
- quality control of the components.

(3) In order to avoid spreading of damage in functionally redundant systems due to structural interconnection of components, the appropriate parts should be functionally isolated.

(4) In case of important facilities vulnerable to earthquakes, of which damage recovery is difficult or time consuming, replacement parts or subassemblies should be provided.

2.2 Seismic action

(1)P The seismic action to be used for the design of silos, tanks and pipelines shall be that defined in EN 1998-1:2004, **3.2** in the various equivalent forms of site-dependent elastic response spectra (EN 1998-1:2004, **3.2.2**), and time-history representation (EN 1998-1:2004, **3.2.3.1**). Additional provisions for the spatial variation of ground motion for buried pipelines are given in Section 6.

(2)P The seismic action for which the ultimate limit state shall be verified is specified in **2.1.2(4)P**. If the determination of the seismic action effects is based on linear-elastic analysis with a behaviour factor q larger than 1 according to EN 1998-1:2004, **3.2.2.5(2)**, the design spectrum for elastic analysis shall be used in accordance with EN 1998-1: 2004, **3.2.2.5** (see also **2.1.2(6)P**).

(3) A reduction factor ν may be applied to the design seismic action corresponding to the ultimate limit state, to take into account the lower return period of the seismic action associated with the damage limitation state, as mentioned in EN 1998-1:2004, **2.1(1)P**. The value of the reduction factor ν may also depend on the Importance Class of the structure. Implicit in its use is the assumption that the elastic response spectrum of the seismic action under which the damage limitation state should be verified has the same shape as the elastic response spectrum of the design seismic action corresponding to the ultimate limit state according to EN 1998-1:2004, **2.1(1)P** and **3.2.1(3)** (See EN 1998-1:2004, **3.2.2.1(2)** and **4.4.3.2(2)**).

NOTE The values to be ascribed to ν for use in a country may be found in its National Annex. Different values of ν may be defined for the various seismic zones of a country, depending on the seismic hazard conditions and on the protection of property objective. The recommended values of ν are 0,5 for importance classes I and II and $\nu = 0,4$ for importance classes III and IV. Different values may result from special zoning studies.

2.3 Analysis

2.3.1 Methods of analysis

(1) For the structures within the scope of this standard the seismic actions effects should be determined on the basis of linear behaviour of the structures and of the soil in their vicinity.

(2) Nonlinear methods of analysis may be used to obtain the seismic action effects for those special cases where consideration of nonlinear behaviour of the structure or of the surrounding soil is dictated by the nature of the problem, or where the elastic solution would be economically unfeasible.

(3)P Analysis for the evaluation of the effects of the seismic action relevant to the damage limitation state shall be linear-elastic, using the elastic spectra defined in EN 1998-1: 2004, **3.2.2.2** and **3.2.2.3**, multiplied by the reduction factor ν referred to in **2.2(3)**. The elastic spectra should be entered with a weighted average value of the viscous damping that takes into account the different damping values of the different materials/elements according to **2.3.5** and to EN 1998-1: 2004, **3.2.2.2(3)**.

(4) Analysis for the evaluation of the effects of the seismic action relevant to the ultimate limit state may be linear-elastic in accordance with **2.1.2(6)** and EN 1998-1:2004, **3.2.2.5**,

using the design spectra which are specified in EN 1998-1:2004, **3.2.2.5** for a damping ratio of 5%. They make use of the behaviour factor q to account for the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, as well as the influence of viscous damping different from 5%(see also **2.1.2(6)P**).

(5)P Unless otherwise specified for particular types of structures in the relevant parts of this standard, the types of analysis that may be applied are those indicated in EN 1998-1: 2004, **4.3.3**, namely:

- a) the ‘lateral force method’ of (linear-elastic) analysis (see EN 1998-1:2004, **4.3.3.2**);
- b) the ‘modal response spectrum’ (linear-elastic) analysis (see EN 1998-1:2004, **4.3.3.3**);
- c) the non-linear static (pushover) analysis (see EN 1998-1:2004, **4.3.3.4.2**);
- d) the non-linear time history (dynamic) analysis (see EN 1998-1:2004 **4.3.3.4.3**).

(6)P Clauses **4.3.1(1)P**, **4.3.1(2)**, **4.3.1(6)**, **4.3.1(7)**, **4.3.1(9)P**, **4.3.3.1(5)** and **4.3.3.1(6)** of EN 1998-1:2004 shall apply for the modelling and analysis of the types of structures covered by the present standard.

(7) The ‘lateral force method’ of linear-elastic analysis should be performed according to clauses **4.3.3.2.1(1)P**, **4.3.3.2.2(1)** (with $\lambda=1,0$), **4.3.3.2.2(2)** and **4.3.3.2.3(2)P** of EN 1998-1: 2004. It is appropriate for structures that respond to each component of the seismic action approximately as a Single-Degree-of-Freedom system: rigid (i.e. concrete) elevated tanks or silos on relatively flexible and almost massless supports.

(8) The ‘modal response spectrum’ linear-elastic analysis should be performed according to Clauses **4.3.3.3.1(2)P**, **4.3.3.3.1(3)**, **4.3.3.3.1(4)** and **4.3.3.3.2** of EN 1998-1: 2004. It is appropriate for structures whose response is significantly affected by contributions from modes other than that of a Single-Degree-of-Freedom system in each principal direction.

(9) Non-linear analysis, static (pushover) or dynamic (time history), should satisfy EN 1998-1: 2004, **4.3.3.4.1**.

(10) Non-linear static (pushover) analysis should be performed according to **4.3.3.4.2.2(1)**, **4.3.3.4.2.3**, **4.3.3.4.2.6** of EN 1998-1:2004.

(11) Non-linear dynamic (time history) analysis should satisfy EN 1998-1:2004, **4.3.3.4.3**.

(12) The relevant provisions of EN 1998-1:2004 apply to the analysis of tanks, silos and individual facilities or components of pipeline systems that are base isolated.

(13) The relevant provisions of EN 1998-2:2005 apply to the analysis of above-ground pipelines provided with seismic isolation devices between the pipeline and its supports.

2.3.2 Interaction with the soil

(1)P Soil-structure interaction effects shall be addressed in accordance with EN 1998-5: 2004, Section 6.

NOTE Additional information on procedures for accounting for soil-structure interaction is presented in Informative Annex A, as well as in EN 1998-6:2005, Informative Annex C.

2.3.3 Damping

2.3.3.1 Structural damping

(1) If the damping values are not obtained from specific information, the following values of the damping ratio should be used in linear analysis:

- a) damage limitation state: the values specified in EN 1998-2:2005, **4.1.3(1)**;
- b) ultimate limit state: $\xi = 5\%$

2.3.3.2 Contents damping

(1) The value $\xi = 0,5\%$ may be adopted for the damping ratio of water and other liquids, unless otherwise determined.

NOTE: Reference to additional information for the determination of damping ratios of liquids is given in Informative Annex B.

(2) For granular materials an appropriate value for the damping ratio should be used. In the absence of more specific information a value of $\xi = 10\%$ may be used.

2.3.3.3 Foundation damping

(1) Material damping varies with the nature of the soil and the intensity of shaking. When more accurate determinations are not available, the values given in Table 4.1 of EN 1998-5:2004 should be used.

(2)P Radiation damping depends on the direction of motion (horizontal translation, vertical translation, rocking, etc.), on the geometry of the foundation, on soil layering and soil morphology. The values adopted in the analysis shall be compatible with actual site conditions and shall be justified with reference to acknowledged theoretical and/or experimental results. The values of the radiation damping used in the analysis shall not exceed a maximum value $\xi_{\max}$.

NOTE: The value to be ascribed to $\xi_{\max}$ for use in a country may be found in its National Annex. Guidance for the selection and use of damping values associated with different foundation motions is provided in EN 1998-6:2005. The recommended value is $\xi_{\max} = 25\%$.

2.3.3.4 Weighted damping

(1) The global average damping of the whole system should account for the contributions of the different materials/elements to damping.

NOTE: Procedures for accounting for the contributions of the different materials/elements to the global average damping of the system are presented in EN 1998-2:2005, **4.1.3(1)**, Note and in EN 1998-6:2005, Informative Annex B.

2.4 Behaviour factors

(1)P For the damage limitation state, the behaviour factor q shall be taken as equal to 1,0.

NOTE: For structures covered by this standard significant energy dissipation is not expected for the

damage limitation state.

(2) Use of q factors greater than 1,5 in ultimate limit state verifications is only allowed, provided that the sources of energy dissipation are explicitly identified and quantified and the capability of the structure to exploit them through appropriate detailing is demonstrated.

(3)P If seismic protection is provided through seismic isolation, the value of the behaviour factor at the ultimate limit state shall be taken as not greater than $q = 1,5$, except as provided in (4)P.

(4)P If seismic protection is provided through seismic isolation, q shall be taken as equal to 1 for the following:

- a) For the design of the substructure (i.e. of the elements below the plane of isolation).
- b) For the part of the superstructure response of tanks which is due to the convective part of the liquid response (sloshing).
- c) For the design of the isolators.

2.5 Safety verifications

2.5.1 General

(1)P Safety verifications shall be carried out for the limit states defined in 2.1, following the specific provisions in 3.5, 4.5, 5.6 and 6.5.

(2) If plate thickness is increased to account for future corrosion effects, the verifications should be made for both the non-increased and the increased thickness. Analysis may be based on a single value of the plate thickness.

2.5.2 Combinations of seismic action with other actions

(1)P The design value E_d of the effects of actions in the seismic design situation shall be determined according to EN 1990: 2002, 6.4.3.4, and the inertial effects of the design seismic action shall be evaluated according to EN 1998-1:2004, 3.2.4(2)P.

(2) In partially backfilled or buried tanks, permanent loads include, in addition to the weight of the structure, the weight of earth cover and any permanent external pressures due to groundwater.

(3)P The combination coefficients $\psi_{2,i}$ (for the quasi-permanent value of variable action i) shall be those given in EN 1991-4. The combination coefficients ψ_{Ei} , introduced in EN 1998-1: 2004 3.2.4(2)P for the calculation of the effects of the seismic actions, shall be taken as being equal to $\psi_{2,i}$ multiplied by a factor ϕ

NOTE: The values to be ascribed to ϕ for use in a country may be found in its National Annex. The recommended values of ϕ are $\phi = 1$ for full silo, tank or pipeline and $\phi = 0$ for empty silo, tank or pipeline.

(4)P The effects of the contents shall be considered in the variable loads for two levels of filling: empty or full. In batteries of silo or tank cells, different likely distributions of full and empty cells shall be considered according to the operation rules of the facility. At least, the design situations where all cells are either empty or full shall be considered. Only the

symmetrical filling loads of silos or silo cells shall be considered in the seismic design situation.

3 SPECIFIC PRINCIPLES AND APPLICATION RULES FOR SILOS

3.1 Introduction

(1) A distinction is made between:

- silos directly supported on the ground or on the foundation, and
- elevated silos, supported on a skirt extending to the ground, or on a series of columns, braced or not.

The main effect of the seismic action on on-ground silos are the stresses induced in the shell wall due to the response of the contents of the silo (see (3) and 3.3(5) to (12) for the additional normal pressures on the shell walls). The main concern in the seismic design of elevated silos is the supporting structure and its ductility and energy dissipation capacity (see 3.4(4) and (5)).

(2)P The determination of the properties of the particulate solid stored in the silo, including its unit weight, γ , shall be in accordance with EN 1991-4:2006, Section 4.

NOTE: The values to be ascribed to γ for use in a country in the seismic design situation may be found in its National Annex. For the stored materials listed in EN 1991-4:2006, Table E1, the recommended value of γ is the upper characteristic value of unit weight γ_u specified in that table.

(3) Under seismic conditions, the pressure exerted by the particulate material on the walls, the hopper and the bottom, may increase over the value relative to the condition when there is no seismic action. For design purposes this increased pressure is deemed to be found only from the inertia forces acting on the stored material due to the seismic action (see 3.3(5)).

(4)P The equivalent surface of the stored contents (as defined in EN 1991-4:2006, 1.5), in the seismic design situation shall be consistent with the value of the combination coefficients ψ_{Ei} used for the calculation of the effects of the seismic actions in accordance with 2.5.2(3)P.

3.2 Combination of ground motion components

(1)P In axisymmetric silos or parts thereof, only one horizontal component of the seismic action may be taken to act together with the vertical component. In all other cases, silos shall be designed for simultaneous action of the two horizontal components and of the vertical component of the seismic action.

(2) When the structural response to each component of the seismic action is evaluated separately, EN1998-1:2004, 4.3.3.5.2(4) may be applied for the determination of the most unfavourable effect of the application of the simultaneous components.

(3)P If expressions (4.20), (4.21), (4.22) in EN1998-1:2004, 4.3.3.5.2(4) are applied for the calculation of the action effects of the simultaneous components, the sign of the action effect due to each individual component shall be taken as the most unfavourable for the particular action effect under consideration.

(4)P If the analysis is performed simultaneously for the three components of the seismic

action using a spatial model of the structure, the peak values of the total response under the combined action of the horizontal and vertical components obtained from the analysis shall be used in the structural verifications.

3.3 Analysis of silos

(1) Analysis of silos should be accordance with **2.3** and **3.3**.

(2)P The model to be used for the determination of the seismic action effects shall reproduce accurately the stiffness, the mass and the geometrical properties of the containment structure, shall account for the response of the contained particulate material and for the effects of any interaction with the foundation soil. The modelling and analysis of steel silos shall be in accordance with EN 1993-4-1:2006, Section 4.

(3)P Silos shall be analysed by considering elastic behaviour of the silo shell and of its supporting structure, if any, unless proper justification is given for performing a nonlinear analysis.

(4) Unless more accurate evaluations are undertaken, the global seismic response and the seismic action effects in the supporting structure may be calculated assuming that the particulate contents move together with the silo shell and modelling them with their effective mass at their centre of gravity and its rotational inertia with respect to it. Unless a more accurate evaluation is made, the contents of the silo may be taken to have an effective mass equal to 80% of their total mass.

(5) Unless the mechanical properties and the dynamic response of the particulate solid are explicitly and accurately accounted for in the analysis (e.g. by using finite elements to model the mechanical properties and the dynamic response of the particulate solid), the effect on the shell of the response of the particulate solid to the horizontal component of the seismic action may be represented through an additional normal pressure on the wall, $\Delta_{ph,s}$ (positive for compression) specified in **(6)** to **(10)**, under the conditions of **(11)** and **(12)**. This additional pressure should be applied only over the part of the wall that is in contact with the stored contents, i.e. up to the equivalent surface of the stored contents, in the seismic design situation (see **3.1(4)P**).

(6) In circular silos (or silo compartments) the additional normal pressure on the wall may be taken as equal to:

$$\Delta_{ph,s} = \Delta_{ph,so} \cos \theta \quad (3.1)$$

where

$\Delta_{ph,so}$ is the reference pressure, see **(8)**;

θ is the angle ($0^\circ \leq \theta < 360^\circ$) between the radial line to the point of interest on the wall and the direction of the horizontal component of the seismic action.

(7) In rectangular silos (or silo compartments)) the additional normal pressure on the wall due to a horizontal component of the seismic action parallel or normal to the silo walls may be taken as equal to:

On the 'leeward' wall which is normal to the horizontal component of the seismic action:

$$\Delta_{ph,s} = \Delta_{ph,so} \quad (3.2)$$

On the 'windward' wall which is normal to the horizontal component of the seismic action:

$$\Delta_{ph,s} = -\Delta_{ph,so} \quad (3.3)$$

On the walls which are parallel to the horizontal component of the seismic action:

$$\Delta_{ph,s} = 0 \quad (3.4)$$

(8) At points on the silo wall at a vertical distance x from a flat bottom or the apex of a conical or pyramidal hopper, the reference pressure $\Delta_{ph,so}$ may be taken as:

$$\Delta_{ph,so} = \alpha(z)\gamma \min(r_s^*; 3x) \quad (3.5)$$

where:

$\alpha(z)$ is the ratio of the response acceleration of the silo at a vertical distance z from the equivalent surface of the stored contents, to the acceleration of gravity;

γ is the bulk unit weight of the particulate material in the seismic design situation (see **3.1(1)P**) and

r_s^* is defined as:

$$r_s^* = \min(h_b, d_c/2) \quad (3.6)$$

where:

h_b is the overall height of the silo, from a flat bottom or the hopper outlet to the equivalent surface of the stored contents, and

d_c is the inside 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).

(9) Expression (3.6) applies for vertical silo walls. Within the height of a hopper the reference pressure $\Delta_{ph,so}$ may be taken as:

$$\Delta_{ph,so} = \alpha(z)\gamma \min(r_s^*; 3x)/\cos\beta \quad (3.7)$$

where:

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

(10) If only the value of the response acceleration at the centre of gravity of the particulate material is available (see, e.g., **(4)** and **2.3.1(7)**) the corresponding ratio of response acceleration to the acceleration of gravity may be used in expression (3.7) for $\alpha(z)$.

(11)P At any point on the silo wall the sum of the static pressure of the particulate material on the wall and of the seismic action effect, $\Delta_{ph,s}$, shall not be taken less than zero.

- (12) If at any location on the silo wall the sum of
- $\Delta_{ph,s}$ given by (6) to (10) and expressions (3.1) to (3.3) and
 - the static pressure of the particulate material on the wall
- is negative (implying net suction on the wall), then (6) or (7) may not be considered to apply. In that case, the additional normal pressures on the wall, $\Delta_{ph,s}$, should be redistributed to ensure that their sum with the static pressure of the particulate material on the wall is everywhere non-negative, while maintaining the same force resultant over the same horizontal plane as the values of $\Delta_{ph,s}$ given in (6) or (7).

3.4 Behaviour factors

(1)P Non-base-isolated silos shall be designed according to one of the following concepts (see EN 1998-1:2004, 5.2.1, 6.1.2, 7.1.2):

- a) low-dissipative structural behaviour;
- b) dissipative structural behaviour.

(2) In concept a) the seismic action effects may be calculated on the basis of an elastic global analysis without taking into account significant non-linear material behaviour. When using the design spectrum defined in EN 1998-1:2004, 3.2.2.5, the value of the behaviour factor q may be taken up to 1,5. Design according to concept a) is termed design for ductility class Low (DCL). Selection of materials, evaluation of resistance and detailing of members and connections should be as specified in EN 1998-1:2004, Section 5 to 7, for ductility class Low (DCL).

(3) Silos directly supported on the ground or on the foundation should be designed according to concept a) and (2).

(4) Concept b) may be applied to elevated silos. According to this concept, the capability of parts of the supporting structure to resist earthquake actions beyond their elastic range (its dissipative zones), is taken into account. Supporting structures designed according to this concept should belong to ductility class Medium (DCM) or High (DCH) defined and described in EN 1998-1: 2004, Section 5 to 7, depending on the structural material of the supporting structure. They should meet the requirements specified therein regarding structural type, materials and dimensioning and detailing of members or connections for ductility. When using the design spectrum for linear-elastic analysis defined in EN 1998-1:2004, 3.2.2.5, the behaviour factor q may be taken as being greater than 1,5. The value of q depends on the selected ductility class (DCM or DCH).

(5) Due to the limited redundancy, the high axial forces due to the weight of the silo contents and the absence of non-structural elements contributing to earthquake resistance and energy dissipation, the energy dissipation capacity of the structural types commonly used to support elevated silos is, in general, less than that of a similar structural type when used in buildings. Therefore, in concept b) the upper limit value of the q factors for elevated silos are defined in terms of the q factors specified in EN 1998-1:2004, Sections 5 to 7, for the selected ductility class (DCM or DCH), as follows :

- For skirt-supported silos, with the skirt designed and detailed to ensure dissipative behaviour; the upper limit values of the q factor defined in EN 1998-1: 2004, Sections 5 to

7 for inverted pendulum structures may be used. If the skirt is not detailed for dissipative behaviour, it should be designed according to concept a) and (2).

- For silos supported on moment resisting frames or on frames with bracings, and for cast-in-place concrete silos supported on concrete walls which are continuous to the foundation, the upper limit of the q factors are those defined for the corresponding structural system in EN 1998-1:2004, Sections 5 to 7, times a factor equal to 0,7 for irregularity in elevation.

3.5 Verifications

3.5.1 Damage limitation state

(1)P In the seismic design situation relevant to the damage limitation state the silo structure shall be checked to satisfy the relevant serviceability limit state verifications required by EN 1992-1-1, EN 1992-3 and EN 1993-4-1.

NOTE: For steel silos, adequate reliability with respect to the occurrence of elastic or inelastic buckling phenomena is considered to be provided in the seismic design situation relevant to the damage limitation state, if the verifications regarding these phenomena are satisfied under the seismic design situation for the ultimate limit state.

3.5.2 Ultimate limit state

3.5.2.1 Global stability

(1)P Overturning or bearing capacity failure of the soil shall not occur in the seismic design situation. The resisting shear force at the interface of the base of the structure and the foundation, shall be evaluated taking into account the effects of the vertical component of the seismic action. Limited sliding 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 are taken into account in the analysis and the verifications (see also EN 1998-5: 2004, **5.4.1.1(7)**).

(2)P For uplift of on-ground silos to be considered acceptable, it shall be taken into account in the analysis and in the subsequent verifications of the structure, of any piping and of the foundation (e.g. in the assessment of overall stability).

3.5.2.2 Shell

(1)P The maximum action effects (membrane forces and bending moments, circumferential or meridional, and membrane shear) induced in the seismic design situation shall be less or equal to the resistance of the shell evaluated as in the persistent or transient design situations. This includes all types of failure modes.

(a) For steel shells:

- yielding (plastic collapse),
- buckling in shear, or

- buckling by vertical compression with simultaneous transverse tension ('elephant foot' mode of failure), etc.

(see EN 1993-4-1:2006, Sections 5 to 9).

(b) For concrete shells:

- the ULS in bending with axial force,
- the ULS in shear for in-plane or radial shear, etc.

(2)P The calculation of resistances and the verifications shall be carried out in accordance with EN 1992-1-1, EN 1992-3, EN 1993-1-1, EN 1993-1-5, EN 1993-1-6, EN 1993-1-7 and EN 1993-4-1.

3.5.2.3 Anchors

(1)P Anchoring systems shall generally be designed to remain elastic in the seismic design situation. However, they shall also be provided with sufficient ductility, so as to avoid brittle failures. The connection of anchoring elements to the structure and to its foundation shall have an overstrength factor of not less than 1,25 with respect to the resistance of the anchoring elements.

(2) If the anchoring system is part of the dissipative mechanisms, then it should be verified that it possesses the necessary ductility capacity.

3.5.2.4 Foundations

(1)P The foundation shall be verified according to EN 1998-5:2004, **5.4** and to EN 1997-1.

(2)P The action effects for the verification of the foundation and of the foundation elements shall be derived in accordance with EN 1998-5:2004, **5.3.1**, EN 1998-1:2004, **4.4.2.6** and **5.8**.

4 SPECIFIC PRINCIPLES AND APPLICATION RULES FOR TANKS

4.1 Compliance criteria

4.1.1 General

(1)P The general requirements specified in **2.1** are deemed to be satisfied if, in addition to the verifications specified in **4.4**, tanks conform to the complementary measures specified in **4.5**.

(2) The compliance criteria and application rules given in this Section do not fully cover the case of steel tanks with floating roofs.

NOTE: Special attention is needed to avoid damage to the shell due to local effects of the impact by the floating roof. Such effects may cause a fire in tanks with combustible contents.

4.1.2 Damage limitation state

(1)P In order to satisfy the ‘integrity’ requirement under the seismic action relevant to the damage limitation state:

- Leak tightness of the tank system shall be verified;
- adequate freeboard shall be provided in the tank under the maximum vertical displacement of the liquid surface, 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;
- the hydraulic systems which are part of, or are connected to the tank, shall be verified to accommodate stresses and distortions due to relative displacements between tanks or between tanks and soil, without their functions being impaired.

(2)P In order to satisfy the ‘minimum operating level’ requirement under the seismic action relevant to the damage limitation state, it shall be verified that local buckling, if it occurs, does not trigger collapse and is reversible.

4.1.3 Ultimate limit state

(1)P The following conditions shall be verified in the seismic design situation:

- The overall stability of the tank shall be verified in accordance with EN 1998-1: 2004, **4.4.2.4**. The overall stability refers to rigid body behaviour and may be impaired by sliding or overturning. A limited amount of sliding may be accepted in accordance with EN 1998-5: 2004, **5.4.1.1(7)**, if tolerated by the pipe system and if the tank is not anchored to the foundation.
- Inelastic behaviour is restricted to well-defined parts of the tank, in accordance with the provisions of the present standard.
- The ultimate deformations of the materials are not exceeded.

- The nature and the extent of buckling phenomena in the shell are controlled according to the relevant verifications.
- The hydraulic systems which are part of, or connected to the tank are designed so as to prevent loss of the contents of the tank in the event of failure of any of its components.

4.2 Combination of ground motion components

- (1)P Tanks shall conform to 3.2(1)P.
- (2) Tanks should conform to 3.2(2).
- (3)P Tanks shall conform to 3.2(3)P.

4.3 Methods of analysis

4.3.1 General

(1)P The model to be used for the determination of the seismic effects shall reproduce properly the stiffness, the strength, the damping, the mass and the geometrical properties of the containment structure, and shall account for the hydrodynamic response of the contained liquid and, where necessary, for the effects of the interaction with the foundation soil.

NOTE The parameters of soil-liquid-structure-interaction may have a significant influence on the natural frequencies and the radiation damping in the soil. With increasing shear wave velocity of the soil, the vibration behaviour changes from a horizontal vibration combined with rocking influenced by the soil to the typical vibration mode of a tank on rigid soil. For highly stressed tank structures or for the case of dangerous goods a global (three-dimensional) analysis may be necessary.

(2) Tanks should be generally analysed assuming linear elastic response. In particular cases nonlinear response may be justified by appropriate methods of analysis.

NOTE Information on methods for seismic analysis of tanks of usual shapes is provided in Informative Annex A.

(3) Possible interaction between different tanks due to connecting piping should be considered whenever relevant.

4.3.2 Hydrodynamic effects

(1)P A rational method based on the solution of the hydrodynamic equations with the appropriate boundary conditions shall be used for the evaluation of the response of the tank system to the seismic action.

(2)P In particular, the analysis shall properly account for the following, where relevant:

- the convective and the impulsive components of the motion of the liquid;
- the deformation of the tank shell due to the hydrodynamic pressures and the interaction effects with the impulsive component;
- the deformability of the foundation soil and the ensuing modification of the response

- the effects of a floating roof, if relevant.

(3) For the purpose of evaluating the dynamic response under seismic actions, the liquid may be generally assumed as incompressible.

(4) Determination of the maximum hydrodynamic pressures induced by horizontal and vertical excitation requires, in principle, use of nonlinear dynamic (time-history) analysis. Simplified methods allowing for a direct application of the response spectrum analysis may be used, provided that suitable conservative rules for the combination of the peak modal contributions are adopted.

NOTE Informative Annex A gives information on acceptable procedures for the combination of the peak modal contributions in response spectrum analysis. It also gives expressions for the calculation of the sloshing wave height

4.4 Behaviour factors

(1)P Tanks of type other than those mentioned in (4)P and (5) shall be either designed for elastic response (q up to 1,5, accounting for overstrength), or, in properly justified cases, for inelastic response (see 2.3.1(2)), provided that it is demonstrated that inelastic response is acceptable.

(2)P The energy dissipation corresponding to the selected value of q shall be properly substantiated and the necessary ductility provided through ductile design.

(3)P The convective part of the liquid response (sloshing) shall always be evaluated on the basis of elastic response (i.e. with $q = 1,0$) and of the associated spectra (see EN 1998-1: 2004, 3.2.2.2 and 3.2.2.3).

(4) The behaviour factors specified in 3.4 should be applied also to the part of the response of elevated tanks which is not due to sloshing of the liquid. For that part, the rules specified in 3.4(4) for skirt-supported silos apply also to elevated tanks on a single pedestal.

(5) Steel tanks (unless base-isolated) which have a vertical axis and are supported directly on the ground or on the foundation, may be designed with a behaviour factor q greater than 1,5, subject to the following:

- the part of the response which is due to sloshing of the liquid, should be taken with $q = 1,0$.
- the tank or its foundation is designed to allow uplift and/or sliding
- localisation of plastic deformations in the shell wall, the bottom plate or their intersection is prevented.

Under these conditions, the behaviour factor q may be taken as not larger than the following values, unless the inelastic response is evaluated by a more refined approach:

- 2,0 for unanchored tanks, provided that the design rules of EN 1993-4-2:2006 are fulfilled, especially those concerning the thickness of the bottom plate, which should be less than the thickness of the lower part of the shell.

- 2,5 for tanks with specially designed ductile anchors allowing an increase in anchor length without rupture equal to $R/200$, where R is the tank radius.

4.5 Verifications

4.5.1 Damage limitation state

4.5.1.1 General

(1)P Under the seismic action relevant to the damage limitation state, the tank structure shall satisfy the serviceability limit state verifications specified in EN 1992-3 and EN 1993-4-2, as relevant.

4.5.1.2 Shell

4.5.1.2.1 Reinforced and prestressed concrete shells

(1) Under the seismic action relevant to the damage limitation state, crack widths should be verified against the limit values specified in EN 1992-1-1: 2004, **4.4.2**, taking into account the appropriate environmental exposure class and the sensitivity of the steel to corrosion.

(2) In case of lined concrete tanks, transient concrete crack widths should not exceed a value that might induce local deformation in the liner exceeding 50% of its ultimate uniform elongation.

4.5.1.2.2 Steel shells

(1) Steel tanks should conform to **3.5.1(2)**.

4.5.1.3 Piping

(1) Unless special requirements are specified for active on-line components, such as valves or pumps, piping does not need to be verified for the damage limitation state.

(2)P Relative displacements due to differential seismic movements of the ground shall be accounted for, if the piping and the tank(s) are supported on different foundations.

(3) The region of the tank where the piping is attached to should be designed to remain elastic under the forces transmitted by the piping amplified by a factor γ_{p1} .

NOTE The value to be ascribed to the amplification factor γ_{p1} for use in a country, may be found in its National Annex. The recommended value is: $\gamma_{p1} = 1,3$.

4.5.2 Ultimate limit state

4.5.2.1 Stability

(1)P Tanks shall conform to **3.5.2.1(1)P**.

(2)P Tanks shall conform to **3.5.2.1(2)P**.

4.5.2.2 Shell

(1)P Tanks shall conform to **3.5.2.2(1)P**.

NOTE Information for the ultimate strength capacity of the shell, as controlled by various failure modes, is given in Informative Annex A.

4.5.2.3 Piping

(1) If reliable data are not available or more accurate analyses are not made, a relative displacement between the first anchoring point of the piping and the tank should be postulated to take place in the most adverse direction, with a minimum value of:

$$\Delta = \frac{x}{x_o} d_g \quad (4.1)$$

where:

x = distance between the anchoring point of the piping and the point of connection with the tank (in meters);

x_o = 500 m; and

d_g = design ground displacement as given in EN 1998-1: 2004, **3.2.2.4(1)**.

(2)P It shall be verified that in the seismic design situation, including the postulated relative displacements of **(1)**, yielding is restricted to the piping and does not extend to its connection to the tank, even when an overstrength factor γ_{p2} on the design resistance of the piping is taken into account.

NOTE The value to be ascribed to the overstrength factor γ_{p2} for use in a country, may be found in its National Annex. The recommended value is: $\gamma_{p2} = 1,3$.

(3)P The design resistance of piping elements shall be evaluated as in the persistent or transient design situations.

4.5.2.4 Anchorages

(1)P Tanks shall conform to **3.5.2.3(1)P**.

4.5.2.5 Foundations

(1)P Tanks shall conform to **3.5.2.4(1)P**.

(2)P Tanks shall conform to **3.5.2.4(2)P**.

4.6 Complementary measures

4.6.1 Bunding

(1)P Tanks, single or in groups, which are designed to control or avoid leakage in order to prevent fire, explosions and release of toxic materials shall be bunded (i.e. shall be surrounded by a ditch and/or an embankment).

(2)P If tanks are built in groups, bunding may be provided either to every individual tank or

to the whole group. If the consequences associated with potential failure of the bund are considered to be severe, individual bunding shall be used.

(3)P The bunding shall be designed to retain its full integrity (absence of leaks) under the design seismic action relevant to the ultimate limit state of the enclosed system.

4.6.2 Sloshing

(1)P In the absence of explicit justifications (see **4.1.2(1)P**), a freeboard shall be provided having a height not less than the calculated height of the slosh waves.

NOTE: Information on procedures to determine the sloshing wave height are presented in Informative Annex A.

(2)P Freeboard at least equal to the calculated height of the slosh waves shall be provided, if the contents are toxic, or if spilling could cause damage to piping or scouring of the foundation.

(3) Freeboard less than the calculated height of the slosh waves may be sufficient, if the roof is designed for the associated uplift pressure or if an overflow spillway is provided to control spilling.

(4) Damping devices, as for example grillages or vertical partitions, may be used to reduce sloshing.

4.6.3 Piping interaction

(1)P The piping shall be designed to minimize unfavourable effects of interaction between tanks and between tanks and other structures.

5 SPECIFIC PRINCIPLES AND APPLICATION RULES FOR ABOVE-GROUND PIPELINES

5.1 General

(1) This section aims at providing principles and application rules for the seismic design of the structural aspects of above-ground pipeline systems. This section may also be used as a basis for evaluating the resistance of existing above-ground piping and to assess any required strengthening.

(2) The seismic design of an above-ground pipeline comprises the establishment of the location and characteristics of the supports in order to limit the strain in the piping components and to limit the loads applied to the equipment located on the pipeline, such as valves, tanks, pumps or instrumentation. Those limits are not defined in this standard and should be provided by the owner of the facility or the manufacturer of the equipment.

(3) Pipeline systems usually comprise several associated facilities, such as pumping stations, operation centres, maintenance stations, etc., each of them housing different types of mechanical and electrical equipment. Since these facilities have a considerable influence on the continued operation of the system, it is necessary to give them adequate consideration in the seismic design process aimed at satisfying the overall reliability requirements. Explicit treatment of these facilities, however, is not within the scope of this standard. In fact, some of those facilities are covered in EN 1998-1, while the seismic design of mechanical and electrical equipment requires additional specific criteria that are beyond the scope of Eurocode 8 (see **1.1(8)** for the seismic protection of individual facilities or components of pipeline systems through seismic isolation).

(4)P For the formulation of the general requirements to follow, as well as for their implementation, pipeline systems shall be distinguished as follows:

- single lines
- redundant networks.

(5)P A pipeline shall be considered as a single line when its behaviour during and after a seismic event is not influenced by that of other pipelines, and if the consequences of its failure relate only to the functions demanded from it.

5.2 Safety requirements

5.2.1 Damage limitation state

(1)P Pipeline systems shall be constructed in such a way as to be able to maintain their supplying capability as a global servicing system, after the seismic action relevant to the ‘minimum operating level’ (see **2.1.3**), even with considerable local damage.

(2) A global deformation of the piping not greater than 1,5 times its yield deformation is acceptable, provided that there is no risk of buckling and the loads applied to active equipment, such as valves, pumps, etc., are within its operating range.

5.2.2 Ultimate limit state

(1)P The main safety hazard directly associated with the pipeline rupture during a seismic event is explosion and fire, particularly with regard to gas pipelines. The remoteness of the location and the exposure of the population to the impact of rupture shall be taken into account in establishing the level of the seismic action relevant to the ultimate limit state.

(2)P For pipeline systems in environmentally sensitive areas, the damage to the environment due to pipeline ruptures shall also be taken into account in the definition of the acceptable risk.

5.3 Seismic action

5.3.1 General

(1)P The following direct and indirect seismic hazard types are relevant for the seismic design of above-ground pipeline systems:

- Movement due to the inertia of the pipelines induced by the seismic movement applied to their supports.
- Differential movement of the supports of the pipelines.

(2) For differential movement of supports two different situations may exist:

- For supports which are directly on the ground, significant differential movement is possible only if there are soil failures and/or permanent deformations
- For supports which are located on different structures, the seismic response of the structure may create differential movements on the pipeline;

5.3.2 Seismic action for inertia movements

(1)P The quantification of the horizontal components of the seismic action shall be carried out in terms of the response spectrum (or a compatible time history representation) as specified in EN 1998-1: 2004, **3.2.2**.

(2) Only the three translational components of the seismic action should be taken into account (i.e., the rotational components may be neglected).

5.3.3 Differential movement

(1) When the pipeline is supported directly on the ground, the differential movement may be neglected, except when soil failures or permanent deformations are likely to occur. In that case the amplitude of the movement should be evaluated with appropriate techniques.

(2) When the pipeline is supported on different structures, their differential movement should be defined from their seismic response analysis or by simplified envelope approaches.

5.4 Methods of analysis

5.4.1 Modelling

(1)P The model of the pipeline shall be able to represent the stiffness, the damping and the mass properties, as well as the dynamic degrees of freedom of the system, with explicit consideration of the following aspects, as appropriate:

- flexibility of the foundation soil and foundation system;
- mass of the fluid inside the pipeline;
- dynamic characteristics of the supporting structures;
- type of connection between pipeline and supporting structure;
- joints along the pipeline and between the supports.

5.4.2 Analysis

(1) Above ground pipelines may be analysed by means of the modal response spectrum analysis with the associated design response spectrum as given in EN 1998-1: 2004, **3.2.2.5**, combining the modal responses according to EN 1998-1: 2004, **4.3.3.3.2**.

NOTE Additional rules regarding the combination of modal responses, namely for the use of the Complete Quadratic Combination is given in EN 1998-2:2005, **4.2.1.3**.

(2) Time history analysis with spectrum compatible accelerograms in accordance with EN 1998-1: 2004, **3.2.3** may also be applied.

(3) The “lateral force method” of (linear-elastic) analysis may also be applied, provided that the value of the applied acceleration is justified. A value equal to 1,5 times the peak of the spectrum applying at the support is acceptable. The principles and application rules specified in EN 1998-1: 2004, **4.3.3.2**, may be applied if considered appropriate.

(4)P The seismic action shall be applied separately along two orthogonal directions (transverse and longitudinal, for straight pipelines); the maximum combined response shall be obtained in accordance with EN 1998-1: 2004, **4.3.3.5.1(2)** and **(3)**.

(5)P Spatial variability of the motion shall be considered whenever the length of the pipeline exceeds 600 m or when geological discontinuities or marked topographical changes are present.

(6) The principles and application rules in EN 1998-2:2005, **3.3** may be used to take into account the spatial variability of the motion.

NOTE Additional models to take into account the spatial variability of the motion are given in EN 1998-2:2005, Informative Annex D.

5.5 Behaviour factors

(1) The dissipative capacity of an above-ground pipeline, if any, is restricted to its supporting structure, since it is both difficult and inconvenient to develop energy dissipation in the supported pipes, except for welded steel pipes. On the other hand, shapes and material

used for the supports vary widely, which makes it unfeasible to establish values for the behaviour factors with general applicability.

(2) For the supporting structures of non-seismically-isolated pipelines, appropriate values of q may be taken from EN 1998-1 and EN 1998-2, on the basis of the specific layout, material and level of detailing.

(3) Welded steel pipelines exhibit significant deformation and dissipation capacity, provided that their thickness is sufficient. For non-seismically-isolated pipelines which have a radius over thickness ratio (r/t) of less than 50, the behaviour factor, q , to be used for the verification of the pipes may be taken as equal to 3,0. If the r/t ratio is less than 100, q may be taken as equal to 2,0. Otherwise, the value of q for the design of the pipeline may not be taken greater than 1,5.

(4) For the verification of the supports, the seismic action effects derived from the analysis should be multiplied by $(1+q)/2$, where q is the behaviour factor of the pipeline used in its design.

5.6 Verifications

(1)P The load effect induced in the supporting elements (piers, frames, etc) in the seismic design situation shall be less than or equal to the design resistance evaluated as for the persistent or transient design situation.

(2)P Under the most unfavourable combination of axial and rotational deformations, due to the application of the seismic action relevant to the 'minimum operating level' requirement, it shall be verified that the joints do not suffer damage that may cause loss of tightness.

6 SPECIFIC PRINCIPLES AND APPLICATION RULES FOR BURIED PIPELINES

6.1 General

(1) This Section aims at providing principles and application rules for the seismic design and for the evaluation of the earthquake resistance of buried pipeline systems..

(2) 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.

(3) Networks are often too extensive and complex to be treated as a whole, and it is both feasible and convenient to identify separate networks 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.

(4) As an example of (3), an urban water distribution system may 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.

(5) The design of pipeline networks involves additional reliability requirements and design approaches with respect to those provided in the present standard.

6.2 Safety requirements

6.2.1 Damage limitation state

(1)P Buried pipeline systems shall be designed and constructed in such a way as to maintain their integrity or some of their supplying capacity after the seismic events relevant to the damage limitation state (see 2.1.3), even with considerable local damage.

6.2.2 Ultimate limit state

(1)P Buried pipelines shall conform to 5.2.2(1)P.

(2)P Buried pipelines shall conform to 5.2.2(2)P.

6.3 Seismic action

6.3.1 General

(1)P The seismic design of buried pipeline systems shall take into account the following direct and indirect seismic hazard types:

a) seismic waves propagating on firm ground and producing different ground shaking intensity at distinct points on the surface and spatial soil deformation patterns within the soil medium;

b) permanent deformations induced by earthquakes such as seismic fault displacements, landslides, ground displacements induced by liquefaction.

(2)P The general requirements regarding damage limitation and the ultimate limit state shall be satisfied for all of the types of hazards specified in (1)P.

(3) For the hazards of type (b) specified in (1)P it may be generally assumed that satisfaction of the ultimate limit state provides also fulfilment of the damage limitation requirements, so that only one verification may be performed.

(4) The fact that pipeline systems traverse or extend over large geographical areas and need to connect certain locations, does not always allow the best choices regarding the nature of the supporting soil. Furthermore, it may not be feasible to avoid crossing potentially active faults, or avoid soils susceptible to liquefaction or areas that might be affected by seismically induced landslides and large differential permanent deformations of the ground.

(5) The situation described in (4) is clearly at variance with that of other structures, for which a requisite for the very possibility to build is that the probability of soil failures of any type be negligible. Accordingly, in most cases, the occurrence of hazards of type (b) specified in (1)P cannot be ruled out. Based on available data and experience, reasoned assumptions should be used to define a model for that hazard.

6.3.2 Seismic action for inertia movements

(1)P The quantification of the components of the earthquake vibrations shall be in accordance with 2.2.

6.3.3 Modelling of seismic waves

(1)P A model for the seismic waves shall be established, from which soil strains and curvatures affecting the pipeline can be derived

NOTE: Informative Annex B provides methods for the calculation of strains and curvatures in the pipeline for some cases, under certain simplifying assumptions.

(2) Ground vibrations in earthquakes are caused by a mixture of shear, dilatational, Love and Rayleigh waves. Wave velocities are a function of their travel path through lower and higher velocity material. Different particle motions associated with these wave types make the strain and curvature in the pipeline also depend upon the angle of incidence of the waves. A general rule is to assume that sites located in the proximity of the epicentre of the earthquake are more affected by shear and dilatational waves (body waves), while for sites at a larger distance, Love and Rayleigh waves (surface waves) tend to be more significant.

(3)P The selection of the waves to be taken into account and of the corresponding wave propagation velocities shall be based on geophysical considerations.

6.3.4 Permanent soil movements

(1)P The ground rupture patterns associated with earthquake induced ground movements, either due to surface faulting or landslides, are likely to be complex, showing substantial variations in displacements as a function of the geologic setting, soil type and the magnitude and duration of the earthquake. The possibility of such phenomena occurring at given sites shall be established and appropriate models shall be defined (see EN 1998-5).

6.4 Methods of analysis (wave passage)

(1)P It is acceptable to take advantage of the post-elastic deformation of pipelines. The deformation capacity of a pipeline shall be evaluated.

NOTE An acceptable analysis method for buried pipelines on stable soil, based on approximate assumptions of the characteristics of ground motion, is given in Informative Annex B.

6.5 Verifications

6.5.1 General

(1) Pipelines buried in stable and sufficiently homogeneous soil may be checked only for the soil deformations due to wave passage.

(2)P Buried pipelines crossing areas where soil failures or concentrated distortions are possible, like lateral spreading, liquefaction, landslides and fault movements, shall be designed to resist these phenomena.

6.5.2 Buried pipelines on stable soil

(1)P The response quantities to be obtained from the analysis shall include the maximum values of axial strain and curvature and, for unwelded joints (reinforced concrete or prestressed pipes) the rotations and the axial deformations at the joints.

(2)P In welded steel pipelines the combination of axial strain and curvature due to the design seismic action shall be compatible with the available ductility of the material in tension and with the local and global buckling resistance in compression:

- allowable tensile strain: 3%;
- allowable compressive strain: $\min \{1\%; 20t/r (\%)\}$;

where t and r are the thickness and radius of the pipe respectively.

(3)P In concrete pipelines, under the most unfavourable combination of axial strain and curvature due to the design seismic action, the limiting strains specified in EN 1992-1-1 for concrete and steel shall not be exceeded.

(4)P In concrete pipelines, under the most unfavourable combination of axial strain and curvature due to the seismic action relevant to the damage limitation state, the tensile strain of the reinforcing steel shall not exceed values that may result in residual crack widths incompatible with the leak-tightness requirements.

(5)P Under the most unfavourable combination of axial and rotational deformations, the joints in the pipeline shall not suffer damage incompatible with the specified damage

limitation requirements.

6.5.3 Buried pipelines under differential ground movements (welded steel pipes)

(1)P The segment of the pipeline deformed by the displacement of the ground, either due to fault movement or caused by a landslide or by lateral spreading, shall be verified not to exceed the available ductility of the material in tension and not to buckle locally or globally in compression. The limit strains shall be in accordance with **6.5.2**.

6.6 Design measures for fault crossings

(1) The decision to apply special fault crossing designs for pipelines where they cross potentially active fault zones depends upon cost, fault activity, consequences of rupture, environmental impact and possible exposure to other hazards during the life span of the pipeline.

(2) In the design of a pipeline for fault crossing, the considerations in **(3)** to **(9)** will generally improve the capability of the pipeline to sustain differential movements along the fault.

(3) Where practical, a pipeline crossing a strike-slip fault should be oriented in such a way as to place the pipeline in tension.

(4) The angle of intersection of reverse faults should be as small as possible, to minimize compression strains. If significant strike-slip displacements are also anticipated, the fault crossing angle of the pipeline should be chosen to promote tensile elongation of the line.

(5) In fault zones the depth at which the pipeline is buried should be minimized in order to reduce soil restraint on the pipeline during fault movement.

(6) An increase in pipe wall thickness will increase the pipeline's capacity for fault displacement at a given level of maximum tensile strain. Within 50 m on each side of the fault relatively thick-walled pipe should be used.

(7) Reduction of the angle of interface friction between the pipeline and the soil increases the pipeline's capacity for fault displacement at a given level of maximum strain. The angle of interface friction can be reduced through a hard, smooth coating.

(8) Close control should be exercised over the backfill surrounding the pipeline over a distance of 50 m on each side of the fault. In general, a loose to medium granular soil without cobbles or boulders will be a suitable backfill material. If the existing soil differs substantially from this, oversize trenches should be excavated for a distance of approximately 15 m on each side of the fault.

(9) For welded steel pipelines, fault movement can be accommodated by utilising the ability of the pipeline to deform well into the inelastic range in tension, in order to conform without rupture to the ground distortions. Wherever possible, pipeline alignment at a fault crossing should be selected such that the pipeline will be subjected to tension plus a moderate amount of bending. Alignments which might place the pipeline in compression should be avoided to the extent possible, because the ability of the pipeline to withstand compressive

strain without rupture is significantly less than that for tensile strain. Any compressive strains should be limited to that strain which would cause wrinkling or local buckling of the pipeline.

(10) In all areas of potential ground rupture, pipelines should be laid in relatively straight sections, avoiding sharp changes in direction and elevation. To the extent possible, pipelines should be constructed without field bends, elbows and flanges that tend to anchor the pipeline to the ground.

ANNEX A (INFORMATIVE)

SEISMIC ANALYSIS PROCEDURES FOR TANKS

A.1 Introduction and scope

This Annex provides information on seismic analysis procedures for tanks subjected to horizontal or vertical seismic action, having the following characteristics:

- a) cylindrical shape, with vertical axis and circular or rectangular cross-section;
- b) rigid or flexible foundation;
- c) full or partial anchorage to the foundation.

Extensions for elevated tanks or cylindrical tanks with horizontal axis are briefly discussed.

A rigorous analysis of the phenomenon of dynamic interaction between the motion of the contained fluid, the deformation of the tank walls and that of the underlying foundation soil, including possible uplift, is a problem of considerable analytical complexity requiring unusually high computational resources and efforts. Several analysis procedures have been proposed, valid for specific design situations. Since their accuracy is problem-dependent, a proper choice requires a certain amount of specialized knowledge from the designer. Attention is called to the importance of a uniform level of accuracy across the design process: it would not be consistent, for example, to use an accurate solution for the determination of the hydrodynamic pressures, and then not to use a correspondingly refined mechanical model of the tank (e.g., a finite element model) for evaluating the stresses due to the pressures.

A.2 Rigid vertical circular tanks on-ground, fixed to the foundation

A.2.1 Horizontal seismic action

A.2.1.1 General

The motion of the fluid contained in a rigid cylinder may be expressed as the sum of two separate contributions, called ‘rigid impulsive’, and ‘convective’, respectively. The ‘rigid impulsive’ component satisfies exactly the boundary conditions at the walls and the bottom of the tank, but gives (incorrectly, due to the presence of the waves in the dynamic response) zero pressure at the original position of the free surface of the fluid in the static situation. The ‘convective’ term does not alter those boundary conditions that are already satisfied, while fulfilling the correct equilibrium condition at the free surface. Use is made of a cylindrical coordinate system: r, z, θ , with origin at the centre of the tank bottom and the z axis vertical. The height of the tank to the original of the free surface of the fluid and its radius are denoted by H and R , respectively, ρ is the mass density of the fluid, while $\xi = r/R$ and $\varsigma = z/H$ are the nondimensional coordinates.

A.2.1.2 Rigid impulsive pressure

The spatial-temporal variation of the ‘rigid impulsive’ pressure is given by the expression:

$$p_i(\xi, \varsigma, \theta, t) = C_i(\xi, \varsigma) \rho H \cos \theta A_g(t) \quad (\text{A.1})$$

where:

$$C_i(\xi, \varsigma) = 2 \sum_{n=0}^{\infty} \frac{(-1)^n}{I_1'(v_n / \gamma) v_n^2} \cos(v_n \varsigma) I_1\left(\frac{v_n}{\gamma} \xi\right) \quad (\text{A.2})$$

in which:

$$v_n = \frac{2n+1}{2} \pi; \quad \gamma = H / R$$

$I_1(\cdot)$ and $I_1'(\cdot)$ denote the modified Bessel function of order 1 and its derivative⁵.

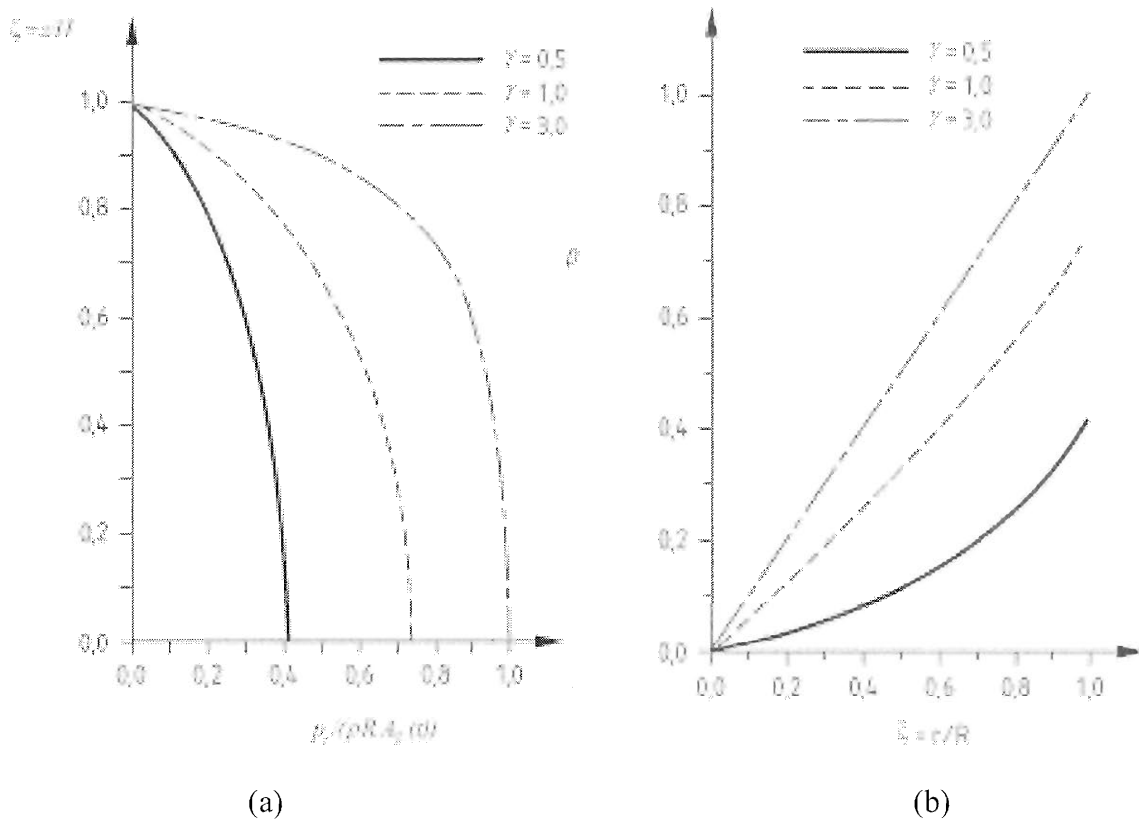


Figure A.1— Variation of the impulsive pressure (normalized to $\rho R a_g$) for three values of $\gamma = H/R$. a) variation along the height; b) radial variation on the tank bottom.

$A_g(t)$ in expression (A.1) is the ground acceleration time-history in the free-field (with peak value denoted by a_g). The function C_i gives the distribution along the height of p_i . It is shown in Figure A.1a) for $\xi = 1$ (i.e. at the wall of the tank) and $\cos \theta = 1$ (i.e. in the plane of the horizontal seismic action), normalized to $\rho R a_g$, for three values of the slenderness parameter $\gamma = H/R$. Figure A.1b) shows the radial variation of p_i on the tank bottom as a function of γ . For large values of γ the pressure distribution on the bottom becomes linear.

⁵ The derivative can be expressed in terms of the modified Bessel functions of order 0 and 1 as:

$$I_1'(x) = \frac{dI_1(x)}{dx} = I_0(x) - \frac{I_1(x)}{x}$$

Pressure resultants: The horizontal resultant of the 'rigid impulsive' pressure from expression (A.1) at the base of the wall, Q_i , is:

Impulsive base shear:

$$Q_i(t) = m_i A_g(t) \quad (\text{A.3})$$

m_i , termed *impulsive mass*, denotes the mass of the contained fluid which moves together with the walls and is given by the expression:

$$m_i = m 2\gamma \sum_{n=0}^{\infty} \frac{I_1(v_n/\gamma)}{v_n^3 I_1'(v_n/\gamma)} \quad (\text{A.4})$$

where $m = \rho \pi R^2 H$ is the total contained mass of the fluid.

The total moment with respect to an axis orthogonal to the direction of the seismic action motion, M'_i , immediately below the tank bottom includes the contributions of the pressures on the walls from expression (A.1) and of those on the tank bottom. The total moment M_i immediately above the tank bottom includes only the contributions of the pressures on the walls.

Impulsive base moment (immediately below the tank bottom):

$$M'_i(t) = m_i h'_i A_g(t) \quad (\text{A.5a})$$

where

$$h'_i = H \frac{\frac{1}{2} + 2\gamma \sum_{n=0}^{\infty} \frac{v_n + 2(-1)^{n+1} I_1(v_n/\gamma)}{v_n^4 I_1'(v_n/\gamma)}}{2\gamma \sum_{n=0}^{\infty} \frac{I_1(v_n/\gamma)}{v_n^3 I_1'(v_n/\gamma)}} \quad (\text{A.6a})$$

Impulsive base moment (immediately above the tank bottom):

$$M_i(t) = m_i h_i A_g(t) \quad (\text{A.5b})$$

with

$$h_i = H \frac{\sum_{n=0}^{\infty} \frac{(-1)^n I_1(v_n/\gamma)}{v_n^4 I_1'(v_n/\gamma)} (v_n (-1)^n - 1)}{\sum_{n=0}^{\infty} \frac{I_1(v_n/\gamma)}{v_n^3 I_1'(v_n/\gamma)}} \quad (\text{A.6b})$$

Figure A.2 shows the quantities m_i , h'_i and h_i as functions of $\gamma = H/R$. m_i increases with γ , tending asymptotically to the total mass, while both h_i and h'_i tend to stabilize to values around midheight. For squat tanks h_i is a little less than midheight, while h'_i is significantly larger than H due to the predominant contribution to M'_i of the pressures on the bottom.

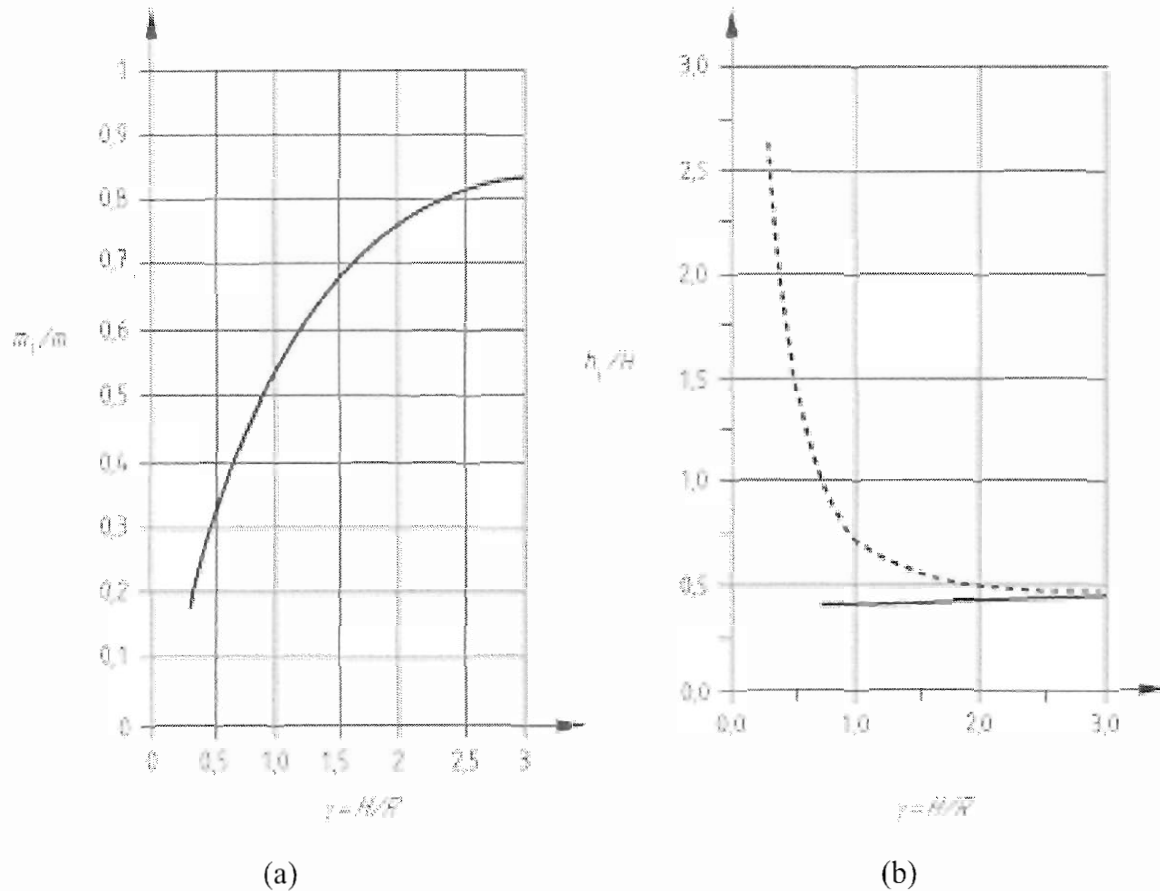


Figure A.2 — Ratios m_i/m , h_i/H and h'_i/H as functions of the tank slenderness (see also Table A.2, columns 4, 6 and 8)

Key to Figure A.2(b): — : above base plate; - - - - : below base plate

A.2.1.3 Convective pressure component

The spatial-temporal variation of the ‘convective’ pressure component is given by:

$$p_c(\xi, \zeta, \theta, t) = \rho \sum_{n=1}^{\infty} \psi_n \cosh(\lambda_n \gamma \zeta) J_1(\lambda_n \xi) \cos \theta A_{cn}(t) \quad (\text{A.7})$$

where:

$$\psi_n = \frac{2R}{(\lambda_n^2 - 1) J_1(\lambda_n) \cosh(\lambda_n \gamma)} \quad (\text{A.8})$$

J_1 = Bessel function of the first order,

$\lambda_1 = 1,841$, $\lambda_2 = 5,331$, $\lambda_3 = 8,536$, and

$A_{cn}(t)$ = acceleration time-history of the response of a single degree of freedom oscillator having a circular frequency ω_{cn} equal to:

$$\omega_{cn} = \sqrt{g \frac{\lambda_n}{R} \tanh(\lambda_n \gamma)} \quad (\text{A.9})$$

and a damping ratio appropriate for the sloshing of the fluid (see [1] for procedures for the calculation of damping).

Only the first oscillating, or sloshing, mode and frequency of the oscillating liquid ($n = 1$) needs to be considered in expression (A.7) for design purposes.

The vertical distribution of the sloshing pressures for the first two modes is shown in Figure A.3a), while Figure A.3b) gives the values of the first two frequencies, as functions of the H/R . In squat tanks the sloshing pressures maintain relatively high values down to the bottom, while in slender tanks the sloshing effect is limited to the vicinity of the surface of the liquid. The sloshing frequencies become almost independent of γ , for γ larger than about 1. For such values of γ , ω_{c1} is approximately equal to:

$$\omega_{c1} = 4,2 / \sqrt{R} \quad (R \text{ in meters}) \quad (\text{A.10})$$

which, for the usual values of R yields periods of oscillation of the order of few seconds.

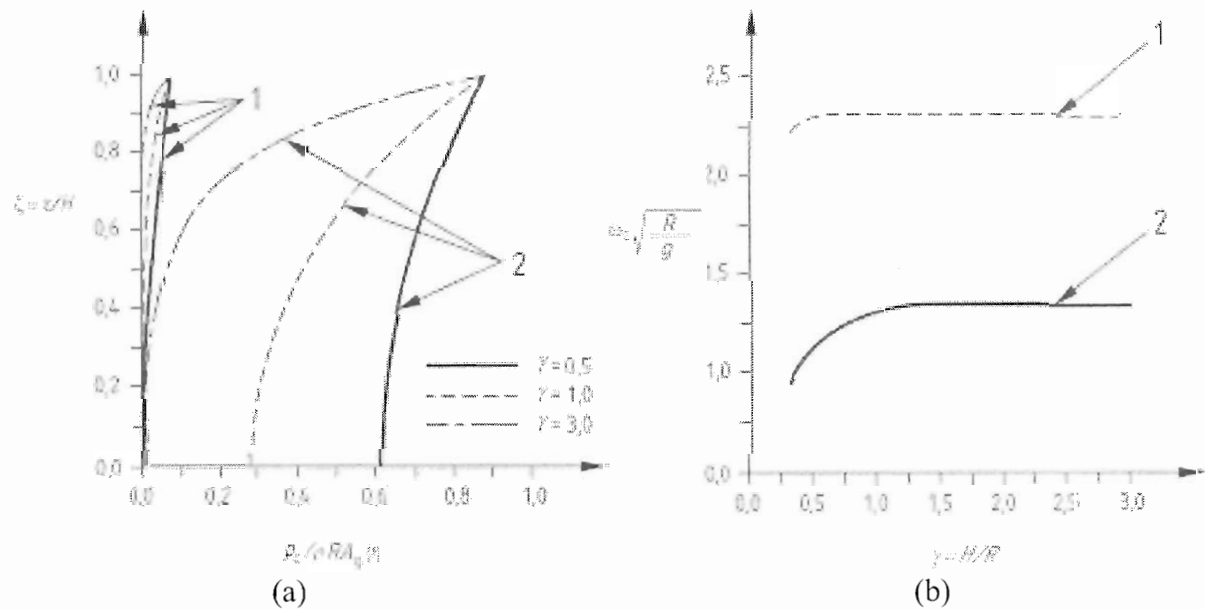


Figure A.3 — a) Variation of sloshing pressures along the height in the first two modes and b) values of the first two sloshing frequencies as functions of γ

Key : 1: 2nd mode; 2: 1st mode

Pressure resultants:

Convective base shear:

$$Q_c(t) = \sum_{n=1}^{\infty} m_{cn} A_{cn}(t) \quad (\text{A.11})$$

where the n -th modal convective mass is:

$$m_{cn} = m \frac{2 \tanh(\lambda_n \gamma)}{\gamma \lambda_n (\lambda_n^2 - 1)} \quad (\text{A.12})$$

Moment immediately below the bottom plate of the tank:

$$M'_c(t) = \sum_{n=1}^{\infty} (m_{cn} A_{cn}(t)) h'_{cn} = \sum_{n=1}^{\infty} Q_{cn}(t) h'_{cn} \quad (\text{A.13a})$$

where:

$$h'_{cn} = H \left(1 + \frac{2 - \cosh(\lambda_n \gamma)}{\lambda_n \gamma \sinh(\lambda_n \gamma)} \right) \quad (\text{A.14a})$$

The values of m_{c1} and m_{c2} and the corresponding values of h_{c1} , h_{c2} , h'_{c1} and h'_{c2} are shown in Figure A.4 as functions of γ .

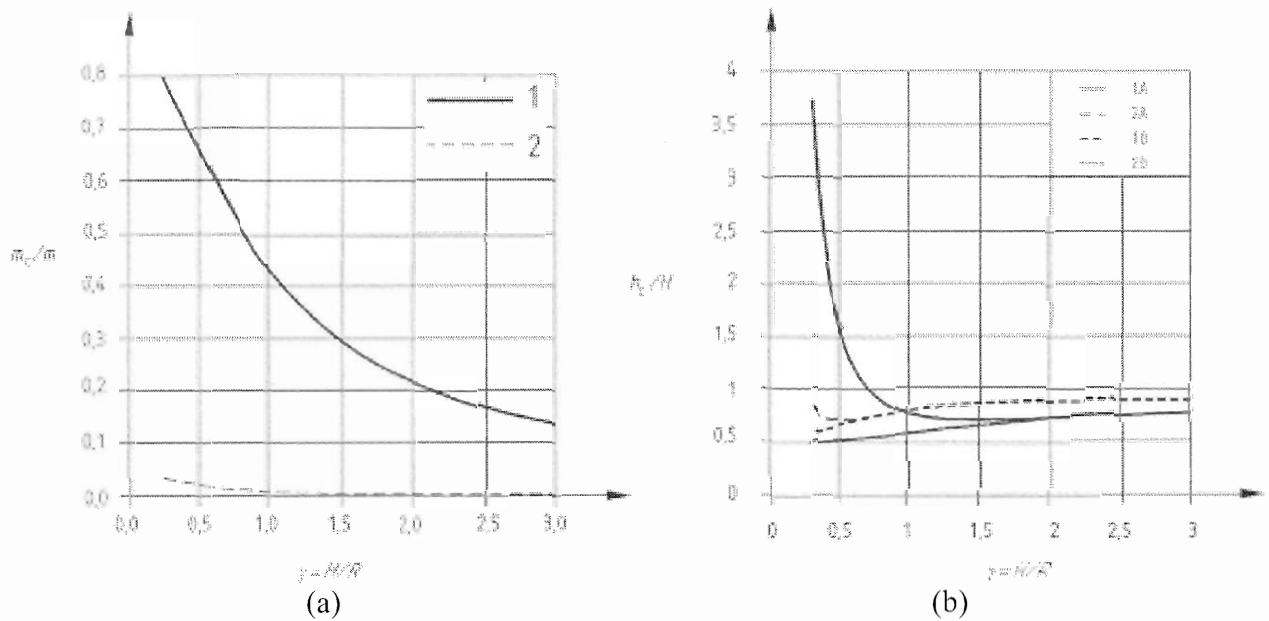


Figure A.4 — a) First two sloshing modal masses and b) corresponding heights h_{c1} , h_{c2} , h'_{c1} and h'_{c2} as functions of γ (see also Table A.2, columns 5, 7 and 9)

Key to Figure A.4(a): 1: 1st mode; 2: 2nd mode.

Key to Figure A.4(b): 1A: 1st mode, below base plate;
2A: 2nd mode, below base plate;
1B: 1st mode, above base plate;
2B: 2nd mode, above base plate.

Moment in the tank wall immediately above the bottom plate:

$$M_c(t) = \sum_{n=1}^{\infty} (m_{cn} A_{cn}(t)) h_{cn} = \sum_{n=1}^{\infty} Q_{cn}(t) h_{cn} \quad (\text{A.13b})$$

where h_{cn} is:

$$h_{cn} = H \left(1 + \frac{1 - \cosh(\lambda_n \gamma)}{\lambda_n \gamma \sinh(\lambda_n \gamma)} \right) \quad (\text{A.14b})$$

The convective component of the response may be obtained from that of oscillators having masses m_{cn} , attached to the rigid tank through springs having stiffnesses: $K_n = \omega_n^2 m_{cn}$. (one oscillator for each mode considered significant, normally only the first one). The tank is subjected to the ground acceleration time-history $A_g(t)$ and the masses respond with accelerations $A_{cn}(t)$. h'_{cn} or h_{cn} is the level where the oscillator needs to be applied in order to give the correct value of M'_{cn} or M_{cn} , respectively.

A.2.1.4 Height of the convective wave

The sloshing wave height is provided mainly by the first mode; the expression for the peak height at the edge is:

$$d_{\max} = 0,84 R S_e(T_{c1}) / g \quad (\text{A.15})$$

where $S_e(\cdot)$ is the elastic response spectral acceleration at the 1st convective mode of the fluid for damping a value appropriate for the sloshing response and g is the acceleration of gravity.

A.2.1.5 Effect of the inertia of the walls

For steel tanks, the inertia forces on the shell due to its own mass are small compared with the hydrodynamic forces and may be neglected. For concrete tanks, they should not be neglected. Inertia forces are parallel to the horizontal seismic action, inducing a pressure normal to the surface of the shell given by:

$$p_w = \rho_s s(\zeta) \cos \theta A_g(t) \quad (\text{A.16})$$

where:

ρ_s = mass density of the wall material

$s(\zeta)$ = wall thickness

The action effects of this pressure component, which follows the variation of wall thickness along the height, should be added to those of the impulsive component given by expression (A.1).

The total shear at the base due to the inertia forces of the tank wall and roof may be taken equal to the total mass of the tank walls and roof, times the acceleration of the ground. The contribution to the base overturning moment in a similar way: it is equal to the wall mass times the wall midheight (for constant wall thickness), plus the roof mass times its mean distance from the base, times the acceleration of the ground.

A.2.1.6 Combination of action effects of impulsive and convective pressures

The time-history of the total pressure is the sum of the following two time-histories:

- the impulsive one being driven by $A_g(t)$ (including the inertia of the walls);
- the convective one driven by $A_{c1}(t)$ (neglecting higher order components).

In the same way that the dynamic response associated with the two pressure components is characterized by different damping ratios, it may also be associated with different hysteretic energy dissipation mechanisms. No energy dissipation can be associated with the convective response of the liquid, whereas some hysteretic energy dissipation may accompany the response due to the impulsive pressures and the inertia of the tank walls, arising from the tank itself and the way it is supported on (or anchored to) the ground. If energy dissipation is taken into account through modification of the elastic spectrum by the behaviour factor q , a different value of q should be used in the derivation of the action effects of the two components: i.e. $q = 1,0$ for the action effects of the convective pressures and $q = 1,5$ (or a higher value) for the action effects of the impulsive pressures and of the inertia of the tank walls.

If, as it is customary in design practice, the response spectrum approach is used for the calculation of the maximum dynamic response, the maxima of the two time-histories of seismic action effects given by the response spectrum should be suitably combined. Due to the generally wide separation between the dominant frequencies in the ground motion and the sloshing frequency, the 'square root of the sum of squares' rule may be unconservative, so that the alternative, upper bound, rule of adding the absolute values of the two maxima may be preferable in design. Each of these two maxima will be derived for the value of q and of the damping ratio considered appropriate for the corresponding component.

The value of the moment and shear force immediately above the bottom plate of the tank should be used for the calculation of the stresses and stress resultants in the tank walls and at the connection to the base, for the verifications. The value of the moment immediately below the bottom plate of the tank should be used for the verification of its support structure, base anchors or foundation.

Due to the long period of the convective component of the response of the liquid, only the moment below the bottom plate of the tank which is due to this component of the pressure is relevant to the static equilibrium verification of the tank (overturning). Due to their high frequency, the impulsive pressures and the inertia of the tank walls may be considered not to contribute to the destabilising moment in the verification of the tank against overturning.

A.2.2 Vertical component of the seismic action

The hydrodynamic pressure on the walls of a rigid tank due to vertical ground acceleration $A_v(t)$ is given by:

$$p_{vr}(\zeta, t) = \rho H(1 - \zeta) A_v(t) \quad (\text{A.17})$$

Being axisymmetric, this hydrodynamic pressure does not produce a shear force or moment resultant at any horizontal level of the tank, or immediately above or below the base.

A.2.3 Combination of the effects of the horizontal and the vertical components of the seismic action, including the effects of other actions

The peak combined pressure on the tank walls due to horizontal and vertical seismic action may be obtained by applying the rule in 3.2. The combined pressure should be added to the hydrostatic pressure on the wall at the one side of the tank (where the wall accelerates into the liquid) and subtracted as suction at the opposite. Dynamic earth and ground water pressures should be considered to act against any buried part of the tank on the side of the tank where the seismic pressure is considered as suction. Earth pressures there should be estimated on the

basis of the coefficient of earth pressure at rest.

A.3 Deformable vertical circular tanks on-ground, fixed to the foundation

A.3.1 Horizontal components of the seismic action

It is normally unconservative to consider the tank as rigid (especially for steel tanks). In flexible tanks the fluid pressure is usually expressed as the sum of three contributions, referred to as: 'rigid impulsive', 'sloshing' and 'flexible'. The third satisfies the condition that the radial velocity of the fluid along the wall equals the deformation velocity of the tank wall, as well as the conditions of zero vertical velocity at the tank bottom and zero pressure at the free surface of the fluid. The dynamic coupling between the sloshing and the flexible components is very weak, due to the large differences between the frequencies of the sloshing motion and of the deformation of the wall, which allows determining the third component independently of the others. The rigid impulsive and the sloshing components in A.2 remain therefore unaffected.

The flexible pressure distribution depends on the modes of vibration of the tank-fluid system, among which only those with one circumferential wave, of the following type, are of interest:

$$\phi(\zeta, \theta) = f(\zeta) \cos \theta \quad (\text{A.18})$$

In the following, the term fundamental or first frequency, or first mode, is not related to the real fundamental modes of the full tank, but only to eigenmodes of the type of expression (A.18).

The radial distribution of the flexible impulsive pressure on the tank bottom is qualitatively the same as for the rigid impulsive pressure. Assuming the modes as known, the flexible pressure distribution on the walls has the form:

$$p_f(\zeta, \theta, t) = \rho H \psi \cos \theta \sum_{n=0}^{\infty} d_n \cos(\nu_n \zeta) A_{fn}(t) \quad (\text{A.19})$$

where:

$$\psi = \frac{\int_0^1 f(\zeta) \left[\frac{\rho_s}{\rho} \frac{s(\zeta)}{H} + \sum_{n=0}^{\infty} b'_n \cos(\nu_n \zeta) \right] d\zeta}{\int_0^1 f(\zeta) \left[\frac{\rho_s}{\rho} \frac{s(\zeta)}{H} f(\zeta) + \sum_{n=0}^{\infty} d_n \cos(\nu_n \zeta) \right] d\zeta} \quad (\text{A.20})$$

$$b'_n = 2 \frac{(-1)^n I_1(\nu_n / \gamma)}{\nu_n^2 I_1'(\nu_n / \gamma)} \quad (\text{A.21})$$

$$d_n = 2 \frac{\int_0^1 f(\zeta) \cos(\nu_n \zeta) d\zeta}{\nu_n} \frac{I_1(\nu_n / \gamma)}{I_1'(\nu_n / \gamma)} \quad (\text{A.22})$$

ρ_s is the mass density of the shell, $s(\zeta)$ is its thickness and $A_{fn}(t)$ is the response acceleration (relative to its base) of a simple oscillator having the period and damping ratio of mode n . The fundamental mode ($n=1$) is normally sufficient, so that in expressions (A.19), (A.21), (A.22),

the mode index, n , and the summation over all modal contributions are dropped.

In most cases of flexible tanks, the pressure $p_t(\cdot)$ in expression (A.19) provides the predominant contribution to the total pressure, due to the fact that, while the rigid impulsive term - expression (A.1) - varies with the ground acceleration $A_g(t)$, the flexible term - expression (A.19) - varies with the response acceleration $A_{fn}(t)$, which, for the usual range of periods of the tank-fluid systems, is considerably amplified with respect to $A_g(t)$.

For the determination of the first mode shape of the tank, the following iterative procedure is suggested in [2], [3]. A trial shape is selected for $f(\zeta)$, in expressions (A.18)-(A.22) (a shape proportional to ζ is usually a good approximation, especially for slender tanks). Denoting with $f^i(\zeta)$ the shape used in the i -th iteration, an 'effective' mass density of the shell is evaluated as:

$$\rho^i(\zeta) = \frac{p_f^i(\zeta)}{2gs(\zeta)f^i(\zeta)} + \rho_s \quad (\text{A.23})$$

where $p_f^i(\zeta)$ is the value of the pressure evaluated from expression (A.19) at the i -th step. The effective mass density from expression (A.23) may then be used in a structural analysis of the tank to evaluate the mode shape in the $(i+1)$ -th iteration, and so forth until convergence.

The fundamental circular frequency of the tank-fluid system may be evaluated by means of the following approximate expression, derived in [4] for steel tanks:

$$\omega_f = 2\pi \frac{\sqrt{Es(\zeta)/\rho H}}{R(0,157\gamma^2 + \gamma + 1,49)} \quad (\text{for } \zeta = 1/3) \quad (\text{A.24})$$

where E is the elastic modulus of the material of the tank wall.

The base shear is:

$$Q_f(t) = m_f A_f(t) \quad (\text{A.25})$$

where:

$$m_f = m\psi\gamma \sum_{n=0}^{\infty} \frac{(-1)^n}{v_n} d_n \quad (\text{A.26})$$

The moment immediately above the tank bottom may be calculated as:

$$M_f(t) = m_f h_f A_f(t) \quad (\text{A.27})$$

where:

$$h_f = H \frac{\left[\gamma \sum_{n=0}^{\infty} d_n \frac{(-1)^n v_n - 2}{v_n^2} + \sum_{n=0}^{\infty} \frac{d_n I_1'(v_n / \gamma)}{v_n} \right]}{\gamma \sum_{n=0}^{\infty} d_n \frac{(-1)^n}{v_n}} \quad (\text{A.28})$$

A.3.2 Combination of the pressure terms due to horizontal components of the seismic action

A.3.2.1 General procedures

The time-history of the total pressure in flexible tanks is the sum of the time-histories of the rigid impulsive pressure (expression (A.1)), of the convective one (expression (A.7)), and of the flexible pressure (expression (A.19)), each of them differently distributed along the height and having a different variation with time. The time-history of the base shear produced by these pressures (expressions (A.3), (A.11) and (A.25)) is:

$$Q(t) = m_i A_g(t) + \sum_{n=1}^{\infty} m_{cn} A_{cn}(t) + m_f A_f(t) \quad (\text{A.29})$$

where $A_{cn}(t)$ is the total or absolute response acceleration of a simple oscillator with circular frequency ω_{cn} (expression (A.9)) and damping ratio appropriate for the sloshing response subjected to a base acceleration $A_g(t)$, while $A_f(t)$ is the response acceleration (relative to the base) of a simple oscillator of circular frequency ω_f (expression (A.24)) and damping appropriate for the tank-fluid system, also subjected to $A_g(t)$.

If the individual maxima of the terms in expression (A.29) are known, e.g. from a response spectrum of absolute and relative accelerations, the corresponding pressures on the tank needed for a detailed stress analysis may be obtained by spreading the resultant of each of the three terms in expression (A.29) over the tank walls and floor according to the relevant distribution of pressures. To expedite the design process, the masses m_i , m_{cn} and m_f , the latter based on assumed first mode shapes, have been calculated as functions of the ratio γ , and are available in tabular form or in diagrams (see, for example, Figures A.2(a), A.4(a), columns 4 and 5 in Table A.2 and [4]). Use of expression (A.29) in combination with response spectra, however, poses the question of the combination of the maxima. Apart from the need to derive a relative acceleration response spectrum for $A_f(t)$, there is no accurate way of combining the peak of $A_g(t)$ with that of $A_f(t)$. As a matter of fact, since the input and its response cannot be assumed as independent in the range of relatively high frequencies under consideration, the 'square root of the sum of squares' rule is not sufficiently accurate. On the other hand, addition of the individual maxima could lead to overconservative estimates.

Given these difficulties, various approximate approaches based on the theory above have been proposed. Three of these, presented in detail in [4], [5], are due to Veletsos and Yang, Haroun and Housner, or Scharf [4].

The Veletsos and Yang approach consists in replacing expression (A.29) with the following:

$$Q(t) = m_i A_{fa}(t) + \sum_{n=1}^{\infty} m_{cn} A_{cn}(t) \quad (\text{A.30})$$

i.e., in assuming the entire impulsive mass to respond with the amplified absolute response acceleration of the flexible tank system ($A_{fa}(t) = A_f(t) + A_g(t)$) with circular frequency ω_f (expression (A.24)) and damping appropriate for the tank-fluid system. The maximum of $A_{fa}(t)$ is obtained directly from the appropriate response spectrum. The total base shear may be evaluated approximately by the expression:

$$Q_w(t) = (\varepsilon_o \cdot m) \cdot A_{fa}(t) \quad (\text{A.31})$$

where $(\varepsilon_0 \cdot m)$ is the effective participating mass of the tank wall in the first mode, where m is the total mass of the tank-fluid system and the factor ε_0 may be determined from Table A.1:

Table A.1 — Effective participating mass of tank wall in first mode as fraction of the total, in the Veletsos and Yang procedure

H/R	0,5	1,0	3,0
ε_0	0,5	0,7	0,9

The Veletsos and Yang procedure provides an upper bound estimate, acceptable for H/R ratios not much larger than 1. Above this value, corrections to reduce the conservativeness have been suggested. In view of the conservative nature of the method, the effects of tank inertia may generally be neglected.

In the Haroun and Housner approach expression (A.29) is written in a form suitable for the use of the response spectrum, as:

$$Q(t) = (m_i - m_f) A_g(t) + \sum_{n=1}^{\infty} m_{cn} A_{cn}(t) + m_f A_{fa}(t) \quad (\text{A.32})$$

The masses m_i and m_f are given in graphs as functions of H/R and s/R , together with the heights at which these masses should be located to yield the correct value of the base moment [5]. The effects of the inertia of the tank wall are incorporated in the values of the masses and of their heights.

The ‘square root of the sum of squares’ rule is used to combine the maximum values of the three components in expression (A.32).

Finally, based on the fact that absolute and relative response accelerations do not differ appreciably in the relevant frequency range, in the Scharf [4] approach expression (A.29) is written as:

$$Q(t) = m_i A_g(t) + \sum_{n=1}^{\infty} m_{cn} A_n(t) + m_f A_{fa}(t) \quad (\text{A.33})$$

The “square root of the sum of squares” rule is used to combine the maximum values of the three components in expression (A.34).

$$Q = \sqrt{(m_i a_g)^2 + (m_f a_{fa})^2 + \left(\sum_{n=1}^{\infty} m_{cn} a_{cn}\right)^2} \quad (\text{A.34})$$

An even more simplified approach has been proposed in [6] along lines similar to those of Veletsos and Yang, as summarized below.

A.2.1.4 applies here as well, regarding the different hysteretic energy dissipation mechanisms (and associated behaviour factor values q) characterising the different pressure components.

A.3.2.2 Simplified procedure for fixed base cylindrical tanks [6]

A.3.2.2.1 Model

The tank-liquid system is modeled by two single-degree-of-freedom systems, one corresponding to the impulsive component, moving together with the flexible wall, and the other corresponding to the convective component. The impulsive and convective responses are combined by taking their numerical-sum.

The natural periods of the impulsive and the convective responses, in seconds, are taken as:

$$T_{\text{imp}} = C_i \frac{\sqrt{\rho} H}{\sqrt{s/R} \sqrt{E}} \quad (\text{A.35})$$

$$T_{\text{con}} = C_c \sqrt{R} \quad (\text{A.36})$$

where:

H = height to the free surface of the liquid;

R = tank's radius;

s = equivalent uniform thickness of the tank wall (weighted average over the wetted height of the tank wall, the weight may be taken proportional to the strain in the wall of the tank, which is maximum at the base of the tank);

ρ = mass density of liquid; and

E = Modulus of elasticity of tank material.

Table A.2 — Coefficients C_i and C_c for the natural periods, masses m_i and m_c and heights h_i and h_c from the base of the point of application of the wall pressure resultant, for the impulsive and convective components

H/R	C_i	C_c (s/m ^{1/2})	m_i/m	m_c/m	h_i/H	h_c/H	h'_i/H	h'_c/H
0,3	9,28	2,09	0,176	0,824	0,400	0,521	2,640	3,414
0,5	7,74	1,74	0,300	0,700	0,400	0,543	1,460	1,517
0,7	6,97	1,60	0,414	0,586	0,401	0,571	1,009	1,011
1,0	6,36	1,52	0,548	0,452	0,419	0,616	0,721	0,785
1,5	6,06	1,48	0,686	0,314	0,439	0,690	0,555	0,734
2,0	6,21	1,48	0,763	0,237	0,448	0,751	0,500	0,764
2,5	6,56	1,48	0,810	0,190	0,452	0,794	0,480	0,796
3,0	7,03	1,48	0,842	0,158	0,453	0,825	0,472	0,825

The coefficients C_i and C_c are obtained from Table A.2. Coefficient C_i is dimensionless, while

if R is in meters C_c is expressed in $\text{s/m}^{1/2}$.

The impulsive and convective masses m_i and m_c are given in Table A.2 as fractions of the total liquid mass m , along with the heights from the base of the point of application of the resultant of the impulsive and convective hydrodynamic wall pressure, h_i and h_c .

A.3.2.2.2 Seismic response

The total base shear is

$$Q = (m_i + m_w + m_r) S_e(T_{\text{imp}}) + m_c S_e(T_{\text{con}}) \quad (\text{A.37})$$

where:

m_w = mass of the tank wall;

m_r = mass of tank roof;

$S_e(T_{\text{imp}})$ = impulsive spectral acceleration, obtained from an elastic response spectrum for a value of damping consistent with the limit state considered according to 2.3.3.1;

$S_e(T_{\text{con}})$ = convective spectral acceleration, from a 0,5%-damped elastic response spectrum.

The overturning moment immediately above the base plate is

$$M = (m_i h_i + m_w h_w + m_r h_r) S_e(T_{\text{imp}}) + m_c h_c S_e(T_{\text{con}}) \quad (\text{A.38})$$

h_w and h_r are heights of the centres of gravity of the tank wall and roof, respectively.

The overturning moment immediately below the base plate is given by

$$M' = (m_i h'_i + m_w h_w + m_r h_r) S_e(T_{\text{imp}}) + m_c h'_c S_e(T_{\text{con}}) \quad (\text{A.39})$$

The vertical displacement of liquid surface due to sloshing is given by expression (A.15).

A.3.3 Vertical component of the seismic action

In addition to the pressure $p_{\text{vf}}(\zeta, t)$ given by expression (A.17), due to the tank moving rigidly in the vertical direction with acceleration $A_v(t)$, there is a contribution to the pressure, $p_{\text{vf}}(\zeta, t)$, due to the deformability (radial ‘breathing’) of the shell [7]. This additional term may be calculated as:

$$p_{\text{vf}}(\zeta, t) = 0,815 f(\gamma) \rho H \cos\left(\frac{\pi}{2} \zeta\right) A_{\text{vf}}(t) \quad (\text{A.40})$$

where:

$$f(\gamma) = 1,078 + 0,274 \ln \gamma \quad \text{for } 0,8 \leq \gamma < 4 \quad (\text{A.41a})$$

$$f(\gamma) = 1,0 \quad \text{for } \gamma < 0,8 \quad (\text{A.41b})$$

$A_{\text{vf}}(t)$ is the acceleration response of a simple oscillator having a frequency equal to the fundamental frequency of the axisymmetric vibration of the tank with the fluid.

The fundamental frequency may be estimated from the expression:

$$f_{vd} = \frac{1}{4R} \left[\frac{2EI_1(\gamma_1) \zeta(\zeta)}{\pi \rho H (1-\nu^2) I_0(\gamma_1)} \right]^{1/2} \quad (\text{for } \zeta = 1/3) \quad (\text{A.42})$$

where:

$$\gamma_1 = \pi/(2\gamma);$$

$I_0(\cdot)$ and $I_1(\cdot)$ denote the modified Bessel function of order 0 and 1, respectively;

E and ν are Young's modulus and Poisson ratio of the tank material, respectively.

The maximum value of $p_{vi}(t)$ is obtained from the vertical acceleration response spectrum for the appropriate values of period and damping. If soil flexibility is neglected (see A.7) the applicable damping values are those of the material of the shell. The behaviour factor value, q , adopted for the response due to the impulsive component of the pressure and the tank wall inertia may be used for the response to the vertical component of the seismic action. The maximum value of the pressure due to the combined effect of $p_{vi}(\cdot)$ and $p_{vf}(\cdot)$ may be obtained by applying the 'square root of the sum of squares' rule to the individual maxima.

A.3.4 Combination of the effects of the horizontal and vertical components of the seismic action, including the effects of other actions

The pressure on the tank walls should be determined in accordance with A.2.3.

A.4 Rectangular tanks

A.4.1 Rigid rectangular tanks on-ground, fixed to the foundation

For tanks with walls assumed as rigid, the total pressure is again given by the sum of an impulsive and a convective contribution:

$$p(z, t) = p_i(z, t) + p_c(z, t) \quad (\text{A.43})$$

The impulsive component follows the expression:

$$p_i(z, t) = q_o(z) \rho L A_g(t) \quad (\text{A.44})$$

where:

L is the half-width of the tank in the direction of the seismic action;

$q_o(z)$ is a function giving the variation of $p_i(\cdot)$ along the height as plotted in Figure A.5 ($p_i(\cdot)$ is constant in the direction orthogonal to the seismic action). The trend and the numerical values of $q_o(z)$ are very close to those of a cylindrical tank with radius $R = L$ (see Figure A.6).

The convective pressure component is given by a summation of modal terms (sloshing modes). As for cylindrical tanks, the dominant contribution is that of the fundamental mode:

$$p_{c1}(z, t) = q_{c1}(z) \rho L A_1(t) \quad (\text{A.45})$$

where

$q_{c1}(z)$ is a function shown in Figure A.7 together with the 2nd mode contribution $q_{c2}(z)$ and $A_1(t)$ is the acceleration response function of a simple oscillator with the frequency of the first mode and the appropriate value of damping, when subjected to an input acceleration $A_g(t)$.

The period of oscillation of the first sloshing mode is:

$$T_1 = 2\pi \left(\frac{L/g}{\frac{\pi}{2} \tanh\left(\frac{\pi}{2} \frac{H}{L}\right)} \right)^{1/2} \quad (\text{A.46})$$

The base shear and the moment on the foundation may be evaluated on the basis of expressions (A.44) and (A.45). The values of the masses m_i and m_{c1} , as well as of the corresponding heights above the base, h_i and h_{c1} , calculated for cylindrical tanks and given by expressions (A.4), (A.12) and (A.6), (A.14), respectively, may be adopted for the design of rectangular tanks as well (with L replacing R), with an error less than 15% [8].

A.4.2 Flexible rectangular tanks on-ground, fixed to the foundation

As in cylindrical tanks with circular section, wall flexibility generally produces a significant increase of the impulsive pressures, while leaving the convective pressures practically unchanged. Studies on the seismic response of flexible rectangular tanks are few and their results are not in a form suitable for direct use in design [9]. An approximation for design purposes is to use the same vertical pressure distribution as for rigid walls [8], see expression (A.44) and Figures A.5, A.6, but to replace the ground acceleration $A_g(t)$ in expression (A.44) with the response acceleration of a simple oscillator having the frequency and the damping ratio of the first impulsive tank-liquid mode.

This period of vibration may be approximated as:

$$T_f = 2\pi(d_f/g)^{1/2} \quad (\text{A.47})$$

where:

d_f is the deflection of the wall on the vertical centre-line and at the height of the impulsive mass, when the wall is loaded by a load uniform in the direction of the ground motion and of magnitude: $m_i g / 4BH$;

$2B$ is the tank width perpendicular to the direction of the seismic action.

The impulsive mass m_i may be obtained as the sum of that from expression (A.4), Figure A.2(a) or column 4 in Table A.2, plus the wall mass.

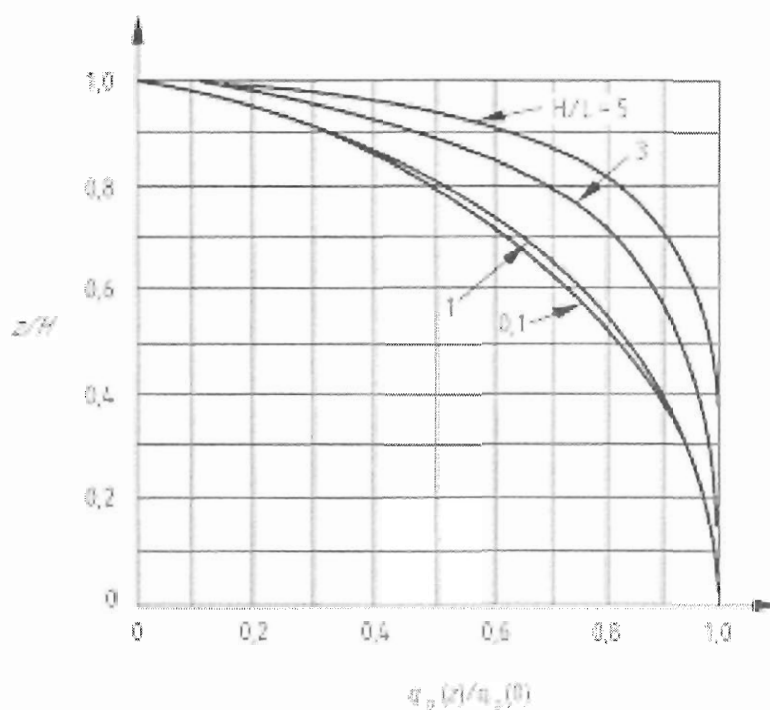


Figure A.5 — Distribution along the height of dimensionless impulsive pressures on rectangular tank wall which is perpendicular to the horizontal component of the seismic action [8]

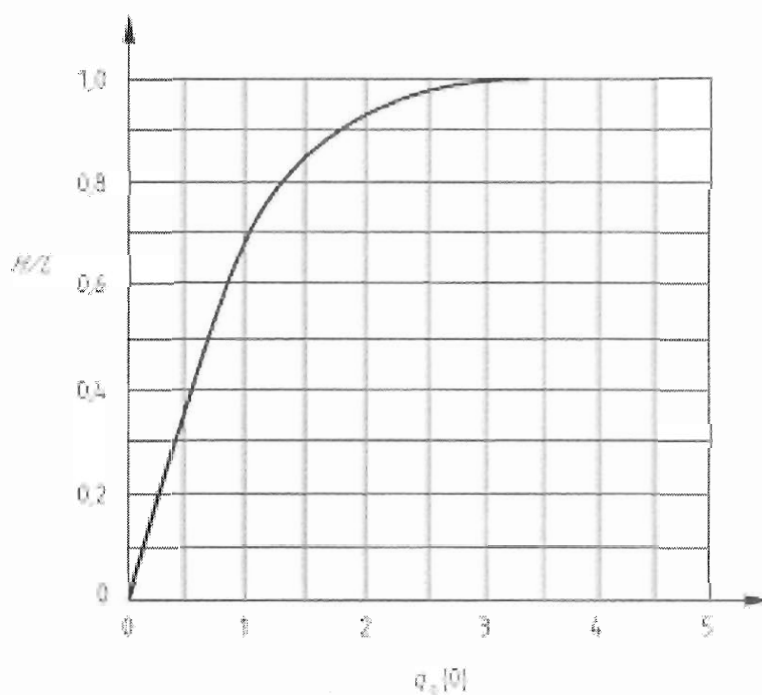


Figure A.6 — Peak value of dimensionless impulsive pressures on a rectangular wall which is perpendicular to the horizontal component of the seismic action [8]

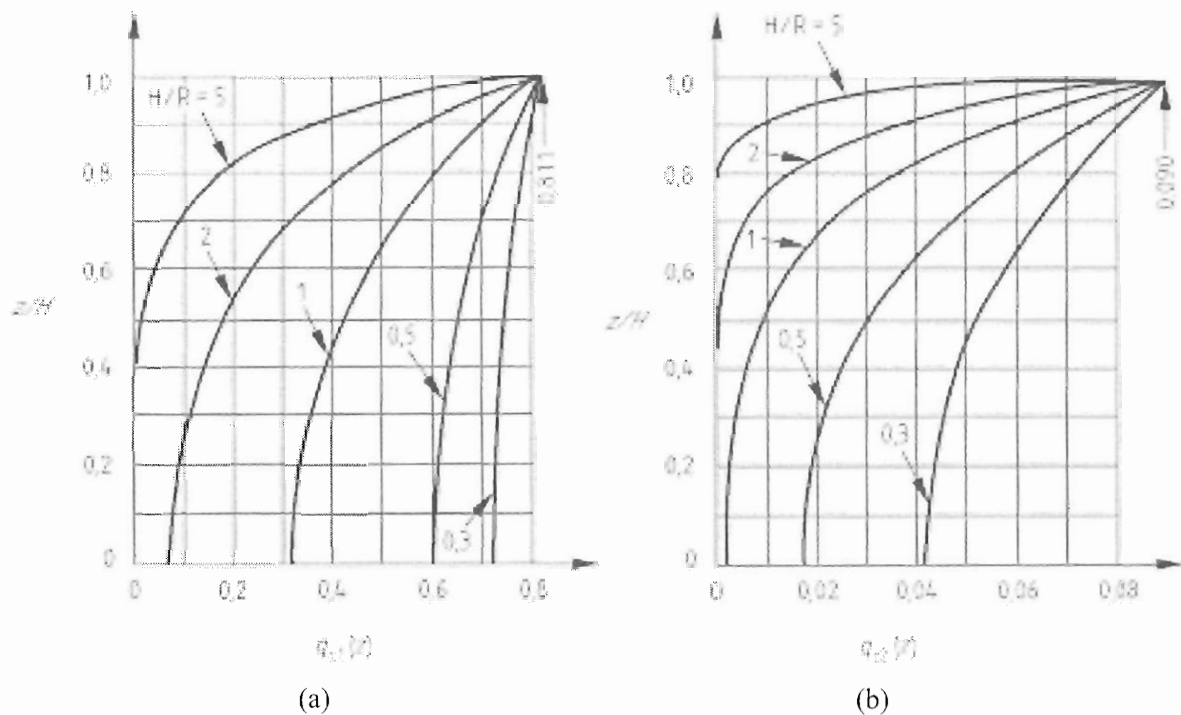


Figure A.7 — Dimensionless convective pressures on rectangular tank wall which is perpendicular to the horizontal component of the seismic action ([8])

A.4.3 Combination of action effects due to the different components and actions

A.2.1.6 applies regarding the different hysteretic energy dissipation mechanisms (and associated behaviour factor values q) for the different pressure components. A.2.2 may be applied for the evaluation of the effects of the vertical component of the seismic action and A.2.3 for the combination of the effects of the horizontal and vertical components, including the effects of other actions in the seismic design situation.

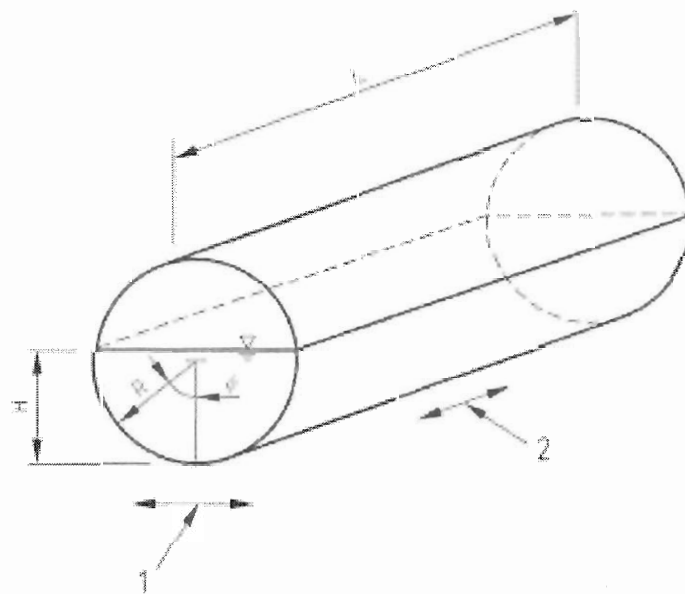


Figure A.8 — Notations for horizontal axis cylindrical tank [8]

Key: 1: seismic action in transverse direction; 2: seismic action in longitudinal direction.

A.5 Horizontal cylindrical tanks on-ground [8]

Horizontal cylindrical tanks should be analyzed for seismic action along the longitudinal and along the transverse axis (see Figure A.8 for notations).

Approximate values for hydrodynamic pressures induced by seismic action in either the longitudinal or transverse direction may be obtained considering a 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 third dimension (width) such that the liquid volume is maintained. This approximation is sufficiently accurate for design purposes over the range of H/R between 0,5 and 1,6. If H/R exceeds 1,6, the tank should be assumed to behave as if it were full, i.e., with the total mass of the fluid acting solidly with the tank.

For a seismic action in the transverse direction (perpendicular to the axis) a more accurate solution is described below for partially full tanks.

The impulsive pressure distribution is given by:

$$p_i(\phi) = q_o(\phi) \gamma R A_g(t) \quad (\text{A.48})$$

For $H = R$ the pressure function $q_o(\cdot)$ takes the form:

$$q_o(\phi) = \frac{4}{\pi} \sum_{n=1}^{\infty} \frac{(-1)^{n-1}}{(2n)^2 - 1} \sin 2n\phi \quad (\text{A.49})$$

and is plotted in Figure A.9.

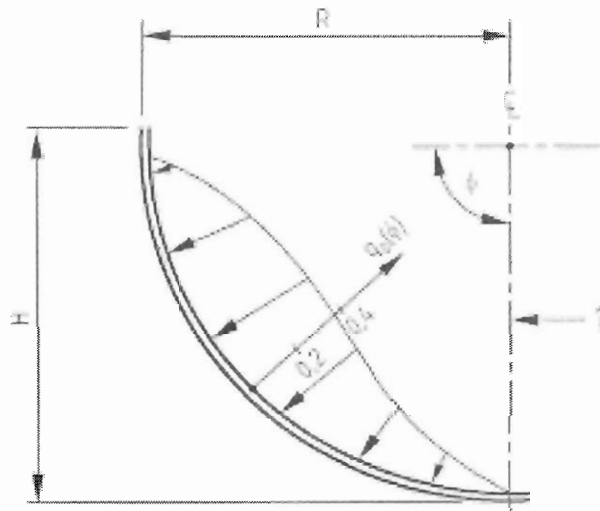


Figure A.9 — Impulsive pressures on horizontal cylinder with $H = R$. Transverse seismic action [8]

Key: 1: Pressure anti-symmetric about centreline

By integrating the pressure distribution the impulsive mass for $H = R$ is evaluated to be:

$$m_i = 0,4m \quad (\text{A.50})$$

As the pressures are in the radial direction, the forces on the cylinder pass through the centre

of the circular section. Both the impulsive and the convective masses should be assumed to be at that point.

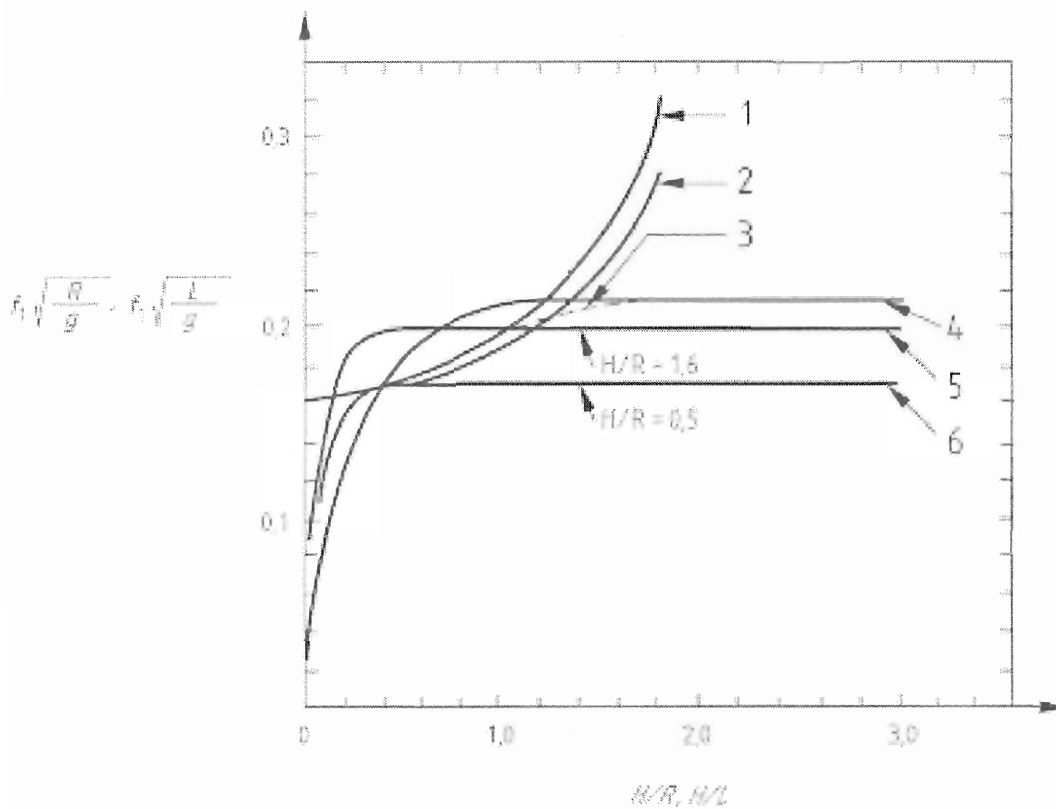


Figure A.10 — Dimensionless first convective mode frequency for rigid tanks of various shapes [8]

Key: 1: Sphere;
 2: Horizontal cylinder, transverse seismic action;
 3: Vertical cylinder, spherical bottom;
 4: Vertical cylinder;
 5: Rectangular tank (length: $2L$);
 5 & 6: Horizontal cylinder, longitudinal seismic action (length: $2L$).

Solutions for the convective pressures are not available in a convenient form for design. When the tank is approximately half full ($H \cong R$), the first sloshing mode mass may be evaluated as:

$$m_{cl} = 0,6m \quad (\text{A.51})$$

Expressions (A.50), (A.51) are considered as reasonable approximations for H/R from 0,8 to 1,2.

The first mode sloshing frequencies for rigid tanks of various shapes, including horizontal cylinders for seismic action along and transverse to the axis, are shown in Figure A.10.

A.6 Elevated tanks

In the structural model that includes also the supporting structure, the liquid in the tank may be accounted for by considering two masses:

- an impulsive mass m_i rigidly connected to the tank walls, located at a height h'_i or h_i above the tank bottom (expressions (A.4) and (A.6a), (A.6b), respectively);
- a mass m_{c1} , connected to the walls through a spring of stiffness $K_{c1} = \omega_{c1}^2 m_{c1}$, where ω_{c1} is given by expression (A.9), located at a height h'_{c1} or h_{c1} (expressions (A.12) and (A.14a), (A.14b), respectively).

The response of the system may be evaluated using standard modal analysis and response spectra methods.

In the simplest case, the global model has only two degrees-of-freedom, corresponding to the masses m_i and m_{c1} . A mass Δm equal to the mass of the tank and an appropriate portion of the mass of the support should be added to m_i . The mass $(m_i + \Delta m)$ should be connected to the ground by a spring representing the stiffness of the support.

Normally, the rotational inertia of the mass $(m_i + \Delta m)$, and the corresponding additional degree of freedom, should also be included in the model.

Elevated tank in the shape of a truncated inverted cone may be considered in the model as an equivalent cylinder of the same volume of liquid and a diameter equal to that of the cone at the level of the liquid.

A.7 Soil-structure interaction effects for tanks on-ground

A.7.1 General

For tanks founded on relatively deformable soils, the base motion can be significantly different from the free-field motion; in general the translational component is modified and there is also a rocking component. Moreover, for the same input motion, as the flexibility of the ground increases, the fundamental period of the tank-fluid system and the total damping increase, reducing the peak force response. The increase in the period is more pronounced for tall, slender tanks, because the contribution of the rocking component is greater. The reduction of the peak force response, however, is in general less for tall tanks, since the damping associated with rocking is smaller than that associated with horizontal translation.

A simple procedure, proposed for buildings in [10] and consisting of an increase of the fundamental period and of the damping of the structure, which is considered to rest on a rigid soil and subjected to the free-field motion, has been extended to the impulsive (rigid and flexible) components of the response of tanks in [11], [12], [13]. The convective periods and pressures are assumed not to be affected by soil-structure interaction. A good approximation can be obtained through the use of an equivalent simple oscillator with parameters adjusted to match frequency and peak response of the actual system. The properties of this substitute oscillator are given in [11], [13] in the form of graphs, as functions of the ratio H/R , for fixed values of the wall thickness ratio s/R , the initial damping, etc.

A.7.2 Simple procedure

A.7.2.1 Introduction

A more rough procedure [8], summarized below, may be adopted. The procedure operates by changing separately the frequency and the damping of the impulsive rigid and the impulsive flexible pressure contributions in **A.2** to **A.5**. In particular, for the rigid impulsive pressure

components, whose time-histories are given by the free-field horizontal, $A_g(t)$, and vertical, $A_v(t)$ accelerations, consideration of soil-structure interaction effects amounts to replacing these time-histories with the response acceleration histories of a single degree of freedom oscillator having natural period and damping as specified below.

A.7.2.2 Modified natural periods:

- ‘rigid tank’ impulsive effect, horizontal

$$T_i^* = 2\pi \left(\frac{m_i + m_o}{k_x \alpha_x} + \frac{m_i h_i^2}{k_\theta \alpha_\theta} \right)^{1/2} \quad (\text{A.52})$$

- ‘deformable tank’ impulsive effect, horizontal

$$T_f^* = T_f \left(1 + \frac{k_f}{k_x \alpha_x} \cdot \left[1 + \frac{k_x h_f^2}{k_\theta \alpha_\theta} \right] \right)^{1/2} \quad (\text{A.53})$$

- ‘rigid tank’, vertical

$$T_{vt}^* = 2\pi \left(\frac{m_{\text{tot}}}{k_v \alpha_v} \right)^{1/2} \quad (\text{A.54})$$

- “deformable tank”, vertical

$$T_{vd}^* = T_{vd} \left(1 + \frac{k_l}{k_v \alpha_v} \right)^{1/2} \quad (\text{A.55})$$

where:

m_i, h_i are the mass and height of the impulsive component;

m_o is the mass of the foundation;

k_f is the stiffness of the “deformable tank” = $4\pi^2 \frac{m_f}{T_f^2}$;

m_{tot} is the total mass of the filled tank, including the foundation;

$k_l = 4\pi^2 \frac{m_l}{T_{vd}^2}$, with m_l = mass of the liquid;

k_x, k_θ, k_v are the horizontal, rocking and vertical stiffness of the foundation; and

$\alpha_x, \alpha_\theta, \alpha_v$ are frequency-dependent factors converting static stiffnesses into dynamic ones [14].

A.7.2.3 Modified damping values:

The general expression for the effective damping ratio of the tank-foundation system is:

$$\xi = \xi_s + \frac{\xi_m}{(T^*/T)^3} \quad (\text{A.56})$$

where:

ξ_s is the radiation damping in the soil; and

ξ_m is the material damping in the tank.

Both ξ_s and ξ_m depend on the specific vibration mode.

In particular for ξ_s :

– for the horizontal impulsive ‘rigid tank’ mode:

$$\xi_s = \frac{2\pi^2 m_i}{k_x T_i^{*2}} a \left(\frac{\beta_x}{\alpha_x} + \frac{k_x h_i'^2 \beta_\theta}{k_\theta \alpha_\theta} \right) \quad (\text{A.57})$$

– for the horizontal impulsive ‘deformable tank’ mode:

$$\xi_s = \frac{2\pi^2 m_f}{k_x T_f^{*2}} a \left(\frac{\beta_x}{\alpha_x} + \frac{k_x h_f'^2 \beta_\theta}{k_\theta \alpha_\theta} \right) \quad (\text{A.58})$$

– for the vertical ‘rigid tank’ mode:

$$\xi_s = \frac{2\pi^2 m_{tot}}{k_v T_{vr}^{*2}} a \frac{\beta_v}{\alpha_v} \quad (\text{A.59})$$

where:

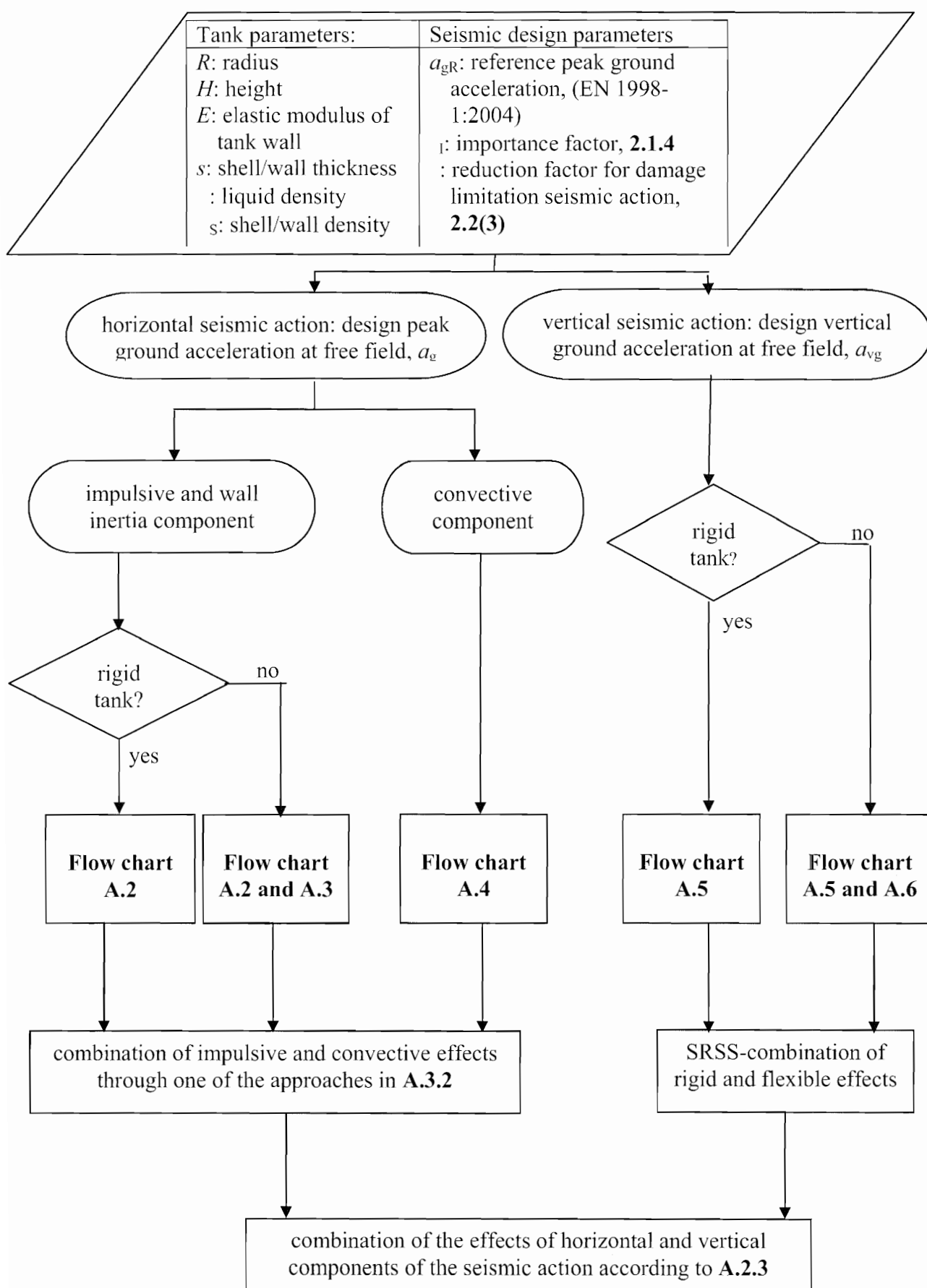
a is the dimensionless frequency function = $\frac{2\pi R}{V_s T}$ (V_s = shear wave velocity of the soil);

$\beta_x, \beta_\theta, \beta_v$ are the frequency-dependent factors providing radiation damping values for horizontal, vertical and rocking motions [14].

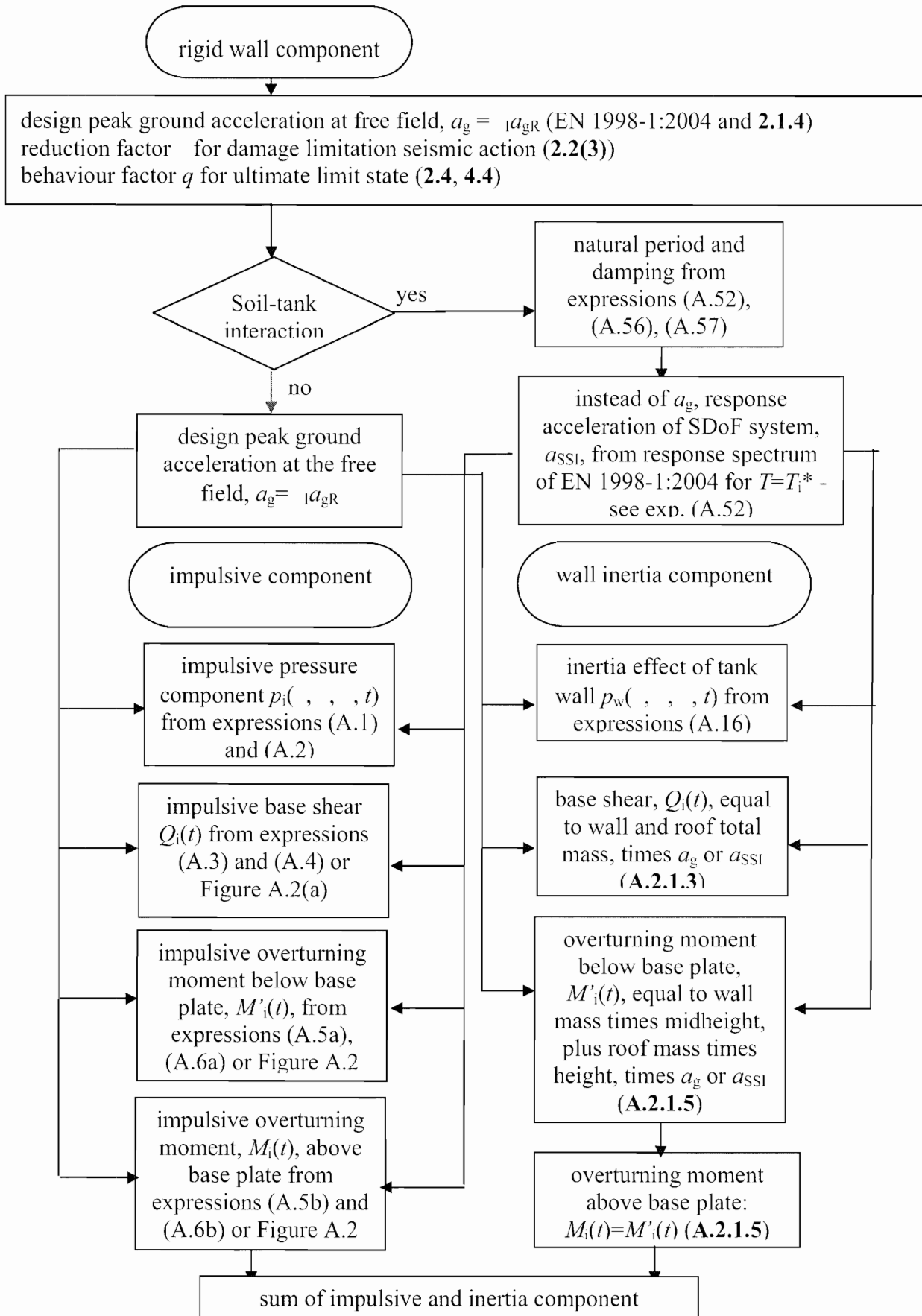
A.8 Flow charts for calculation of hydrodynamic effects in vertical cylindrical tanks

The following flow charts provide an overview of the determination of hydrodynamic effects in vertical cylindrical tanks subjected to horizontal and vertical seismic actions. The flow charts essentially address the application of the response spectra method.

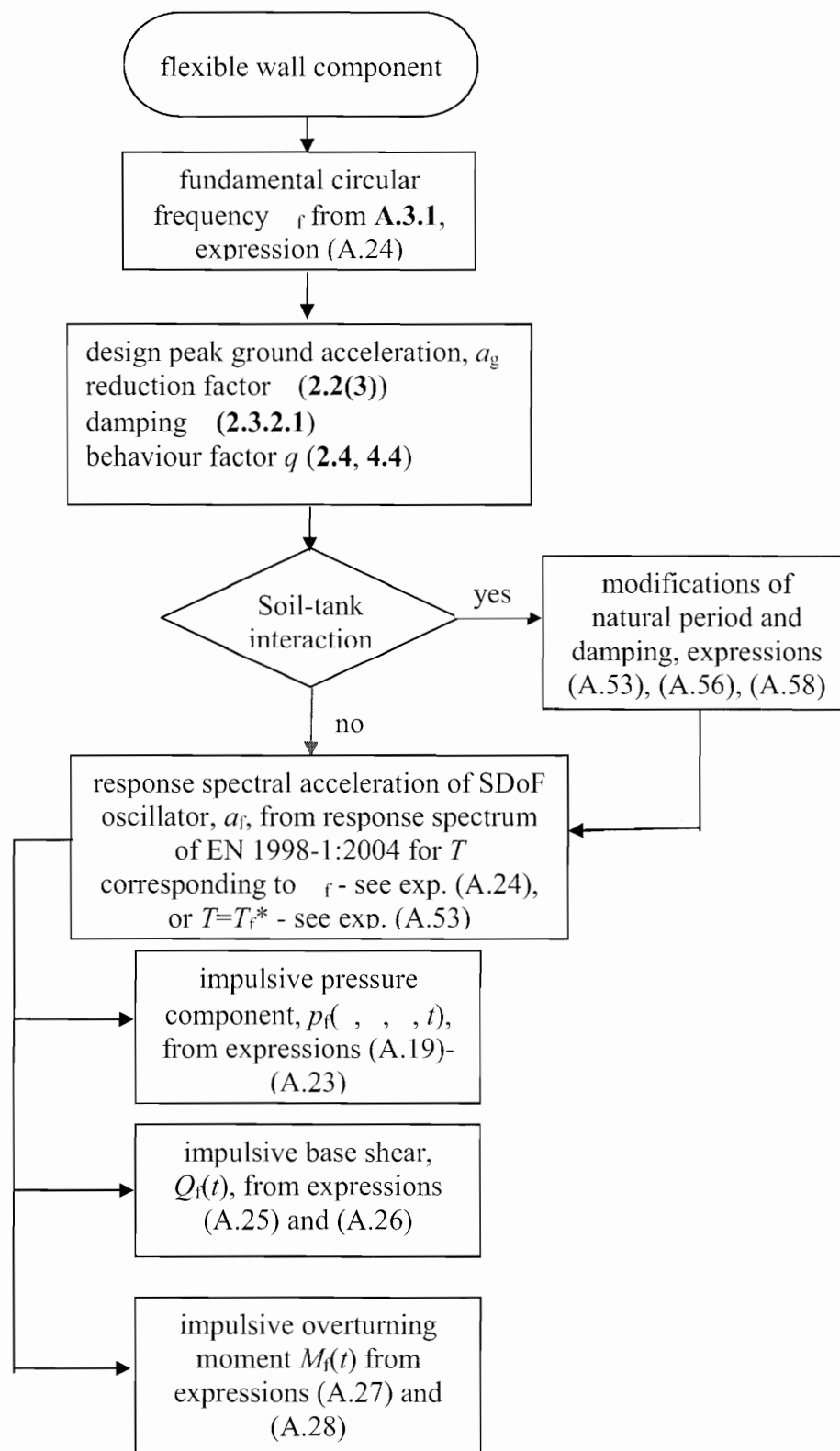
Flow chart 1 gives an overview of the calculation process and of the combination of the various components of the response. **Flow charts 2 to 6** address the different hydrodynamic components or seismic action components.



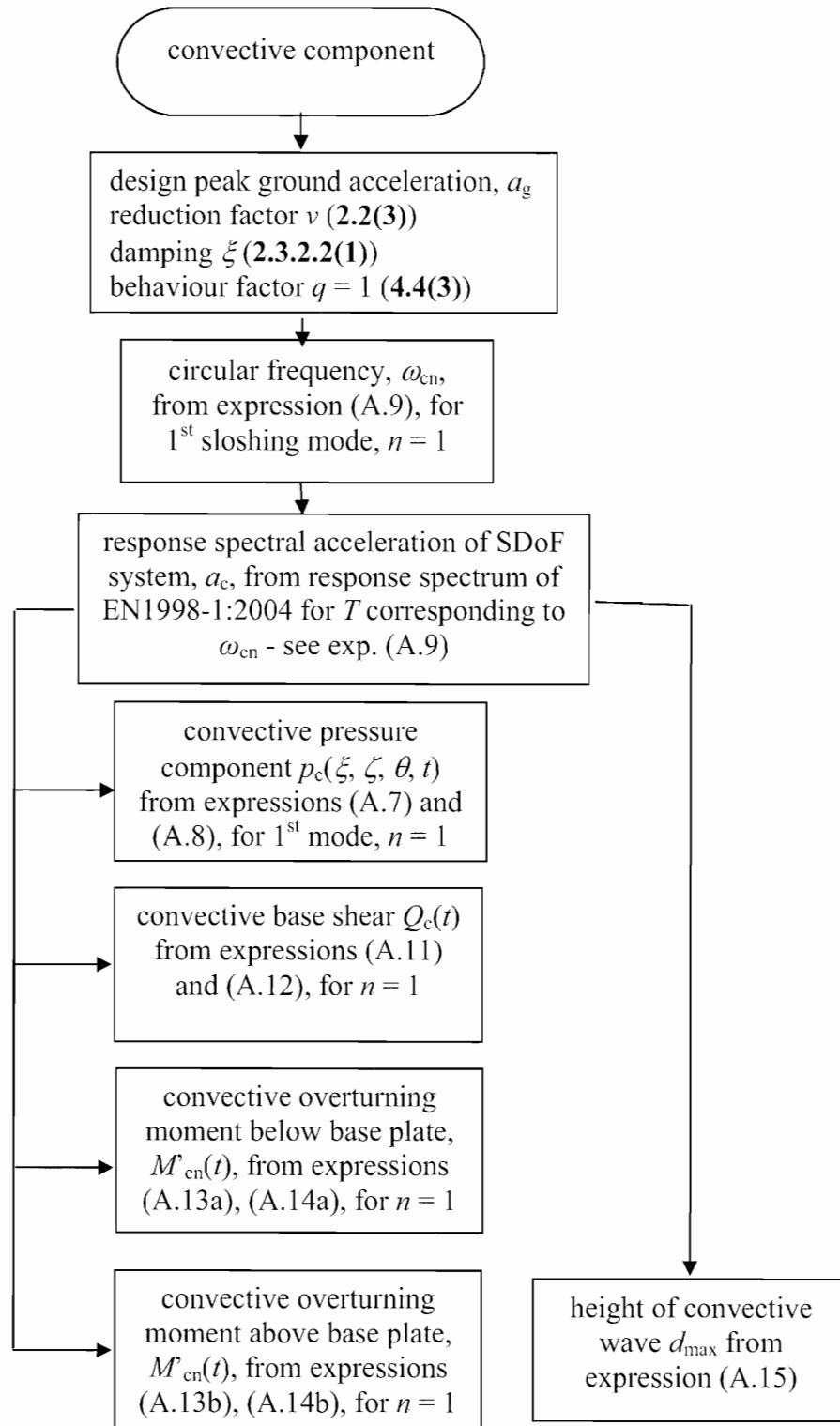
Flow chart A.1: Overview of determination of hydrodynamic effects in anchored vertical cylindrical tanks on ground, considering soil-structure interaction



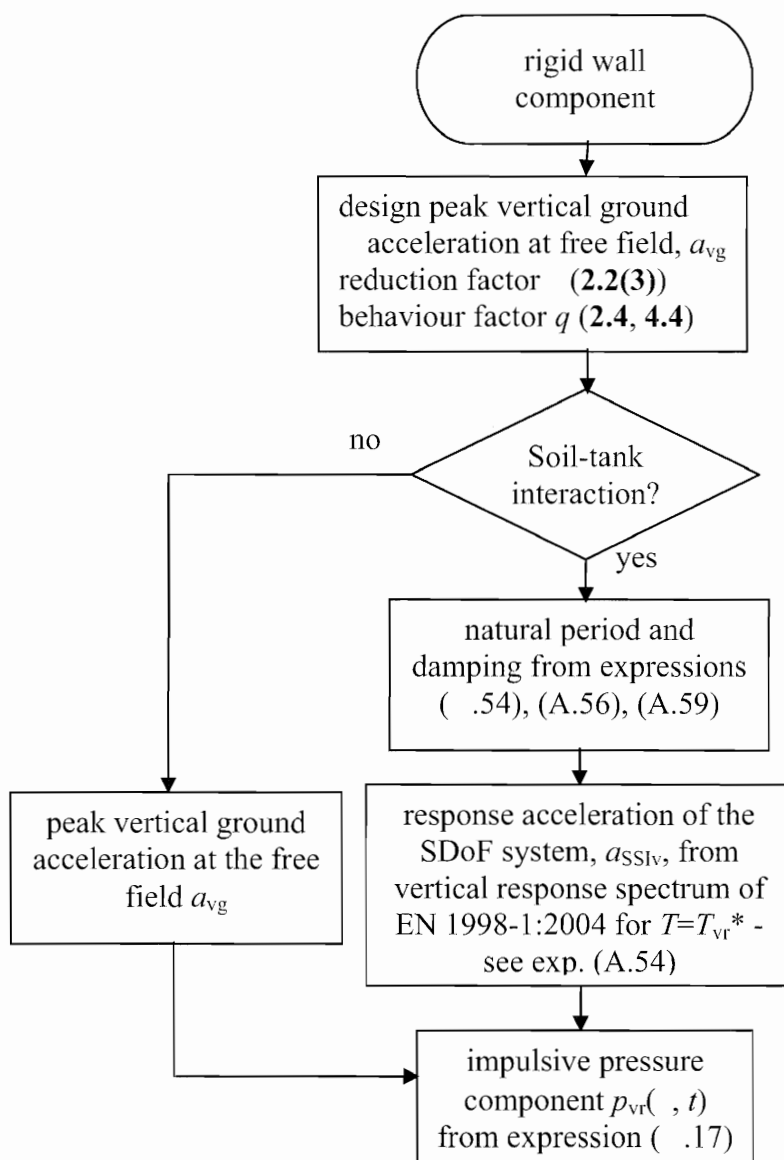
Flow chart A.2: Horizontal seismic action, rigid wall impulsive component (see A.2.1, A.7.2)



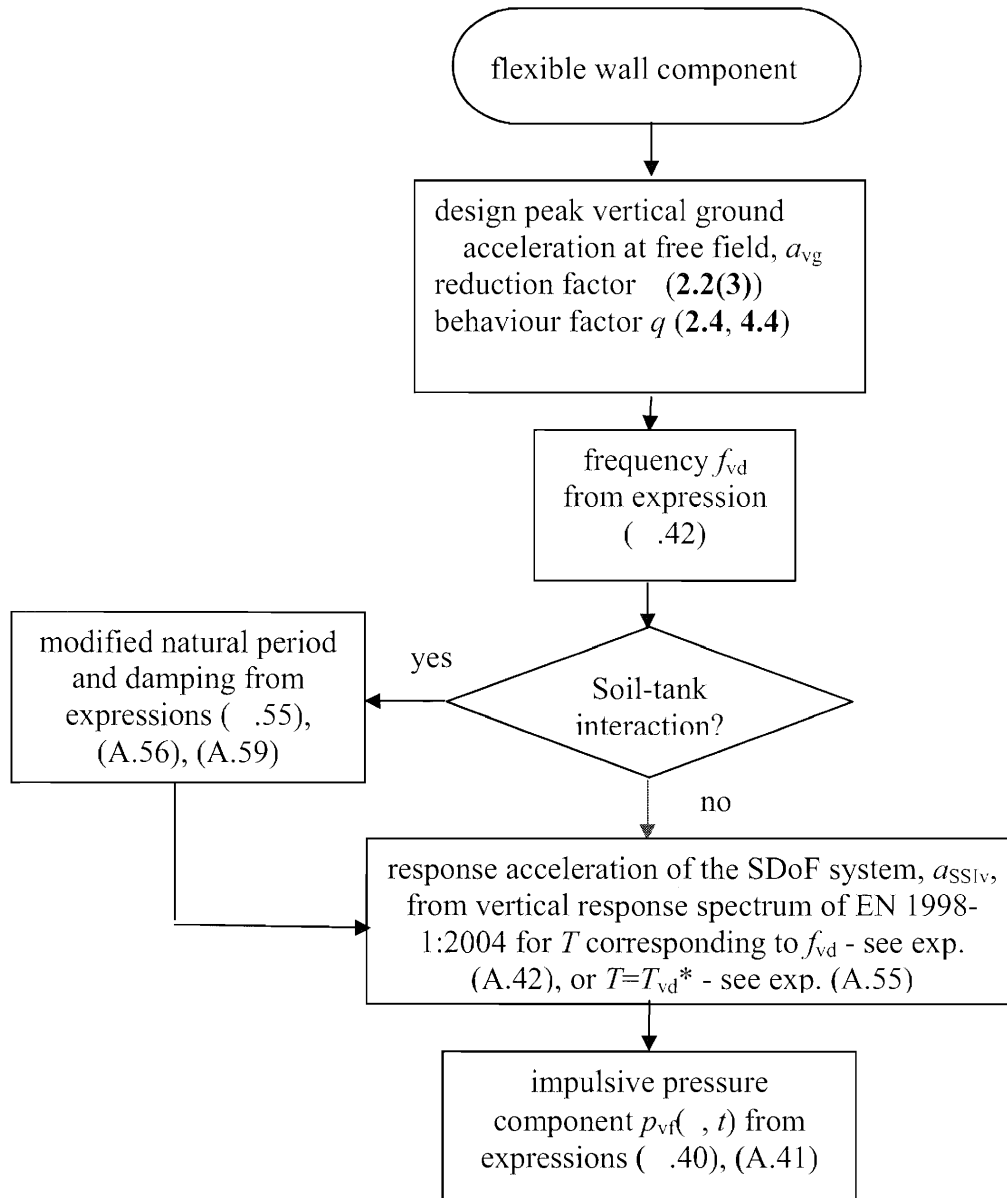
Flow chart A.3: Horizontal seismic action, flexible wall impulsive component (see A.3.1, A.7.2)



Flow chart A.4: Horizontal seismic action, convective component (see A.2.1)



Flow chart A.5: Vertical seismic action, rigid wall component (see A.2.2, A.7.2)



Flow chart A.6: Vertical seismic action, flexible wall component (see A.3.3, A.7.2)

A.9 Unanchored tanks on-ground

A.9.1 General

In tanks on-ground which are not anchored to the foundation, uplift of the tank bottom from the ground will occur due to the seismic overturning moment. Uplift is more pronounced in tanks with open top. Uplift may cause plastic deformations in the tank, especially in its base plate. Tearing and leakage of the liquid, however, should be prevented by design.

In most cases, the effects of uplift and of the accompanying rocking motion on the magnitude and the distribution of the pressures are disregarded. For most purposes this is conservative, as rocking increases the flexibility of the system and shifts the period into a range of less dynamic amplification of forces.

An approximate and iterative analysis procedure for vertical cylindrical tanks, accounting for uplift and for the dynamic nature of the problem, is given in [2], [4]. Design charts from this procedure apply to tanks with fixed roof and refer to specific parameters values, such as the ratio of wall thickness to radius, the soil stiffness, the wall foundation type, etc.

Once the peak hydrodynamic pressures are known, whether determined ignoring or considering uplift, calculation of the stresses in the tank is a matter of static structural analysis, where the designer has certain freedom in selecting the level of sophistication of the method. For an uplifting tank, an accurate model would necessarily involve a non-linear finite element model of the tank, the soil and their interface. Simplified but comprehensive computer methods have been proposed recently in the literature [15], [16]. Crude methods, not requiring the use of computer and proposed for example in [8], have been proven by experiments and more refined analyses to be unconservative and inadequate for accounting of all the variables entering the problem.

The principal effect of uplift is to increase the compressive vertical stress in the shell, which is critical for buckling-related modes of failure. At the wall which is on the side opposite to the uplifting one, vertical compression is maximum and hoop compressive stresses are generated in the shell, due to the membrane action of the base plate.

Flexural yielding is accepted to take place in the base plate, and a check of the maximum tensile stress is appropriate.

A.9.2 Compressive vertical membrane forces and stress in the wall due to uplift

The increase of the vertical membrane force due to uplift (N_u) with respect to that stress in the anchored case (N_a) may, for the usual fixed-roof cylindrical steel tanks on-ground in the petrochemical industry, be estimated from Figure A.11 [4], as a function of the nondimensional overturning moment, M/WH (W = total weight of the liquid). For slender tanks the increase is very significant. For fixed roofs, the values in Figure A.11 are on the safe side, since they have been calculated (using static finite element analysis) assuming that the underlying soil to be quite stiff (Winkler springs with a subgrade reaction modulus $k = 4000 \text{ MN/m}^3$) which is unfavourable for vertical membrane forces.

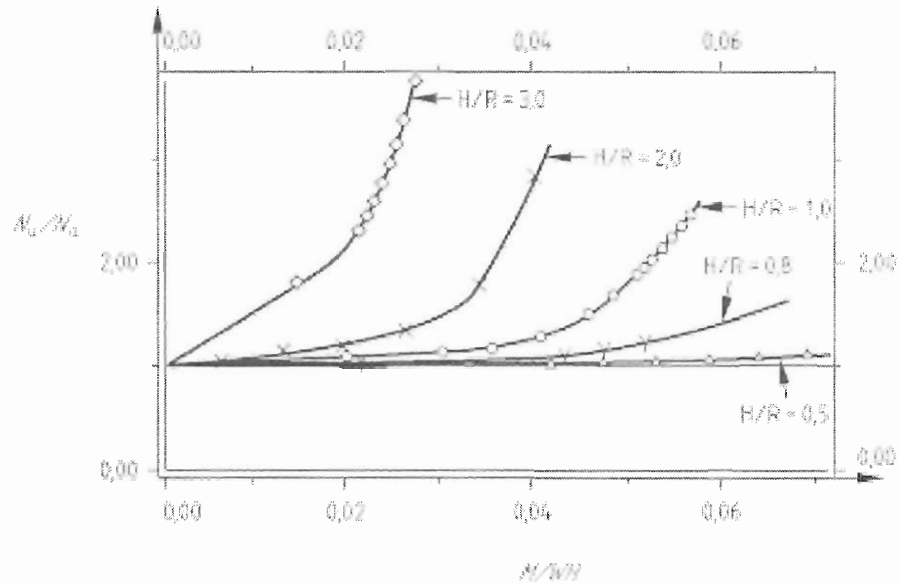


Figure A.11 — Ratio of maximum compressive axial membrane force for unanchored cylindrical tanks on ground with fixed-roof to value for anchored tank, versus overturning moment [4]

A.9.3 Shell uplift and uplifted length of the base plate

The vertical uplift at the edge of the base, w , as derived from a parametric study with finite element models of unanchored cylindrical steel tanks on-ground of commonly used geometry and fixed, fairly heavily loaded roof [4], is given in Figure A.12 as a function of the overturning moment M/WH , for different values of H/R . The results in Figure A.12 would underestimate uplift in tanks with open top or floating roof.

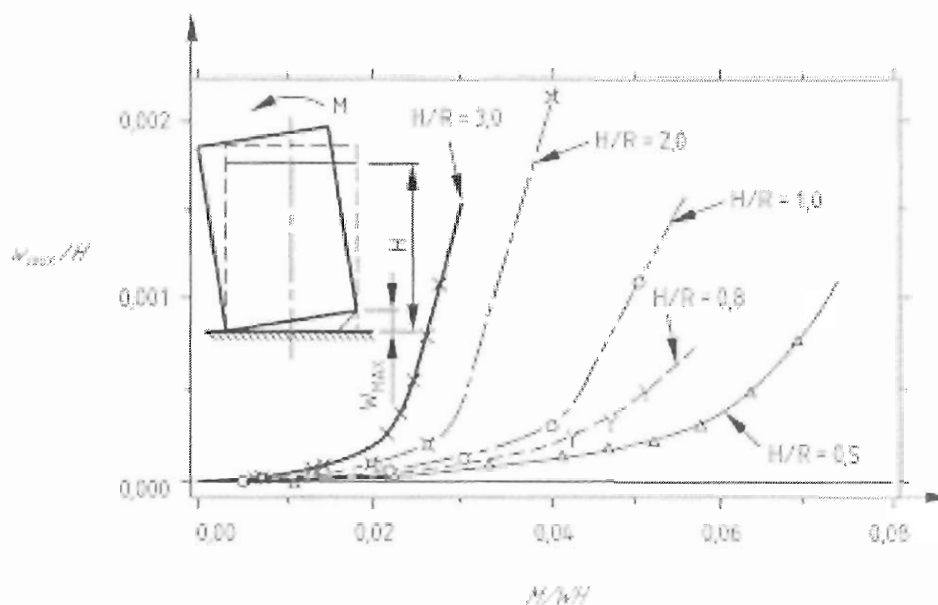


Figure A.12 — Maximum vertical uplift of fixed-roof unanchored cylindrical tanks on ground versus overturning moment M/WH [4]

For the estimation of the radial membrane stresses in the plate, the length L of the uplifted part

of the tank bottom is necessary. Results from [4] for fixed-roof tanks are shown in Figure A.13. Once uplift occurs, the dependence of L on the vertical uplift w is almost linear.

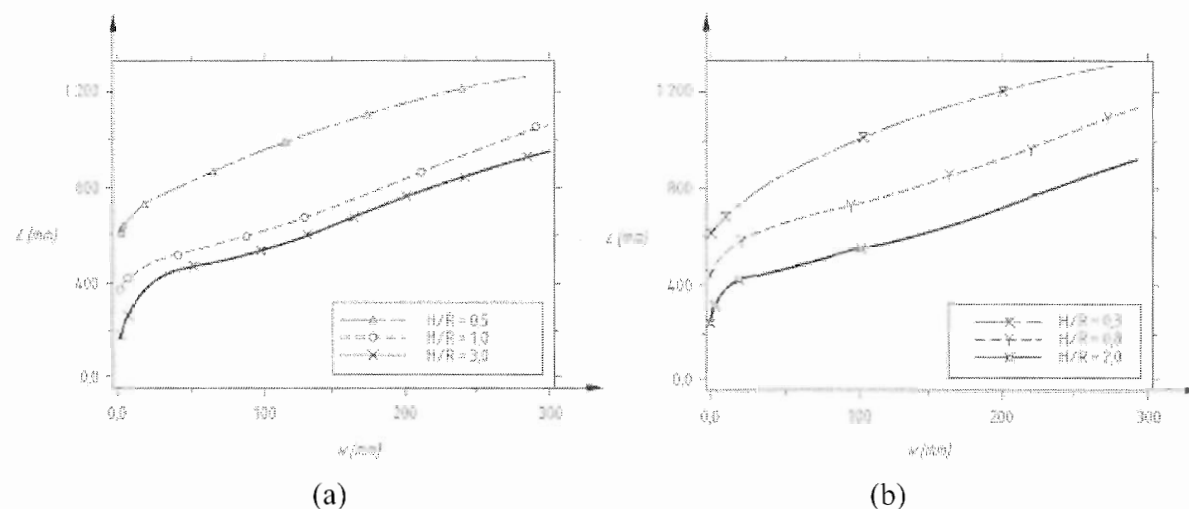


Figure A.13 — Length of uplifted part of the base in fixed-roof unanchored cylindrical tanks on ground as a function of the vertical uplift at the edge [4]

A.9.4 Radial membrane stresses in the base plate [17], [18]

An estimate of the membrane stress σ_{rb} in the base plate due to uplift is given in [17]:

$$\sigma_{rb} = \frac{1}{s} \left(\frac{2}{3} \frac{E}{1-\nu^2} s p^2 R^2 (1-\mu)^2 \right)^{1/3} \quad (\text{A.60})$$

where:

s is the thickness of the base plate;

p is the pressure on the base;

$\mu = 1 - L/(2R)$, with L = uplifted part of the base.

When significant uplift takes place in large diameter tanks, the state of stresses in the uplifted part of the base plate at the ultimate limit state is dominated by plate bending (including the effect of the pressure acting on the tank base), not by membrane stresses. In such cases the finite element method should be used for the calculation of the state of stresses.

A.9.5 Plastic rotation of the base plate

It is recommended to design the bottom annular ring with a thickness less than the wall thickness, so as to avoid flexural yielding at the base of the wall.

The rotation of the plastic hinge in the tank base should be compatible with the available flexural deformation capacity. For a maximum allowable steel strain of 0,05 and a postulated length of the plastic hinge equal to $2s$, the maximum allowable rotation is 0,20 rads. From Figure A.14 the rotation associated to an uplift at the edge w and a base separation of L is:

$$\theta = \left(\frac{2w}{L} - \frac{w}{2R} \right) \quad (\text{A.61})$$

which should be less than the estimated rotation capacity of 0,20 radians.

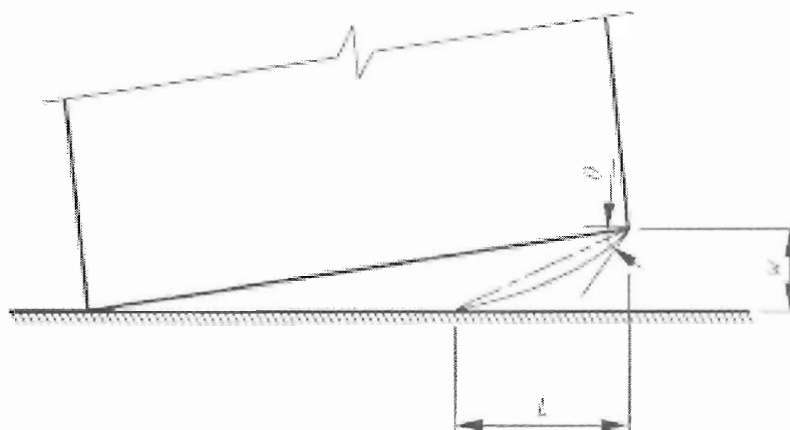


Figure A.14 — Plastic rotation of base plate of uplifting tank [8]

A.10 Verifications for steel tanks

A.10.1 Introduction

The integrity of the corner region between the base plate and the wall of anchored or unanchored tanks should be verified under the stresses and strains predicted there from the analysis for the seismic design situation. In addition, the stability of the tank wall near the base and above the base should be verified for two possible failure modes.

A.10.2 Verification of elastic buckling

This form of buckling has been observed in those parts of the shell where the thickness is reduced with respect to the thickness of the base and/or the internal pressure (which has a stabilising effect) is also reduced with respect to the maximum value attained at the base. For tanks of constant or varying wall thickness, the verification for elastic buckling should take place at the base as well as in the wall above the base. Due to the stabilising effect of the internal pressure, the verification should be based on the minimum possible value of the interior pressure in the seismic design situation.

The verification may be performed in accordance with EN 1993-1-6:200X.

As an alternative, the following inequality may be verified [19]-[23]:

$$\frac{\sigma_m}{\sigma_{cl}} \leq 0,19 + 0,81 \frac{\sigma_p}{\sigma_{cl}} \quad (\text{A.62})$$

where:

σ_m is the maximum vertical membrane stress,

$$\sigma_{cl} = 0,6 \cdot E \cdot \frac{s}{R} \quad (\text{A.63})$$

is the ideal critical buckling stress for cylinders loaded in axial compression, and

$$\sigma_p = \sigma_{cl} \left[1 - \left(1 - \frac{\bar{p}}{5} \right)^2 \left(1 - \frac{\sigma_o}{\sigma_{cl}} \right)^2 \right]^{1/2} \leq \sigma_{cl} \quad (\text{A.64})$$

where:

$$\frac{\bar{p}}{p} = \frac{pR}{s\sigma_{cl}} < 5 \quad (\text{A.65})$$

with p denoting the minimum possible interior pressure in the seismic design situation,

$$\sigma_o = f_y \left(1 - \frac{\lambda^2}{4} \right) \text{ if } \lambda^2 = \frac{f_y}{\sigma \sigma_{cl}} \leq 2 \quad (\text{A.66a})$$

$$\sigma_o = \bar{\sigma} \sigma_{cl} \quad \text{if: } \lambda^2 \geq 2 \quad (\text{A.66b})$$

$$\text{with: } \bar{\sigma} = 1 - 1,24 \left(\frac{\delta}{s} \right) \left[\left(1 + \frac{2}{1,24 \left(\frac{\delta}{s} \right)} \right)^{1/2} - 1 \right] \quad (\text{A.67})$$

and δ/s denoting the ratio of maximum imperfection amplitude to wall thickness, which may be taken as [8]:

$$\left(\frac{\delta}{s} \right) = \frac{0,06}{a} \sqrt{\frac{R}{s}} \quad (\text{A.68})$$

where:

$a = 1$ for normal construction

$a = 1,5$ for quality construction

$a = 2,5$ for very high quality construction

A.10.3 Elastic-plastic collapse

This form of buckling ('elephant's foot') normally occurs close to the base of the tank, due to a combination of vertical compressive stresses and tensile hoop stresses inducing an inelastic biaxial state of stress. In tanks with variable wall thickness, verification for this mode of buckling should not be limited to the section close to the base of the tank, but should extend to the bottom section of all parts of the wall which have constant thickness.

The empirical equation developed in [24]-[25] to check this form of instability is:

$$\sigma_m = \sigma_{cl} \left[1 - \left(\frac{pR}{sf_y} \right)^2 \right] \left(1 - \frac{1}{1,12 + r^{1,15}} \right) \left[\frac{r + f_y / 250}{r + 1} \right] \quad (\text{A.69})$$

where:

$$r = \frac{R / s}{400};$$

f_y is the yield strength of the tank wall material in MPa; and

p is the maximum possible interior pressure in the seismic design situation, in MPa.

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ANNEX B (INFORMATIVE)

BURIED PIPELINES

B.1 General design considerations

(1) As a rule, 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.

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

(3) Soil liquefaction, whenever it did occur, has been a major contributor to pipelines distress in past earthquakes.

(4) Depending on the circumstances, the solution may require either increasing the burial depth, possibly also encasing the pipes in larger stiff conduits, or in placing the pipeline above-ground, supporting it at rather large distances on well founded piers. In the latter 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. Given these parameters, the simplest way of modelling the phenomenon is to consider a rigid displacement between the soil masses interfacing at the fault.

(6) 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.

(7) In the case under consideration this can be done:

- by decreasing the burial depth so as to reduce the soil restraint;
- by providing a large ditch for the pipes, to be filled with soft material;
- by putting the pipeline above ground, and introducing flexible and extensible piping elements.

B.2 Seismic actions on buried pipelines

(1) The ground motion propagating beneath the soil surface is made up of a mixture of body (compression, shear) and surface (Rayleigh, Love, etc) waves: the actual composition depending most significantly on the focal depth and on the distance between the focus and the site.

(2) The various types of waves have different propagation velocities, and different motions of the particles (i.e. parallel to the propagation of the wave, orthogonal to it, elliptical, etc.). Although geophysical-seismological studies can provide some insight, they are generally unable to predict the actual wave pattern, so that conservative assumptions have to be made.

(3) One often made assumption is to consider in turn the wave pattern to consist entirely of a single type of wave, whatever is more unfavourable for a particular effect on the pipeline.

(4) The wave trains can in this case be easily constructed on the basis of the frequency content underlying the elastic response spectrum appropriate for the site, by assigning to each frequency component an estimated value of the propagation velocity.

(5) Theoretical arguments and a number of numerical simulations indicate that the inertia forces arising from the interaction between pipe and soil are much smaller than the forces induced by the soil deformation. This fact allows the soil-pipeline interaction problem to be reduced to a static one, i.e., one where the pipeline is deformed by the passage of a displacement wave, without consideration of dynamic effects.

(6) The forces on the pipeline can therefore be obtained by a time-history analysis, where time is a parameter whose function is to displace the wave along or across, the structure, which is connected to the soil through radial and longitudinal springs.

(7) A much simpler method is often used, whose accuracy has been proved to be comparable with the more rigorous approach described above, and which yields in any case an upper bound estimate of the strains in the pipeline, since it assumes it to be flexible enough to follow without slippage nor interaction the deformation of the soil.

(8) According to this method [1] the soil motion is represented by a single sinusoidal wave:

$$u(x,t) = d \sin \omega(t - \frac{x}{c}) \quad (\text{B.1})$$

where d is the total displacement amplitude, and c is the apparent wave speed.

(9) The particle motion is assumed in turn to be along the direction of propagation (compression waves), and normal to it (shear waves) and, for simplicity and in order to take the worst case, the pipeline axis and the direction of propagation coincide.

(10) The longitudinal particle movement produces strains in the soil and in the pipeline given by the expression:

$$\varepsilon = \frac{\partial u}{\partial x} = -\frac{\omega d}{c} \cos \omega(t - \frac{x}{c}) \quad (\text{B.2})$$

whose maximum value is:

$$\varepsilon_{\max} = \frac{v}{c} \quad (\text{B.3})$$

where:

$v = \omega d$ the peak soil velocity

(11) The transverse particle movement produces a curvature χ in the soil and in the pipe given by the expression:

$$\chi = \frac{\partial^2 u}{\partial x^2} = -\frac{\omega^2 d}{c^2} \sin \omega(t - \frac{x}{c}) \quad (\text{B.4})$$

whose maximum value is:

$$\chi_{\max} = \frac{a}{c^2} \quad (\text{B.5})$$

where:

$a = \omega^2 d$ the peak soil acceleration.

(12) For the condition of perfect bond between pipe and soil to be satisfied, the available friction force per unit length should equilibrate the variation of the longitudinal force leading to:

$$\tau_{av} = s E \frac{a}{c^2} \quad (\text{B.6})$$

where:

E Modulus of Elasticity of the pipe;

s thickness of the pipe; and

τ_{av} average shear stress between pipe and soil which depends on the friction coefficient between soil and pipe, and on the burial depth.

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