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Eurocode 8 : Design of structures for earthquake resistance

Part 4: **Silos, tanks and pipelines**

Calcul des structures pour leur résistance aux séismes

Partie 4 : Silos, réservoirs et réseaux de tuyauteries

Auslegung von Bauwerken gegen Erdbeben

Teil 4 : Silos, Tankbauwerke und Rohrleitungen

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~~English version~~

~~Eurocode 8: Design of structures for earthquake resistance~~

~~Part 4: Silos, tanks and pipelines~~

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Contents

Foreword

This document (EN 1998-4:200X) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

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of an identical text or by endorsement, at the latest by MM-200Y, and conflicting national standards shall be withdrawn at the latest by MM-20YY.

This document supersedes ENV 1998-4:1997.

CEN/TC 250 is responsible for all Structural Eurocodes.

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 harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised 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 agreement[1] 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:

EN 1990 Eurocode: Basis of structural design

EN 1991 Eurocode 1: Actions on structures

EN 1992 Eurocode 2: Design of concrete structures

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

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 recognise 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 recognise 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 harmonised 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 Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product

[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, etc. ;

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.

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 harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. 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

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[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.

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**.

EN 1998-4:200X 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 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 1998-4:200X through clauses:

Reference	**Item**

1 GENERAL

1.1 Scope

(1)P This standard aims at providing 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. This standard may also be used as a basis for evaluating the resistance of existing facilities and to assess any required strengthening.

(2) P 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, ~~however,~~ it also provides detailed methods of assessment and verification rules.

~~-~~(3) P This standard may not be complete for those facilities associated with large risks to the population or the environment, for which additional requirements shall be established by 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.

(4) The nature of lifeline systems which often characterises 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 ~~of~~ soil-structure and stored material (either ~~-~~fluid or granular)~~interaction~~, 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 organisation of this standard is to some extent different from that of companion 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, B and C for a number of typical situations.

(5) P For the formulation of the general requirements as well as for ~~their~~ its implementation, a distinction ~~can~~ shall be made between independent structures and redundant systems, via the choice of importance factors and/or through the definition of ~~adapted~~ specific verification criteria.

(6)~~P~~ A structure may~~can~~ be considered as independent when its structural and functional behaviour during and after a seismic event is not influenced by that of other structures, and if the consequences of its failure relate only to the functions demanded from it.

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 1998-1 : 2004 Eurocode 8 - Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings

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

EN 1998-6 : 200X Eurocode 8 - Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys

1.3 Assumptions

(1)P The general assumptions of EN 1990:2002, **1.3** apply.

1.4 Distinction between principles and applications rules

(1)P The rules of EN 1990:2002, **1.4** apply.

1.5 Terms and ~~d~~Definitions

1.5.1 Terms common to all Eurocodes

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

(2)P EN 1998-1: ~~200X~~2004, **1.5.1** applies for terms common to all Eurocodes.

1.5.2 Additional terms used in the present standard

(1) For the purposes of this standard the terms defined in EN 1998-1:2004, **1.5.2** apply.

1.6 Symbols

(1) For the purposes of this European Standard the following symbols apply. All symbols used in Part 4 are defined in the text when they first occur, for ease of use. In addition, a list of the symbols is given below. Some symbols occurring only in the annexes are defined therein:

NOTE: The list of symbols shall be added later on

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.

~~1.2~~2 GENERAL RULES~~SAFETY REQUIREMENTS~~

2.1 Safety requirements

~~1.2.1~~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 stored product 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.

~~1.2.2~~2.1.2 Damage limitation ~~limit~~ state

(1) P Depending on the characteristics and the purposes of the structures considered one or both of the two following damage limitation states may need to be satisfied:

– full integrity;

– minimum operating level.

(2) P In order to satisfy t~~T~~he "full integrity" requirement, ~~implies that~~ the considered system, including a specified set of accessory elements integrated with it, shall remain~~s~~ fully serviceable and leak proof under a seismic event having an annual probability of exceedance whose value is to be established based on the consequences of its loss of function and/or of the leakage of the content.

(3) P Satisfaction of the~~The~~ "minimum operating level" requirement, means that ~~implies that~~ the considered system may suffer a certain amount of damage to some of its components, to an extent, however, that after the damage control operations have been carried out, the capacity of the system can be restored up to a predefined level of operation. The seismic event 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 losses related to the reduced capacity of the system and to the necessary repairs.

PT NOTE: A more clear definition of the seismic events for the verification of these two damage limitation states has to be provided. It may become a NDP

~~1.2.3~~2.1.3 Ultimate limit state

(1)P The ultimate limit state of a system which shall be checked is defined as that corresponding to the loss of operational capacity of the system, with the possibility of partial recovery (in the measure defined by the responsible authority) conditional to an acceptable amount of repairs. ~~the limit state that guarantees the non collapse of the facility and the avoidance of uncontrolled loss of stored products.~~

(2)P For particular elements of the network, as well as for independent structures whose complete collapse would entail high risks, the ultimate limit state is defined as that of a state of damage 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 involve appreciable risks to life and property, the ultimate limit state can be defined as corresponding to total collapse.

(3)P The design seismic action for which the ultimate limit state must not be exceeded shall be established based on the direct and indirect costs caused by the collapse of the system

~~1.2.4~~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 and environmental 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, as defined in EN 1998-1:2004~~X~~, **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 with a seismic event having the reference return period indicated in EN 1998-1:200X, **3.2.1(3).**

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

(4)P For the structures within the scope of this standard it is appropriate to consider three different Importance Classes, depending on the potential ~~exposure to~~ loss of life due to the failure of the particular structure and on the environmental, 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 ~~ramifications~~ implications for public safety.

NOTE Importance classes I, II and III correspond roughly to consequences classes CC1~~3~~, CC2 and CC3~~1~~, respectively, defined in EN 1990:2002, Annex B.

(5)P Class III refers to situations with a high risk to life and large environmental, economic and social consequences.

(6)P Situations with medium risk to life and considerable environmental, economic or social consequences belong to Class II.

(7)P Class I~~II~~ refers to situations where the risk to life is low and the environmental, economic and social consequences of failure are small or negligible.

(8) A more detailed definition of the classes, specific for pipeline systems, is given in **4.2.1**

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~~X~~, **2.1(4)**) and on the public safety considerations detailed in **~~1.2.~~2.1.4**. The recommended values of γ_I are given in Table 1.1N. In the column at left there is a classification of the more common uses of these structures, while the three columns at right contain the recommended levels of protection in terms of the values of the importance factor γ_I –for three Importance Classes.

Table 2~~1~~.1N Importance factors

Use of the structure/facility	Importance Class		
	I	II	III~~3~~
Potable water supply Non-toxic, non inflammable material	0,8~~1,2~~	1,0	~~0,8~~1,2
Fire fighting water Non-volatile toxic material Low flammability petrochemicals	1,0 ~~1,4~~	1,2	~~1,0~~1,4
Volatile toxic chemicals Explosive and other high flammability liquids	1,2~~1,6~~	1,4	~~1,2~~1,6

~~1.2.5~~2.1.5 System versus element reliability

(1) P The reliability requirements set forth in **1.2.2** and **1.2.3** refer 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 shall 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 ~~shall~~ 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 should 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 can be achieved by assigning these elements to a class of reliability (expressed in terms of Importance Class) one level higher than that proper to the system as a whole.

~~1.2.6~~2.1.6 Conceptual design

(1) P Even when the overall seismic response is specified to be elastic ~~(corresponding to a value q = 1,5 for the behaviour factor)~~, 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:

– 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 redundant systems due to structural interconnection of components, the ~~necessary~~ appropriate parts should be 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.

~~1.3~~2.2 Seismic action

(1) P The seismic action to be used in the determination of the seismic action effects for the design of silos, tanks and pipelines shall be that defined in EN 1998-1:~~-~~2004~~X~~, **3.2** in the various equivalent forms of elastic, site-dependent response spectra (EN 1998-1:~~-~~2004~~X~~, **3.2.2**), and time-history representation (EN 1998-1: 200X, **3.2.3.1**). In those cases where a behaviour factor q larger than ~~the value of~~ 1,5 (considered as resulting~~derived~~ from overstrength alone) is acceptable (see **~~1.10~~2.34.2**), the design spectrum for elastic analysis shall be used (EN 1998-1:~~-200X~~2004, **3.2.2.5**). Additional provisions for the spatial variation of ground motion for buried pipelines are given in Section **5**.

(2) P The two seismic actions to be used for checking the damage limitation state and the ultimate limit state, respectively, shall be established by the competent National Authority on the basis of the seismicity of the different seismic zones and of ~~the level of~~ the ~~importance~~ Importance ~~category~~ Class of the specific facility.

(3) A reduction factor ν applied to the design seismic action, to take into account the lower return period of the seismic event associated with the damage limitation state may be considered as mentioned in EN 1998-1:~~-~~2004~~X~~, **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 requirement" should be met has the same shape as the elastic response spectrum of the design seismic action corresponding to the " ultimate limit state requirement" according to EN 1998-1:~~-~~2004,~~X~~ ~~(~~**2.1(1)**P and **3.2.1(3)**~~)~~ (See EN 1998-1:~~-~~2004,~~X~~ ~~(~~**3.2.2.1(2)**)). In the absence of more precise information, the reduction factor ν ~~applied on the design seismic action~~with the value according to EN 1998-1:~~-~~2004,~~X~~ ~~(~~**4.4.3.2(2)**~~)~~ may be used to obtain the seismic action for the verification of the damage limitation requirement.

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 0,54 for importance class~~es~~ I and ~~II and~~ ν = 0,4~~5~~ for importance classes II and III.

~~1.4~~2.3 Analysis

~~1.4.1~~2.3.1 ~~Methods of Analysis~~Methods of analysis

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

(2) P Nonlinear methods of ~~analyses~~ 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. In those cases it shall be proved that the design obtained possesses at least the same amount of reliability as the structures explicitly covered by this standard.

(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~~0X~~, **3.2.2.2** and ~~EN 1998-1: 20040X,~~ **3.2.2.3**, multiplied by the reduction factor ν ~~of~~ referred to in **~~1.9~~2.2~~3~~(3)** and 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 **~~1.10~~2.34.5** and to EN 1998-1:~~-~~2004~~0X~~, **3.2.2.2(3).**

(4)P Analysis for the evaluation of the effects of the seismic action relevant to the ultimate limit state may be elastic, using the design spectra which are specified in EN 1998-1: 2004~~0X~~, **3.2.2.5** for a damping ratio of 5% and 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%.

(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:~~-~~200~~0X~~4, **4.3.3**, namely:

a) the “lateral force method” of (linear-elastic) analysis (see EN 1998-1:~~-~~2004~~0X~~ **4.3.3.2**);

b) the “modal response spectrum” (linear-elastic) analysis (see EN 1998-1:~~-~~2004~~0X~~, **4.3.3.3);**

c) the non-linear static (pushover) analysis (see EN 1998-1:~~-~~2004~~0X~~ **4.3.3.4.2**);

d) the non-linear time history (dynamic) analysis (see EN 1998-1:~~-~~2004~~0X~~ **4.3.3.4.3).**

~~-~~(6) 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~~0X~~ apply for the modelling and analysis of the types of structures covered by the present standard.

PT NOTE: The conditions for use of each type of analysis (regularity criteria, etc.), the possible use of two planar models instead of a spatial model and the consideration of accidental eccentricity, etc., will be addressed in the 3rd Draft.

(7) The “lateral force method” of linear-elastic analysis should be performed according to ~~Clauses~~ clauses **4.3.3.2.1(1)**P, **4.3.3.2.2(1)** (with λ=1,0), **4.3.3.2.2(2)** and **4.3.3.2.3(2)**P of EN

1998-1:~~-~~2004~~0X~~. 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 "m~~M~~odal 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~~0X~~. 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. ~~This includes tanks, silos or pipelines which are not sufficiently stiff to be considered to respond to the seismic action as a rigid body.~~

(9) Non-linear analysis, static (pushover) or dynamic (time history), should satisfy EN 1998-1:~~-~~2004~~0X~~, **4.3.3.4.1.**

(10) Non-linear static (pushover) analysis should be performed according to ~~Clauses~~ clauses **4.3.3.4.2.2(1)**, **4.3.3.4.2.3**, **4.3.3.4.2.6** of EN 1998-1: 2004~~0X~~.

(11) Non-linear dynamic (time history) analysis should satisfy EN 1998-1: 2004~~0X~~, **4.3.3.4.3.**

~~1.4.2~~2.3.2 Behaviour factors

(1)P For structures covered by this standard~~, except welded steel above groung piping systems, and for the damage limitation state,~~ significant energy dissipation is not expected for the damage limitation state~~,~~. Hence, for the damage limitation state, the behaviour ~~coefficient~~ factor q shall be taken as equal to 1.

(2) ~~-~~Use of q factors greater than 1,5 ~~is only allowed~~ 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.

PT NOTE: The value of q was modified to align with the general rule in EC8 in which q =1,5 may always be used in ULS verifications due to the effect of overstrength. However this has to be checked by the PT.

~~1.4.3~~2.3.3 Damping

~~1.4.3.1~~2.3.3.1 Structural damping

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

a) Damage limitation state: ξ = 2%

b) Ultimate limit state: ξ = 5%

~~1.4.3.2~~2.3.3.2 Contents damping

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

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

~~1.4.3.3~~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~~X~~ 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 the value~~:~~ ξ = 20 %.

NOTE Guidance for the selection and use of damping values associated with different foundation motions is given in Informative Annex B~~ of EN 1998-6: 200X,~~ and in Informative Annex B~~A~~ of EN 1998-6~~4~~: 200X .

~~1.4.4~~2.3.4 Interaction with the soil

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

NOTE Additional information on procedures for accounting for soil-structure interaction is given in Informative Annex B and in Informative Annex C of EN 1998-6: 200X~~, and Informative Annex A of EN 1998-4: 200X~~.

~~1.4.5~~2.3.5 Weighted damping

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

NOTE A procedure for accounting for the contributions of the different materials/elements to the global average damping of the ~~whole~~ system is given in Informative Annex B of EN 1998-6.

~~1.5~~2.4 Safety verifications

~~1.5.1~~2.4.1 General

(1) P Safety verifications shall be carried out for the limit states defined in ~~**1.2**~~**2.1**, following the specific provisions in ~~**2.4**~~**3.5**, ~~**3.5**~~**4.4, 5.5** and ~~**4.5**~~**6.4**.

(2) If plate thickness is increased to account for future corrosion effects, the verifications shall be made for both the non-increased and the increased thickness.

~~1.5.2~~2.4.2 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~~X~~, **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) -The combination coefficients ψ_{2i} (for the quasi-permanent value of variable action q_i) shall be those given in EN 1990:2002, Annex A4. 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 ψ_{2i}.

NOTE : Informative Annex A of EN1991-4 provides information for the combination coefficients ψ_{2i} (for the quasi-permanent value of variable action q_i) to be used for silos and tanks in the seismic design situation.

PT NOTE: The Note and the text may have to be adjusted at a later stage, in view of the final contents of the Annexes of EN1990 and EN1991-4.

-(~~2~~4) P The effects of the contents shall be considered in the variable loads for various levels of filling. In groups of silos and tanks, different likely distributions of full and empty compartments shall be considered according to the operation rules of the facility. At least, the design situations where all compartments are either empty or full shall be considered.

~~2~~3 SPECIFIC RULES FOR SILOS

~~2.1~~3.1 Properties of stored solids and d~~D~~ynamic ~~over~~pressures

(1)P Annexes C, D and E of EN1991-4~~-~~:~~-~~200X apply for the determination of the properties of the particulate solid stored in the silo. The upper characteristic value of the solid unit weight presented in EN1991-4~~-~~:~~-~~200X, Table E1, shall be used in all calculations.

(2)P 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 at rest. For design purposes this increased pressure is deemed to be included in the effects of the inertia forces acting on the stored material due to the seismic excitation (see 3.3(5).~~This increased pressure is deemedassumed to be covered by the the effects of the inertia forces due to the seismic excitation.~~

~~2.2~~3.2 Combination of ground motion components

(1) P Silos shall be designed for simultaneous action of the two horizontal components and of the vertical component of the seismic action. If the structure is axisymmetric, it is allowed to consider only one horizontal component.

(2) When the structural response to each component of the seismic action is evaluated separately, EN1998-1:~~-~~2004~~X~~, **4.3.3.5.2(4)** may be applied for the determination of the most unfavourable effect of the application of the simultaneous components. If expressions (4.20), (4.21), (4.22) in EN1998-1:~~-~~2004~~X~~, **4.3.3.5.2(4)** are applied for the computation of the action effects of the simultaneous components, the sign of the action effect ~~of~~ due to each individual component shall be taken as ~~being~~ the most unfavourable for the particular action effect under consideration.

(3) 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.

~~2.3~~3.3 Analysis

~~NOTE Information on seismic analysis of vertical cylindrical silos are given in Informative Annex A.~~

(1) The –following subclauses provide rules additional to those of ~~1.10~~**2.34** which are specific to silos.

NOTE Additional information on seismic analysis of vertical cylindrical silos is given in Informative Annex A.

(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 provisions of EN 1993-4-1~~-~~:~~-~~200X,

Section 4, apply ~~rules~~ for the modelling and analysis of steel silos. ~~Numerical values for characteristics of infilled materials are given in EN1991-4: Annex E.~~

(3) P Silos shall be analysed considering elastic behaviour, 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 of the silo 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 ~~through~~ to modelling the mechanical properties and the dynamic response of the particulate solid ~~with Finite Elements~~), the effect on the shell of the~~its~~ 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 the following paragraphs.~~:~~

(6) For circular silos (or silo compartments):

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

where

~~the reference pressure~~ $\Delta_{ph,so}$ is the reference pressure given in **(8)** of this subclause

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

(7) For rectangular silos (or silo compartments) with walls parallel or normal to the horizontal component of the seismic action:

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

$$\Delta_{ph,s} = \Delta_{ph,so}$$

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

$$\Delta_{ph,s} = -\Delta_{ph,so}$$

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

$$\Delta_{ph,s} = 0$$

(8) At points on the wall with a vertical distance, z, from the hopper greater or equal to one-third of R_s* defined as:

$$R_s^* = \min(H, B_s/2)$$

where:

H~~:~~ is the silo height;

B_s~~:~~ is the horizontal dimension of the silo parallel to the horizontal component of the seismic action (Diameter, D=2R, in circular silos or silo compartments, width b parallel to the horizontal component of the seismic action in rectangular ones),

the reference pressure $\Delta_{ph,so}$ may be taken as:

$$\Delta_{ph,so} = \alpha\cancel{a}(z)\,\gamma R_s^*$$

where:

$\alpha\cancel{a}(z)$~~:~~ is the ratio of the response acceleration ~~(in g's)~~ of the silo at the level of interest, z to the acceleration of gravity

γ~~:~~ is the bulk unit weight of the particulate material (upper characteristic value, see EN1991-4~~-~~:-200X Table E1).

(9) At the top of the silo, f~~Due to the transfer of inertia forces to the bottom of the silo, rather than to its walls, within the part of the height of the silo f~~rom z = 0 to z = R_s*/3, the value of $\Delta_{ph,so}$ increases linearly from $\Delta_{ph,so}$ =0 at z = 0 to the full value of expression (2.6) at z = R_s*/3.

(10) If only the value of the response acceleration at the centre of gravity of the particulate material is available (see, e.g., ~~1.10~~2.34.1(7) and paragraph **(4)** of the present subclause) the corresponding ratio ~~at value~~ to the acceleration of gravity may be used in expression (2.~~,~~6) for $\alpha\cancel{a}(z)$.

(11) The value of $\Delta_{ph,s}$ at any ~~certain vertical distance z from the hopper and~~ location on the silo wall is limited by the condition that the sum of the static pressure of the particulate material on the wall and ~~of~~ the additional pressure~~one~~ given by expressions (2.1) to ~~-~~(2.4) may not be taken less than zero.

~~2.4~~3.4 Behaviour factors

(1)P The supporting structure of ~~earthquake-resistant~~ silos shall be designed according to one of the following concepts (see **5.2.1**, **6.1.2**, **7.1.2** in EN 1998-1:~~-~~2004~~X~~):

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~~X~~, **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~~Low~~) and is recommended only for low seismicity cases (see EN 1998-1: 2004~~X~~, **3.2.1(4)**). Selection of materials, evaluation of resistance and detailing of members

and connections should be as specified in EN 1998-1: 2004~~X~~, Section **5** to **7,** for ductility class Low (DCL).

(3) In concept b) the capability of parts of the supporting structure ~~(its dissipative zones)~~ 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~~X~~, Section **5** to **7**, depending on the structural material of the ~~the~~ supporting structure. They should meet the specific requirements specified therein regarding structural type, materials and dimensioning and detailing of members or connections for ductility. When using the design spectrum for elastic analysis defined in EN 1998-1:~~-~~2004~~X~~, **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).

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

- For silos supported on a single pedestal or skirt, or on irregular bracings, the upper limit of the q factors are those defined for inverted pendulum structures.

- For silos supported on moment resisting frames or on regular bracings, the upper limit of the q factors are 1,25 times the values defined ~~applying~~ for inverted pendulum structures.

- For cast-in-place concrete silos supported on concrete walls which are continuous to the foundation, the upper limit of the q factors are 1,5 times the values ~~applying~~ defined for inverted pendulum structures.

~~2.5~~3.5 Verifications

~~2.5.1~~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 serviceability limit state verifications required by EN 1992-1-1, EN 1992-3 and EN 1993-4-1.

~~-~~(2) For steel silos, adequate reliability with respect to the occurrence of elastic or inelastic buckling phenomena is assured, if the verifications regarding these phenomena are satisfied under the seismic design situation for the ultimate limit state.

~~2.5.2~~3.5.2 Ultimate limit state

~~2.5.2.1~~3.5.2.1 Global stability

(1) P Overturning, sliding 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~~of its~~ foundation, shall be evaluated taking into account the effects of the vertical component of the seismic action. A limited sliding may be acceptable, if the structure is monolithic and is not connected to any piping (see also EN 1998-5:~~-~~2004~~X~~, **5.4.1.1(7)**).

(2) P Uplift is acceptable if it is adequately taken into account in the analysis and in the subsequent verifications of both the structure and of the foundation.

~~2.5.2.2~~3.5.2.2 Shell

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

~~-:~~ F~~: f~~or steel shells:~~,~~ yielding (plastic collapse), buckling in shear or by vertical compression with simultaneous transverse tension ("elephant foot" mode of failure), etc. (see EN 1993-4-1 : 200X, Sections **5** to **9**).

- F~~; f~~or 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, EN1993-1-1, EN1993-1-5, EN1993-1-6, EN1993-1-7 and EN 1993-4-1.

~~2.5.2.3~~3.5.2.3 Anchors

(1) Anchoring systems should 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 should 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.

~~(1) P Anchoring systems shall be designed to remain elastic in the seismic design situation. They shall also be provided with sufficient ductility, so as to avoid brittle failures. If the anchorage system is part of the dissipating mechanisms, then it shall be appropriately verified. Their 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 anchoring elements.~~

~~2.5.2.4~~3.5.2.4 Foundations

(1) P The foundation shall be verified according to EN 1998-5:~~-~~200X, **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 according to EN 1998-5: 2004~~X~~, **5.3.1,** to EN 1998-1: 2004~~X~~, **4.4.2.6** and to EN 1998-1: 2004~~X~~, **5.8**.

~~3~~4 SPECIFIC RULES FOR TANKS

~~3.1~~4.1 Compliance criteria

~~3.1.1~~4.1.1 General

(1) P The general requirements set forth in ~~1.8~~2~~2~~.1 are deemed to be satisfied if, in addition to the verifications specified in ~~3~~4.4, the complementary measures indicated in ~~3~~4.5 are also satisfied.

~~3.1.2~~4.1.2 Damage limitation state

(1) P It shall be ensured that under the ~~relevant~~ seismic ~~design situation~~actions relevant ~~and in respect~~ to the "full integrity" limit state ~~or~~ and to the "minimum operating level" limit state:

a) Full integrity

– The tank system maintains its tightness against leakage of the contents. Adequate freeboard shall be provided, in order to prevent damage to the roof due to the pressures of the sloshing liquid or, if the tank has no rigid roof, to prevent the liquid from spilling over;

– The hydraulic systems which are part of, or are connected to the tank, are capable of accommodating stresses and distortions due to relative displacements between tanks or between tanks and soil, without their functions being impaired;

b) Minimum operating level

– Local buckling, if it occurs, does not trigger collapse and is reversible; for instance, local buckling of struts due to stress concentration is acceptable.

NOTE: The final wording of this clause may have to be adjusted in view of the Note presented in 2.1.2 and a NDP may be needed here.

~~3.1.3~~4.1.3 Ultimate limit state

(1) P It shall be ensured that under the ~~relevant~~ seismic design situation:

– The overall stability of the tank is ensured according to EN 1998-1:~~-~~2004~~X~~, **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 EN according to 1998-5: 2004~~X~~, **5.4.1.1(7)** if tolerated by the pipe system and the tank is not anchored to the ground;

– Inelastic behaviour is restricted within limited portions of the tank, and the ultimate deformations of the materials are not exceeded;

– The nature and the extent of buckling phenomena in the shell are adequately controlled;

– The hydraulic systems which are part of, or connected to the tank are designed so as to prevent loss of the tank content following failure of any of its components;

~~3.2~~4.2 Combination of ground motion components

(1) P Clause ~~2~~3.2(1)P applies to tanks.

(2) Clause ~~2~~3.2(2) applies to tanks.

(3) P Clause ~~2~~3.2(3)P applies to tanks.

~~3.3~~4.3 Methods of analysis

~~3.3.1~~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 ~~, and~~the interaction with the foundation soil, ~~when necessary~~.

(2) P Tanks shall be generally analysed considering elastic behaviour, unless proper justification is given for the use of nonlinear analysis in particular cases.

NOTE Information on m~~M~~ethods for seismic analysis of tanks of usual shapes are given in Informative Annex B.

(3) P The ~~localized~~localised non linear phenomena, admitted in the seismic design situation for which the ultimate limit state is verified (see ~~3~~4.1.3), shall be restricted so as to not affect the global dynamic response of the tank to any significant extent.

(4) Possible interaction between different tanks due to connecting piping~~s~~ shall be considered whenever appropriate.

~~3.3.2~~4.3.2 Behaviour factors

(1) P Tanks of type other than those mentioned below shall be either designed for elastic response (q up to 1,5, accounting for overstrength), or, for properly justified cases, for inelastic response (see ~~1.10~~2.3~~4~~4.1(2)), provided that its~~the~~ acceptability ~~of their inelastic response~~ is~~shall be~~ adequately demonstrated.

(2)P Clause ~~2~~3.4 applies also to elevated tanks.

(3)P For non-elevated tanks ~~other than those of (2)~~, the energy dissipation corresponding to the selected value of q shall be properly substantiated and the necessary ductility provided through ductile design. However, t~~T~~he ~~full~~ elastic response spectra (see EN 1998-1:2004, 3.2.2.2 and 3.2.2.3) ~~elastic design action (i.e., q = 1), however,~~ shall, in all cases, be used for the evaluation of the convective part of the liquid response.

(5) Steel tanks with vertical axis, supported directly on the ground or on the foundation may be designed with a behaviour factor q greater than~~>~~ 1 provided that the tank is designed in such way to allow uplift. Unless~~If~~ the inelastic behaviour is ~~not justified~~ evaluated by any more refined ~~scientifically proven~~ approach, the behaviour factor q ~~may~~ should not be ~~be~~ taken larger than~~equal to~~:

– 1,5 for unanchored tanks, provided that the design rules of EN 1993-4-2 are fulfilled, especially those concerning the thickness of the bottom plate, which shall be less than the thickness of the lower shell course.

–2 for tanks with specially designed ductile anchors allowing an ~~elongation~~ increase in length without rupture~~,~~ equal to R/200, where R is the tank radius.

–

~~3.3.3~~4.3.3 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 design seismic actions defined in ~~1.9~~2.2~~3~~.

(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.

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

(4) Determination of the ~~critical~~ 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 B gives information on acceptable procedures for the combination of the peak modal contributions in response spectrum analysis.~~.~~ It ~~nformative Annex B~~ gives also ~~appropriate~~ expressions for the calculation of the sloshing wave height.

~~3.4~~4.4 Verifications

~~3.4.1~~4.4.1 Damage limitation state

(1) P Under the ~~In the~~ seismic action ~~design situation~~ relevant to the damage limitation state, ~~if it is specified,~~ the tank structure shall be checked to satisfy the -serviceability limit state verifications of the relevant material Eurocodes for tanks or liquid-retaining structures.

NOTE: The issue of damage limitation states has to be re-checked, as there are no explicit compliance criteria.

~~3.4.1.1~~4.4.1.1 Shell

4~~3~~.4.1.1.1 Reinforced and prestressed concrete shells

(1) Calculated crack widths in the seismic design situation relevant to the damage limitation state, may be compared to the values specified in ~~clause 4.4.2 of~~ 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 shall not exceed a value that might induce local deformation in the liner exceeding 50% of its ultimate uniform elongation.

4~~3~~.4.1.1.2 Steel shells

(1) Clause 2~~3~~.5.1(2) applies to tanks.

~~3.4.1.2~~4.4.1.2 Piping

(1) Piping needs to be verified for the damage limitation state only if special requirements are imposed to active on-line components, such as valves or pumps

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

(3) If reliable data are not available or accurate analyses are not made, a minimum value of the imposed relative displacement between the first anchoring point of the piping and the tank may be assumed as:

$$\Delta = \frac{x d_g}{500} \tag{3.1}$$

where x (in m~~m~~) is the distance between the anchoring point of the piping and the point of connection with the tank, and d_g is the design ground displacement as given in EN 1998-1: 2004~~X~~, **3.2.2.4(1)**.

(4)P The resistance of piping elements shall be evaluated as ~~taken equal to that applying~~in the ~~in the~~ persistent or transient design situations.

(5) 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 $\gamma_p = 1{,}3$.

~~3.4.2~~4.4.2 Ultimate limit state

~~3.4.2.1~~4.4.2.1 Stability

(1) P Clause 2~~3~~.5.2.1(1)P applies to tanks.

(2) P Clause 2~~3~~.5.2.1(2)P applies to tanks.

~~3.4.2.2~~4.4.2.2 Shell

(1) P Clause 3~~2~~.5.2.2(1) applies to tanks.

NOTE : Information on ~~Appropriate~~ expressions for checking the ultimate strength capacity of the shell, as controlled by various failure modes are given in Informative Annex B~~A~~.

~~3.4.2.3~~4.4.2.3 Piping

(1) P Under the combined effects of inertia and service loads, as well as under the imposed relative displacements, yielding of the piping at the connection to the tank shall not occur. The connection of the piping to the tank shall have an overstrength factor of not less than 1,~~.~~3 with respect to the piping.

~~3.4.2.4~~4.4.2.4 Anchorages

(1) P Clause 2.5.2.3(1) applies to tanks.

~~3.4.2.5~~4.4.2.5 Foundations

(1) P Clause ~~2~~3.5.2.4(1)P applies to tanks.

(2) P Clause 3~~2~~.5.2.4(2)P applies to tanks.

~~3.5~~4.5 Complementary measures

~~3.5.1~~4.5.1 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), if the seismic action used for the verification of the damage limitation state is smaller than the design seismic action (used for the verification of the ultimate limit state).

(2) P If tanks are built in groups, bunding ~~shall~~ may be provided either to every individual tank or to the whole group. However, if the consequences ~~, depending on the risk~~ associated with the failure of the bund are 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 considered for the ultimate limit state of the enclosed system.

~~3.5.2~~4.5.2 Sloshing

(1) P In the absence of explicit justifications, a freeboard shall be provided having a height not less than the calculated height of the sloshing waves (see ~~referred to in~~ ~~3~~4.3.3(4~~5~~)).

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

~~3.5.3~~4.5.3 Piping interaction

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

~~4~~5 SPECIFIC RULES FOR ABOVE-GROUND PIPELINES

~~4.1~~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~~ 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 ~~determination~~ of the ~~supports~~ 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.

(4)~~-~~ Explicit treatment of these facilities, however, is not within the scope of this standard. ~~; i~~In fact, some of those facilities are ~~already~~ 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.

(4) P For the formulation of the general requirements to follow, as well as for their implementation, a distinction ~~needs to be~~is made among the pipeline systems covered by the present standard i.e.:

- single lines
- ~~and~~ redundant networks.

(5) P For this purpose, a pipeline is 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.

~~4.2~~5.2 Safety r~~R~~equirements

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 event defined for the "Minimum operating level" (see **2.1.2**), even if with considerable local damage.

(2) A global deformation up to 1,5 times the 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 size of the population that is exposed to the impact of rupture shall be considered in establishing the level of protection.

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

~~4.2.1~~5.2.3 Reliability differentiation

(1) P For purposes of reliability differentiation the different components in a pipeline system are classified as follows:~~.~~

Importance Class I: Buildings, facilities and equipment that may deform inelastically to a moderate extent without unacceptable loss of function (non-critical piping support structures, buildings enclosing process operations, etc). It is unlikely that failure of the component will cause extensive loss of life.~~Structures and equipment performing vital functions that shall remain nearly elastic. Items that are essential for the safe operation of the pipeline or any facility, or components that would cause extensive loss of life or a major impact on the environment in case of damage. Other items, which are required to remain functional to avoid damage that would cause a lengthy shutdown of the facility (emergency communications systems, leak detection, fire control, etc.).~~

Importance Class II: Items that ~~shall~~ must remain operational after an earthquake, but need not operate during the event; Structures that may deform slightly in the inelastic range; Facilities that are ~~vital~~important, but whose service may be interrupted until minor repairs are made. It is unlikely that failure of the component will cause extensive loss of life.

Importance Class III: Structures and equipment performing vital functions that must remain nearly elastic. Items that are essential for the safe operation of the pipeline or any facility. Components that would cause extensive loss of life or have a major impact on the environment in case of damage. Other items, which are required to remain functional to avoid damage that would cause a lengthy shutdown of the facility (emergency communications systems, leak detection, fire control, etc.).~~Buildings, facilities and equipment that may deform inelastically to a moderate extent without unacceptable loss of function (noncritical piping support structures, buildings enclosing process operations, etc). It is unlikely that failure of the component will cause extensive loss of life.~~

(2) The values of the importance factors appropriate to each class and as function of the use of the facility are given in Table 2~~1~~.1N of ~~1.8~~**2.1**~~2~~**.4 (4)**.

~~4.2.2 Damage limitation requirements~~

~~(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 as much as possible, even under considerable local damage due to high intensity earthquakes.~~

~~For this, a global deformation up to 1.5 times the yield deformation is acceptable, provided there is no risk of buckling and the loads applied to active equipment, such as valves, pumps, etc.; are acceptable.~~

~~4.2.3 Safety requirements~~

~~(1) P The principal safety hazard directly associated with the pipeline rupture under a seismic event is explosion and fire, particularly with regard to gas pipelines. The remoteness of the location and the size of the population that is exposed to the impact of rupture shall be considered in establishing the level of protection.~~

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

~~4.3~~5.3 Seismic action

~~4.3.1~~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:

~~a)~~- Shaking of the pipelines due to the seismic movement applied to their supports.

~~b)~~- 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 present only if there are soil failures and/or permanent deformations

- For supports which are located on different structures its seismic response may create differential movements on the pipeline;

~~4.3.2~~5.3.2 Earthquake vibrations

(1) P The quantification of the~~one~~ horizontal components of the earthquake vibrations shall be carried out in terms of the~~a~~ response spectrum, (or a compatible time history representation ~~(mutually consistent~~) as presented in ~~of~~ EN 1998-1:~~200X~~2004, **3.2.2**~~, which is referred to as containing the basic definitions~~.

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

~~4.3.3~~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 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 analysis or by simplified envelope approaches.

~~4.4~~5.4 Methods of analysis

~~4.4.1Above ground pipelines~~

~~4.4.1.1~~5.4.1 ~~Modeling~~Modelling

(1) P The model of the pipeline shall be able to represent the stiffness, the ~~and~~ 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

~~4.4.1.2~~5.4.2 Analysis

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

NOTE Additional information regarding the combination of modal responses, namely for the use of the Complete Quadratic Combination is given in EN 1998-2: 2004, **4.2.1.3**.

(2) Time history analysis with spectrum compatible accelerograms according to EN~~V~~ 1998-1:2004~~X~~, **3.2.3** is also allowed.

(3) Simplified ~~static~~ lateral force analyses are acceptable, provided that the value of the applied acceleration is justified. A value equal to 1~~.~~,5 times the peak of the support spectrum is acceptable.

PT NOTE: This rule is under discussion. Possible link to cl.4.3.5.2 of EN1998-1:2004.

(~~2~~4)P The seismic action shall be applied separately along two orthogonal directions (transverse and longitudinal, for straight pipelines) and the maximum combined response shall be obtained according to ~~, if the response spectrum approach is used, by using the SRSSrule~~EN 1998-1:2004, **4.3.3.5.1(2)** and **(3)**.

~~(3) Guidance on the choice between the two methods is given in EN 1998-2: 200X, 4.2.1.3.~~

(4~~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.

NOTE Appropriate models to take into account the spatial variability of the motion are given in Informative Annex D of EN 1998-2: 200X

~~4.4.1.3~~5.4.3 Behaviour factors

(1) The dissipative capacity of an above-ground pipeline, if any, is restricted to its supporting structure, since it ~~would be~~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 of the behaviour factors of general applicability.

(2) For the supporting structures, 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.

~~(2)~~(3) Welded steel pipelines exhibit significant deformation and dissipation capacity, ~~as soon as~~provided that their thickness is sufficient. For pipelines which have a radius over thickness ~~(R/t)~~ ratio (R/t) less than 50, the behaviour factor, q, to be used for the verification of the pipes ~~shall~~ may be taken equal to 3. If this ratio is less than 100, q ~~shall~~ may be taken equal to 2. Otherwise, q ~~is~~ may be taken equal to 1.

PT NOTE: Possible use q=1,5 as the minimum on account for overstrength is under discussion .

(4) For the verification of the supports, the seismic loads derived from the analysis should be multiplied by $(1+q)/2$.

PT NOTE: It has to be clarified whether the q factor takes the value of the behaviour factor used for the verification of the pipelines or of the supporting structure.

~~(3)For other cases, 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.~~

~~4.5~~5.5 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 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 defined for the "Minimum operating level" requirement, it shall be verified that the joints do not suffer damage inducing loss of tightness.~~the joints shall not suffer damage incompatible with the specified serviceability requirements.~~

~~5~~6 SPECIFIC RULES FOR BURIED PIPELINES

~~5.1~~6.1 General

(1) P This Section aims at providing principles and application rules for the evaluation of the earthquake resistance of buried pipeline systems. ~~This wording allows for~~It applies both for the design of new and for the evaluation of existing systems.

(2) P Although large diameter pipelines are within the scope of this standard, the corresponding design criteria may not be used for apparently similar facilities, like tunnels and large underground cavities.

(3) Even though ~~various~~ distinction~~s~~ ~~can~~ could 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.

(4) 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.

(5) As an example of the latter situation, 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.

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

~~5.2~~6.2 Safety r~~R~~equirements

6.2.1 Damage limitation state

(1)P Buried pipeline systems shall be constructed in such a way as to maintain their integrity or some of their supplying capacity after the seismic events defined for the “Full integrity” or “Minimum operating level” (see **2.1.2**), even if with considerable local damage..

6.2.2 Ultimate limit state

(1)P Clause **5.2.2(1)**P applies to buried pipelines.

(2)P Clause **5.2.2(2)**P applies to buried pipelines.

~~5.2.1~~6.2.3 Reliability differentiation

(1) 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 amount of damage is acceptable, need not be designed to such stringent criteria (see **2.1.4**)~~Under such circumstances, where seismic resistance is deemed to be important, critical components (pumps, compressors, control equipment, etc.) shall be designed under criteria that provide for sufficient integrity in the event of a major severe earthquake. Other components, that are less essential and are allowed to sustain greater amounts of damage, need not be designed to such stringent criteria.~~

(2) P Clause **5.2.3(1)**P applies to buried pipelines. ~~In order to adapt the reliability to the importance of the stakes, the different elements in a pipeline system shall be classified as follows.~~

~~Class I:~~ ~~Two types of pipeline system elements are considered: those for which integrity shall be assured due to the risk they represent for their environment, and those which shall remain operational after the earthquake (significant example: water supply for fire fighting). The elements of this class may undergo limited plastic deformations, which are compatible with the above requirements.~~

~~Class II:~~ ~~The elements of pipeline systems which present a limited or negligible risk. The elements of this class may undergo moderate plastic deformations.~~

(3) Clause **5.2.3(2)** applies to buried pipelines.

~~5.2.2 Damage limitation requirements~~

~~(1)P Buried pipeline systems shall be constructed in such a way as to maintain their integrity, or in special cases, when absolutely needed, some of their supplying capacity, specifically identified for given purposes, even under considerable local damage due to high intensity earthquakes.~~

~~5.2.3 Safety requirements~~

~~(1)P The risks to which goods, people and the environment are exposed in the vicinity of a pipeline system depend on various factors, either linked to the pipeline, like the transported fluid, its pressure, the pipeline diameter, etc., or linked to the environment of the pipeline: all the human, economical and environmental factors in the considered site, which are also designated by "what is at stake".~~

~~(2) P The importance of what is at stake, together with the importance of the seismic hazard, define the risk level. It's the latter which is managed by means of the pipeline design.~~

~~5.3~~6.3 Seismic action

~~5.3.1~~6.3.1 General

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

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 ~~the~~ damage limitation and the ultimate limit state shall, ~~in principle, to~~ be satisfied for all of the types of hazards listed above.

(3) However, for~~-~~ the hazards of type b) listed above it can be generally assumed that satisfaction of the ultimate limit state provides the satisfaction of the damage limitation requirements, so that only one check has to be performed.

~~The general requirements regarding the damage limitation state shall only be satisfied for utilities which need to remain functional after an earthquake (fire-fighting for example).~~

(~~3~~4) The fact that pipeline systems traverse or extend over large geographical areas, and the necessity of connecting certain locations, does not always allow for the best choices regarding the nature of the supporting soil. Furthermore, it may not be feasible to avoid crossing potentially active faults, or to avoid laying the pipelines in soils susceptible to liquefaction, as well as in areas that can be affected by seismically induced landslides and large differential permanent ground deformations.

(5)~~-~~ This situation 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, i~~(4) In~~n most cases, the occurrence of hazards of type b) in **(1)**P ~~simply~~ cannot be ruled out. Based on available data and experience, reasoned assumptions ~~may~~ should be used to define a model for that~~e~~ hazard.

~~5.3.2~~6.3.2 Earthquake vibrations

(1)P The quantification of the components of the earthquake vibrations is given in ~~1.9~~2.2~~3~~.

~~5.3.3~~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 C 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 also dependent 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 considered and of the corresponding wave propagation velocities shall be based on geophysical considerations.

~~5.3.4~~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 adequately evaluated.

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

~~5.4~~6.5 Verifications

~~5.4.1~~6.5.1 General

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

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

~~5.4.1.1~~6.5.1.1 Buried pipelines on stable soil ~~(Ultimate limit state)~~

(1) The response quantities to be obtained from the analysis are 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.

~~a) welded steel pipelines~~

(2)P In welded steel pipelines t~~T~~he 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: 5%

– allowable compressive strain: min~~imum~~ ~~(~~{1~~%,~~ %; 40.t / D (%)}~~)~~

where t and D are the thickness and diameter of the pipe respectively.

~~b) Concrete pipelines~~

(3)P In concrete pipelines, u~~U~~nder the most unfavourable combination of axial strain and curvature, due to the design seismic action, the section of the pipe ~~:~~shall not exceed the ~~ultimate compressive~~limiting strains of concrete and steel.

~~– shall not exceed a tensile strain of steel such as to produce residual crack widths incompatible with the specified requirements.~~

(4)P In concrete pipelines, under the most unfavourable combination of axial strain and curvature, due to the seismic action for the damage limitation state, the tensile strain of the reinforcing steel shall not exceed the limiting values as to produce residual crack widths incompatible with the tightness requirements.

(4~~5~~)P Under the most unfavourable combination of axial and rotational deformations, the joints shall not suffer damage incompatible with the specified serviceability requirements.~~.~~

~~5.4.1.2~~6.5.1.2 Buried pipelines under differential ground movements (welded steel pipes) (~~ultimate limit state)~~

(1)~~P The load effects induced in the supporting elements (piers, frames, etc) by the seismic design situation shall be less than or equal to the resistance evaluated as for the persistent or transient design situation~~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 checked not to exceed the available ductility of the material in tension and not to buckle locally or globally in compression. The limit strains are those indicated in **6.5.1.1**.

~~(2)P Under the most unfavourable combination of axial and rotational deformations, the joints shall not suffer damages incompatible with the specified serviceability requirements.~~

~~(3) For the pipeline itself the relevant provisions in 5.5.1.1 apply.~~

~~5.5~~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 following considerations will generally improve the capability of the pipeline to withstand differential movements along the fault:

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

b) Reverse faults should be intersected at an oblique angle, which should be as small as possible, to ~~minimize~~minimise 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.

(3) The depth of pipeline burial should be minimised in fault zones in order to reduce s~~S~~oil restraint on the pipeline during fault movement.

(4) An increase in pipe wall thickness will increase the pipeline's capacity for fault displacement at a given level of maximum tensile strain. It would be appropriate to use relatively thick-walled pipe within 50 m on each side of the fault.

(5) Reduction of the angle of interface friction between the pipeline and the soil also increases the pipeline's capacity for fault displacement at a given level of maximum strain. One way to accomplish this is to use a hard, smooth coating.

(6) 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.

(7) For welded steel pipelines, the most common approach to accommodate fault movement is to ~~utilize~~utilise 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 are to 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. When compressive strains exist, they should be limited to that strain which would cause wrinkling or local buckling of the pipeline.

(8) In all areas of potential ground rupture, pipelines should be laid in relatively straight sections taking care to avoid 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

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ANNEX A (INFORMATIVE) SEISMIC ANALYSIS OF SILOS

A.1 Introduction and scope

This annex provides information on seismic analysis procedures for vertical cylindrical silos subjected to horizontal seismic action.

Unlike liquid storage tanks, silos containing solid-granular material and subjected to earthquake excitation have not been studied intensively. The literature in the subject is scarce (a list of few relevant publications is given below) and in spite of the rather complex mathematics involved, the available solutions are based on a number of simplifying assumptions and idealisations, leaving thus to the designer the decision on to what extent they are relevant for the case at hand. Further, again unlike the case of liquid storage tanks, the available analytical solutions are not of the form allowing an analogy to be established with simpler mechanical problems, whose solution can be rapidly obtained with the ordinary tools of earthquake engineering. Hence, when the data, or the other characteristics of a specific problem, such as for example the geometry of the silo or the properties of the insulated material, differ from those for which solution graphs and tables are provided in the references below, recourse has presently to be made to a ad-hoc modelling of both the material and the structure containing it.

This annex presents the essential features of the results given in the references 1-4, for selected combinations of parameters, without analytical derivations and formulas, with the purpose of allowing the user to check whether they are of use for the case at hand.

A.2 System considered and materials modelling

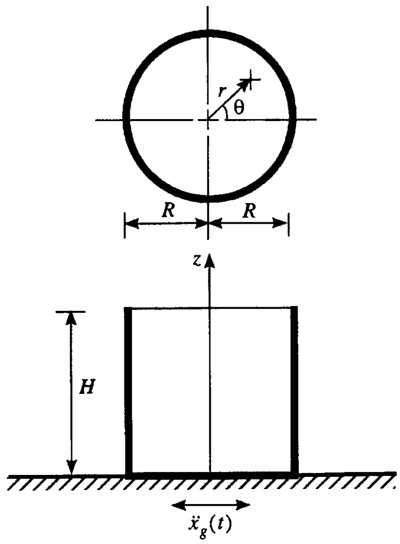

Figure A.1: System considered.

The system considered shown in ~~Fig.~~Figure A.1, is a vertical cylindrical silo assumed to be fixed to a rigid base to which the seismic motion is imposed.

The parameters defining the silo are: the height H, the radius R, the constant thickness t_w, the mass density ρ_w, the shear modulus G_w, the Poisson ratio ν_w and the damping ratio $2\xi_w$. The tank is filled with a homogeneous viscoelastic solid whose material properties are denoted by

ρ, G, ν and 2ξ in the same order as for the silo. These data completely define the elastic behaviour of the system, in particular periods and mode shapes of both the separate parts (i.e., the silo and the column of the filling material) and the combined system. In what follows the main results will be shown for the (more unfavourable) case in which the internal solid can be assumed as fully bonded to the internal face of the cylinder (rough interface).

A.3: Maximum responses of the system

The response parameters considered are: the profile along the height of the maximum pressures on the wall (these pressures vary along the circumference as cosθ), the maximum base shear, and the height (from the base) at which the resultant of the inertia forces is located. In the results shown subsequently, the mass of the silo is assumed to be negligible compared to the mass of the retained material. Corrections to account for walls inertia, when the above assumption is not satisfied, are given in ref. 4.

Following the approach used in ref. 1-4, the results are given as the product of two terms. The first one represents the response to a constant acceleration acting at the base. This part of the total response is indicated "static". The total reponse (due to an arbitrary seismic excitation) is obtained by multiplying the static component by an appropriate amplification factor.

Static effects

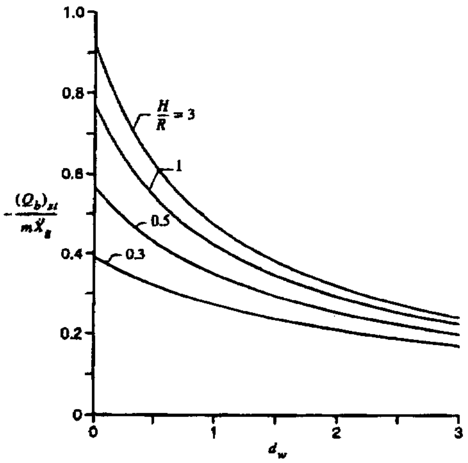

Figure A.2: Normalised values of base shear for statically excited systems with different wall flexibilities and slenderness ratios; m_w = 0 and ν = 1/3.

The static value of the maximum base shear in the silo wall: $(Q_b)_{st}$ is plotted in ~~Fig.~~<u>Figure</u> A.2 as function of the relative flexibility factor:

$$d_w = \frac{G \cdot R}{G_w t_w} \tag{A.1}$$

for different slenderness ratios H/R. The values are normalised with respect to the product: $m\ddot{X}_g$, where m is the total contained mass and $\ddot{X}_g$ is the constant acceleration value. The normalising factor is thus the inertia force that would act on the mass if it were a rigid body. The results in ~~Fig.~~<u>Figure</u> A.2 are for ν = 1/3. It is observed from ~~Fig.~~<u>Figure</u> A.2 that the base shear, and hence the proportion of the contained mass contributing to this shear, is highly dependent on both the slenderness ratio H/R and the relative flexibility factor d_w. For rigid (d_w

≅ 0), tall silos with values of H/R of the order 3 or more, the inertia forces for all the retained material are effectively transmitted to the wall by horizontal shearing action, and practically the entire mass of the silo content may be considered to contribute to the wall force. With decreasing H/R, a progressively larger portion of the inertia forces gets transferred by horizontal shearing action to the base, and the effective portion of the retained mass is reduced.

The effect of wall flexibility (increasing values of d_w) is to reduce the horizontal extensional stiffness of the contained material relative to its shearing stiffness, and this reduction, in turn, reduces the magnitude of the resulting pressures on and associated forces in the silo wall.

It is observed that the reduced response of the flexible silos is in sharp contrast to the well-established behaviour of liquid containing tanks, for which the effect of wall flexibility is to increase rather than decrease the impulsive components of the wall pressures and forces that dominate the response of such systems.

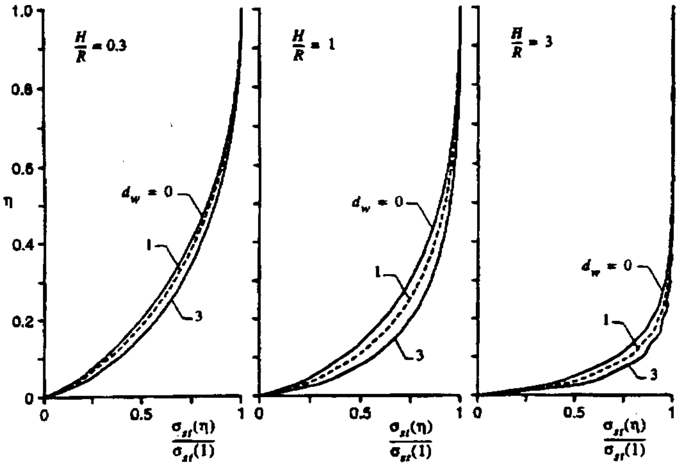

Figure A.3: Heightwise variations of static values of normal wall pressures induced in silos of different flexibilities and slenderness ratios; m_w = 0 and ν = 1/3.

The height wise variation of the maximum pressures is shown in ~~Fig.~~Figure A.3 for different values of H/R and d_w. It is observed that for broad silos these pressures increase from base to top approximately as a quarter-sine, whereas for the rather slender silos, the distribution becomes practically uniform.

Table A.1 collects the values of the maximum pressures at the top of the silo as well as the maximum base shear with the accompanying location of the centre of pressure for different combinations of the parameters H/R and d_w.

Table A.1: Static values of top radial pressure $\sigma_{st}(1)$, base shear $(Q_b)_{st}$, and effective height h; $m_w = 0$, $\nu = 1/3$ and rough interface.

H/R (1)	$d_w = 0$ (2)	$d_w = 0.5$ (3)	$d_w = 1$ (4)	$d_w = 1.5$ (5)	$d_w = 2$ (6)	$d_w = 3$ (7)
			(a) Values of $-\sigma_{st}(1)/\rho \bar{X}_g R$			
0.30	0.365	0.293	0.245	0.210	0.183	0.146
0.50	0.540	0.398	0.315	0.260	0.222	0.170
0.80	0.671	0.466	0.356	0.288	0.241	0.182
1.00	0.709	0.483	0.365	0.294	0.246	0.185
1.25	0.731	0.492	0.370	0.297	0.247	0.186
1.50	0.740	0.495	0.372	0.298	0.248	0.186
1.75	0.744	0.496	0.372	0.298	0.248	0.186
2.00	0.746	0.497	0.373	0.298	0.248	0.186
2.50	0.745	0.497	0.372	0.298	0.248	0.186
3.00	0.744	0.497	0.372	0.298	0.248	0.186
			(b) Values of $-(Q_b)_{st}/m\bar{X}_g$			
0.30	0.391	0.320	0.272	0.236	0.209	0.170
0.50	0.567	0.430	0.348	0.293	0.253	0.199
0.80	0.715	0.513	0.402	0.331	0.282	0.217
1.00	0.770	0.543	0.421	0.345	0.292	0.224
1.25	0.816	0.567	0.436	0.355	0.300	0.229
1.50	0.846	0.583	0.447	0.362	0.305	0.232
1.75	0.868	0.594	0.454	0.367	0.309	0.234
2.00	0.884	0.603	0.459	0.371	0.312	0.236
2.50	0.906	0.615	0.467	0.376	0.315	0.238
3.00	0.921	0.623	0.472	0.380	0.318	0.240
			(c) Values of h/H			
0.30	0.595	0.589	0.584	0.580	0.576	0.570
0.50	0.590	0.582	0.575	0.570	0.565	0.558
0.80	0.580	0.570	0.562	0.557	0.552	0.546
1.00	0.573	0.562	0.555	0.550	0.546	0.539
1.25	0.565	0.555	0.548	0.543	0.539	0.534
1.50	0.559	0.548	0.542	0.538	0.534	0.529
1.75	0.553	0.543	0.537	0.533	0.530	0.526
2.00	0.548	0.539	0.534	0.530	0.527	0.523
2.50	0.540	0.533	0.528	0.525	0.523	0.519
3.00	0.535	0.528	0.524	0.521	0.519	0.517

Total seismic response

Base shear

The maximum total dynamic base shear: $(Q_b)_{max}$, is obtained by multiplying the corresponding static value $(Q_b)_{st}$ times the so-called dynamic amplification factor *AF*. Numerical studies show that this latter factor is essentially a function of the flexibility ratio d_w, of the slenderness ratio H/R and of the fundamental period of the solid-silo system. With respect to this latter parameter, *AF* remains close to unity in the range of very short periods (i.e. for rigid tanks), then increases sharply and remains practically constant up to the periods of 0,5-0,6sec, beyond which it decreases rapidly to values lower than unity. Since the range of periods of practical importance is 0,1-0,5sec, within which *AF* does not vary significantly, the average value of *AF* in this range has been evaluated (using as input motion the El Centro N-S, 1940 record) and is reported in ~~Fig.~~Figure A.4 as a funcion of d_w and for a number of H/R values. The figure shows that for rigid silos *AF* increases fast with the increase of the slenderness ratio H/R, while the dependence tends to vanish for flexible silos.

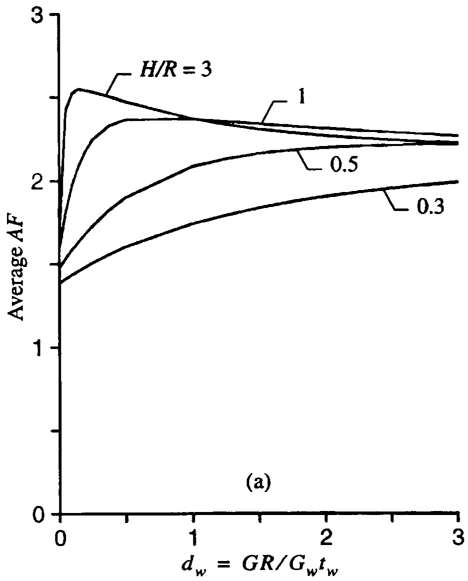

Figure A.4: Effects of silo flexibility on average amplification factor of base shear in wall of system with $0{,}1 \leq T \leq 0{,}5$, $\rho_w = 0$, $2\xi_w = 0{,}04$, $2\xi = 0{,}10$ **subjected to ElCentro record**

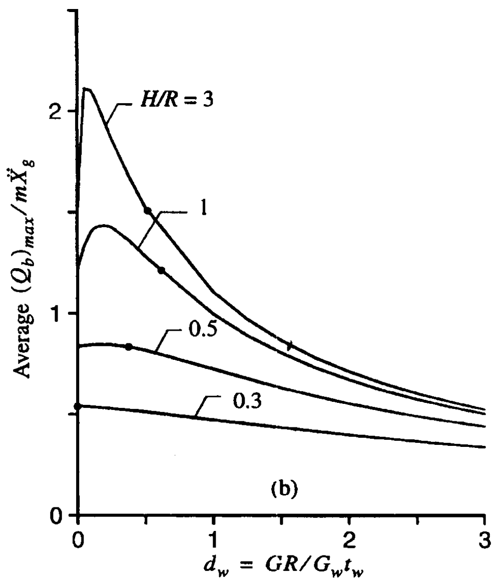

Figure A.5: Effects of silo flexibility on $(Q_b)_{\max} / m\ddot{X}_g$ **average amplification factor of base shear in wall of system with** $0{,}1 \leq T \leq 0{,}5$, $\rho_w = 0$, $2\xi_w = 0{,}04$, $2\xi = 0{,}10$ **subjected to ElCentro record**

The normalised dynamic base shear $(Q_b)_{max}$ corresponding to the *AF* values in ~~Fig.~~Figure A.4 is reported in ~~Fig.~~Figure A.5 as a function of d_w and for a number of *H*/*R* values. The contained material has $\nu = 1/3$ and $2\xi = 0{,}1$.

There are two main points worth commenting. The maximum response does not vary monotonically with d_w, i.e. with wall flexibility. Speifically, for systems represented by points to the right of the dots in ~~Fig.~~Figure A.5, the effect of wall flexibility is to reduce the response below the level for rigid silos (d_w = 0). Only for slender systems with moderate wall flexibility is the response likely to be higher than for the corresponding rigid silos. As already noted, this reducion of the maximum response with wall flexibility is completely at difference with what occurs with liquid-containing tanks, where wall flexibility systematically increases the response.

The second observation from the figure is that, depending on the slenderness and the flexibility of the silo, the base shear may significantly exceed the rigid-body inertia drag force: $m\ddot{X}_g$, implying that the effective mass can be considerably in excess of the total mass of the contained solid.

Overturning moment

The value of overturning base moment may be conveniently expressed as the product of the total base shear and an appropriate height *h*. The variation of the ratio *h*/*H* is not very sensitive to the wall flexibility and slenderness parameters, and does not change significantly from the static to the total response. The values are comprised between 0,5 for slender silos, for which the heightwise variation of the pressure is practically uniform, and 0,6 for squat silos whose heightwise variation is close to a quarter-sine (see ~~Fig.~~Figure A.3).

Wall pressures

References 1-4 do not contain explicitly the values of the amplification *AF* applicable to wall pressures. However, taking into account that the vertical distribution of the total wall pressures does not deviate appreciably from that of the static case, one can infer that the *AF* value appropriate for the base shear can be used in approximation also for obtaining the total wall pressures.

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ANNEX B~~A~~ (I~~I~~NFORMATIVE) SEISMIC ANALYSIS PROCEDURES FOR TANKS

B.1 Introduction and scope

This Annex provides information on seismic analysis procedures for tanks subjected to horizontal and vertical excitation and having the following characteristics:

a) cylindrical shape, with vertical axis and circular or rectangular cross-section;

b) rigid or flexible foundation;

c) fully or partially anchored to the foundation.

Extensions required for dealing with elevated tanks are briefly discussed, as it is the case for cylindrical tanks with horizontal axis.

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 and requiring unusually high computational resources and efforts. Although solutions to the more simple cases of seismic response of tanks are known from the early seventies, progress in the treatment of the more complex ones is continuing up to the present, and it is still incomplete.

Numerous studies are being published, offering new, more or less approximate, procedures valid for specific design situations. Since their accuracy is problem-dependent, a proper choice requires a certain amount of specializ~~a~~zed 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 ex., to select 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 F.E. model) for evaluating the stresses due to the pressures.

The necessary limitations in the scope and space of this Annex do not allow to go beyond a detailed presentation of the seismic design procedure for the simplest of all cases: rigid circular tanks anchored to a rigid base. For all the situations which make the problem more complex, as for example~~.~~ the flexibility of the tank, and/or that of the foundation soil, and/or that of the anchoring system, since exact solutions are either complicated and lengthy, or non existing, a brief explanation is given of the physical phenomena distinguishing the particular situation from the reference case, and approximate solutions are either summarized or reference is made to pertinent literature.

At present, the most comprehensive documents giving guidelines for the seismic design of tanks are the ASCE volume: "Guidelines for the seismic design of oil and gas pipeline systems", 1984, ref. [5], and the Recommendations of a New Zealand Study Group: "Seismic Design of Storage Tanks", 1986, ref. [10]. Although more than ten years old they are still valuable in that they cover in detail a wide range of cases. Both documents are used as sources for the present Annex.

B.2 Vertical rigid circular tanks

B.2.1 Horizontal earthquake excitation

The complete solution of the Laplace equation for the motion of the fluid contained in a rigid cylinder can be expressed as the sum of two separate contributions, called "rigid impulsive", and "convective", respectively. The "rigid impulsive" component of the solution satisfies exactly the boundary conditions at the walls and at the bottom of the tank (compatibility between the velocities of the fluid and of the tank), but gives (incorrectly, due to the presence of the waves) zero pressure at the free surface of the fluid. A second term must therefore be added, which does not alter those boundary conditions that are already satisfied, and re-establishes the correct equilibrium condition at the top.

Use is made of a cylindrical coordinate system: r, z, θ, with origin at the cent~~e~~re of the tank bottom, and the z axis vertical. The height and the radius of the tank are denoted by H and R, respectively, ρ is the mass density of the fluid, and $\xi = r/R$, $\varsigma = z/H$, are the non~~a~~dimensional coordinates.

B.2.1.1 Rigid impulsive pressure

The spatial-temporal variation of this component is given by the expression:

$$p_{\mathrm{i}}(\xi,\varsigma,\theta,t) = C_{\mathrm{i}}(\xi,\varsigma)\rho H \cos\theta A_{\mathrm{g}}(t) \tag{B.1}$$

where:

$$C_{\mathrm{i}}(\xi,\varsigma) = \sum_{n=o}^{\infty} \frac{(-1)^{\mathrm{n}}}{I_1'(\nu_{\mathrm{n}}/\gamma)\nu_{\mathrm{n}}^2} \cos(\nu_{\mathrm{n}}\varsigma) I_1\left(\frac{\nu_{\mathrm{n}}}{\gamma}\xi\right) \tag{B.2}$$

in which:

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

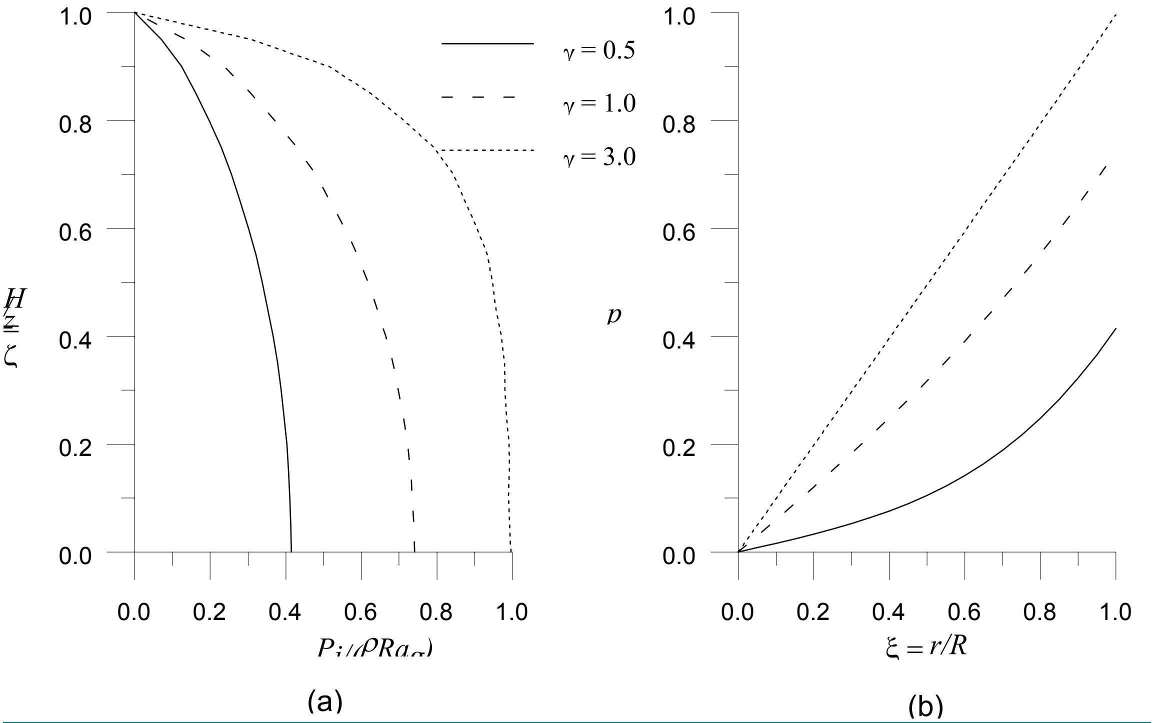
$I_1(\cdot)$ and $I_1'(\cdot)$ denote the modified Bessel function of order 1 and its derivative[5].

The time-dependence of the pressure p_{i} in eq. (B.1) is given by the function $A_{\mathrm{g}}(t)$, which represents here the free-field motion of the ground (the peak value of $A_{\mathrm{g}}(t)$ is denoted by a_{g}). The distribution along the height of p_{i} in eq. (B.1) is given by the function C_{i} and is represented in ~~Fig.~~Figure B.1(a) for $\xi = 1$ (i.e. at the wall of the tank) and $\cos\theta = 1$ (i.e. on the plane which contains the motion), normalized to $\rho R\ a_{\mathrm{g}}$ and for three values of $\gamma = H/R$.

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

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

The circumferential variation of p_i follows the function $\cos\theta$ ~~Fig.~~ Figure B.1(b) shows the radial variation of p_i on the tank bottom as a function of the slenderness parameter γ. For increasing values of γ the pressure distribution on the bottom tends to become linear.



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

Pressure resultants

For a number of purposes it is useful to evaluate the horizontal resultant of the pressure at the base of the wall: Q_i, as well as the moment of the pressures with respect to an axis orthogonal to the direction of the motion: M_i. The total moment M_i immediately below the tank bottom includes the contributions of the pressures on the walls and of those on the bottom.

By making use of eq. (B.1) and (B.2) and performing the appropriate integrals one gets:

– impulsive base shear:

$$Q_i(t) = m_i A_g(t) \tag{B.3}$$

where m_i indicates the mass of the contained fluid which moves together with the walls, is called *impulsive mass,* and has the expression:

$$m_i = m 2\gamma \sum_{n=0}^{\infty} \frac{I_1(\nu_n/\gamma)}{\nu_n^3 I_1'(\nu_n/\gamma)} \tag{B.4}$$

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

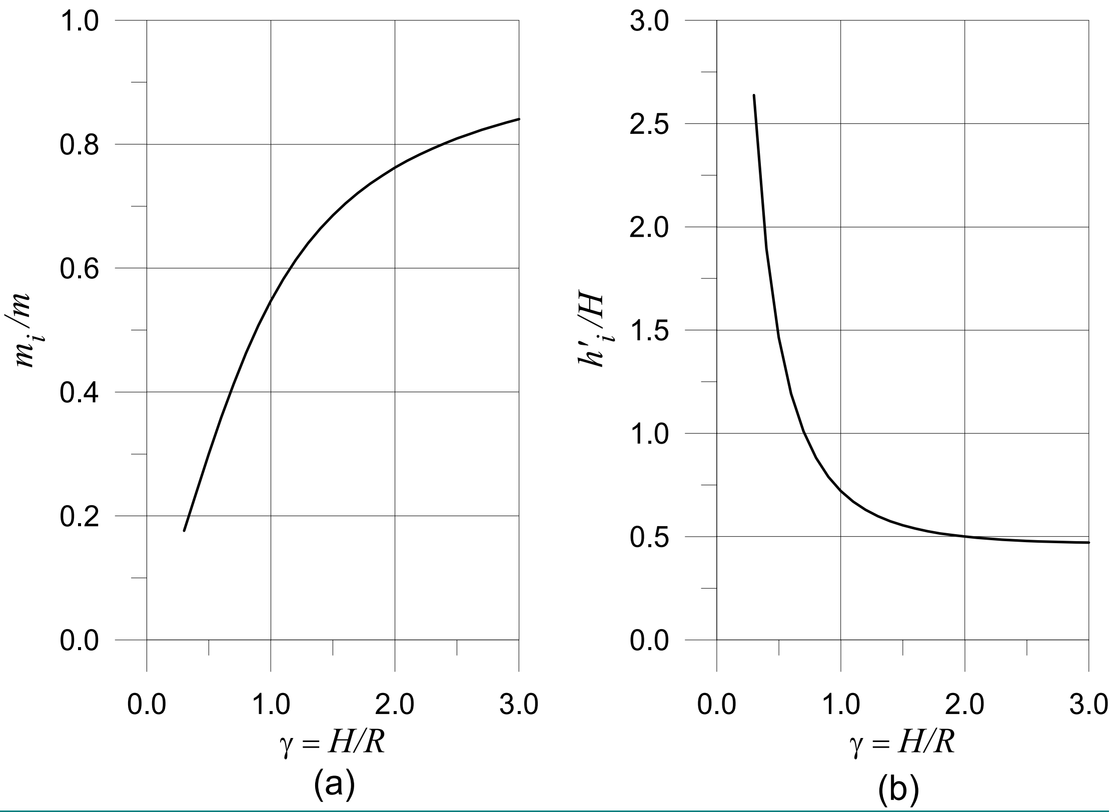
– impulsive base moment:

$$M_{\mathrm{i}}(t) = m_{\mathrm{i}} h_{\mathrm{i}}^{'} A_{\mathrm{g}}(t) \tag{B.5}$$

with

$$h_{\mathrm{i}}^{'} = H \frac{\dfrac{1}{2} + 2\gamma \sum_{\mathrm{n}=0}^{\infty} \dfrac{\nu_{\mathrm{n}} + 2(-1)^{\mathrm{n}+1}}{\nu_n^4} \dfrac{I_1(\nu_{\mathrm{n}}/\gamma)}{I_1^{'}(\nu_{\mathrm{n}}/\gamma)}}{2\gamma \sum_{n=0}^{\infty} \dfrac{I_1(\nu_{\mathrm{n}}/\gamma)}{\nu_{\mathrm{n}}^3 I_1^{'}(\nu_{\mathrm{n}}/\gamma)}} \tag{B.6}$$

The two quantities m_{i} and $h_{\mathrm{i}}^{'}$ are plotted in ~~Fig.~~Figure B.2 as functions of the ratio $\gamma = H/R$.



~~Fig.~~Figure B.2: Ratios m_{i} / m and h'_{i} / H as functions of the slenderness of the tank

It is noted from ~~Fig.~~Figure B.2 that m_{i} increases with γ, to become close to the total mass for high values of this parameter, while $h_i^{'}$ tends to stabilize at about mid height. Values of $h_i^{'}$ larger than H for squat tanks are due to the predominant contribution of the pressures on the bottom.

B.2.1.2 Convective pressure component

The spatial-temporal variation of this component is given by the expression:

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

with

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

$$\lambda_1 = 1{,}8112 \qquad \lambda_2 = 5{,}3314 \qquad \lambda_3 = 8{,}5363$$

J_1 = Bessel function of the first order

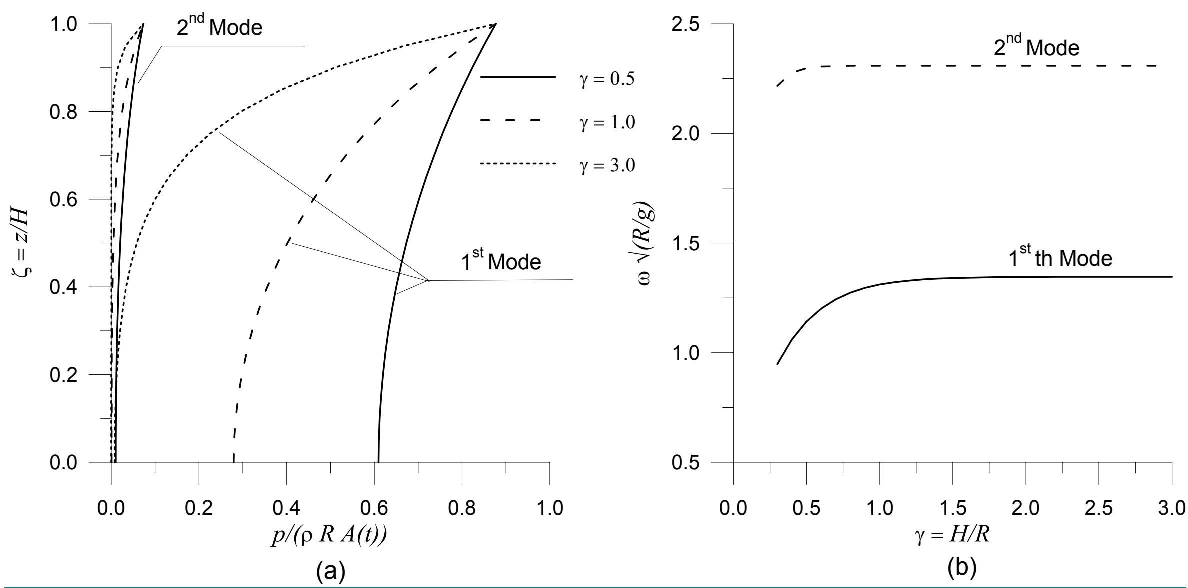
$A_n(t)$ = response acceleration of a single degree of freedom oscillator having a frequency ω_{cn}:

$$\omega_{cn}^2 = g \frac{\lambda_n}{R} \tan h(\lambda_n \gamma) \quad \text{(B.9)}$$

and a damping factor value appropriate for the fluid.

Eq. (B.7) shows that the total pressure is the combination of an infinite number of modal terms, each one corresponding to a wave form of the oscillating liquid. Only the first oscillating, or sloshing, mode and frequency, needs in most cases to be considered for design purposes.

The vertical distribution of the sloshing pressures for the first two modes are shown in ~~Fig.~~Figure B.3(a), while ~~Fig.~~Figure B.3(b) gives the values of the first two frequencies, as functions of the ratio H/R.



~~Fig.~~Figure B.3: Variation of the first two modes sloshing pressures along the height (a) and values of the first two sloshing frequencies as functions of γ (b)

One can observe from ~~Fig.~~Figure B.3 that in squat tanks the sloshing pressures maintain relatively high values down to the bottom, while in slender tanks the sloshing effect is superficial.

For the same value of the response acceleration, the contribution of the second mode is seen to be negligible. The other interesting result from ~~Fig.~~Figure B.3(b) is that the sloshing frequencies become almost independent of the parameter γ, when this is larger than about 1.

The value of ω_{c1} in this case is approximately given by the expression:

$$\omega_{c1} = 4{,}2/\sqrt{R} \qquad (R \text{ in metres}) \tag{B.10}$$

which, for the usual values of R in petrochemical plants yields periods of oscillation of the order of few seconds (for instance, T_{c1} = 4,7 sec for R = 10 m).

Pressure resultants

In a way analogous to that followed for the impulsive component one may arrive at the expressions for the base shear resultant and the total moment immediately below the bottom plate of the tank.

The base shear is given by:

$$Q_c(t) = \sum_{n=1}^{\infty} m_{cn} A_n(t) \tag{B.11}$$

with the nth modal convective mass:

$$m_{cn} = m \frac{2 \tan h(\lambda_n \gamma)}{\gamma \lambda_n (\lambda_n^2 - 1)} \tag{B.12}$$

From eq. (B.11) one can note that the total shear force is given by the instantaneous sum of the forces contributed by the (infinite) oscillators having masses m_{cn}, attached to the rigid tank by means of springs having stiffnesses: $K_n = \omega_n^2 m_{cn}$. The tank is subjected to the ground acceleration $A_g(t)$ and the masses respond with accelerations $A_n(t)$.

From ~~Fig.~~Figure B.3 (and the following, ~~Fig.~~Figure B.4) one can verify that only the first of the sloshing masses needs to be considered.

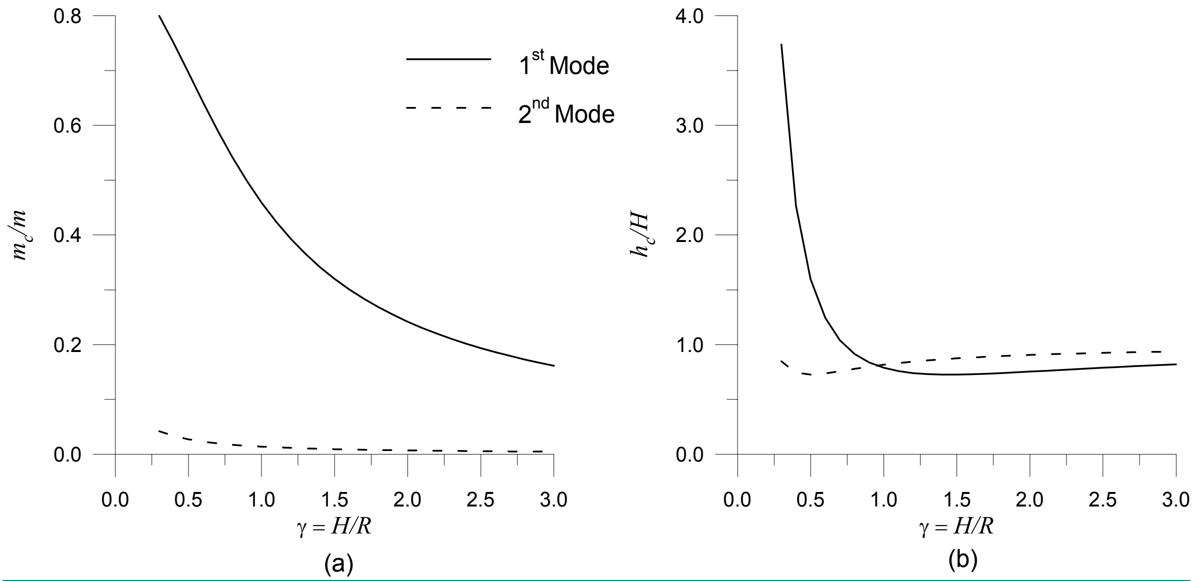
The total moment can be expressed as:

$$M_c(t) = \sum_{n=1}^{\infty} (m_{cn} A_n(t)) h_{cn} = \sum_{n=1}^{\infty} Q_{cn}(t) h_{cn} \tag{B.13}$$

where h_{cn} is the level where the equivalent oscillator has to be applied in order to give the correct value of M_{cn}:

$$h_{cn} = H\left(1 + \frac{2 - \cos h(\lambda_n \gamma)}{\lambda_n \gamma \sinh(\lambda_n \gamma)}\right) \tag{B.14}$$

The values of m_{c1} and m_{c2}, and the corresponding values of h_{c1} and h_{c2} are shown in ~~Fig.~~ Figure B.4, as functions of γ.



~~Fig.~~ Figure B.4: First two sloshing modal masses ~~Fig.~~(a) and corresponding heights h_{c1} and h_{c2} ~~Fig.~~(b) as functions of γ

B.2.1.3 Height of the convective wave

The predominant contribution to the sloshing wave height is provided by the first mode, and the expression of the peak at the edge is:

$$d_{\max} = 0{,}84 R S_e(T_{c1}) \tag{B.15}$$

where $S_e(\cdot)$ is the appropriate elastic acceleration response spectrum, expressed in g (acceleration of gravity).

B.2.1.4 Combination of impulsive and convective pressures

The time-history of the total pressure is the sum of the two time-histories, the impulsive one being driven by $A_g(t)$, the convective one by $A_{c1}(t)$ (neglecting higher order components).

If, as it is customary in design practice, a response spectrum approach is preferred, the problem of suitably combining the two maxima arises. Given the generally wide separation between the central frequencies of the ground motion and the sloshing frequency, the "square root of the sum of squares" rule may become unconservative, so that the alternative, upper bound, rule of adding the absolute values of the two maxima is recommended for general use.

B.2.1.5 Effect of walls inertia

For steel tanks, the inertia forces acting on the shell due to its own mass are small in comparison with the hydrodynamic forces, and can normally be neglected. For concrete tanks however, the wall inertia forces may not be completely negligible. The inertia forces are contained in the same vertical plane of the seismic excitation; considering their component normal to the surface of the shell one has for the pressure the following expression:

$$p_{\mathrm{w}} = \rho_{\mathrm{w}} s \cos\theta A_{\mathrm{g}}(t) \tag{B.16}$$

with

ρ_{w} = mass density of the wall material

s = wall thickness

This pressure component, which is constant along the height, has to be added to the impulsive component given by eq. (B.1). The total shear at the base is obtained by simply considering the total mass of the tank multiplied by the acceleration of the ground.

B.2.2 Vertical earthquake excitation

The hydrodynamic pressure on the walls of a rigid tank due to ~~a~~ vertical ground acceleration $A_{\mathrm{v}}(t)$ is given by:

$$p_{\mathrm{vr}}(\varsigma, t) = \rho H(1-\varsigma) A_{\mathrm{v}}(t) \tag{B.17}$$

B.2.3 Combination of pressures due to horizontal and vertical excitation

The peak combined pressure due to horizontal and vertical excitation can be obtained by applying the rule given in 3.2.

B.3 Vertical deformable circular tanks

B.3.1 Horizontal earthquake excitation

When the tank cannot be considered as rigid (this is almost always the case for steel tanks) the complete solution of the Laplace equation is ordinarily sought in the form of the sum of three contributions, referred to as: "rigid impulsive", "sloshing" and "flexible".

The third contribution is new with respect to the case of rigid tanks: it satisfies the condition that the radial velocity of the fluid along the wall equals the deformation velocity of the tank wall, plus the conditions of zero vertical velocity at the tank bottom and zero pressure at the free surface of the fluid.

Since the deformation of the wall is also due to the sloshing pressures, the sloshing and the flexible components of the solution are theoretically coupled, a fact which makes the determination of the solution quite involved. Fortunately, the dynamic coupling is very weak, due to the separation which exists between the frequencies of the two motions, and this allows to determine the third component independently of the others with almost complete accuracy. The rigid impulsive and the sloshing components examined in **B.2** remain therefore unaffected.

No closed-form expression is possible for the flexible component, since the pressure distribution depends on the modes of vibration of the tank-fluid system, and hence on the geometric and stiffness properties of the tank. These modes cannot be obtained directly from usual eigenvalue algorithms, since the participating mass of the fluid is not known a priori and also because only the modes of the type: $f(\varsigma,\theta) = f(\theta)\cos\theta$ are of interest (and these modes may be laborious to find among all other modes of a tank).

Assuming the modes as known (only the fundamental one is normally sufficient, so that in the following expressions both the mode index and the summation over all modal contributions are dropped) the flexible pressure distribution has the form:

$$p_{\mathrm{f}}(\varsigma,\theta,t) = \rho H \psi \sum_{n=0}^{\infty} d_{\mathrm{n}} \cos(\nu_{\mathrm{n}}\varsigma)\cos\theta A_{\mathrm{f}}(t) \tag{B.18}$$

with:

$$\psi = \frac{\int_0^1 f(\varsigma)\left[\frac{\rho_{\mathrm{s}}}{\rho}\frac{s}{H} + \sum_{n=0}^{\infty} b_{\mathrm{n}}^{'} \cos(\nu_{\mathrm{n}}\varsigma)\right]d\varsigma}{\int_0^1 f(\varsigma)\left[\frac{\rho_{\mathrm{s}}}{\rho}\frac{s}{H} f(\varsigma) + \sum_{n=0}^{\infty} d_{\mathrm{n}} \cos(\nu_{\mathrm{n}}\varsigma)\right]d\varsigma} \tag{B.19}$$

$$b_{\mathrm{n}}^{'} = 2\frac{(-1)^{\mathrm{n}}}{\nu_{\mathrm{n}}^2}\frac{I_1(\nu_{\mathrm{n}}/\gamma)}{I_1^{'}(\nu_{\mathrm{n}}/\gamma)} \tag{B.20}$$

$$d_{\mathrm{n}} = 2\frac{\int_0^1 f(\varsigma)\cos(\nu_{\mathrm{n}}\varsigma)d\varsigma}{\nu_{\mathrm{n}}}\frac{I_1(\nu_{\mathrm{n}}/\gamma)}{I_1^{'}(\nu_{\mathrm{n}}/\gamma)} \tag{B.21}$$

ρ_{s} is the mass density of the shell, s is its thickness and $A_{\mathrm{f}}(t)$ is the response acceleration (relative to its base) of a simple oscillator having the fundamental frequency and damping factor of the first mode.

In most cases of flexible tanks, the pressure $p_{\mathrm{f}}(\cdot)$ in eq. (B.18) provides the predominant contribution to the total pressure, due to the fact that, while the rigid impulsive term (eq. (B.1)) varies with the ground acceleration $A_{\mathrm{g}}(t)$, the flexible term (eq. (B.18)) varies with the response acceleration which, given the usual range of periods of the tank-fluid systems, is considerably amplified with respect to $A_{\mathrm{g}}(t)$.

For the determination of the first mode shape of the tank, the following iterative procedure is suggested in ref. [2]. Starting from a trial shape $f(\varsigma)$ and denoting with $f^i(\varsigma)$ the one corresponding to the i-th iteration step, an "effective" mass of the shell is evaluated as:

$$\rho^i(\varsigma) = \frac{p_s^i(\varsigma)}{2g\, s(\varsigma)\, f^i(\varsigma)} + \rho_s \tag{B.22}$$

where $p_s^i(\varsigma)$ is the amplitude of the pressure evaluated with eq. (B.18) at the i-th step, and $s(\varsigma)$ is the thickness of the shell, respectively.

The effective density from eq. (B.22) can then be used in a structural analysis of the tank to evaluate the (i+1)th mode shape, and so forth until convergence is achieved.

The fundamental frequency of the tank-fluid system can be evaluated by means of the following approximate expression:

$$f_s = \left(E\, s\,(\varsigma)/\rho\, H\right)^{1/2} / 2R\, g(\gamma) \qquad \text{(with } \varsigma = 1/3\text{)} \tag{B.23}$$

with

$$g(\gamma) = 0{,}01675\gamma^2 - 0{,}15\gamma + 0{,}46 \tag{B.24}$$

Pressure resultants

Starting from eq. (B.18), the resultant base shear and total moment at the base can be evaluated, arriving at expressions in the form:

- base shear $Q_f(t) = m_f A_f(t) \quad (1\text{st mode only})$ (B.25)

with

$$m_f = m\psi\gamma \sum_{n=0}^{\infty} \frac{(-1)^n}{\nu_n} d_n \tag{B.26}$$

- total moment $M_f(t) = m_f h_f A_f(t)$ (B.27)

with

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

B.3.2 Combination of pressures terms due to horizontal excitation

The time-history of the total pressure is, in the case of flexible tanks, the sum of three time-histories: of the rigid impulsive one (eq. (B.1)), of the convective one (eq. (B.7)), and of the flexible one (eq. (B.18)) each of them differently distributed along the height and having a different variation with time.

Referring for simplicity to the base shears produced by these pressures (eqs. (B.3), (B.11) and (B.25)) one has:

$$Q(t) = m_i A_g(t) + \sum_{n=1}^{\infty} m_{cn} A_n(t) + m_f A_f(t) \tag{B.29}$$

where, it is recalled, $A_n(t)$ is the total or absolute response acceleration of a simple oscillator of frequency ω_n (eq. (B.9)) 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 frequency ω_f (eq. (B.23)), and damping appropriate for the tank-fluid system, also subjected to $A_g(t)$.

If the individual maxima of the terms in eq. (B.29) are known, which can be achieved by using a response spectrum of absolute and relative accelerations, the corresponding pressures on the tank needed for a detailed stress analysis can be obtained by spreading the resultant over the tank walls and floor according to the relevant distribution.

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, for ex. in ref. [5] and [10].

Use of eq. (B.29) in combination with response spectra, however, poses the problem of how to superimpose the maxima~~B~~. Apart from the necessity of deriving 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 $A_f(t)$.

In fact, since the input and its response cannot be assumed as independent in the relatively high range of frequency under consideration, the "square root of the sum of squares" rule is unconservative. On the other hand, the simple addition of the individual maxima can lead to overconservative estimates.

Given these difficulties, various approximate approaches based on the theory previously discussed have been proposed.

Two of these, presented as alternatives and illustrated in detail in ref. [5], are due to Veletsos-Yang (V.Y.) and Haroun-Housner (H.H.).

The V.Y. proposal consists essentially in replacing eq. (B.29) with the equation:

$$Q(t) = m_i A_{fa}(t) + \sum_{n=1}^{\infty} m_{cn} A_n(t) \tag{B.30}$$

i.e., in assuming the entire impulsive mass to respond with the amplified absolute response acceleration of flexible tank system ($A_{fa}(t) = A_f(t) + A_g(t)$). The maximum of $A_{fa}(t)$ is obtained directly from the appropriate response spectrum.

The V.Y. procedure is an upper bound solution, whose approximation has been proven to be acceptable for H/R ratios not much larger than 1. Above this value, corrections to decrease the conservativeness are suggested. In view of the conservative nature of the method, the effects of tank inertia may generally be neglected. If desired, the total base shear can be evaluated approximately by the expression:

$$Q_w(t) = (\varepsilon_o \cdot m) \cdot A_{fa}(t) \tag{B.31}$$

where $A_{fa}(t)$ is the pseudoacceleration response of the tank-fluid system, and $(\varepsilon_o \cdot m)$ is the effective participating mass of the tank wall in the first mode, where m is the total mass of the tank and the factor ε_o may be determined approximately from:

H/R	0,5	1,0	3,0
ε_o	0,5	0,7	0,9

The H.H. proposal starts by writing eq. (B.29) in the form:

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

which can be re-arranged as:

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

i.e., in a form suitable for the use of the response spectrum.

The masses m_i and m_j are given in graphs as functions of H/R and s/R, together with the heights at which these masses must be located to yield the correct value of the moment (see ref. [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 eq. (B.33).

In the H.H. approach, the problem of distributing heightwise the total shear force at the base is solved by assuming a uniform pressure distribution over the tank height, which leads to a value of the hoop stress σ equal to:

$$\sigma_{max} = \frac{1}{\pi} \frac{Q_{max}}{H\, s} \tag{B.34}$$

Along lines similar to those of Veletsos-Yang, an even more simplified approach has been elaborated by Malhotra (1997) [8], which is reported in full below.

B.3.2.1 Simplified procedure for fixed base cylindrical tanks (Malhotra, 1997)[8]

Model

The hydrodynamic effects in a tank are evaluated by the superposition of these two components: (1) The impulsive component, which represents the action of the liquid near the base of the tank that moves rigidly with the flexible wall of the tank; and (2) the convective component, which represents the action of the liquid that experiences sloshing motion near the free-surface. In this analysis, the tank-liquid system is modeled by two single-degree-of-freedom systems, one corresponding to the impulsive and the other corresponding to the convective action. The impulsive and convective responses are combined by taking their numerical-sum rather than their root-mean-square value.

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

$$T_{imp} = C_i \frac{\sqrt{\rho}\, H}{\sqrt{s/R}\, \sqrt{E}} \tag{B.35}$$

$$T_{\text{con}} = C_{\text{c}}\sqrt{R} \tag{B.36}$$

where H = design liquid height, R = tank's radius, s = equivalent uniform thickness of the tank wall, ρ = mass density of liquid, and E = Young's modulus of elasticity of tank material. The coefficients C_i and C_c are obtained from Table B.1. The coefficient C_i is dimensionless, while C_c is expressed in $s/m^{1/2}$; substituting R in meters in eq. (B.36), therefore, gives the correct value of the convective period. For tanks with nonuniform wall thickness, s may be computed by taking a weighted average over the wetted height of the tank wall, assigning highest weight to the thickness near the base of the tank where the strain is maximum.

Impulsive and convective masses: The impulsive and convective masses m_{i} and m_{c} are given in Table B.1 as fractions of the total liquid mass m.

Table B.1:

H/R	C_1	C_c	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

Note: C_c is expressed in $s/m^{1/2}$

Seismic response

Base shear: The total base shear is

$$Q = (m_{\text{i}} + m_{\text{w}} + m_{\text{r}})\,\text{S}_{\text{e}}(T_{\text{imp}}) + m_{\text{c}}\,\text{S}_{\text{e}}(T_{\text{con}}) \tag{B.37}$$

where, m_{w} = the mass of tank wall, m_{r} = the mass of tank roof; $\text{S}_{\text{e}}(T_{\text{imp}})$ = the impulsive spectral acceleration, obtained from a 2 percent damped elastic response spectrum for steel or prestressed concrete tanks and a 5 percent damped elastic response spectrum for concrete tanks; $\text{S}_{\text{e}}(T_{\text{con}})$ = the convective spectral acceleration, obtained from a 0,5 percent damped elastic response spectrum.

Overturning moment above the base plate: The overturning moment above the base plate, in combination with ordinary beam theory, gives the axial stress at the base of the tank wall.

The net overturning moment immediately above the base plate is

$$M = \left(m_{\mathrm{i}}\, h_{\mathrm{i}} + m_{\mathrm{w}}\, h_{\mathrm{w}} + m_{\mathrm{r}}\, h_{\mathrm{r}}\right) S_{\mathrm{e}}\left(T_{\mathrm{imp}}\right) + m_{\mathrm{c}}\, h_{\mathrm{c}}\, S_{\mathrm{e}}\left(T_{\mathrm{con}}\right) \quad \text{(B.38)}$$

where, h_{i} and h_{c} are the heights of the centroid of the impulsive and convective hydrodynamic wall pressure; they are obtained from Table B1; h_{w} and h_{r} are heights of the centres of gravity of the tank wall and roof, respectively.

Overturning moment below the base plate: The overturning moment immediately below the base plate is on account of the hydrodynamic pressure on the tank wall as well as that on the base plate. It is given by

$$M' = \left(m_{\mathrm{i}}\, h'_{\mathrm{i}} + m_{\mathrm{w}}\, h_{\mathrm{w}} + m_{\mathrm{r}}\, h_{\mathrm{r}}\right) S_{\mathrm{e}}\left(T_{\mathrm{imp}}\right) + m_{\mathrm{c}}\, h'_{\mathrm{c}}\, S_{\mathrm{e}}\left(T_{\mathrm{con}}\right) \quad \text{(B.39)}$$

where heights h'_{i} and h'_{c} are obtained from Table B.1.

If the tank is supported on a ring foundation, moment M should be used to design the tank wall, base anchors and the foundation. If the tank is supported on a mat foundation, moment M should be used to design the tank wall and anchors, while M' should be used to design the foundation.

Free-surface wave-height: The vertical displacement of liquid surface due to sloshing is given by eq (B.15).

B.3.3 Vertical earthquake excitation

In addition to the pressure $p_{\mathrm{vr}}(\varsigma,t)$ given by eq. (B.17), due to the tank moving rigidly in the vertical direction with acceleration $A_{\mathrm{v}}(t)$, a pressure contribution $p_{\mathrm{vf}}(\varsigma,t)$ resulting from the deformability (radial "breathing") of the shell must be considered. This additional term has the expression:

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

where:

$f(\gamma)$ = 1,078 + 0,274 ln γ for $0{,}8 \leq \gamma < 4$

$f(\gamma)$ = 1,0 for $\gamma < 0{,}8$

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

The fundamental frequency can be estimated by means of the expression:

$$f_{\mathrm{vd}} = \frac{1}{4R}\left[\frac{2EI_1(\gamma_1)s(\varsigma)}{\pi\rho H\left(1-\nu^2\right)I_o(\gamma_1)}\right]^{1/2} \qquad \text{(with } \varsigma = 1/3\text{)} \quad \text{(B.41)}$$

in which $\gamma_1 = \pi / (2\gamma)$ and where E and ν are Young modulus and Poisson ratio of the tank material, respectively.

The maximum value of $p_{vf}(t)$ is obtained from the vertical acceleration response spectrum for the appropriate values of the period and the damping. If soil flexibility is neglected (see **B.**7) the applicable damping values are those of the material (steel, concrete) of the shell.

The maximum value of the pressure due to the combined effect of the rigid: $p_{vr}(\cdot)$ and flexible: $p_{vf}(\cdot)$ contributions can be obtained by applying the "square root of the sum of squares" rule to the individual maxima~~B~~.

B.3.4 Combination of pressures due to horizontal and vertical excitation

The maximum value of the pressure due to the combined effect of horizontal and vertical excitation can be obtained by applying the "square root of the sum of squares" rule to the maximum pressures produced by each type of excitation.

B.4 Rectangular tanks

For tanks whose walls can be assumed as rigid, a solution of the Laplace equation for horizontal excitation can be obtained in a form analogous to that described for cylindrical tanks, so that the total pressure is again given by the sum of an impulsive and a convective contribution:

$$p(z,t) = p_{\mathrm{i}}(z,t) + p_{\mathrm{c}}(z,t) \tag{B.42}$$

The impulsive component has the expression:

$$p_{\mathrm{i}}(z,t) = q_o(z)\rho L A_{\mathrm{g}}(t) \tag{B.43}$$

where L is the half-width of the tank in the direction of the seismic action, and the function $q_o(z)$, which gives the variation of $p_i(\cdot)$ along the height ($p_i(\cdot)$ is constant in the direction orthogonal to the seismic action), is plotted in ~~Fig.~~Figure B.5.

The trend and the numerical values of the function $q_o(z)$ are quite close to those of a cylindrical tank with radius $R = L$.

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

$$p_{\mathrm{c1}}(z,t) = q_{\mathrm{c1}}(z)\rho L A_1(t) \tag{B.44}$$

where the function $q_{c1}(z)$ is shown in ~~Fig.~~Figure B.6 together with the 2nd mode contribution $q_{c2}(z)$ and $A_1(t)$ is the acceleration response function of a simple oscillator having the frequency of the first mode, the appropriate value of the damping, and subjected to an input acceleration $A_g(t)$.

The period of oscillation of the first sloshing mode is:

$$T_1 = 2\pi \left(\frac{L/g}{\frac{\pi}{2}\tanh\left(\frac{\pi}{2}\frac{H}{L}\right)} \right)^{1/2} \quad \text{(B.45)}$$

Pressure resultants

The base shear and the moment on the foundation could be evaluated on the basis of expressions (B.43) and (B.44).

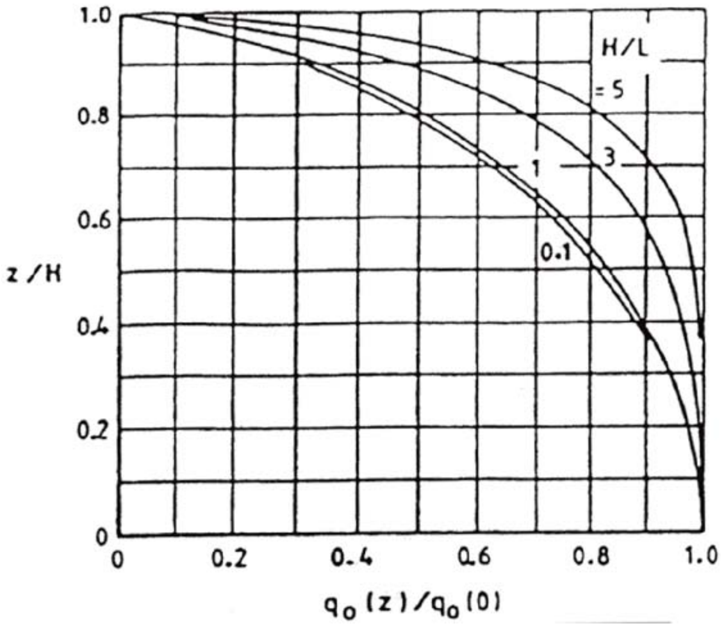
According to reference [10], for design purposes the values of the masses m_i and m_{c1}, as well as of the corresponding heights above the base: h'_i and h_{c1}, calculated for cylindrical tanks and given by the expressions (B.4), (B.12) and (B.6), (B.14), respectively, may be adopted for rectangular tanks as well (with L replacing R), with a margin of approximation not exceeding 15%.

Flexible walls

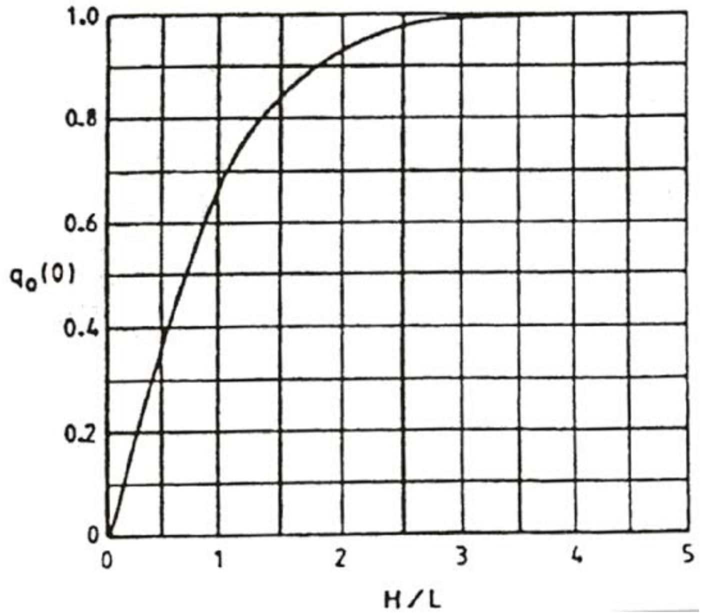
Wall flexibility produces generally a significant increase of the impulsive pressures, while leaving the convective pressures practically unchanged. The reason for this difference is the same discussed previously for the case of cylindrical tanks, i.e., the uncoupling of the sloshing oscillations from the dynamic deformations of the walls, due to the separation of their respective periods.

Studies on the behaviour of flexible rectangular tanks are not numerous, and the solutions are not amenable to a form suitable for direct use in design: for a recent treatment of the subject see for example. ref. [6].

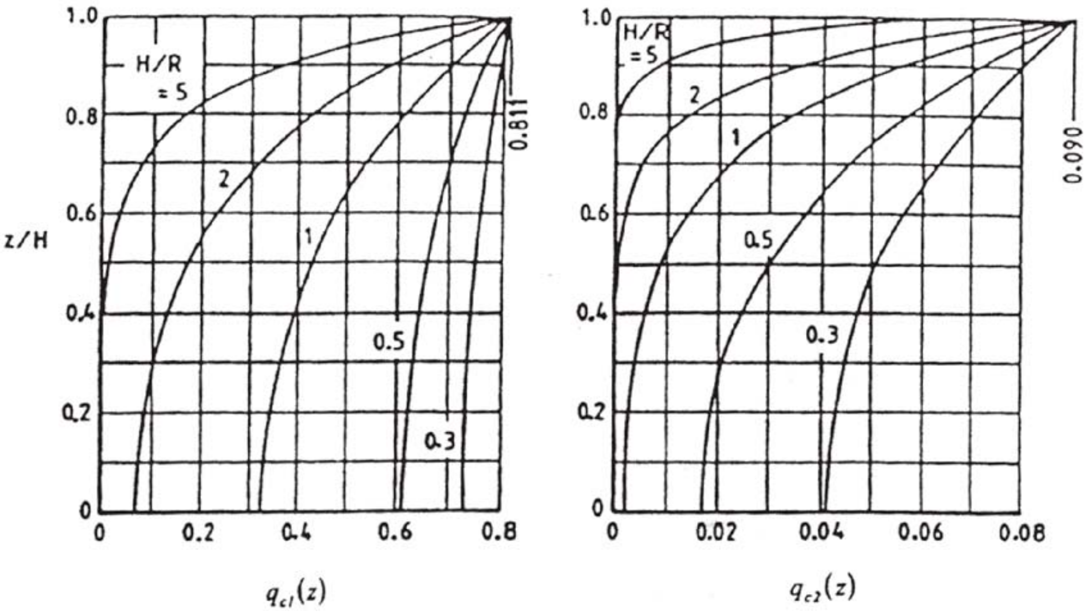
For design purposes, an approximation which is suggested in ref. [10] is to use the same vertical pressure distribution valid for rigid walls, see eq. (B.43) and ~~Fig.~~Figure B.5, but to replace the ground acceleration $A_g(t)$ in eq. (B.43) with the response acceleration of a simple oscillator having the frequency and the damping factor of the first impulsive tank-liquid mode.



~~Fig.~~Figure B.5(a): Dimensionless impulsive pressures on rectangular tank wall perpendicular to direction of earthquake (from ref. [10])



~~Fig.~~Figure B.5(b): Peak value of dimensionless impulsive pressures on rectangular wall perpendicular to direction of earthquake (from ref. [10])



~~Fig.~~Figure B.6: Dimensionless convective pressures on rectangular tank wall perpendicular to direction of earthquake (from ref. [10])

This period of vibration is given approximately by:

$$T_{\mathrm{f}} = 2\pi\left(d_{\mathrm{f}} / g\right)^{1/2} \qquad \text{(B.46)}$$

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: $m_{\mathrm{i}}\, g/4BH$.

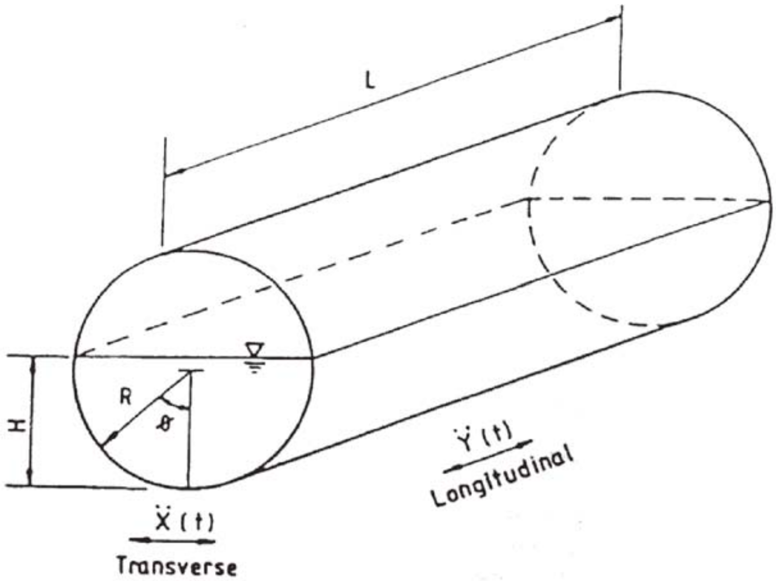
$2B$ is the tank width perpendicular to the direction of loading.

The impulsive mass m_{i} can be obtained from eq. (B.4), but should include the wall mass.

B.5 Horizontal circular cylindrical tanks

The information contained in this section **B.**5 is taken from ref. [10].

Horizontal cylindrical tanks need to be analyzed both along the longitudinal and the transverse axis: see ~~Fig.~~Figure B.7 for nomenclature.



~~Fig.~~Figure B.7: Nomenclature for horizontal axis cylindrical tank (from ref. [10])

Approximate values for hydrodynamic pressures induced by horizontal excitation in either the longitudinal or transverse direction can be obtained from solutions for the rectangular tank of equal dimension at the liquid level and in the direction of motion, and of a depth required to give equal liquid volume. This approximation is sufficiently accurate for design purposes over the range of H/R between 0,5 and 1,6. When 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 excitation perpendicular to the axis, a more accurate solution is available for partially full tanks.

The impulsive pressure distribution is given in this case by:

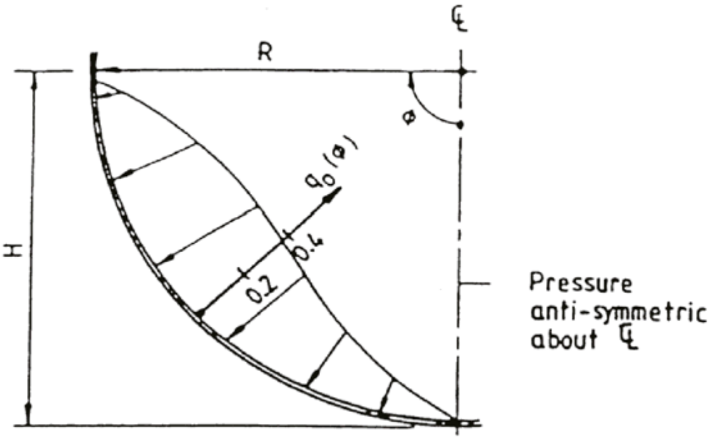
$$p_{\mathrm{i}}(\phi) = q_o(\phi)\gamma R A_{\mathrm{g}}(t) \tag{B.47}$$

For $H = R$ the pressure function $q_{\mathrm{o}}(\cdot)$ takes the form:

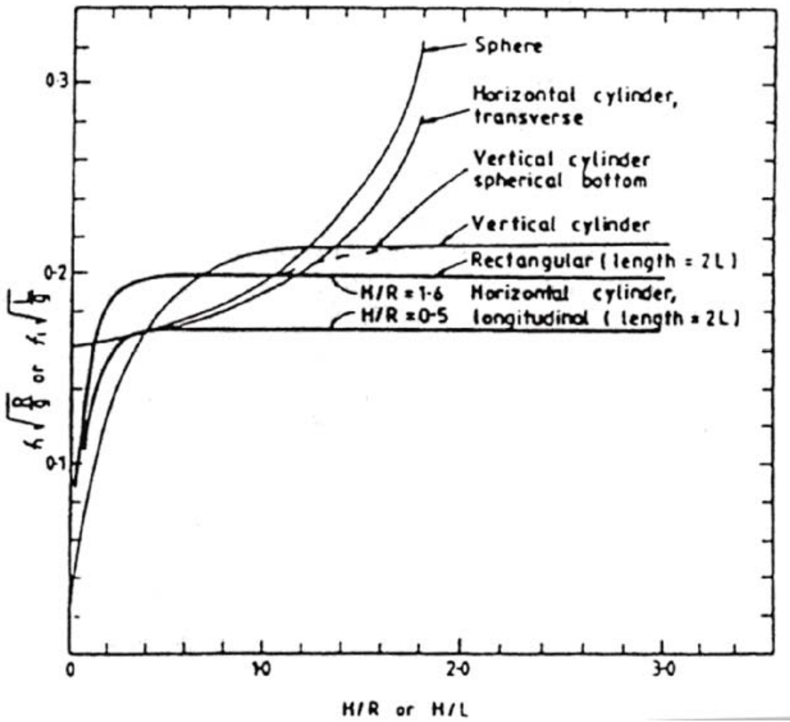
$$q_o(\phi) = \frac{H}{\pi}\sum_{1=n}^{\infty}\frac{(-1)^{n-1}}{(2n)^2-1}\sin 2n\phi \tag{B.48}$$

The function $p_{\mathrm{o}}(\cdot)$ is plotted in ~~Fig.~~Figure B.8. By integrating the pressure distribution the impulsive mass is evaluated to be:

$$m_{\mathrm{i}} = 0{,}4m \tag{B.49}$$



~~Fig.~~Figure B.8: Impulsive pressures on horizontal cylinder with $H = R$. Transverse excitation (from ref. [10])



~~Fig.~~Figure B.9 - Dimensionless first convective mode frequency for rigid tanks of various shapes (from ref. [10])

Because the pressures are in the radial direction, the forces acting on the cylinder pass through the centre of the circular section, and both the impulsive and the convective masses should be assumed to act at this point.

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 can be evaluated as:

$$m_{c1} = 0{,}6m \qquad \text{(B.50)}$$

The two expressions given for the masses m_i and m_{c1} are expected to be reasonable approximations for values of H/R ranging from 0,8 to 1,2.

The first mode sloshing frequencies for tanks of various shapes, including horizontal circular cylinders, with motion along and transverse to the axis, are shown in ~~Fig.~~Figure B.9.

B.6 Elevated tanks

Elevated tanks can have supporting structures of different types, from simple cylindrical towers to frame or truss-like structures. For the purpose of the analysis, the presence of the liquid in the supported tank can be accounted for considering two masses: an impulsive mass m_i located at a height h'_i above the tank bottom (eq. (B.4) and (B.6), respectively), and a mass m_{c1} located at a height h_{c1} (eq. (B.12) and (B.14), respectively).

The mass m_i is rigidly connected to the tank walls, while the mass m_{c1} is connected to the walls through a spring of stiffness: $K_{c1} = \omega^2_{c1}\, m_{c1}$, where ω_{c1} is given by eq. (B.9).

The mass of the tank is included in the structural model which describes also the supporting structure. The response of the system can be evaluated using standard modal analysis and response spectra methods.

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

In some cases, the rotational inertia of the mass ($m_i + \Delta_m$), and the corresponding additional degree of freedom, need also to be considered.

In the relatively common case where the shape of the elevated tank is a truncated inverted cone (or close to it), an equivalent cylinder can be considered, having the same volume of liquid as the real tank, and a diameter equal to that of the cone at the level of the liquid.

B.7 Soil-structure interaction effects

For tanks founded on relatively deformable soils, the resulting base motion can be significantly different from the free-field motion, and it includes generally a rocking component, in addition to a modified translational component.

Accurate solutions for the interaction problem between tank-fluid and soil systems have been developed only recently for the case of tanks with rigid foundation on homogeneous soil: see ref. [14], [15], [16]. The solution procedures are based on the sub-structuring approach, whereby the response of the deformable tank and of the soil beneath the foundation are first expressed separately for an excitation consisting of a horizontal and a rocking motion: equilibrium and compatibility conditions imposed at the interface yield a set of two equations on the unknown ground displacement components.

Analyses performed on tanks of various geometries confirm what was known from previous studies on building systems. Increasing the flexibility of the supporting medium lengthens the period of the tank-fluid system and reduces the peak of the response (for the same input) due to an increase of the total damping. For a given soil flexibility, the increase in the fundamental period is more pronounced for tall, slender tanks, because the contribution of the rocking component is greater for these structures than for short, broad tanks. The reduction of the peak response, however, is in general less significant for tall tanks, since the damping associated with rocking is smaller than the damping associated with horizontal translation.

Although the method in ref. [15] would be easily implemented in a computer code, simpler procedures are desirable for design purposes. One such procedure has been proposed for buildings already several years ago, see ref. [13], and consists of a modification (increase) of the fundamental period and of the damping of the structure, considered to rest on a rigid soil and subjected to the free-field motion.

This procedure has been extended to tanks, see refs. [15] and [16], and more specifically, to the impulsive (rigid and flexible) components of the response. The convective periods and pressures are assumed not to be affected by soil-structure interaction.

The recent study in ref. [15] confirms the good approximation that 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 the replacement oscillator are given in ref. [15] in the form of graphs, as functions of the ratio H/R and for fixed values of the other parameters: wall thickness ratio s/R, initial damping, etc. These graphs can be effectively used whenever applicable.

Alternatively, the less approximate procedure of ref. [2] and [10], as summarised below, can still be adopted.

Since the hydrodynamic effects considered in **B.**2 to **B.**5 and, specifically, the impulsive rigid and impulsive flexible pressure contributions, are mathematically equivalent to a single degree-of-freedom system, and they are uncoupled from each other, the procedure operates by simply changing separately their frequency and damping factors.

In particular, for the rigid impulsive pressure components, whose variation with time is given by the free-field horizontal: $A_g(t)$, and vertical: $A_v(t)$ accelerations, inclusion of soil-structure interaction effects involves replacing the time-histories above with the response acceleration functions of a single degree of freedom oscillator having frequency and damping factors values as specified below.

Modified natural periods

– "rigid tank" impulsive effect, horizontal

$$T_i^* = 2\pi\left(\frac{m_i + m_o}{k_x \alpha_x} + \frac{m_i h_i'^2}{k_\theta \alpha_\theta}\right)^{1/2} \tag{B.51}$$

– "deformable tank" impulsive effect, horizontal

$$T_f^* = T_f\left(1 + \frac{k_f}{k_x \alpha_x} + \left[1 + \frac{k_x h_f^2}{k_\theta \alpha_\theta}\right]\right) \tag{B.52}$$

– "rigid tank", vertical

$$T_{vr}^{*} = 2\pi \left(\frac{m_{tot}}{k_v \alpha_v} \right)^{1/2} \tag{B.53}$$

– "deformable tank", vertical

$$T_{vd}^{*} = T_{vd} \left(1 + \frac{k_1}{k_v \alpha_v} \right)^{1/2} \tag{B.54}$$

where:

m_i , h'_i mass and height of the impulsive component

m_o mass of the foundation

k_f stiffness associated to the "deformable tank" = $4\pi^2 \frac{m_f}{T_f^2}$

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

k_1 $= 4\pi^2 \frac{m_1}{T_{vd}^2}$, with m_l = mass of the contained liquid

where:

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

$\alpha_x, \alpha_\theta, \alpha_v$ frequency dependent factors which convert the static stiffnesses into the corresponding dynamic ones

Modified damping values

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

$$\xi = \xi_s + \frac{\xi_m}{\left(T^* / T\right)^3} \tag{B.55}$$

where:

ξ_s radiation damping in the soil

ξ_m material damping in the tank

Both ξ_s and ξ_m depend on the specific oscillation mode.

In particular for ξ_s one has:

– for the horizontal impulsive "rigid tank" mode

$$\xi_s = 2\pi^2 \frac{a}{T_i^*} \left(\frac{\beta_x}{\alpha_x} + \frac{k_x h_i'^2 \beta_\theta}{k_\theta \alpha_\theta} \right) \tag{B.56}$$

– for the horizontal impulsive "deformable tank" mode

$$\xi_s = \frac{2\pi^2 m_f}{k_x T_f^{*2}} a \left(\frac{\beta_x}{\alpha_x} + \frac{k_x h_f^2 \beta_\theta}{k_\theta \alpha_\theta} \right) \tag{B.57}$$

– for the vertical "rigid tank" mode

$$\xi_s = 2\pi^2 \frac{a \ \beta_v}{T_{vr}^* \alpha_v} \tag{B.58}$$

where:

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

$\beta_x, \beta_v, \beta_\theta$ frequency-dependent factors providing radiation damping values for horizontal vertical and rocking motions

Expressions for the factors $\alpha_x, \alpha_\theta, \alpha_v$ and $\beta_x, \beta_\theta, \beta_v$ can be found for example in ref.[4].

B.8 Unanchored tanks

Tanks are often built with the walls not anchored to the foundation, for reasons of economy. In case of earthquake, if the overturning moment due to the hydrodynamic forces is larger than the stabilizing one some uplift occurs. It is difficult to avoid in this case plastic deformations in the tank, at least in the base plate. Leakage of the liquid, however, can be prevented by proper design.

The mechanism of tank uplift is obviously complex and substant~~c~~ially sensitive to several parameters, both from the point of view of tank response and of the subsequent stress analysis.

In most cases, the effects of the uplift, and of the accompanying rocking motion, on the magnitude and the distribution of the pressures is disregarded, and the pressures calculated for an anchored tank are used. This is believed to be in many a conservative approach, due to the fact that rocking adds flexibility to the tank-fluid system, and hence shifts the period into a range of lesser amplification. This approach is accepted in ref. [5].

The only approximate design procedure elaborated thus far which accounts for the dynamic nature of the problem is presented in ref. [3], and can be used if deemed appropriate.

For the purpose of the present Annex a conceptual outline of the procedure in ref. [3] is adequate.

– The sloshing and the rigid impulsive pressure components are assumed to remain unaffected by the rocking motion.

– The flexible impulsive component is treated using expressions analogous to eq. (B.18) to (B.28), but on the basis of a first mode shape which includes, in addition to the deformation of the shell, the uplift of the base. Modified values of the mass m_f and of its height h_f are obtained, as functions, as before, of the ratio H/R; of course these modified values depend on the amount of uplift, but this dependence is found numerically to be weak so that average values can be used.

– For what concerns the dynamic response, the objective is to find the fundamental period of a system made up of a deformable tank-fluid sub-system, linked to the ground by means of vertical springs characterized by a non-linear force-uplift relationship.

– The non-linearity of the base springs is treated in an "equivalent" linear way by assuming their average stiffness for a vertical deformation going from zero to the maximum value reached during the response. Based on extensive Finite Element analyses on steel tanks typical of oil industry, results have been obtained in the form of graphs, which give the fundamental period of the whole system in the form:

$$T_f = 2\pi\sqrt{\frac{R}{g}}F\left(\frac{d_{max}}{R},\frac{H}{R}\right) \tag{B.59}$$

where d_{max} is the maximum displacement at the level h_f where the mass m_f is located, and $F(\cdot)$ is an empirical function of the two non~~a~~dimensional parameters indicated.

The procedure then works iteratively as follows:

– starting with the fixed-base value of the overturning moment, a value of d_{max} is obtained using a non-dimensional graph prepared for different H/R values;

– based on this value, the period of the system is calculated from eq. (B.59), and using the appropriate response spectrum, the impulsive flexible component of the response is obtained;

– combining the latter response with the sloshing and the rigid-one, a new value of the total overturning moment is obtained, and so forth until convergence is achieved.

The limitation in the use of the procedure described is that available design charts refer to specific values of important parameters, as for ex. the thickness ratio of the wall, the soil stiffness, the wall foundation type, etc., which are known to influence the response to a significant extent.

Once the hydrodynamic pressures are known, whether determined ignoring or considering occurrence of uplift, the following step of calculating the stresses in the critical regions of the tank is a matter of structural analysis, an area in which the designer must have a certain freedom in selecting the level of sophistication of the method he uses, under the condition that the less approximate ones must be clearly on the safe side.

For an uplifting tank, an accurate model would necessarily involve a Finite Element method with non-linear capabilities, a fact which is still out of common practice. At the other extreme, rather crude methods, not requiring the use of computer, have been developed long ago, and they are still proposed in current design standards, as for ex. in ref. [10].

These methods have been proven to be unconservative by experiments and by more refined analyses and, more generally, to be inadequate for accounting of all the variables entering the problem.

Simplified but comprehensive computer methods have been proposed recently in the literature, see for ex. ref. [7] and [9], and they will gradually replace the present ones.

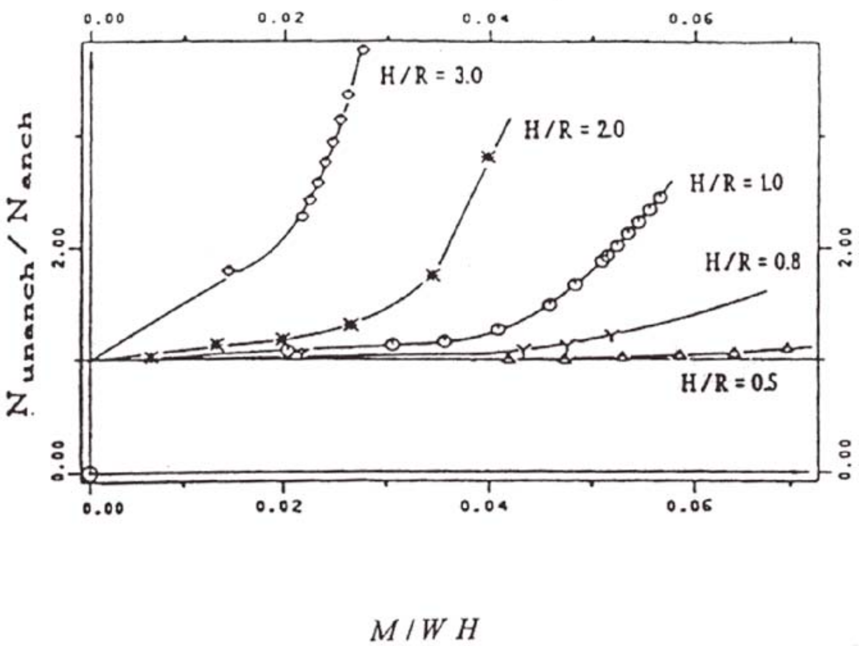
The principal effect of uplift is to increase the compressive vertical stress in the shell, which is critical with regard to buckling-related types of failure. At the opposite side of the wall where the compression is maximum, hoop compressive stresses are generated in the shell, due to the membrane action of the base plate.

These latter stresses, however, in combination with the other stress components, are not critical for the stability of the tank. Finally, flexural yielding is accepted to take place in the base plate, and a check of the maximum tensile stress is appropriate.

Compressive axial stress in the wall due to uplift

The increase of the vertical stress due to uplift (N_u) with respect to the stress in the anchored case (N_a) can be estimated from ~~Fig.~~Figure B.10, taken from ref. [12]. The ratio N_u/N_a is given in ~~Fig.~~Figure B.10 as function of the nona~~a~~dimensional overturning moment: M/WH (W = total weight of the liquid).

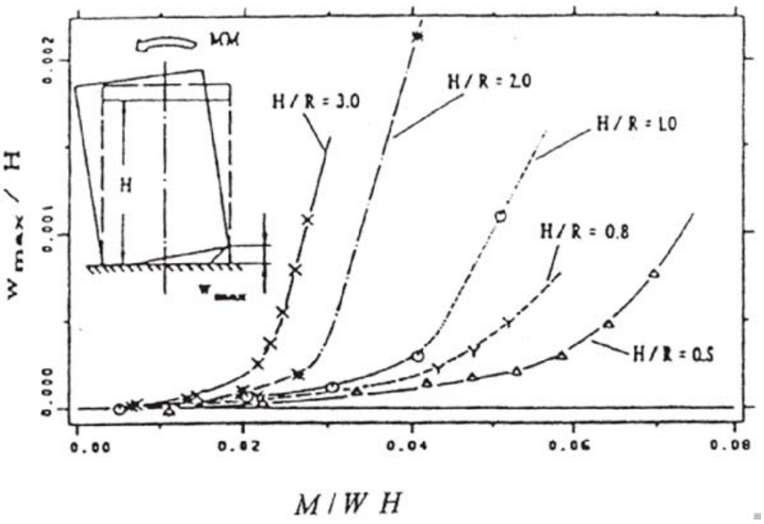
It is seen that for slender tanks the increase is very significant. The values in ~~Fig.~~Figure B.10 should be on the safe side, since they have been calculated (using static Finite Element analysis) assuming the underlying soil to be quite rigid (Winkler coefficient k=4000 N/cm^3) which is an unfavourable situation for the considered effect.



~~Fig.~~Figure B.10: Ratio of maximum compressive axial membrane force for unanchored and anchored tanks versus overturning moment (from ref. [12])

Shell uplift and uplifted length of the base plate

From a parametric study with F.E. models, performed on a number of tanks of commonly used geometry, the amount of uplift has been derived in ref. [12], and it is given in ~~Fig.~~Figure B.11 as a function of the overturning moment *M*/*WH*, for different values of the ratio *H*/*R*. For estimating the radial membrane stresses in the plate, the length *L* of the uplifted part of the tank bottom is also necessary. Results obtained from the parametric study mentioned above are shown in ~~Fig.~~Figure B.12. The dependence of *L* on the uplift *w* is almost linear, the values of *L* being larger (for a given *w*) for squat tanks than for slender ones.



~~Fig.~~Figure B.11: Maximum uplift height versus overturning moment *M*/*WH* (from ref. [12])

Radial membrane stresses in the base plate

An estimate of the membrane stress σ_{rb} in the base plate due to uplift has been derived in ref. [1]:

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

where

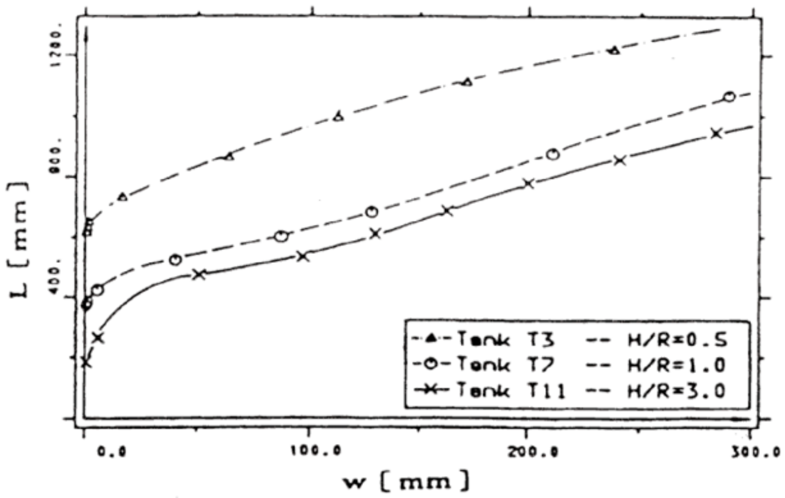
t is the thickness of the plate

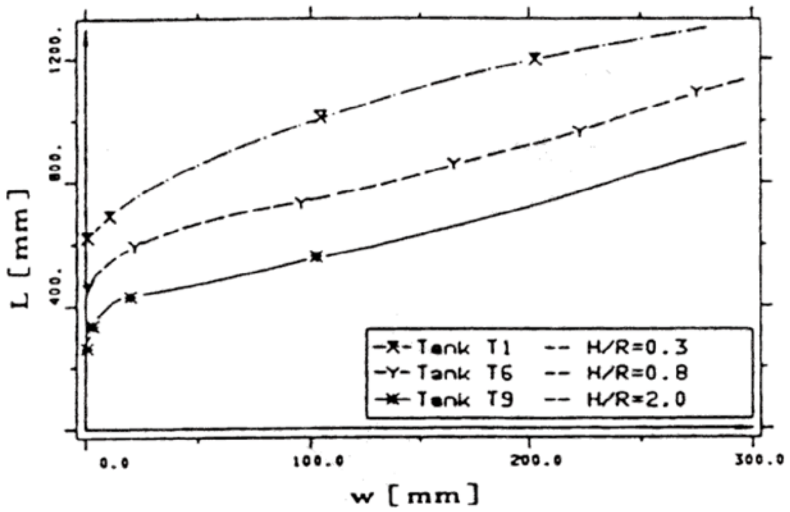
p is the hydrostatic pressure on the base

μ = $(R/L)/R$, with L = uplifted part of the base

Plastic rotation of the base plate

A recommended practice is 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.





~~Fig.~~Figure B.12: Length of the uplifted part as a function of the uplift (from ref. [12])

The rotation of the plastic hinge in the tank base must be compatible with the available flexural ductility.

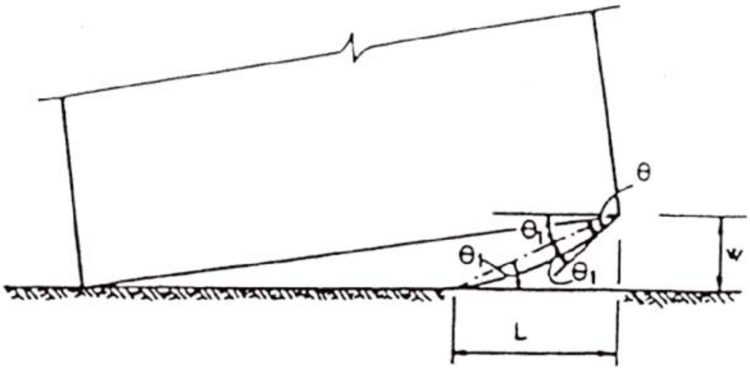
Assuming a maximum allowable steel strain of 0,05 and a length of the plastic hinge equal to 2 t, the maximum allowable rotation is.

$$\theta = \left(\frac{0{,}05}{t/2} \right) 2t = 0{,}20 \text{ radians} \tag{B.61}$$

From ~~Fig.~~Figure B.13 the rotation associated to an uplift w and a base separation of L is:

$$\theta = \left(\frac{2w}{L} - \frac{w}{2R} \right) \tag{B.62}$$

which must be less than 0,20 radians.



~~Fig.~~Figure **B.13: Plastic rotation of base plate of uplifting tank (from ref. [10])**

B.9 Stability verifications for steel tanks

Stability verifications have to be performed with respect to two possible failure modes.

a) Elastic buckling

This form of buckling has been observed to occur in those parts of the shell where the thickness is reduced with respect to the thickness of the base, and the internal pressure (which has a stabilising effect) is also reduced with respect to the maximum value it attains at the base. This verification should be carried out assuming the vertical component of the seismic excitation to give zero contribution to the internal pressure.

Denoting by σ_m the maximum vertical membrane stress, the following inequality shall be satisfied:

$$\frac{\sigma_m}{\sigma_{c1}} \leq 0{,}19 + 0{,}81 \frac{\sigma_p}{\sigma_{c1}} \tag{B. 63}$$

where

$$\sigma_{c1} = 0{,}6 \cdot E \frac{s}{R} \tag{B.64}$$

(ideal critical buckling stress for cylinders loaded in axial compression)

$$\sigma_p = \sigma_{c1} \left[1 - \left(1 - \frac{\bar{p}}{5} \right)^2 \left(1 - \frac{\sigma_o}{\sigma_{c1}} \right)^2 \right]^{1/2} \leq \sigma_{c1} \tag{B.65}$$

$$\bar{p} = \frac{pR}{s\sigma_{c1}} < 5 \tag{B.66}$$

$$\sigma_o = f_y \left(1 - \frac{\lambda^2}{4} \right) \text{if} : \lambda^2 = \frac{f_y}{\sigma \sigma_{c1}} \leq 2 \tag{B.67.a}$$

$$\sigma_o = \bar{\sigma}\,\sigma_{c1} \qquad \text{if:} \qquad \lambda^2 \geq 2 \tag{B.67.b}$$

$$\bar{\sigma} = 1 - 1{,}24\left(\frac{\delta}{s}\right)\left[\left(1 + \frac{2}{1{,}24\left(\frac{\delta}{s}\right)}\right)^{1/2} - 1\right] \tag{B.68}$$

$\left(\frac{\delta}{s}\right)$ is the ratio of maximum imperfection amplitude to wall thickness which can be taken as (see ref. [10]):

$$\left(\frac{\delta}{s}\right) = \frac{0{,}06}{a}\sqrt{\frac{R}{s}} \tag{B.69}$$

with:

$a = 1$ for normal construction

$a = 1{,}5$ for quality construction

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

In eq. (B.65), the second term within square brackets at the right hand side takes into account of the favourable effect of the internal pressure, while the third one (which is set as a factor of the previous one) provides the reduction of the critical stress due to the imperfections.

b) Elastic-plastic collapse

This form of buckling occurs normally close to the base of the tank, due to a combination of vertical compressive stresses, tensile hoop stresses and high shear, inducing an inelastic biaxial state of stress: the mode of collapse is referred to as 'elephant's foot'.

The empirical equation developed in ref. [11] to check for this form of instability is:

$$\sigma_{\mathrm{m}} = \sigma_{\mathrm{c1}}\left[1 - \left(\frac{pR}{s f_{\mathrm{y}}}\right)^2\right]\left(1 - \frac{1}{1{,}12 + r^{1{,}15}}\right)\left[\frac{r + f_{\mathrm{y}}/250}{r+1}\right] \tag{B.70}$$

where

$r = \dfrac{R/s}{400}$ and f_{y} is expressed in MPa~~pB~~.

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ANNEX C (INFORMATIVE) BURIED PIPELINES

C.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 consist either in 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.

C.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 to reduce the soil-pipeline interaction problem 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, due to Newmark,[6] the soil motion is represented by a single sinusoidal wave:

$$u(x,t) = d \sin \omega(t - \frac{x}{c}) \tag{C.1}$$

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:

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

whose maximum value is:

$$\varepsilon_{\max} = \frac{v}{c} \tag{C.3}$$

with $v = \omega d$ being the peak soil velocity

[6] Newmark, N. M. 1967, Problems In Wave Propagation In Soil And Rock, Proc. Intnl. Symp. on Wave Propagation and Dynamic Properties of Earth Materials, Univ. of New Mexico, Albuquerque, New Mexico, 7-26

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

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

whose maximum value is:

$$\chi_{\max} = \frac{a}{c^2} \tag{C.5}$$

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

(12) If the directions of the pipeline and of the propagation do not coincide, in both cases of wave types longitudinal strains and curvatures are produced, which are functioning of the angle ~~ϑ~~ θ formed by the two directions. The longitudinal strains are given in this case by

$$\varepsilon(\theta) = \frac{v}{c} \cdot f_1(\theta) + \frac{a}{c^2} f_2(\theta) \cdot R \tag{C.6}$$

where R is the diameter of the pipe. Since the second term is in general small compared with the first one, the maximum of the sum occurs when the first term is at its maximum, that is, with a value: v/c.

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

$$\tau_{av} = s\, E \frac{a}{c^2} \tag{C.7}$$

where E and s are the Modulus of Elasticity and thickness of the pipe, and τ_{av} is the average shear stress between pipe and soil which depends on the friction coefficient between soil and pipe, and on the burial depth.