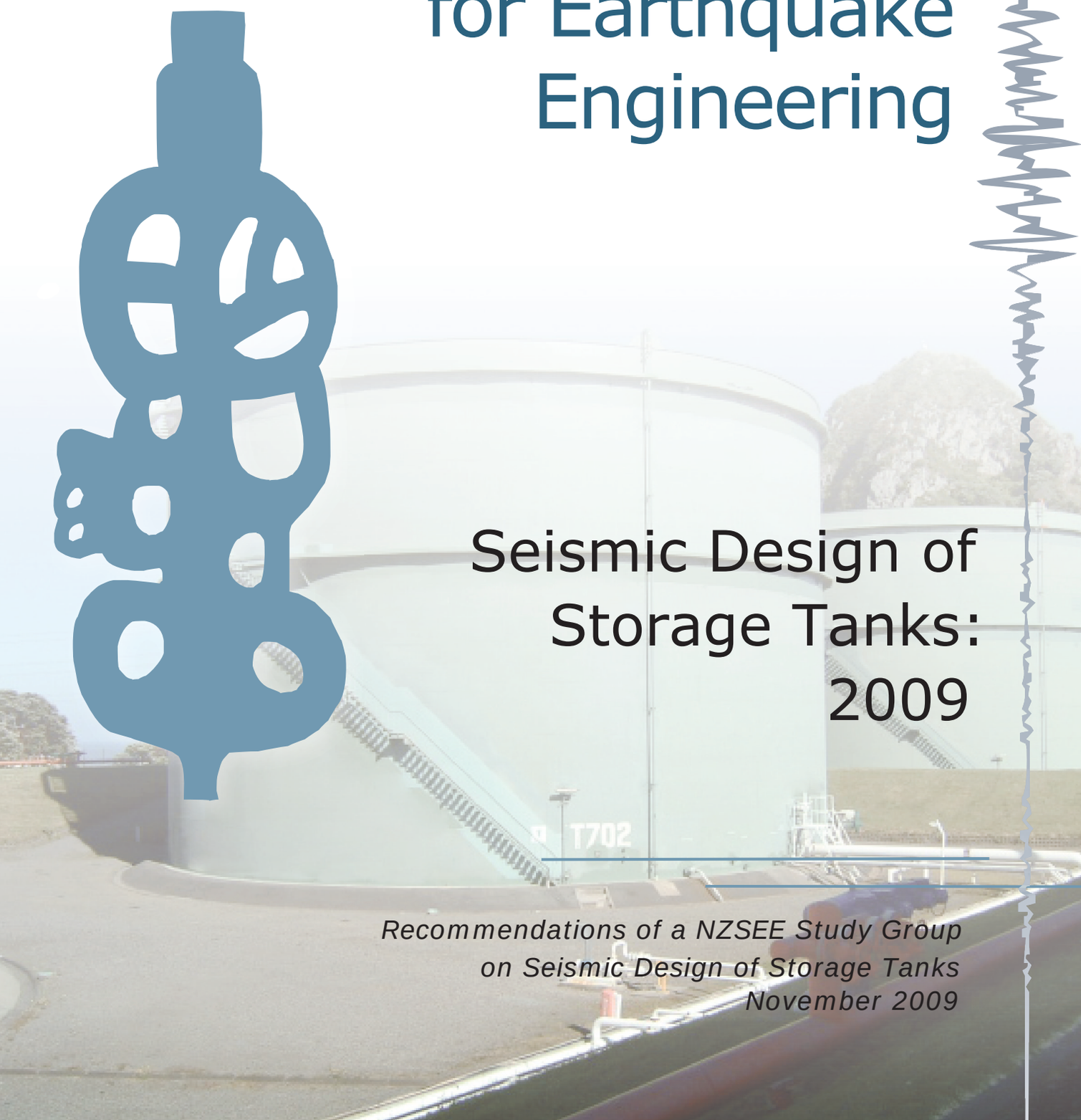
New Zealand Society for Earthquake Engineering



Seismic Design of Storage Tanks: 2009

Recommendations of a NZSEE Study Group on Seismic Design of Storage Tanks
November 2009

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This report does not necessarily reflect the views of the New Zealand Society for Earthquake Engineering, nor that of any of the Sponsors.

Neither the NZSEE, nor any member of the Study Group, accepts any liability for the application of these Guidelines in any specific instance.

Seismic Design of Storage Tanks: 2009

Recommendations of a Study Group of the New Zealand Society for Earthquake Engineering

Foreword

The Recommendations for Seismic Design of Storage Tanks document (sometimes called the "Tank Red Book") was published in 1986. Over the ensuing 20 years this document has been widely used in New Zealand and acknowledged internationally. NZSEE still receives frequent requests for copies of the original document, and there has been wide interest in the publication of this revision.

Since 1986 there have been substantial changes to the regulatory environment and applicable standards in New Zealand. Changes include the following:

- 1992 Building Act (amended 2004) introduced a new performance-based building control system and the New Zealand Building Code. The Building Act defines buildings as including tanks except where covered by the HSNO Act 1996.
- Applicable structural design standards have been revised; NZS 4203:1992 and now NZS 1170.5:2004 specifies seismic loads on building structures in New Zealand.
- New legislative requirements for storage of hazardous liquids have been issued in the HSNO Act 1996.

Expected safety levels and return periods used for earthquake design have increased worldwide. Prior to 1986 recurrence intervals of the order of 150 years were commonly used for earthquake design. Current practice is to design for at least 1 in 500 year events and up to 1 in 2,500 years for critical structures.

Storage tank behaviour is different from buildings in relation to ductility and damping levels. Building structures and design codes are typically based on 5% damping. In tanks a range of damping levels is used. For the convective sloshing mode damping is assumed to be about 0.5%. For the impulsive modes, 5% is common, however, the beneficial effects of additional radiation damping from soil-structure interaction can also provide significantly higher damping, as is recognised in the NZSEE approach.

Some conservatism was found in the practical application of the 1986 document, particularly for design of large steel storage tanks. The design basis assumed no yield or damage being permitted to tanks under the design earthquake loading.

Environmental protection and sustainability issues are of high importance to our community. This document aims to acknowledge the potential effects on the environment as a critical aspect in setting seismic performance for liquid containment systems.

In New Zealand the Hazardous Substances and New Organisms Act 1996 (HSNO) has replaced the previous Dangerous Goods Act and Regulations. This document has been prepared with the intention that it is recognised by the HSNO legislation as an Approved Code of Practice. Other standards are also recognised by HSNO for design of containers for storage of hazardous substances.

API 650 *Welded Steel Tanks for Oil Storage* has been widely used internationally for design of steel storage tanks, and has contained seismic design provisions for several decades. In the 2008 11th edition of API 650 the design seismic loads are linked to ASCE 7, and a procedure is provided to allow appropriate design seismic actions to be determined for other countries. Application of API 650 in New Zealand using seismic loads based on NZS 1170.5 is regarded as a suitable approach for seismic design of steel storage tanks.

Eurocode 8: Design of Structures for Earthquake Resistance – Part 4 Silos, Tanks and Pipelines published in 2006 makes direct reference to the 1986 NZSEE document. The Eurocode uses the 1986 NZSEE methodology for the design of horizontal cylindrical and rectangular tanks, including the method for adjusting the tank response for soil-structure

interaction. The Eurocode methodology for the design of vertical cylindrical tanks uses a similar approach to the recommendations in this NZSEE 2009 document, including an allowance for some reduction in design force from the fully elastic response.

Another design guide which has previously been widely used in New Zealand is the 1981 document *Seismic Design of Petrochemical Plants* prepared for the NZ Ministry of Energy. This was a useful and simple document, based on a modified API 650 approach for use in New Zealand. It is recommended that this approach should no longer be used and should not be recognised by current legislation as the provisions are out of date.

The principal changes in this revised document are:

- Seismic load is derived from the current national standard for derivation of seismic loads for buildings in New Zealand, NZS 1170.5.
- A procedure is presented to allow assessment of an appropriate return period factor, R_u.
- A correction factor based on the ductility and damping applicable to tank behaviour is applied.
- Some limited ductility is permitted in steel tanks on grade. This generally reduces the load demands from those given previously.
- The document is presented in a form that can be interfaced with other international design codes.

In preparing this revised document, benchmarking comparisons have been made against the previous document and other relevant codes. These comparisons suggest that the proposed approach is reasonable.

Design of concrete liquid retaining structures in New Zealand was covered by NZS 3106:1986. The Standard also specifies seismic design loads. Although for many liquid storage structures, seismic load cases will not often govern the design, the loads specified in NZS 3106:1986 were somewhat out of step with more recent seismic loads given in the NZ Loading Standards NZS 1170. Standards New Zealand is currently preparing a revision to NZS 3106.

It is hoped that this updated document will be useful and widely used within the industry.

David Whittaker

(Editor and Convenor)

(Draft page for ERMA review)

ERMA Approved Code of Practice

The process for development and approval of HSNO codes of practice is set out in the Environmental Risk Management Authority (ERMA) information Sheet 13 dated May 2001.

This Code of Practice is approved pursuant to Sections …… of the Hazardous Substance and New Organisms Act. The Environmental Risk Management Authority has delegated power to approve Codes of Practice to the Chief Executive of the Authority, and this Code is approved in accordance with that delegation. It is confirmed that the requirements of Sections ……. have been met.

This code has been developed by the New Zealand Society for Earthquake Engineering, NZSEE.

Notice of approval of this Code has been published in the Gazette dated ……

Pursuant to Section …….. of the Act, the Code may be inspected on request at the Wellington office of ERMA New Zealand (HSNO #.....) or alternatively, can be purchased from NZSEE.

Approved this ……..

Not yet approved for use

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Notation

B	Half tank width perpendicular to direction of earthquake
C	Foundation stiffness factor
$C_d(T_i)$	Horizontal design action coefficient for mode i including correction factor to account for ductility and level of damping
$C_h(T_i)$	Elastic site hazard spectrum coefficient for mode i from NZS 1170.5:2004
C(0)	The site hazard coefficient for T=0 using the modal response spectrum from NZS 1170.5
D_{HD}	Pitch circle diameter of hold-down bolts for anchored tank
E	Modulus of elasticity for tank wall material
E_s	Modulus of elasticity for soil
E_u	Loads resulting from earthquake ultimate limit state
G	Gravity loads resulting from dead weight of tank
G_s	Shear modulus for soil
H	Height of liquid surface above tank base
H_e	Tank embedment depth
K_1	Equivalent stiffness of first convective mode
K_b	Equivalent vertical stiffness of tank-liquid system
K_f	Equivalent horizontal stiffness of tank-liquid system
K_s	Support structure stiffness
K_v	Vertical stiffness for foundation
K_x	Horizontal translation stiffness of foundation
K_θ	Rocking stiffness of foundation
L	Half length of rectangular tank
L_b	Uplifted length of shell base
M_1	Overturning moment just above the base from first convective mode
M'_1	Total overturning moment from first convective mode including base pressure
M_f	Overturning moment just above the base from flexible wall impulsive mode
M'_f	Total overturning moment from flexible wall impulsive mode including base pressure
M_0	Overturning moment just above the base from impulsive mode
M'_0	Total overturning moment from impulsive mode including base pressure
M_{0T}	Total design overturning moment
M_R	Resisting/restoring moment for uplifting tank
M_r	Overturning moment just above the base from rigid impulsive mode
M'_r	Total overturning moment from rigid impulsive mode including base pressure
M_z	Wall vertical bending moment /m

M_{zr}	Maximum impulsive-rigid bending moment/unit length
M_{zf}	Maximum impulsive-flexible bending moment/unit length
M_{zl}	Maximum 1st mode convective bending moment/unit length
M_{zv}	Maximum vertical impulsive bending moment/unit length
N_θ	Maximum resultant hoop force/unit length
$N_{\theta 1}$	Maximum lst mode convective hoop force/unit length
$N_{\theta f}$	Maximum impulsive-flexible hoop force/unit length
$N_{\theta h}$	Maximum hoop force/unit length due to hydrostatic pressure
$N_{\theta i}$	Maximum impulsive hoop-force/unit length
$N_{\theta r}$	Maximum impulsive-rigid hoop force/unit length
$N_{\theta v}$	Maximum hoop force/unit length due to vertical acceleration
N_z	Maximum axial force/unit length
N_{z1}	Maximum lst mode convective axial force/unit length
N_{zf}	Maximum impulsive-flexible axial force/unit length
N_{zr}	Maximum impulsive-rigid axial force/unit length
N_{zv}	Maximum vertical impulsive axial force/unit length
P	Anchorage force per unit circumferential length
Q	Gravity loads resulting from tank contents
R	Radius of tank
R_b	Radius of foundation
R_m	Mean radius of tank wall
R_u	Return period factor for ultimate limit state
S_p	Structural performance factor
T	Period of vibration
T_1	Period of first convective mode
T_b	Period of first vertical (breathing) mode
$\check{T}_b$	Period of first vertical (breathing) mode including soil-structure interaction effects
T_{eq}	Period of equivalent linear system for yielding structure
T_f	Period of first horizontal tank-liquid impulsive mode
$\check{T}_f$	Period of first horizontal tank-liquid impulsive mode including soil-structure interaction effects
T_i	Period of vibration of ith convective mode
T_i	Period of vibration of mode i
$\check{T}_0$	Period of impulsive mode including soil-structure interaction effects
$\check{T}_v$	Period of first vertical mode including soil-structure interaction effects
V	Total horizontal base shear
V_1	Base shear from first convective mode

V_f	Base shear from flexible wall impulsive mode
V_i	Horizontal base shear associated with mode i
V_0	Base shear from impulsive mode
V_r	Base shear from rigid impulsive mode
W	Total weight of fluid
W_f	Weight of fluid supported by part of base that does not uplift
W_s	Total compression reaction in the shell for uplifting circular tank at base
W_w	Total weight of walls and roof of uplifting tank
Z_T	Section modulus for the tank
Z	Seismic zone factor from NZS 1170.5
a	Foundation frequency parameter
a	Quality indicator for tank construction
ä	Peak response acceleration
$ä_o$	Peak ground acceleration
$a_j(z)$	Coefficient of horizontal acceleration response in jth sloshing mode
c_s	Sloshing wave impact coefficient
d_f	Deflection of tank wall on vertical centreline at height of impulsive mass, m_f
d_{max}	Maximum vertical displacement of sloshing wave
f_b	Surface stress under wall vertical bending moment
f_{cl}	Classical membrane compression buckling stress for perfect elastic cylinder under axial load
f_h	Membrane hoop stress
f_m	Vertical membrane compression stress for tank with internal pressure, subject to flexural compression (cantilever mode)
f_0	Membrane compression buckling stress for tank without internal pressure, subject to uniform compression
f_p	Membrane compression buckling stress for tank with internal pressure, subject to uniform compression
f_{rb}	Membrane stress in base plate
f_y	Material yield stress
f_{z1}	Maximum 1st mode convective stress
f_{ze}	Maximum resultant seismic stress from maximum impulsive, convective and vertical stresses combined using SRSS method
f_{zf}	Maximum impulsive-flexible stress
f_{zp}	Wall stress resulting from the overturning forces
f_{zr}	Maximum impulsive-rigid stress
f_{zv}	Maximum vertical impulsive stress
g	Acceleration of gravity
h_1	Height of first convective mode mass above base

h'_1	Height of first convective mode mass above base for total overturning including base pressure
h_f	Height of flexible wall impulsive mass, m_f , above the tank base
h_i	Height of mode i mass above base
h_0	Height of rigid impulsive mass above base, equivalent to h_r for a flexible tank
h'_0	Height of rigid impulsive mass above base for total overturning including base pressure, equivalent to h'_r for a flexible tank
h_r	Height of rigid impulsive mass, m_r , above the tank base
h_r	Head of liquid due to sloshing wave contacting tank roof
h_t	Height to centre of gravity of roof mass
h_w	Height to centre of gravity of wall mass
k_1	Spring stiffness for first convective mode
$k_f(\mu,\xi_i)$	Correction factor for AS/NZS 1170 spectrum to account for ductility and level of damping for mode i
k_h	Period coefficient for first horizontal tank liquid impulsive mode
k_i	Spring stiffness for mode i
k_v	Period coefficient for first vertical breathing mode
m_1	Equivalent mass for first convective mode
m_b	Mass of base
m_f	Impulsive mass for wall horizontal deformation mode
m_i	Equivalent mass of tank and contents in mode i
m_l	Total mass of fluid contained in tank
m_0	Impulsive mass for rigid tank
m_r	Impulsive mass for rigid body mode
m_s	Total mass of tank including base, support system and structural foundation
m_t	Mass of tank roof
m_v	Total mass of liquid and tank including foundation
m_w	Mass of tank walls
p	Pressure
p(z,t)	Total hydrodynamic pressure on rectangular tank wall
$p(z,\theta,t)$	Total hydrodynamic pressure on circular tank wall
p_b	Buoyancy pressure on tank roof
$p_c(z,\theta,t)$	Convective pressure component on circular tank wall
$p_c(z,t)$	Convective pressure component rectangular tank wall
$p_{c1}(z,\theta)$	Maximum first convective mode pressure component on circular tank wall
$p_{c1}(z)$	Maximum first convective mode pressure component on rectangular tank wall
$p_i(\phi)$	Impulsive pressure component on horizontal cylinder
$p_i(z)$	Maximum impulsive pressure component on rectangular tank wall
$p_i(z,t)$	Impulsive pressure component on rectangular tank wall

$p_i(z,\theta)$	Maximum impulsive pressure component on circular tank wall
$p_i(z,\theta,t)$	Impulsive pressure component on circular tank wall
p_o	Hydrostatic pressure on the base
p_r	Hydrodynamic pressure on tank roof
$p_v(z)$	Pressure on tank walls from vertical acceleration
q	Shear force/unit length
q	Linear equivalent pressure distribution
$q_1(z)$	Dimensionless pressure function for first sloshing mode
q_c	Circumferential shear flow
q_h	Equivalent hydrostatic pressure distribution
$q_j(z)$	Dimensionless pressure function for jth sloshing mode
$q_0(z)$	Dimensionless impulsive pressure function
$q_0(\phi)$	Dimensionless impulsive pressure function
$q'_0(z)$	Dimensionless impulsive pressure function
q_r	Maximum shear force per unit circumferential length
q_u	Equivalent uniform pressure distribution
r	Radius of circular part of tank base that does not uplift
t	Tank wall thickness
t	Time dependant variable
t_b	Thickness of base plate
$\bar{u}$	Wave particle velocity
$\lvert u \rvert_{max}$	Maximum displacement at top of free edge of a cylindrical tank from overturning effects
v	Uplift displacement of tank wall/base
v_0	Peak ground velocity
v_s	Shear wave velocity of soil
$\ddot{x}(t)$	Seismic horizontal ground acceleration
w	Rectangular tank wall deflection
w_t	Weight of walls and roof at base of wall, per unit circumferential length
z	Vertical ordinate
$\ddot{z}(t)$	Seismic vertical ground acceleration
α_θ	Dimensionless factor to convert foundation rotational static stiffness to dynamic stiffness
α_i	Period of vibration coefficient for rectangular tank
α_v	Dimensionless factor to convert foundation vertical static stiffness to dynamic stiffness
α_x	Dimensionless factor to convert foundation horizontal translation static stiffness to dynamic stiffness
β_1	Modal participation factor for lateral tank displacement

β_θ	Dimensionless factor for damping in foundation rocking mode
β_x	Dimensionless factor for damping in foundation horizontal translation mode
ϕ	Cylindrical coordinate for horizontal circular tank
ϕ	Strength reduction factor for material
λ_i	Period of vibration coefficient for vertical circular cylindrical tank
σ_x	Increment in at-rest soil pressure due to earthquake
μ	Displacement ductility factor
ν	Poisson's ratio
ν_s	Poisson's ratio for soil
ρ_s	Mass density of soil
γ_ℓ	Unit weight of liquid
γ_s	Unit weight of soil
θ	Cylindrical coordinate for vertical circular tank
θ_p	Plastic rotation
θ^*	Half angle of arc of the shell base in contact with foundation for an uplifting base
δ	Amplitude of tank wall imperfections
ξ	Viscous damping level of elastic system
ξ_f	Effective damping ratio for the flexible mass impulsive mode
ξ_h	Equivalent additional viscous damping to represent hysteretic energy dissipation
ξ_i	Damping level appropriate to mode i
ξ_m	Material damping in the tank structural system for impulsive mode
ξ_s	Soil damping ratio including material and radiation damping
ξ_v	Damping level for vertical mode

All equations set out using SI units.

Section 1 - Background to the Recommendations

1.1 2009 Revision

The 2009 revision of this document introduces several significant changes to the original 1986 version of the document, including the following:

- The load levels have been made consistent with NZS 1170.5:2004: Structural Design Actions (SNZ, 2004b), with suitable adjustments for ductility and damping applicable to tanks.
- Some ductility is permitted for certain types of tank, for example a ductility factor of up to 2.0 is permitted for unanchored steel tanks. A capacity design approach needs to be used to protect these tanks against failure while yielding occurs in the chosen mechanism.
- The seismic return period factor, R_u, is derived based on a more detailed treatment of consequences of failure including life safety, environmental exposure, community significance and design life.
- The Structural Performance Factor, Sp (used in NZS 1170.5), is taken as 1.0 for tanks.
- The document has been reorganised for easier use.
- Guidance has been given to allow interfacing with other international load standards.
- Equation 2.7 of the original 1986 version has been amended as shown in Equation 3.16.

1.2 Legislation for Storage of Hazardous Substances

This document has been prepared with the intention that it is recognised under New Zealand legislation and regulatory controls as an approved code of practice by the Hazardous Substances and New Organisms (HSNO) Act 1996 (ERMA New Zealand, 1996), and associated Regulations.

The HSNO Act has replaced the previous Dangerous Goods Act and Regulations and sets out the regulatory controls required in New Zealand for the safe storage of hazardous substances.

The process for development and approval of HSNO codes of practice is set out in the Environmental Risk Management Authority (ERMA) information Sheet 13 dated May 2001.

1.3 Damage to Storage Tanks in Earthquakes

The seismic performance of storage tanks is a matter of special importance, extending beyond the economic value of the tanks and contents. Without an assured water supply, uncontrolled fires subsequent to a major earthquake may cause substantially more damage than the earthquake itself, as occurred in the great 1906 San Francisco earthquake. Safe supplies of drinking water are also essential immediately following destructive earthquakes to avoid outbreaks of disease. Consequently, water supply reservoirs must remain functional after earthquakes. Failure of tanks containing highly inflammable petroleum products has frequently lead to extensive uncontrolled fires, as occurred, for example, following the Niigata and Alaska earthquakes of 1964. Spillage of toxic chemicals or liquefied gases from damaged tanks could release dangerous liquids or gas clouds with disastrous effects in populous areas, or to sensitive environmental areas.

Serious damage has been caused to both steel and concrete tanks in recent earthquakes. Some of the more common types of failure or damage are briefly reviewed below.

1.3.1 Shell Buckling of Steel Tanks

A common form of damage involves outward buckling of the bottom shell courses, a phenomenon often termed 'elephant's foot' buckling. Figure 1-1 shows a typical example which occurred in the 1989 Loma Prieta earthquake and Figure 1-2 shows an example which occurred in the 1992 Landers earthquake, (Cooper and NIST:BFRL, 1997). Typically, the buckling extends over a large proportion of the diameter of the tank. Buckling of the lower strakes has often, though not always, led to loss of tank contents due to weld or piping fracture, and in some cases has caused total collapse of the tank.

Although damage has been more frequent for full tanks of high aspect ratio (height/diameter), squat tanks, and partially filled tanks have also suffered severe damage. It is difficult to model accurately the 'elephant's foot' buckling behaviour in dynamic shaking table tests. It is possible that vertical accelerations occurring in earthquakes, and typically not modelled in shaking table tests, have an important role in this type of failure.

Tanks with very thin shells, such as the stainless steel shells common for wine, beer and milk storage tanks, have displayed a different form of buckling, involving diamond-shaped buckles a distance above the base of the tank. An example of this form of buckling to stainless steel wine tanks is shown in Figure 1-3, (Moore, 1981).

Recent experience in New Zealand indicates that a wall to base detail commonly used with thin stainless steel tanks is susceptible to damage. The detail uses a radiused joint between the floor and base of the wall (termed a "knuckle joint") that can locally swash due to bending induced by large vertical forces in the walls. This mode of damage can occur in unanchored and anchored tanks where uplift only anchorage has been provided. Figure 1-4 shows an example of knuckle swash from the 2007 Gisborne earthquake.

1.3.2 Roof Damage to Steel Tanks

During earthquake excitation, a sloshing motion of the tank contents is set up, with an amplitude of motion at the tank circumference which has been estimated, on the basis of scratch marks produced by floating roofs to have exceeded several metres in some cases. For near-full tanks, resistance of the roof to the free sloshing results in an upwards pressure distribution on the roof. This has frequently been sufficient to cause failure of the frangible joint between walls and the conical roof. Accompanying spillage of contents over the top of the wall, and sometimes extensive buckling of the upper courses of the shell walls results. Floating roofs have frequently sustained extensive damage to support guides from sloshing of contents. Figure 1-5, (Cooper and NIST:BFRL, 1997), shows an example of roof/shell buckling that resulted in a small loss of contents during the 1952 Kern County earthquake.

1.3.3 Failure of Tanks Support Systems

Steel and concrete storage tanks supported above grade by columns or frames have failed due to the inadequacy of the support system under lateral seismic forces resulting from response of the tank and contents to horizontal ground accelerations under earthquake attack. Figure 1-6, (Rai, 2003), shows this type of failure of an elevated concrete water reservoir damaged in the 2001 Bhuj earthquake. Figure 1-7, (Macelwane, 1933), shows the failure of the support frame of an elevated steel water reservoir damaged in the 1933 Long Beach earthquake.

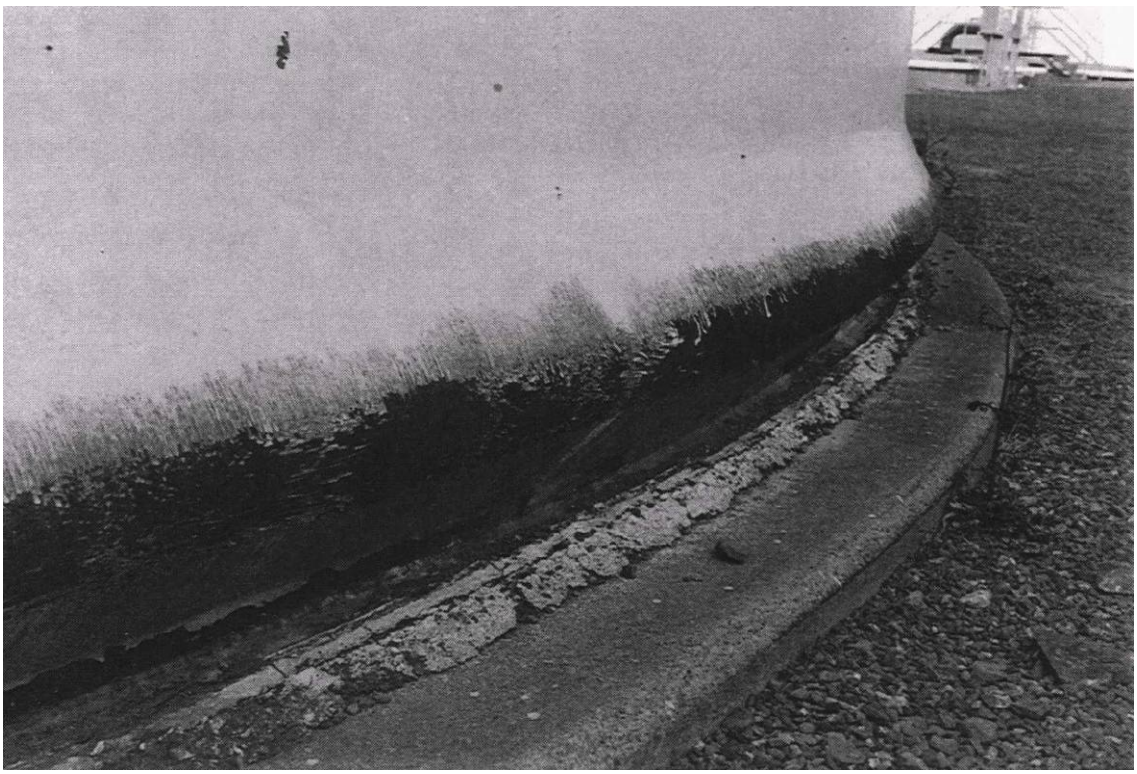


**Figure 1-1: 'Elephant's Foot' Buckling,
1989 Loma Prieta (Cooper and NIST:BRFL, 1997)**

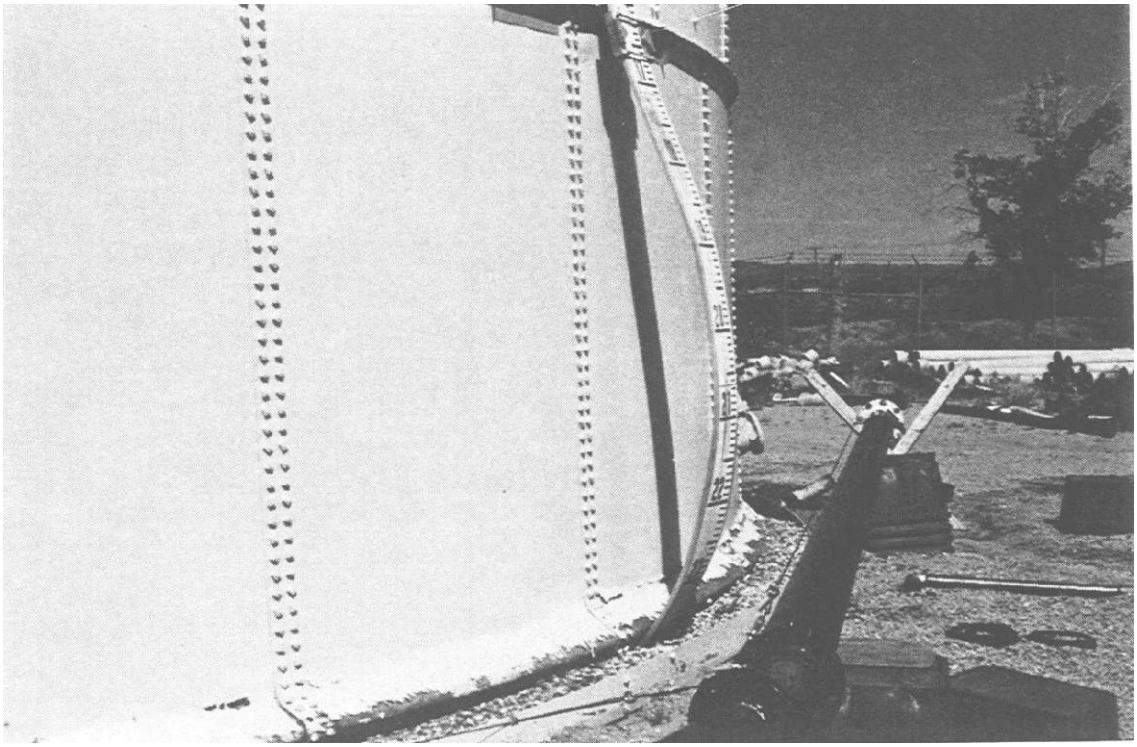


**Figure 1-2: 'Elephant's Foot' Buckling, 1992 Landers
(Cooper and NIST:BRFL, 1997)**

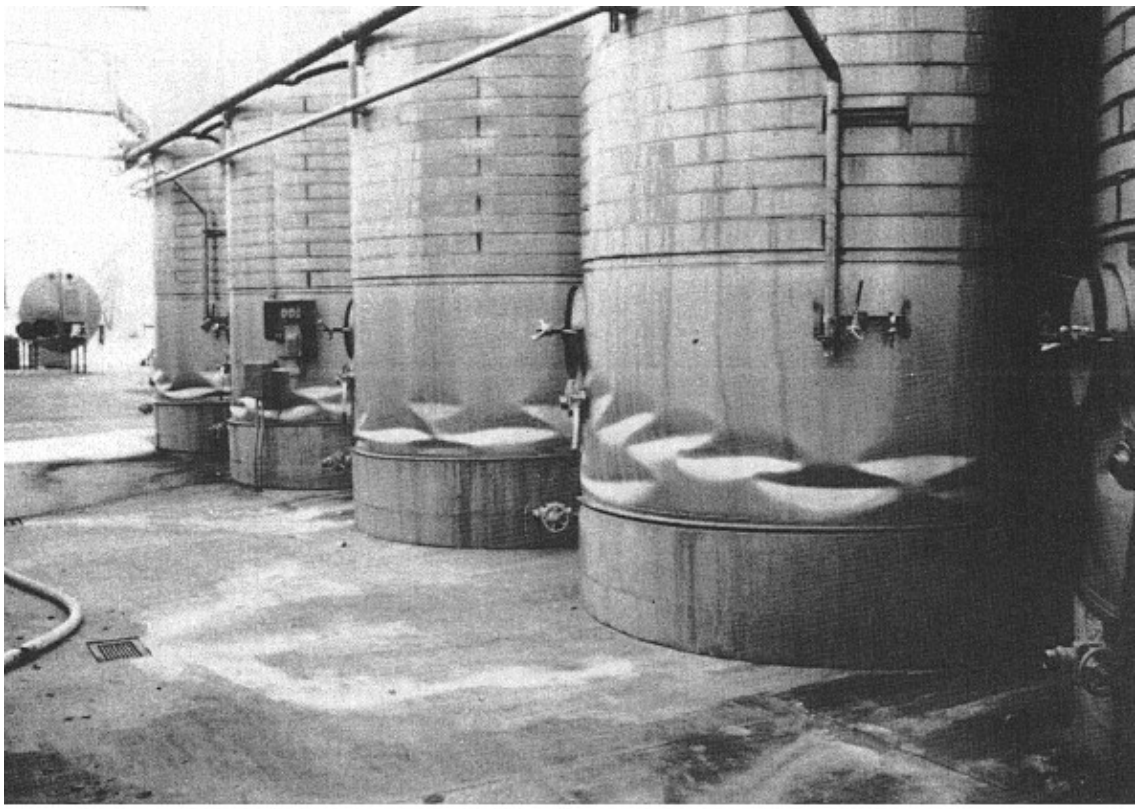


Figure 1-3: 'Diamond' Buckling of Stainless Steel Wine Tanks (Moore, 1981)



Figure 1-4: "Knuckle Squash" of Stainless Steel Tanks, 2007 Gisborne

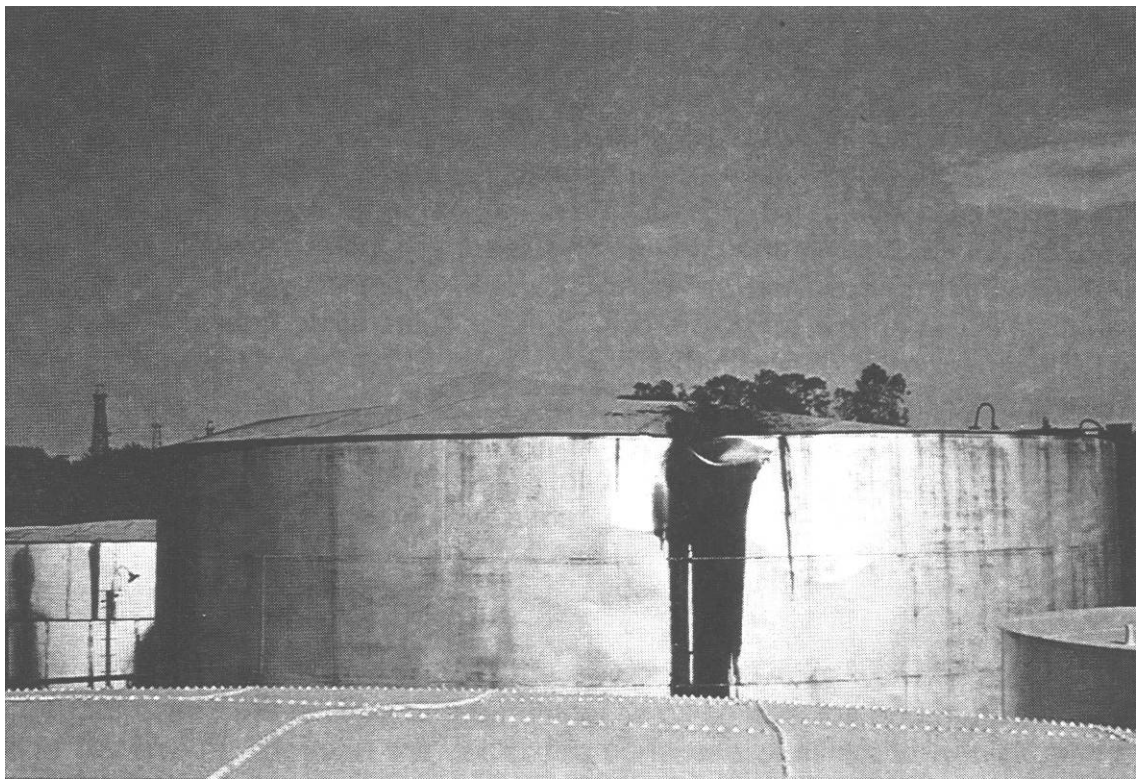


Figure 1-5: Roof/Shell Buckling, 1952 Kern County (Cooper and NIST:BRFL, 1997)



Figure 1-6: Damaged Support to Elevated Concrete Water Reservoir, 2001 Bhuj (Rai, 2003)



Figure 1-7: Support Frame Collapse of Elevated Steel Water Reservoir, 1933 Long Beach (Macelwane, 1933)

1.3.4 Foundation Failures

Tank storage farms have frequently been sited in areas with poor foundation conditions. An example is the use of sites close to harbours or rivers which consist of cohesionless materials with liquefaction potential. In many earthquakes liquefaction of materials under tanks, coupled with seismic moments on the tank base from lateral accelerations, has resulted in base rotation and gross settlements of the order of several metres. Figure 1-8, (Boulanger, 1995), shows a storage tank that tilted during the 1995 Kobe earthquake.

On firm foundations, fracture of steel wall/base-plate welds has occurred in tanks not restrained, or inadequately restrained against uplift. In these cases the seismic accelerations have resulted in large uplift displacements (up to 350 mm was recorded in the moderate San Fernando earthquake) on the tension side of the tank under overturning moments induced by seismic response. Since the base is held down by hydrostatic pressure of the tank contents, the base weld is subject to high stress, and fracture may result. In some cases the resulting loss of liquid (water or petroleum products) resulted in scouring of the foundation materials in the vicinity, reducing support to the tank in the damaged area, and exacerbating the damage. Figure 1-9, (Cooper and NIST:BFRL, 1997), shows separation of the tank floor and shell at the base weld and the resultant scouring due to loss of contents during the 1971 San Fernando earthquake.

1.3.5 Connecting Piping

One of the most common causes of loss of tank contents in earthquakes has been fracture of piping at the connection to the tank. This generally results from large vertical displacements of the tank as a result of buckling, wall uplift, or foundation failure. Failure of rigid piping connecting adjacent tanks has also resulted from relative horizontal displacements of the tanks. These failures would be easily avoided by designing flexibility into the piping system, and ensuring brittle elements such as valves have sufficient strength to ensure damage occurs in ductile elements. Figure 1-10, (Cooper and NIST:BFRL, 1997), shows a rigid pipe connection that has sustained damage and separated at the coupling during the 1992 Landers earthquake.



Figure 1-8: Settlement Initiated Rotation of Storage Tank, 1995 Kobe (Baulanger, 1995)

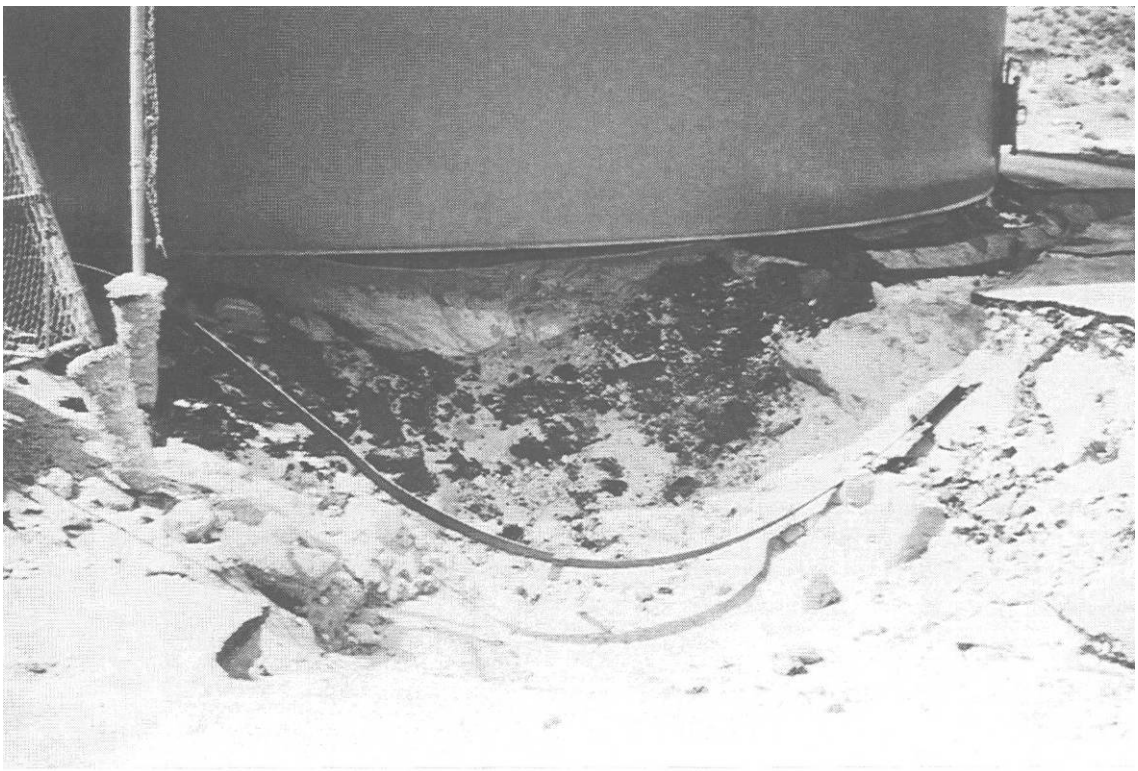


Figure 1-9: Separation of Tank Shell/Floor Weld, 1971 San Fernando (Cooper and NIST:BRFL, 1997)

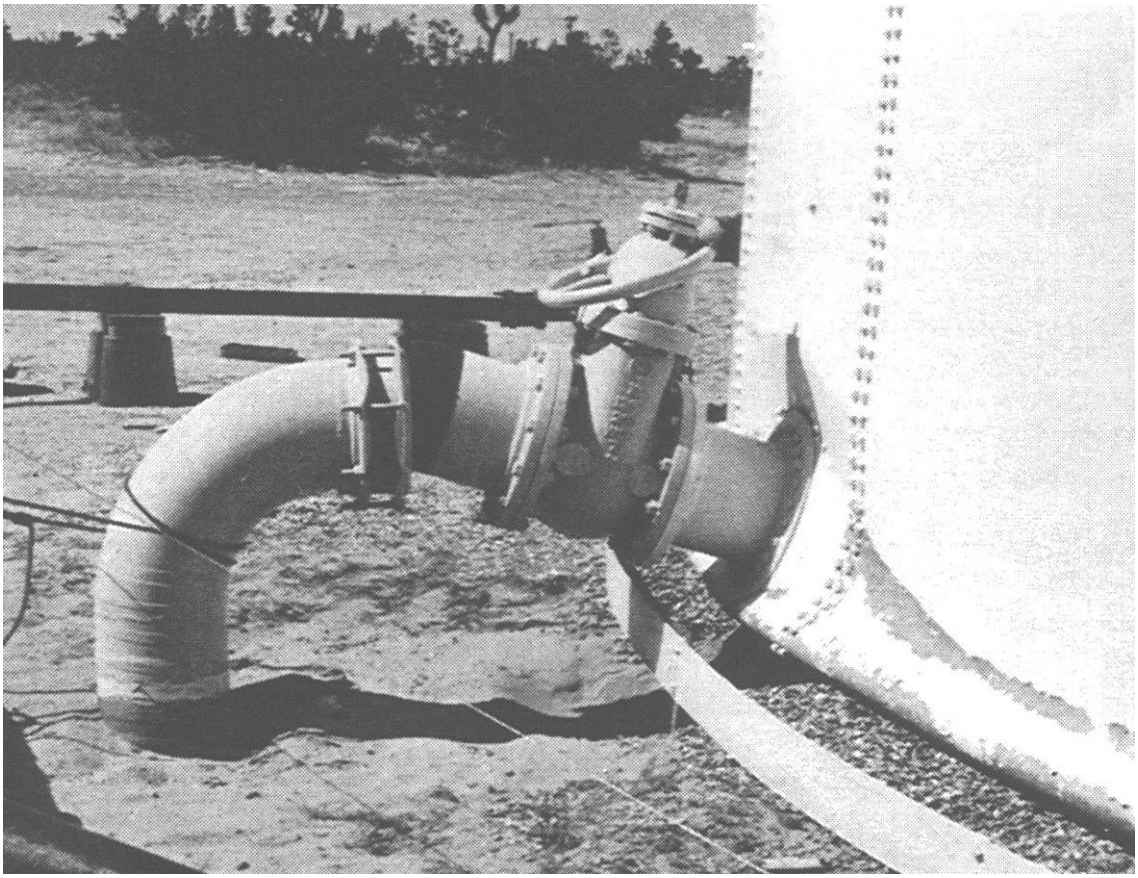


Figure 1-10: Rigid Pipe Connection, 1992 Landers (Cooper and NIST:BRFL, 1997)

1.4 A Simplified Description of Seismic Response of Storage Tanks

Much of the damage described above has occurred because of inadequate understanding of the basic dynamics of fluids in tanks at the time of designing the tanks. A brief qualitative description is given in this section, as a forerunner of the more detailed, and quantitative data included in the accompanying sections.

Consider the elevation of a ground supported tank shown in Figure 1-11(a). Under lateral accelerations, fluid in the upper regions of the tank tends not to displace laterally with the tank walls. The resulting incompatibility in the region of the tank walls is resolved by vertical displacement of the fluid, generating seismic waves, or "convective" mode behaviour (sloshing). Typical periods of the convective mode behaviour are very long (in the range of 6-10 seconds) for large tanks, and as such are more influenced by the level of seismic ground displacements rather than ground accelerations. Nearer the tank base, the fluid is constrained to displace with the tank, and acts as an added mass to the inertial mass of the tank structure, generating an "impulsive" mode behaviour. The proportion of tank contents that act in a convective fashion decreases as tank aspect ratio (height/diameter) increases, with the impulsive mode becoming more dominant. Conversely, for tanks of very low aspect ratio only about 30% of tank contents acts with the walls, with the remainder responding in various convective modes.

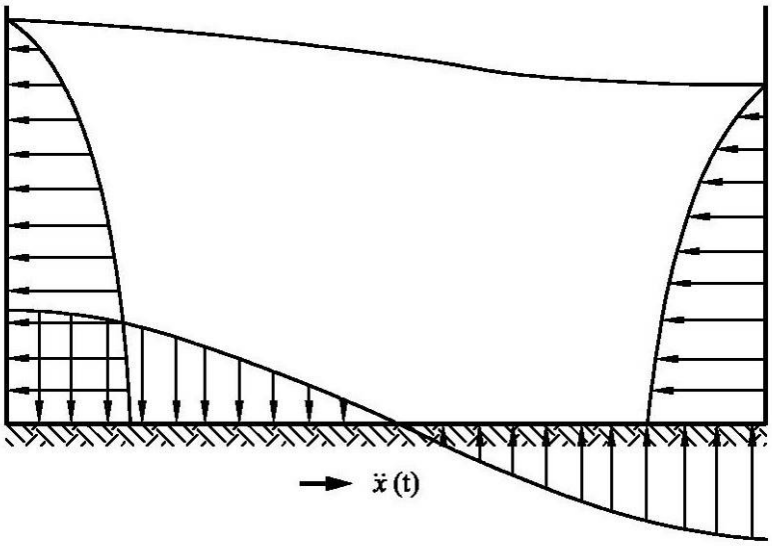
Note that for tanks which are filled to the top and sealed (e.g. commonly the case for wine tanks), the convective mode cannot develop and all contents should be assumed to participate in the impulsive mode. The height at which the impulsive mass acts needs to be increased accordingly to match.

Work by Graham and Rodriguez 1952, Housner 1963, and Jacobsen 1949, has resulted in the simple mechanical analogy of response illustrated in Figure 1-11(b). The impulsive mass m_o is rigidly linked to the walls, whereas the convective mass, divided into a series of sub-masses m_1 ... m_i, is connected to the tank walls by springs of stiffness k_1 ... k_i. The flexibility of these springs attached to masses represents the various antisymmetric slosh frequencies of the fluid in the tank. In practice, seismic forces on the tank walls can generally be found with adequate accuracy from the response of the impulsive mass m_o and the first convective mode (mass m_1) with higher modes typically contributing less than 5% of the total. However, to estimate fluid displacements, it has been shown that higher modes should be considered.

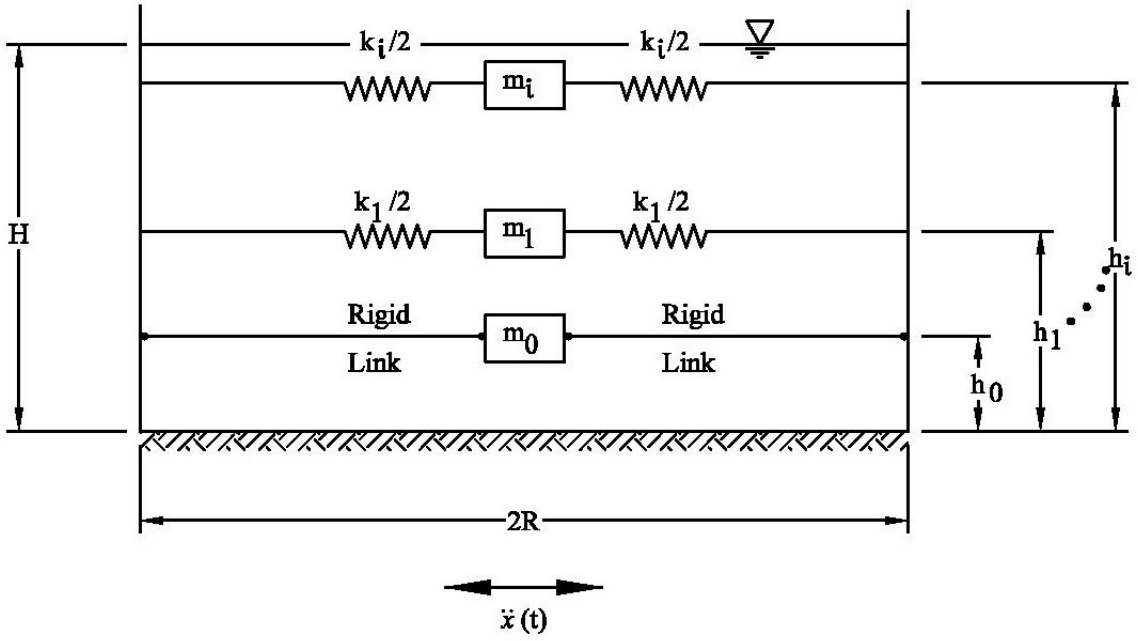
In applying the mechanical analogue of Figure 1-11(b) to the response of elevated tanks it is necessary to include terms representing the overturning effects of pressure changes on the tank base induced by sloshing of the tank contents. It must also be realised that the flexibility of the support structure of an elevated tank will influence the response of the equivalent impulsive mass to the ground accelerations. This influence, for either elastic or ductile support structures, can be calculated using the normal methods of structural analysis adopted for structural response to earthquakes.

For concrete tanks the walls are stiffer than those for a comparable steel tank, however they should not be considered to be fully rigid. Thus for concrete tanks, as for steel tanks, the accelerations of the impulsive mass are determined at the impulsive period and will likely be on the plateau of the hazard spectrum of NZS 1170.5:2004 (SNZ, 2004b).

Should there be any doubt as to the rigidity of a tank, use of the flexible tank design method will be conservative for vertical circular cylindrical and rectangular tanks.



(a) Hydrodynamic Pressure Induced by Ground Accelerations



(b) Mechanical Analogue for Response of Contents of a Rigid Tank

Figure 1-11: Seismic Response of a Ground Supported Tank

Some uncertainty still exists about the response of ground supported or elevated tanks of complex shape. It is possible, however, on the basis of an understanding of the response of simpler shapes to make conservative but realistic recommendations for these more complex cases.

The influence of viscosity of tank contents has been subject to only limited investigation. However, for most fluids, damping resulting from viscosity is minimal, and response is essentially the same as for the inviscid solution generally considered in developing solutions for the mechanical analogue. Shake-table testing of 1 m diameter models with fluids up to 2000 times more viscous than water (furnace oil at 20 ºC) indicated little change in overall response, though amplitudes of sloshing were slightly reduced in comparison with tanks containing water. Since the influence of viscosity is primarily a boundary-layer

phenomenon, it can be shown that the influence will decrease as the tank diameter increases. Consequently, results on the small-scale models tested so far exaggerate effects of viscosity. Tanks containing granular material may behave differently to tanks containing fluid. Fine granular materials such as cement, grains etc may be expected to have some limited convective response, but coarse materials, such as coal presumably respond largely in an inertial mode. Typically tanks containing granular materials will have comparatively high aspect ratios (height/radius > 2), and for these tanks behaviour may be approximated to that of a similar tank containing fluid of the same density as the granular material, since fluid response of such tanks is dominated by the impulsive effects.

1.5 Discussion of Existing Codes

Three codes have commonly been considered, under different conditions, for the seismic design of storage tanks in New Zealand. These are Appendix E of the American Petroleum Institute Standard API-650, for which the 11th Edition Addendum 1 is the current edition (API, 2008), a Ministry of Works and Development document, Seismic Design of Petrochemical Plants, (MWD-SDPP, 1981), and the New Zealand Loadings Code NZS 4203 (SNZ, 1984 and 1992 – both now withdrawn). Another standard of relevance is the American Waterworks Association document ANSI/AWWAD100-96 (AWWA, 1996).

API-650-E has been widely used for design of petroleum storage tanks in New Zealand. This document is based on Housner's representation of the mechanical analogue, and seismicity levels appropriate to the USA. The 2006 update to Appendix E contained within Addendum 4 of the Tenth Edition (API, 2006) contains methods for determining the spectral response for regions outside the USA. This would allow use of AS/NZS 1170 with appropriate effects, for site amplification, importance factor, and damping for impulsive and convective modes. However spectra for the convective mode, (0.5% damping), is not directly accessible and the building importance levels are not applicable to tanks.

Many tanks for New Zealand's major projects completed during the 1980s were designed to the MWD-SDPP document. This document is now well out of date and should no longer be used.

The Loadings Code NZS 4203 (now withdrawn) specifically excluded storage tanks from its scope, and did not deal adequately with the special behaviour of contained liquids, long convective periods, damping levels or appropriate ductility levels for tanks. Since 1986 when this document was first published, the Loadings Code NZS 4203 has been through a complete revision published in 1992 (SNZ, 1992) and is now superseded by another which was completed in 2004 and titled AS/NZS 1170: Structural Design Actions (SNZ, 2004a). The basis of the seismic load derivation from seismic hazard analyses is now well entrenched into the code and forms a suitable basis for the derivation of seismic spectra for tanks. Appropriate adjustment to the NZS 1170.5:2004 spectra are proposed in this document to account for the different performance characteristics, damping levels and ductility factors for tanks.

NZS 3106:1986 (SNZ, 1986 – now withdrawn) gives requirements for design of concrete retaining structures. It includes requirements for seismic design loads which are independent of the Loadings Code (as this Standard was prepared before the NZS 4203 Loadings Code seismic loads were derived). The use of working stress design in NZS 3106:1986 is not consistent with the use of limit states in the current loading codes. This standard is currently being updated and revised.

None of the above codes provide information relevant to the special problems of elevated tanks or tanks of unusual shape or excessive flexibility. Foundation problems are generally inadequately dealt with, and the overall issues of seismic performance criteria and probability of failure are treated in widely different ways.

The 1986 version of this document (commonly known as the "Red Book") has been widely considered in the industry for design of steel bulk storage tanks. A significant feature of the 1986 document was that the design procedure required the tank to remain fully elastic with no allowance for any force reduction due to any form of energy dissipation.

1.6 2009 Recommendations

The recommendations in this revision are based on the following:

- Seismic design spectra from NZS 1170.5:2004
- Modifications to the NZS 1170.5 seismic loads are included for the following:
  - (a) Damping levels, which are different from the normal assumption of 5% for buildings and other civil engineering structures
  - (b) Return period factors to reflect the wide range of life-safety, property and environmental exposures, plus the community significance
  - (c) Ductility factors for tanks as a representative formulation of force reduction appropriate to the response of the tank type
- The analysis and design recommendations remain essentially unchanged from the 1986 recommendations except where errors or omissions have been corrected.

These recommendations have been prepared in the form of Code and Commentary Sections. It is anticipated that this document will be recognised as an Approved Code of Practice by the New Zealand Environmental Risk Management Authority, ERMA.

1.7 Content of Recommendations

The recommendations attempt to produce a unified approach for the seismic design of storage tanks, regardless of material or function, and to provide additional information to that already available in alternative sources. The information relates only to the storage tanks themselves, and their immediate fixings to pipelines, but not to pipeline design itself. Useful design information for pipelines is available in publications by ASCE (1984) and American Lifelines Alliance (2005).

Section 2 defines general design principles, including the level of earthquake to be designed for, and the performance criteria under the design level earthquake.

Section 3, Design Loading, defines the loading corresponding to the design level earthquake including design spectra, inertia and hydrodynamic masses, and pressure distributions for a variety of tank geometries.

Section 4, Design Actions, presents analytical methods for calculating stresses and displacements induced by the design loading.

Section 5, Design Criteria, presents acceptable limits for the design actions calculated in accordance with Section 4.

Section 6, Foundations, discusses problems associated with foundation investigation, and soil structure interaction.

Section 7, Design Details, discusses a number of design details which are likely to improve seismic performance of tanks.

Each section has an accompanying commentary and there are design examples in the appendix.

1.8 Areas Where Further Research is Needed

The following areas identified by the original 1986 publication as needing further research, remain as areas that are not well understood.

- Uplift and buckling characteristics of steel tanks. Although some work has been carried out in this area, more is required to fully understand the mechanisms involved in uplift and buckling, and the resulting modification to response.
- Earthquake spectral characteristics for the very long periods that correspond to tank sloshing modes.
- Interaction between sloshing displacements and roofs. Experimental verification of pressures that can be exerted on roofs by sloshing fluid is needed.
- Bund design. Confidence in bund design for larger-than-design level earthquakes must be high. At present this area has received comparatively little attention.
- Response of other than circular or rectangular vertical-axis tanks. Although some solutions for forces and pressure distributions are available for other tank shapes, they are limited in scope and require experimental verification.
- Response of tanks containing granular materials. Little information is available.
- Behaviour of elevated tanks on ductile supports. The influence of ductility of the support system on the inertia, and particularly convective mode response require theoretical and experimental studies.
- Influence of wall flexibility on the response of rectangular tanks. Although approximate methods are available to allow the influence of wall flexibility on impulsive behaviour to be assessed, more exact methods are needed.

1.9 References

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Recommendations

Section 2 - General Design Principles

2.1 Scope

This section defines the general principles to be applied in the seismic design of storage tanks. Performance criteria under seismic loading are defined.

2.2 Performance Based Determination of Earthquake Actions

The New Zealand Building Act (DBH, 2004), and the HSNO Act 1996 (ERMA New Zealand, 1996), are the governing legislation for the design and construction of storage tanks in New Zealand. The New Zealand Building Act requires compliance with the New Zealand Building Code (BIA, 1992) which is a performance-based code. The overall performance objectives of these recommendations have been established in relation to the New Zealand Building Code and are as follows:

- To safeguard people from injury or loss of amenity caused by structural failure, or spillage of contents.
- To protect the environment and other property from harmful effects caused by spillage of contents.

These objectives are achieved by ensuring that the design level forces have an acceptably low probability of being exceeded during the design life of the tank. The design level forces are determined using the New Zealand Loadings Standard AS/NZS 1170: Structural Design Actions (SNZ, 2004) with appropriate modification factors for use in the seismic design of storage tanks. The overall performance objectives can be quantified by the following performance criteria:

- Under the design level earthquake (ultimate limit state), the tank and any associated bunding shall remain functional.
- A designed hierarchy of failure shall be established to minimise damage and potential for loss of tank contents under earthquakes which cause ground shaking more severe than that corresponding to the design level earthquake.
- Tanks containing hazardous substances (toxic, flammable or explosive) shall be designed to meet the requirements of the HSNO Regulations (ERMA New Zealand, 2004)

2.3 References

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Section 3 - Design Loading

3.1 Scope

This section defines the loading on the tank shell and foundation corresponding to the design level earthquake. The loading is a function of the mass of the tank, impulsive and convective masses representing the fluid, and earthquake accelerations derived from a response spectrum.

In calculating the design earthquake loading, the effects of horizontal and vertical ground acceleration, and the effects of tank and foundation flexibility shall be considered.

3.2 Earthquake Actions

The horizontal seismic shear acting at the base of the tank, associated with a particular mode of vibration, shall be calculated from the expression:

$$V_i = C_d(T_i) m_i g \quad (3.1)$$

where $C_d(T_i) = C(T_i) k_f(\mu,\xi_i) S_p$
but not less than $C(0)$ when $T_i < 0.25$ seconds (3.2)

$$C(T_i) = C_h(T_i) Z R_u N(T_i,D) \quad (3.3)$$

and

V_i = horizontal base shear associated with mode i (impulsive, convective etc).

m_i = equivalent mass of tank and contents responding in particular mode of vibration considered.

$C_d(T_i)$ = horizontal design action coefficient for mode i.

$C(T_i)$ = ordinate of the elastic site hazard spectrum for horizontal loading for the site subsoil type, and the relevant mode. Obtained from NZS 1170.5 (SNZ, 2004b) with the tank importance level as determined by Table 3.3.

$k_f(\mu,\xi_i)$ = correction factor for NZS 1170.5 elastic site hazard spectrum to account for ductility and level of damping, refer Table 3.2.

S_p = structural performance factor, to be taken as 1.0.

$C(0)$ = the site hazard coefficient for T=0 using the modal response spectrum from NZS 1170.5.

$C_h(T_i)$ = spectral shape factor for the site subsoil type and the relevant mode, from Figure 3.1 of NZS 1170.5:2004.

Z = seismic zone hazard factor, from NZS 1170.5.

R_u = return period factor for the ultimate limit state, from AS/NZS 1170 (SNZ, 2004a) with tank importance level as determined by Table 3.3. See Commentary Table C3.1.

$N(T_i,D)$ = near fault factor, from NZS 1170.5.

μ = displacement ductility factor for horizontal impulsive modes, refer Table 3.1.
Taken as 1.0 for the convective and vertical modes.
Taken as 1.25 for the impulsive mode when calculating displacements.

T_i = period of vibration of appropriate mode of response.

ξ_i = damping level appropriate to mode of response, refer Figures 3.1 and 3.2 for impulsive mode. For design purposes damping of the horizontal impulsive mode for H/R ratios greater than 3.0 can be taken as the H/R = 3.0 value. For convective mode, shall be taken as 0.5%.

For calculating the vertical seismic force acting on the tank, $C_d(T_i)$ shall be scaled by 0.7.

For periods greater than 3 seconds the equations 3.4 to 3.7 apply to the value of spectral shape factor $C_h(T_i)$ for the site subsoil types as defined by NZS 1170.5.

Class A Strong Rock and Class B Rock

$T_i > 3$ sec

$$C_h(T_i) = 3.15/T_i^2 \quad (3.4)$$

Class C Shallow Soil

$T_i > 3$ sec

$$C_h(T_i) = 3.96/T_i^2 \quad (3.5)$$

Class D Deep or Soft Soil

$T_i > 3$ sec

$$C_h(T_i) = 6.42/T_i^2 \quad (3.6)$$

Class E Very Soft Soil

$T_i > 3$ sec

$$C_h(T_i) = 9.96/T_i^2 \quad (3.7)$$

The Importance Level of the tank shall be determined from the most severe consequence of failure from Tables 3.4 to 3.7 for each of the following risks.

- life safety
- environmental exposure
- national or community significance
- adjacent property value

The default design life for new installations shall be 50 years. For tanks of importance levels 1 and 2 only, a shorter design life may be acceptable, and the design loadings may be adjusted accordingly based on an equal probability of exceedence of design loadings as indicated in AS/NZS 1170. Where a design life of less than 50 years is assumed, the tank shall be de-commissioned on the expiry of that design life.

Table 3.1 Displacement Ductility Factor, μ, for horizontal impulsive modes

Type of Tank	Ductility Factor μ[1]
Steel Tanks on Grade	
Elastically responding	1.25
Unanchored tank with limited ductile behaviour (e.g. elephant's foot shell buckling or uplift)	2.0 [2][4]
Unanchored tank elastically designed or with a non-ductile mechanism (e.g. elastic, diamond shaped shell buckling)	1.25
Anchored with non-ductile holding down bolts	1.25
Anchored with ductile tensile yielding holding down bolts	2.0 [3][4]
Ductile skirt pedestal	2.0 [3][4]
On concrete base pad designed for rocking	2.0 [3][4]
Concrete Tanks on Grade (Reinforced or prestressed)	1.25
Tanks of other materials on Grade	
Timber Stave	1.25
Non-ductile materials (e.g. fibreglass)	1.25
ductile materials and failure mechanisms	2.0[4]
Elevated Tanks	As appropriate for support structure[3][4]

Notes:
1. The ductility factor is used to determine the force reduction appropriate to the response of the type of tank.
2. Unanchored steel tanks designed assuming an elephant's foot buckling mechanism may occur under seismic overload also need to be checked for the elastic response loading to ensure that elastic (diamond) buckling of the shell does not occur first.
3. A capacity design approach shall be used to protect against other modes of yielding or failure. Elastic diamond-shaped and elephant's foot buckling mechanisms shall be checked against at the ductility factors for the unanchored tank, except where elephant's foot buckling is unacceptable and a ductility factor of 1.25 is required.
4. A ductility factor of 1.25 shall be used when calculating displacements.

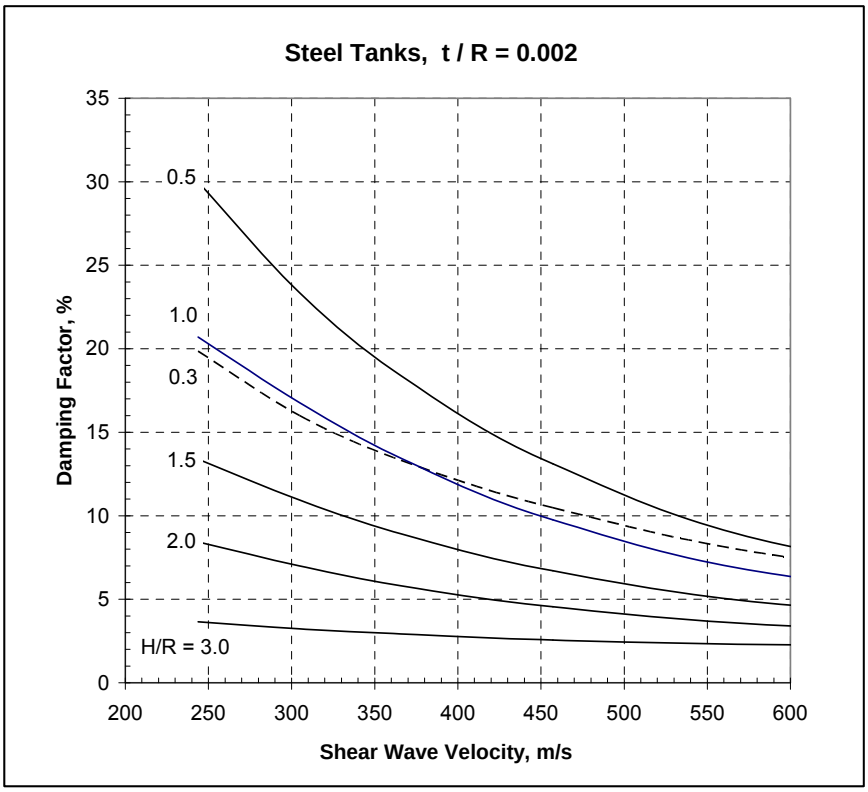


Figure 3.1(a): Damping for Horizontal Impulsive Mode Steel Tanks t/R = 0.002 (Veletsos and Shivakumar, 1997)

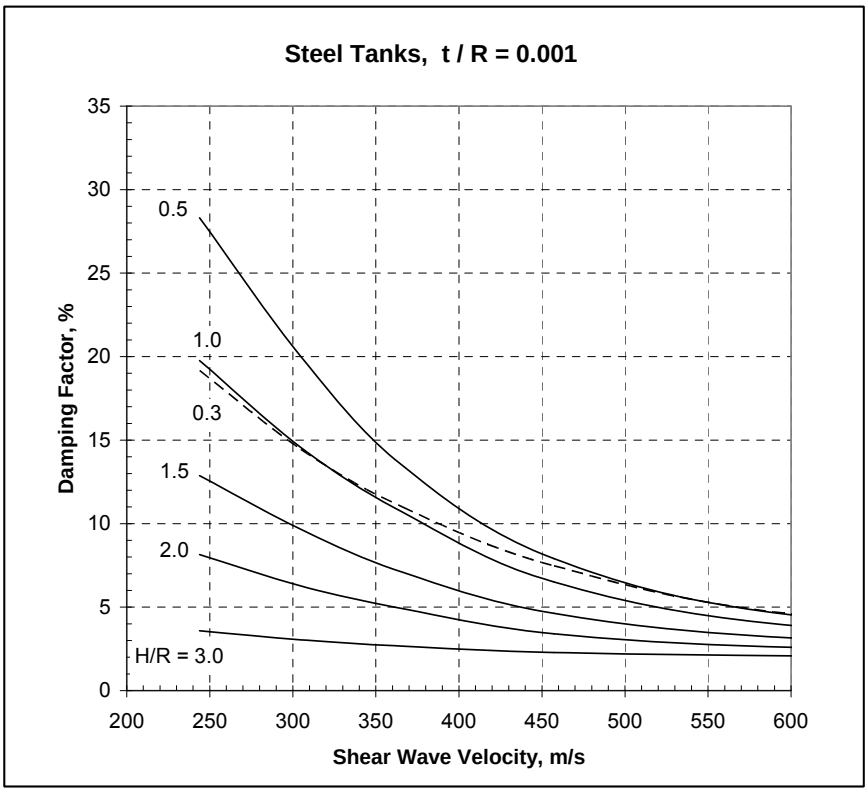


Figure 3.1(b): Damping for Horizontal Impulsive Mode Steel Tanks t/R = 0.001 (Veletsos and Shivakumar, 1997)

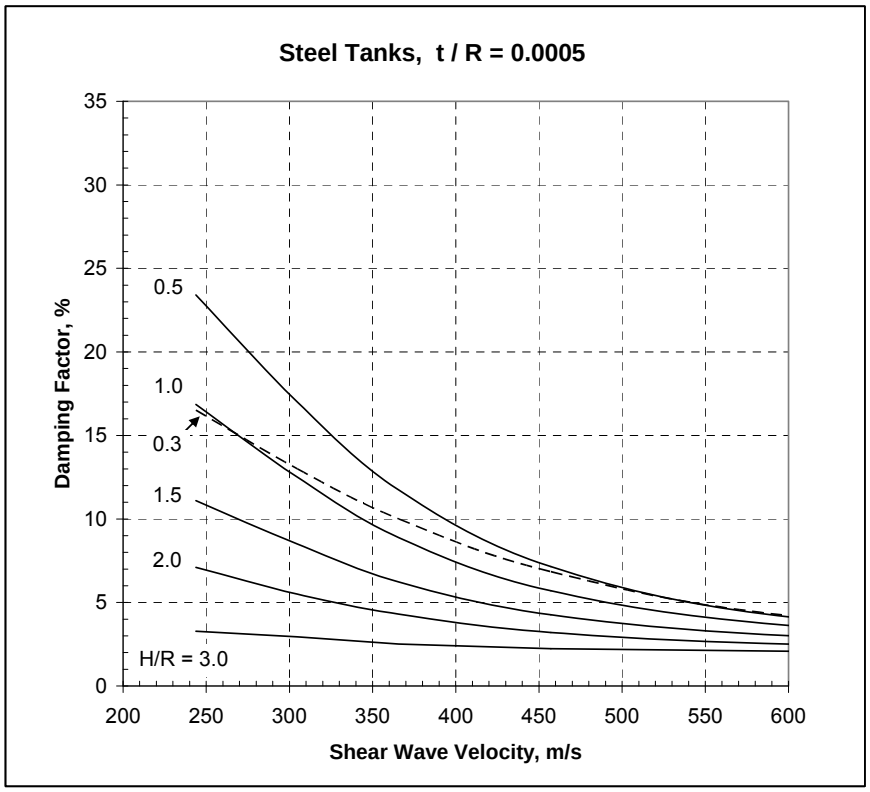


Figure 3.1(c): Damping for Horizontal Impulsive Mode Steel Tanks t/R = 0.0005 (Veletsos and Shivakumar, 1997)

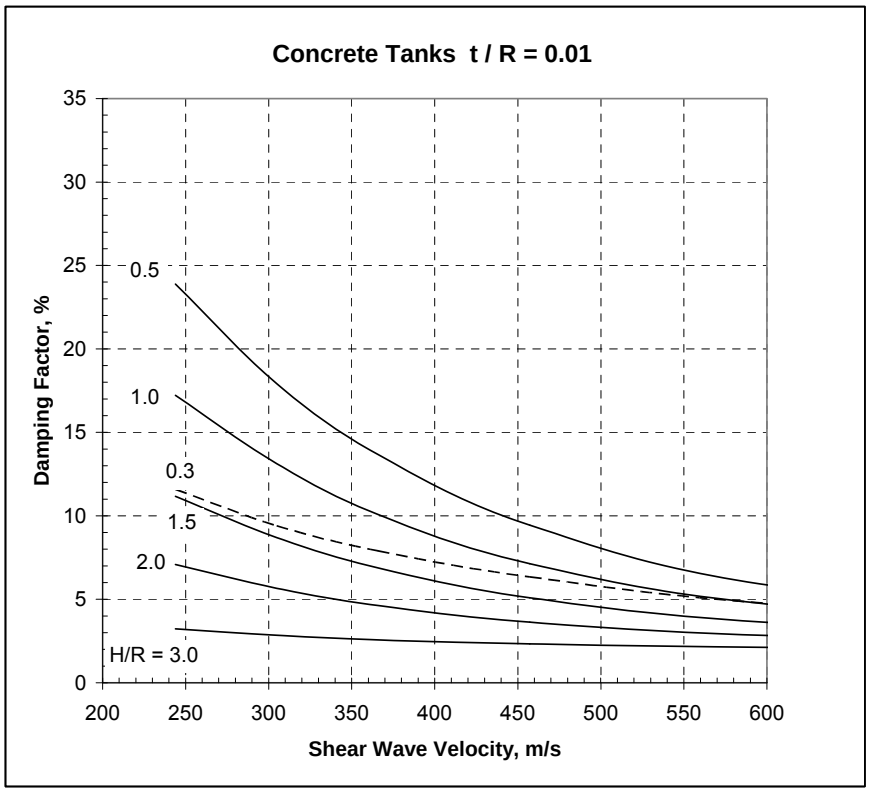


Figure 3.1(d): Damping for Horizontal Impulsive Mode Concrete Tanks t/R = 0.01 (Veletsos and Shivakumar, 1997)

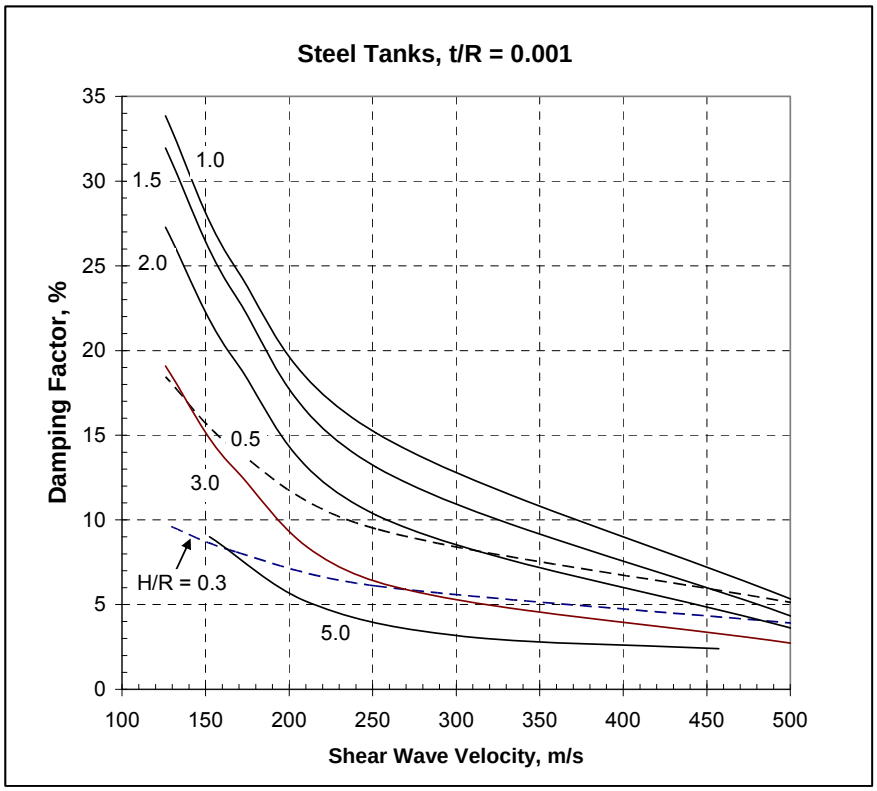


Figure 3.2(a): Damping for Vertical Impulsive Mode Steel Tanks t/R = 0.001 (Veletsos and Tang, 1986)

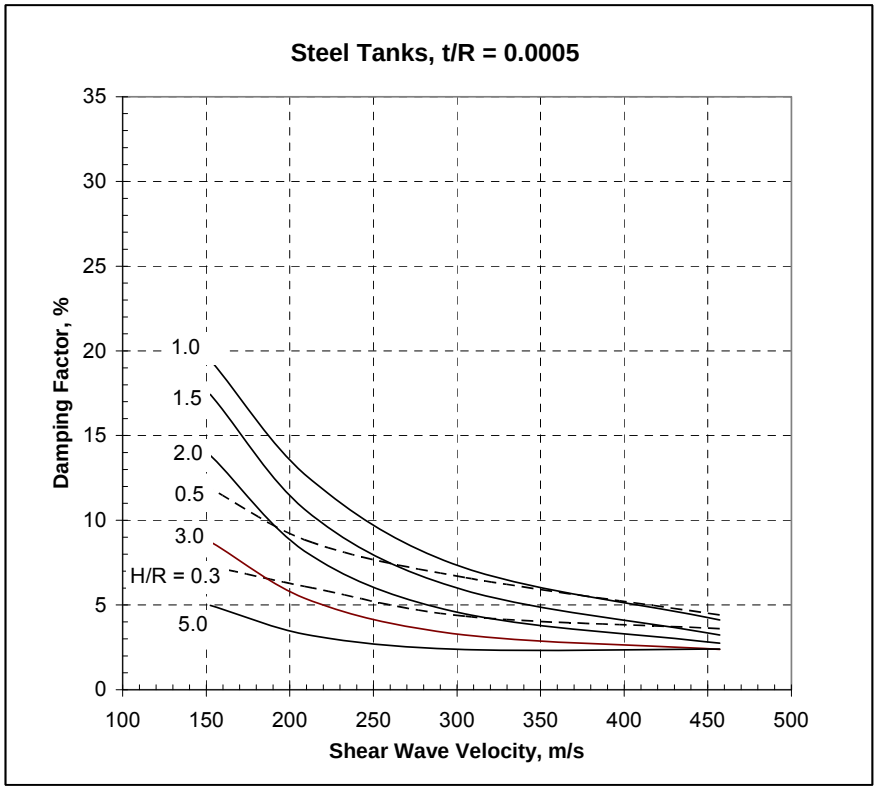


Figure 3.2(b): Damping for Vertical Impulsive Mode Steel Tanks t/R = 0.0005 (Veletsos and Tang, 1986)

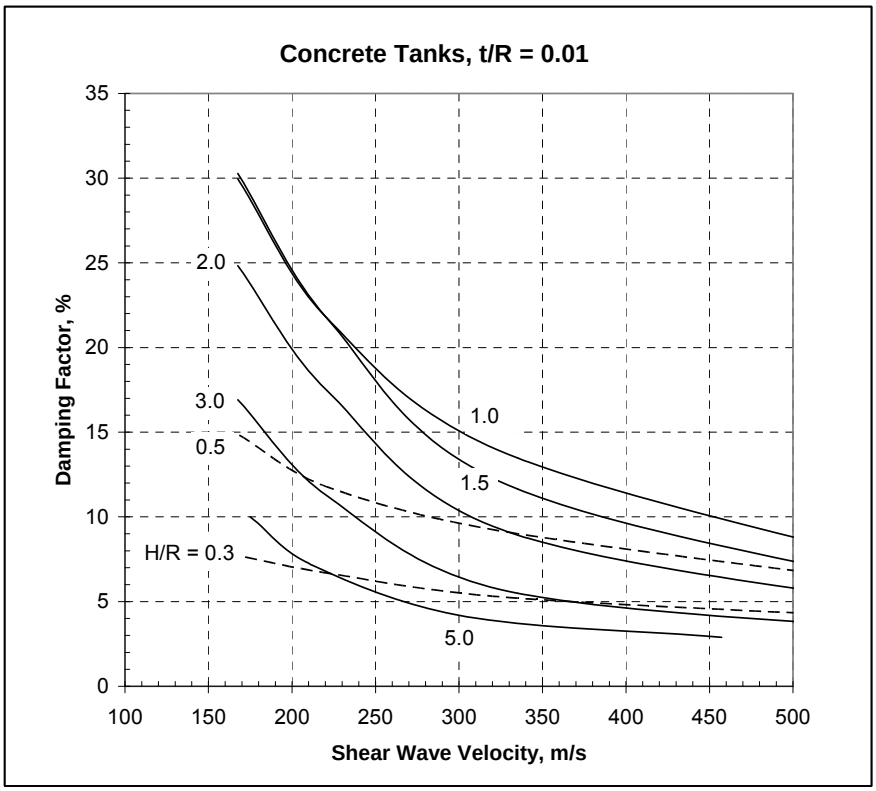


Figure 3.2(c): Damping for Vertical Impulsive Mode Concrete Tanks t/R = 0.01 (Veletsos and Tang, 1986)

Table 3.2 Correction Factor k_f to AS/NZS 1170 elastic site hazard spectrum for damping and ductility

Ductility μ	Teq/T[1]	ξ_h [2] (%)	$k_f(\mu, \xi_i)$[4]							
			ξ [3] = 0.5 %	ξ = 1 %	ξ = 2 %	ξ = 5 %	ξ = 10 %	ξ = 15 %	ξ = 20 %	ξ = 30 %
1.0	1.000	0.0	1.67	1.53	1.32	1.00	0.76	0.64	0.56	0.47
1.25	1.033	3.5	1.08	1.04	0.96	0.82	0.67	0.58	0.52	0.44
2.0	1.120	5.9	0.91	0.89	0.84	0.74	0.63	0.55	0.50	0.43

Notes 1. Teq = equivalent linear period.
T = initial elastic period
2. ξ_h = equivalent additional viscous damping to represent hysteretic energy dissipation (Iwan, 1980)
3. ξ = viscous damping level of elastic system from Figures 3.1 and 3.2
4. Ductility and damping correction factor (NZSEE, 2006)

Table 3.3 Importance Level to be used in application of AS/NZS 1170

Consequence of Failure[1]	AS/NZS 1170 Importance Level
Negligible	1
Slight	1
Moderate	2
Serious	3
Extreme	4

Notes: 1. The importance level for a tank shall be determined from the most severe consequence of failure determined from Tables 3.4 to 3.7.

Table 3.4 Risk Classification Based on Life Safety

No. of Persons Potentially within 50 m of Tank or Perimeter of Bunded Area	Hazard Level of Contents[1], [3]			
	Schedule 4[2]	Schedule 3 UNPG III	Schedule 2 UNPG II	Schedule 1 UNPG I
0	Negligible	Negligible	Negligible	Slight
0 < No. ≤ 1	Negligible	Slight	Slight	Moderate
1 < No. ≤ 10	Slight	Slight	Moderate	Serious
10 < No. ≤ 100	Moderate	Moderate	Serious	Extreme
No. > 100	Serious	Serious	Extreme	Extreme

Notes:
1. Schedules as defined by ERMA New Zealand, (2001).
2. Schedule 4 covers less hazardous substances not covered by UN packing groups, e.g. fresh water.
3. United Nations Packing Groups (UNPG) assigned based on most severe hazard classification of substance (UN, 2005) (ERMA New Zealand, 2001) (ERMA New Zealand, 2004)
   UNPG I: Substances presenting high danger
   UNPG II: Substances presenting medium danger
   UNPG III: Substances presenting low danger

Table 3.5 Risk Classification Based on Environmental Exposure

Environmental Vulnerability[1][2][3]	Hazard Level of Contents			
	Schedule 4	Schedule 3 UNPG III	Schedule 2 UNPG II	Schedule 1 UNPG I
Low	Negligible	Negligible	Slight	Moderate
Moderate	Negligible	Slight	Moderate	Extreme
High	Slight	Moderate	Extreme	Extreme

Notes:
1. Environment includes natural environment only.
2. Environmental vulnerability shall consider the total extent of the surrounding environment affected by an area within 50 m of the tank or of the perimeter of the bunded area.
3. The environment shall be assumed to be vulnerable if it would be adversely affected by coming into contact with the tank contents.

Table 3.6 Risk Classification Based on National or Community Significance

Description	Risk Classification
Facilities of no public significance	Negligible
Facilities of low public significance	Slight
Facilities of moderate public significance	Moderate
Facilities of high national or community significance which are intended to remain functional after a severe earthquake	Serious
Facilities critical to national interest which are vital to remain functional after a severe earthquake	Extreme

Notes:
1. The facilities to be considered include the facility of which the tank is a part and any other facility affected by or within a 50 m radius of the tank or of the perimeter of the bunded area.

Table 3.7 Risk Classification Based on Adjacent Property Value

Adjacent Property value at direct risk from tank failure (year 2007 cost index)	Risk Classification
< $100,000	Negligible
< $1,000,000	Slight
< $10,000,000	Moderate
< $100,000,000	Serious
> $100,000,000	Extreme

Notes: 1. The value of property shall include the value of adjacent property (within 50 m of the tank or of the perimeter of the bunded area).
2. Consideration of risk to property on the same site, based solely on property value, is at the discretion of the owner.

3.3 Hydrodynamic Pressures and Forces

3.3.1 Rigid Tanks

3.3.1.1 Horizontal Earthquake – Rigid Tank

Unless a more rigorous analysis is undertaken, the earthquake induced horizontal hydrodynamic forces on a tank with rigid walls shall be determined from the spring-mass analogy shown in Figure 3.3. The mass m_1 at height h_1, represents the hydrodynamic effects of the first convective (or sloshing) mode of vibration and the mass m_0, rigidly attached to the walls, represents the hydrodynamic effects of the impulsive mode or rigid body displacement of the tank wall.

3.3.1.2 Vertical Earthquake – Rigid Tank

Unless a more rigorous analysis is undertaken, the hydrodynamic pressures induced by the vertical earthquake component shall be determined by the single mass analogy shown in Figure 3.4. The mass m_l shall be the total mass of the fluid and shall be rigidly connected to the tank base. The effect of the vertical earthquake on the hydrodynamic pressures shall be considered as a factored increase in the static pressures. The additional pressure is the product of the earthquake vertical force coefficient (Section 3.2) and the static pressures.

3.3.2 Flexible Tanks

3.3.2.1 Horizontal Earthquake – Flexible Tank

Unless a more rigorous analysis is undertaken, the earthquake induced horizontal hydrodynamic forces on a tank with flexible walls shall be determined from the spring-mass analogy shown in Figure 3.5. The mass m_1 at height h_l represents the hydrodynamic effects of the first convective mode of vibration; the mass m_r at height h_r represents the hydrodynamic effects from the rigid body displacement of the tank wall, and the mass m_f at height h_f represents the influence of the deformation of the tank wall relative to the base. The masses m_r and m_f correspond to two separate impulsive modes of vibration. The mass, m_r, is given by:

$$m_r = m_0 - m_f \tag{3.8}$$

where m_0 = the rigid tank impulsive mass (i.e. the total impulsive mass for the flexible tank)
m_r = tank wall rigid body mode impulsive mass
m_f = flexible tank mode impulsive mass

3.3.2.2 Vertical Earthquake – Flexible Tank

The hydrodynamic pressures induced by the vertical earthquake component shall be determined by the single mass analogy shown in Figure 3.6. The mass m_l shall have the total mass of the fluid and shall be connected to the shell base by a spring representing the flexibility of the tank walls. The hydrodynamic pressures from vertical earthquake shall be determined in a manner similar to that given above for the rigid tank. However, for the flexible tank case the influence of the wall flexibility on the vertical mode period shall be considered.

3.3.3 Base Pressures

The hydrodynamic pressures acting on the base of the tank (as well as on the walls) shall be included in the analysis of the tank support system and the soil foundation.

3.3.4 Foundation Flexibility

The influence of foundation soil-structure interaction on the response of both the horizontal and vertical spring-mass systems shall be considered.

3.3.5 Granular Contents or Full and Sealed Tanks

For granular contents in the tank the convective or sloshing effects may be neglected. If this assumption is made, the total mass of the contents shall be assumed to act in the impulsive mode and taken as rigidly connected to the tank shell.

The same applies to tanks that are full and sealed such as wine vats where the convective mode is constrained from acting.

In both cases the height at which the impulsive mass acts needs to be increased accordingly to match.

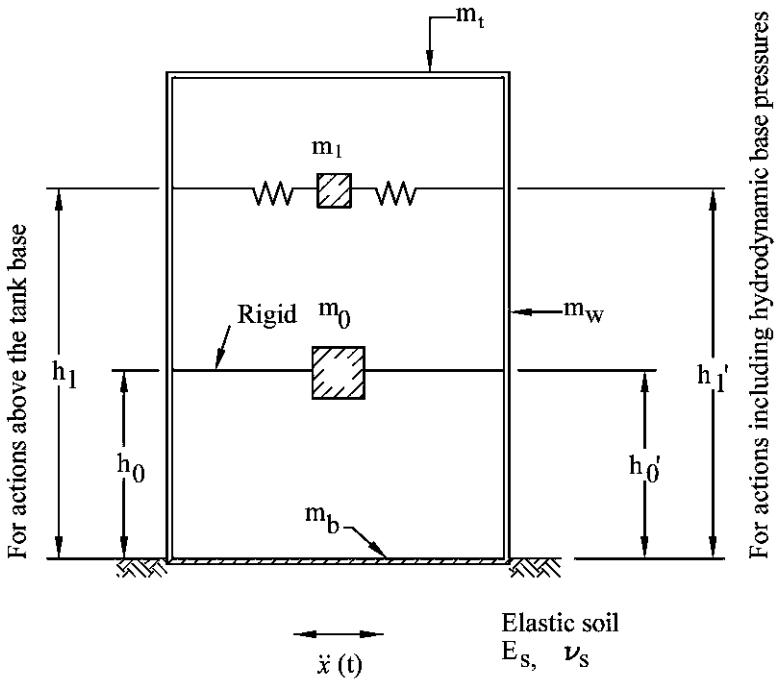


Figure 3.3: Spring-Mass Model for Horizontal Earthquake, Rigid Tank

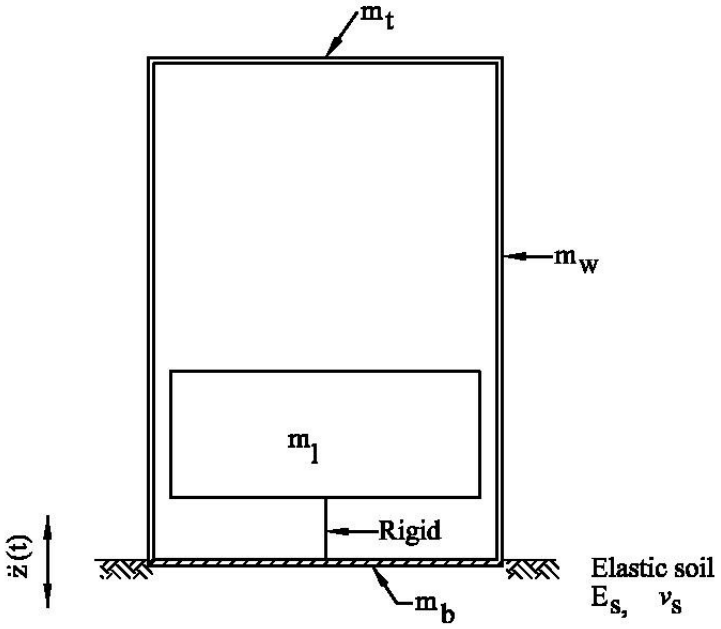


Figure 3.4: Spring-Mass Model for Vertical Earthquake, Rigid Tank

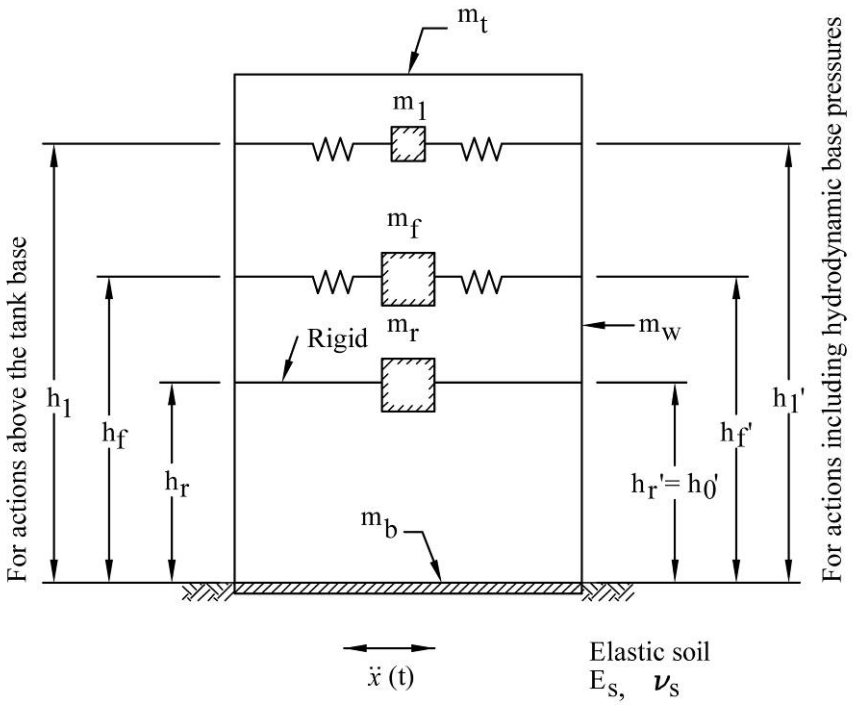


Figure 3.5: Spring-Mass Model for Horizontal Earthquake, Flexible Tank

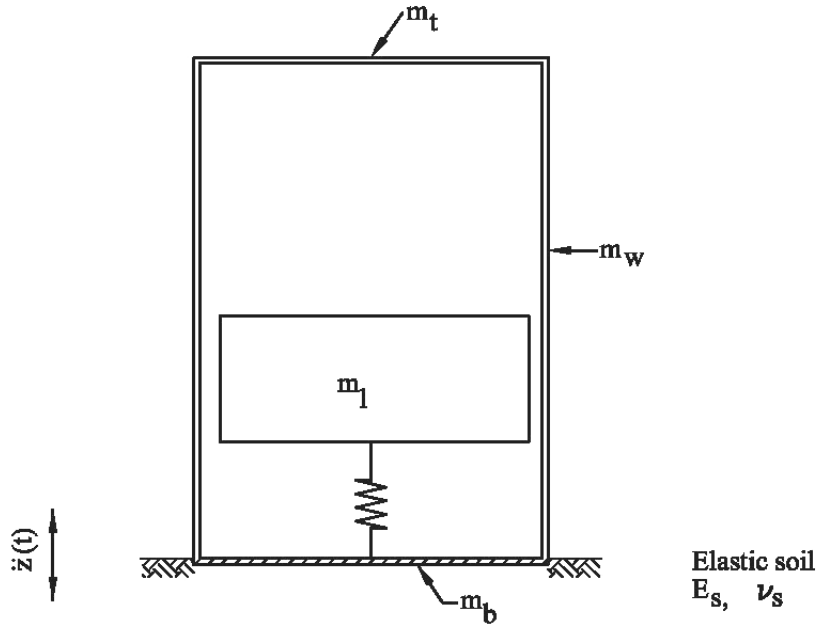


Figure 3.6: Spring-Mass Model for Vertical Earthquake, Flexible Tank

3.4 Shell Inertia Forces

3.4.1 Rigid Tanks

Should there be any doubt as to the rigidity of a tank, use of the flexible tank design method will be conservative for vertical circular cylindrical and rectangular tanks.

3.4.1.1 Horizontal Earthquake - Rigid Tanks

The earthquake forces from the horizontal inertia of the tank walls and roof shall be included in the analysis of the earthquake stresses in the tank shell and foundation. The inertia loads from the tank base and any support system or structural foundation shall be included in the analysis of the stresses in the foundation.

The earthquake forces arising from the inertia of the shell, base and structural foundation act at the centre of gravity of the respective masses. The effect of these masses on the impulsive mode period shall be considered.

3.4.1.2 Vertical Earthquake - Rigid Tank

The vertical inertia loads on the tank roof, walls, base and any support system or structural foundation shall be included in the analysis of the earthquake stresses in the foundation. These masses shall also be used in the calculation of the period of the vertical mode of vibration.

3.4.2 Flexible Tanks

3.4.2.1 Horizontal Earthquake - Flexible Tanks

For flexible tanks, other than concrete tanks, the mass of the shell may be neglected. Where a more rigorous analysis is undertaken, the mass of the shell shall be included in the hydrodynamic mass, m_f, that represents the influence of the deformation of the tank wall. To calculate periods of vibration and stresses in the foundation, the mass from the tank base and any support system or structural foundation shall be included.

3.4.2.2 Vertical Earthquake - Flexible Tank

A detailed analysis of the vertical response of a flexible tank on a flexible foundation requires a two mass analogy. However, this refinement adds considerable complexity to the analysis and will generally be unnecessary for design purposes. Unless a more rigorous analysis is justified, the single mass analogy shall be used with the vertical inertia loads on the tank roof, walls, base and any support system or structural foundation included in the analysis of the earthquake stresses in the foundation. The influence of these masses on the vertical period of vibration may be neglected and the inertia forces on them may be estimated by assuming a vertical force coefficient corresponding to zero period on the acceleration response spectrum.

3.5 Base Shears and Overturning Moments

The following equations give the horizontal base shear components, just above the tank base, from the horizontal inertia forces associated with the masses representing the hydrodynamic effects on the shell and the shell mass:

$V_1 = C_d(T_1)m_1 g$ convective mode (3.9)

$V_r = C_d(\check{T}_0)\,[m_0 - m_f]\,g$ rigid impulsive mode (3.10)

$V_f = C_d(\check{T}_f)\,[m_f + m_w + m_t]\,g$ flexible impulsive mode (3.11)

where T_1 = the period of vibration of the first convective mode of vibration

$\check{T}_0$ = the period of the impulsive mode of vibration for the tank-foundation system. For a rigid foundation $\check{T}_0$ = zero

$\check{T}_f$ = the period of vibration of the first horizontal tank-liquid impulsive mode including the effects of foundation flexibility. For a rigid tank $\check{T}_f = \check{T}_0$ and $m_f = 0$

m_1 = convective mass

m_0 = impulsive mass for rigid tank

m_f = flexible impulsive mass

m_r = rigid impulsive mass = $m_0 - m_f$

m_w = wall mass

m_t = roof mass

g = acceleration of gravity

The overturning moments, just above the tank base, from the inertia forces associated with the mass components are given by:

$M_1 = C_d(T_1)\,m_1\,g\,h_1$ convective mode (3.12)

$M_r = C_d(\check{T}_0)\,[m_0 - m_f]\,gh_0$ rigid impulsive mode (3.13)

$M_f = C_d(\check{T}_f)\,[m_f h_f + m_w h_w + m_t h_t]\,g$ flexible impulsive mode (3.14)

where h_w = height to the centre of gravity of the wall mass

h_t = height to the centre of gravity of roof mass

The overturning moments at the base including the base pressure component are given by:

$M'_1 = C_d(T_1)\,m_1\,g\,h'_1$ convective mode (3.15)

$M'_r = C_d(\check{T}_0)\,[m_0 - m_f]\,gh'_0$ rigid impulsive mode (3.16)

$M'_f = C_d(\check{T}_f)\,[m_f h'_f + m_w h_w + m_t h_t]\,g$ flexible impulsive mode (3.17)

where h'_1 = The equivalent height at which the first mode convective mass is placed to give the total overturning moment arising from pressures on both the walls and base.

h'_0 = The equivalent height at which the rigid impulsive mass is placed to give the total overturning moment arising from pressures on both the walls and base.

h'_f = The equivalent height at which the flexible wall impulsive mass is placed to give the total overturning moment arising from pressures on both the walls and base.

For rigid tanks:

m_f = 0, $\check{T}_f = \check{T}_o$ and Equations 3.11, 3.14 and 3.17 provide the contribution to the base shear and overturning moment of the walls and roof only.

3.6 Natural Periods of Vibration

The deformation of the soil foundation shall be included in the calculation of the impulsive mode periods of vibration for both the horizontal and vertical directions. For flexible tanks this effect shall be included in the estimation of the periods associated with both the impulsive mode masses m_r and m_f.

The mass of the shell, base and any support system or structural foundation shall be included with the impulsive masses, as outlined in Section 3.4, to calculate the impulsive mode periods.

The influence of soil deformation and tank wall flexibility may be neglected when calculating the periods of vibration of the convective modes.

3.7 Damping

The overall damping level for convective modes in all types of tanks shall be taken as 0.5%.

An overall damping level used for the impulsive modes of vibration shall be assessed by a consideration of tank material damping, foundation radiation and soil hysteretic damping.

3.8 Direction of Earthquake

The design earthquake ground motion shall be considered to act in any direction in the horizontal plane that produces the most critical loading on the tank, supports, and foundations. Simultaneous shaking in two orthogonal horizontal directions need not be considered for assessing the strength of the tank or the tank support and foundation system.

Simultaneous shaking in both a single horizontal direction and the vertical direction shall be considered combining the components by the square root of the sum of the squares (SRSS).

3.9 Convective Wave Height

Unless a more rigorous analysis is undertaken the maximum height of convective mode waves in the tank shall be estimated from a spectral modal analysis involving at least the first two antisymmetric modes. The wave heights shall be calculated based on elastic response, i.e. no reduction for any tank ductility shall be made. Interaction of the waves and roof will influence the pressures acting on the roof of the tank. The distribution of the liquid mass between the impulsive and convective masses needs to consider the effect of constraining the convective wave by impact on the tank roof.

3.10 Embedded Tanks

Tanks embedded in the ground shall be designed for earthquake induced soil pressures.

3.11 Elevated Tanks

Unless a more rigorous analysis is undertaken, elevated tanks shall be analysed using a two-mass model representing the convective and the impulsive modes, and including the mass and flexibility of the supporting structure. The flexibility of the tank shell may be neglected.

If the tank support system is loaded beyond the material yield range, the acceleration response of the impulsive modes may be reduced by using an appropriate ductility or equivalent damping factor. The acceleration response of the convective mode shall be computed from the elastic spectrum. That is, no reduction in convective forces results from the ductility of the tank support system.

3.12 References

Environmental Risk Management Authority (ERMA) New Zealand 2001. User Guide to the HSNO Control Regulations, ER-UG-05-1 11/01. ERMA New Zealand, New Zealand.

Environmental Risk Management Authority (ERMA) New Zealand 2004. Hazardous Substances (Dangerous Goods and Scheduled Toxic Substances) Transfer Notice 2004. ERMA New Zealand, New Zealand.

Iwan, W.D., 1980. Estimating Inelastic Response Spectra from Elastic Spectra. Earthquake Engineering and Structural Dynamics, Vol 8, Issue 4, 1980.

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Standards New Zealand (SNZ), 2004b. NZS 1170.5:2004, Structural Design Actions, Part 5: Earthquake Actions – New Zealand. SNZ, New Zealand.

United Nations (UN), 2005. Recommendations on the Transport of Dangerous Goods, Model Regulations, Volume 1, 14th revised edition. United Nations, New York and Geneva.

Veletsos, A.S. and Shivakumar, P., 1997. Computer Analysis and Design of Earthquake Resistant Structures, Chapter 15 – Tanks Containing Liquids or Solids. Computational Mechanics Publications, United Kingdom or United States.

Veletsos, A.S. and Tang, Y., 1986. Dynamics of Vertically Excited Liquid Storage Tanks. Journal of Structural Engineering, Vol 112, No. 6, 1986. ASCE, United States.

Section 4 - Structural Analysis

4.1 Scope

This section provides guidance for the calculation of stresses and displacements induced in tanks by the pressures and forces defined in Section 3.

4.2 Method of Combining Actions

Unless a more rigorous method of analysis is adopted, total tank response resulting from the combined effects of horizontal impulsive, convective, and vertical impulsive actions shall be found by combining the peak spectral responses of each action according to the Square Root of Sum of Squares method (SRSS).

For flexible tanks, the horizontal rigid-impulsive and flexible-impulsive actions should be combined by taking the absolute sum, to give the total horizontal impulsive component.

4.3 Computation of Stresses and Deflection in Tanks with a Vertical Axis

4.3.1 Rigorous Analysis

Except as allowed by 4.3.2, actions induced in tanks with a vertical axis by impulsive or convective seismic pressures shall be calculated using the pressure distributions defined in Section 3.3. Hoop forces and vertical bending moments may be calculated assuming an axially symmetric pressure distribution equal to the maximum value on the axis parallel to the direction of ground acceleration considered.

4.3.2 Approximate Analysis

Where accurate solutions are unavailable, actions induced in tank walls by impulsive or convective pressures may be calculated assuming an equivalent linear pressure distribution. This equivalent pressure shall induce the same base shear and the same overturning moment at the critical section as the original distribution.

4.4 Overturning Effects

The stresses and deflections induced by overturning moments shall be adequately considered.

Tank stresses may be calculated using the correction factor, k_f including the selected displacement ductility factor from Table 3.1. Tank deflections shall be calculated using a displacement ductility factor of 1.25.

4.4.1 Non-Uplifting Tanks

Non-uplifting tanks are tanks which are adequately anchored to the foundation, or are unanchored tanks for which the restoring moment, M_R (based on gravity stresses in the tank walls at the base), is greater than the design overturning moment, M_{OT}, resulting from the resultant seismic force. M_R shall be calculated to determine the weight of the tank and such part of the contents that may assist in resisting overturning. Axial membrane stresses in the wall that result from the overturning forces shall be calculated using an appropriate method.

4.4.2 Uplifting Tanks

Uplifting tanks are those unanchored tanks where M_{OT} is greater than M_R when calculated as described in Section 4.4.1. The response of these tanks shall be assessed considering the modification of behaviour caused by uplift of part of the base.

4.5 Shear Transfer

Tanks shall be designed to adequately transfer shear between roof and wall, and between wall and base.

Section 5 - Design Criteria

5.1 Scope

This section defines the allowable limits to stresses and deflections calculated in accordance with Section 4.

5.2 Load Combinations including Seismic

Design actions resulting from seismic load shall be combined with actions resulting from other loads in accordance with the ultimate limit state load combination given by Equation 5.1.

$$G + Q + E_u \tag{5.1}$$

where

G = Gravity loads resulting from dead weight of tank.

Q = Gravity loads resulting from tank contents.

E_u = Loads resulting from earthquake ultimate limit state.

5.3 Stability

5.3.1 Anchorage Limits

Vertical axis cylindrical tanks shall be anchored to their foundations when

$$\frac{H}{R} > \frac{2}{C_d(\check{T}_f)} \tag{5.2}$$

and

$$R < [C(0)]^2 \frac{H}{R} \tag{5.3}$$

where

$C_d(\check{T}_f)$ = The horizontal design action coefficient at the first horizontal impulsive mode including the effects of foundation flexibility and a displacement ductility factor of 1.25.

$C(0)$ = the site hazard coefficient for T=0 using the modal response spectrum from NZS 1170.5.

R = radius of tank, (metres).

H = height of liquid surface above tank base, (metres).

For vertical axis rectangular tanks Equations 5.2 and 5.3 may be used, substituting L for R,

where

L = half length of rectangular tank in direction of loading.

5.3.2 Required Anchorage

For anchorage provided by non-ductile anchors, the minimum anchorage force per unit circumferential length shall be:

$$P = \frac{4}{\pi}\frac{M_{OT}}{D_{HD}^2} - w_t \quad (5.4)$$

where

D_{HD} = pitch circle diameter of hold-down bolts

M_{OT} = total design overturning moment, for a ductility factor, $\mu = 1.25$.

w_t = weight of walls and roof at base of wall, per unit circumferential length.

For anchorage provided by special ductile anchors the minimum anchorage force per unit circumferential length shall be

$$P = \frac{8}{3\pi}\frac{M_{OT}}{D_{HD}^2} - w_t \quad (5.5)$$

where

M_{OT} = total design overturning moment, for a ductility factor, $\mu = 2.0$.

When the weight of the walls and roof, w_t, is significant a reduction of the wall and roof mass due to upward vertical acceleration should be included.

The minimum anchorage force per unit circumferential length shall be resisted by anchors designed to satisfy the requirements of NZS 3404:1997 (SNZ, 1997). Where special ductile anchors are used, a capacity design approach is required to prevent other potential failure mechanisms, based on the lesser of the overstrength capacity of the anchors and the elastic load. Anchor chairs and their connection to the tank wall shall also be designed using a capacity design approach.

5.4 Displacements

5.4.1 Uplift Displacements

For uplifting circular tanks, the maximum uplift displacement, v, at the wall base shall not exceed that given by

$$v\left(\frac{H}{2}+v\right) \leq \frac{R^2}{2} \quad (5.6)$$

For uplifting rectangular tanks, Equation 5.6 may be used, substituting R = L.

5.4.2 Radial Displacements of Top Stiffening Girder

Radial displacements of stiffening girders at the top of tanks with floating roofs, under the pressure distributions defined in Section 3.3 shall not impede free vertical displacement of the floating roof.

5.4.3 Sliding Displacements

Unless a special study is made to determine the consequences of the tank sliding on the base under seismic response, provision shall be made to transfer the seismic base shear to the

ground by recognised and reliable means. Such means may include friction, holding-down bolts, or the provision of shear keys.

5.4.4 Convective Wave Displacements

Tanks with fixed roofs shall be designed to prevent damage resulting from pressures on the roof caused by convective mode waves. Alternatively allowance shall be made for a freeboard equal to the wave height expected under the design level earthquake. Tanks without roofs or with floating roofs, and where any loss of the contents would be unacceptable, shall also be provided with adequate freeboard to prevent over-topping.

5.5 Specific Requirements for Steel Tanks

5.5.1 Loading

Steel tanks shall be designed for the ultimate limit state load combination of Equation 5.1.

5.5.2 Stress Limits

Stresses due to loads under Equation 5.1, calculated using the methods of analysis given in Section 4, shall not exceed the stress required to induce buckling in membrane compression, buckling in membrane shear, elastic-plastic collapse, nor exceed the material yield stress times the appropriate strength reduction factor (ϕ). For cylindrical tank walls, this requirement may be deemed to be satisfied by the following equations.

Buckling in Membrane Compression

$$\frac{f_m}{f_{cl}} \leq 0.19 + 0.81\,\frac{f_p}{f_{cl}} \tag{5.7}$$

where

f_m = vertical membrane compression stress for tank with internal pressure, subject to flexural compression (cantilever mode).

$$f_p = f_{cl}\sqrt{\left(1-\left(1-\frac{\bar{p}}{5}\right)^2\left(1-\left(\frac{f_0}{f_{cl}}\right)^2\right)\right)} \leq f_{cl} \tag{5.8}$$

$$f_{cl} = 0.6E\frac{t}{R} \tag{5.9}$$

$$\bar{p} = \frac{pR}{tf_{cl}} \leq 5 \tag{5.10}$$

for

$$\left(\lambda^2 = \frac{f_y}{\bar{\sigma}f_{cl}}\right) \leq 2 \;;\; f_0 = f_y\left(1-\frac{\lambda^2}{4}\right) \tag{5.11}$$

$$\geq 2 \;;\; f_0 = \bar{\sigma}f_{cl}$$

where

f_y = material yield stress.

$$\overline{\sigma} = 1-\psi\left(\frac{\delta}{t}\right)\left[\left(1+\frac{2}{\psi\left(\frac{\delta}{t}\right)}\right)^{\frac{1}{2}}-1\right] \tag{5.12}$$

(δ/t) = ratio of maximum imperfection amplitude to wall thickness.

ψ = 1.24 for membrane compression buckling.

Elastic-Plastic Collapse

$$f_m \leq f_{cl}\left[1-\left(\frac{pR}{tf_y}\right)^2\right]\left(1-\frac{1}{1.12+s^{1.5}}\right)\left[\frac{s+\frac{f_y}{250}}{s+1}\right] \tag{5.13}$$

where

$$s = \frac{\left(\frac{R}{t}\right)}{400} \tag{5.14}$$

Material Yield

$$\begin{aligned} f_m &< \phi f_y \\ f_h &< \phi f_y \\ f_b &\leq \phi f_y \end{aligned} \tag{5.15}$$

where

f_m = vertical membrane compression stress

f_h = membrane hoop stress.

f_b = surface stress under wall vertical bending moment.

ϕ = strength reduction factor from NZS 3404 (SNZ, 1997)

5.5.3 Base Plate Design

5.5.3.1

Except as allowed by 5.5.3.2 the thickness of the bottom annular ring of unanchored cylindrical vertical axis tanks shall not exceed the thickness of the bottom shell course.

5.5.3.2

Where 5.5.3.1 requires thickness of less than 6 mm (or 3 mm for stainless steel) a plate thickness of 6 mm (or 3 mm for stainless steel) may be used.

5.5.3.3

Where the bottom annular ring is thicker than the remainder of the bottom, the width of the annular ring shall be not less than the uplift portion of the base under the design seismic load determined using a displacement ductility factor of 1.25.

5.5.3.4

For unanchored tanks, the maximum stress in the base plate, based on catenary action, shall make allowance for the biaxial stress state. The maximum plastic strain in the plastic hinge at the shell/base junction shall not exceed 0.05, based on a plastic hinge length equal to twice the base plate thickness.

5.6 Specific Requirements for Concrete Tanks

The anticipated revision to NZS 3106 shall supersede the recommendations of this section.

Where the ultimate load capacity of a concrete tank can be predicted within tolerances normally expected for ultimate design of concrete structures and liquid containment is not critical, the tank may be designed for the load combination of Equation 5.1, in accordance with provisions of NZS 3101 (SNZ, 2006). Using this approach a ductility of $\mu=2$ may be used in design of the tank. All deflections shall be calculated using a ductility of $\mu=1.25$.

5.7 Tanks Constructed from Materials other than Steel or Concrete

5.7.1 Tanks of Other Metals

Tanks constructed of metals other than steel may be designed in accordance with the provisions of Section 5.5, substituting the 0.2% proof stress for the yield strength, where appropriate.

5.7.2 Tanks of Other Materials

Tanks constructed of non-metallic materials other than concrete shall be designed to a working stress load combination, using material stress limits appropriate for earthquake loading and given by an established relevant materials code.

5.8 References

Standards New Zealand (SNZ), 1986. NZS 3106:1986, Code of Practice for Design of Concrete Structures for the Storage of Liquids. SNZ, New Zealand.

Standards New Zealand (SNZ), 1997. NZS 3404:1997, Steel Structures Standard. SNZ, New Zealand.

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Section 6 - Foundations

6.1 Scope

This section discusses basic considerations in the design, analysis and preparation of foundations for construction of storage tanks where seismic loadings are significant. Types of investigation procedures, analysis methods, and foundation details are identified in general terms.

Design of tank foundations shall be in accordance with the requirements of the New Zealand Building Code Verification Method B1/VM4, Foundations (BIA, 1992).

6.2 Site Investigations

6.2.1 Site Investigation Planning

Site investigations should be planned by a suitably experienced civil or geotechnical engineer. Investigations should include a process of office study of available information, subsurface exploration and laboratory or insitu testing appropriate to the case. Consideration should be given to the involvement of engineering geologists and geophysical survey specialists from the outset of major site investigations. Irrespective of the intended scope or extent of the investigations, the planning and management of the investigations should be flexible and allow the direction of the investigations to be adjusted to suit actual site conditions exposed.

6.2.2 Scope and Extent of Site Investigations

For minor relatively non hazardous tanks on most foundations and for major non hazardous tanks founded on uniform stable soil materials, the site investigations may be directed by the civil designer. For hazardous tanks and for major non hazardous tanks on potentially difficult sites, it is recommended that the site investigations (and foundation design) be undertaken by an experienced geotechnical engineer.

6.3 Foundation Evaluation

In assessing the acceptability of a site for construction of a storage tank, the adequacy of the foundation details, and any site preparation or improvement works, the following factors shall be considered.

- The overall stability of the whole site (e.g. slope stability of a hillside platform) after construction and loading of the tank, under the seismic ground accelerations defined in Section 3 shall be assessed.
- The ability of the foundation to resist, with adequate soil strength reduction factors, the vertical, lateral and overturning loads applied by tank response to the design level earthquake defined in Section 3 shall be ensured.
- The level of differential and total settlements under the design level earthquake defined in Section 3 shall be within such limits as to ensure that the tank continues to meet the performance objectives.
- The probability of disruption of the foundation by gross fault rupture, fracture, or movement shall be less than the acceptable probability of exceedence for the appropriate tank category, defined in Section 3.
- The ability of the soils to resist erosion in the event of tank rupture and loss of contents should be assessed. Where erosion is probable, special details to reduce the extent and impact of erosion shall be adopted.

- The liquefaction potential, collapse potential, and sensitivity of site soils under the design level seismic attack defined in Section 3 shall be assessed. The probability of such failure shall be sufficiently low that the tank continues to meet its performance objectives for the appropriate category of tank.
- The site shall be adequately drained.

6.4 Foundation Analysis

6.4.1 Strength Reduction Factors

Soil strength reduction factors adopted for acceptable foundation conditions should reflect the degree of confidence of the soil parameters used, the consequences of a failure in the particular mode, and the importance of the structure. For all major or hazardous tanks the appropriate reduction factors should be determined by the geotechnical engineer in discussion with the designer and must ensure that the probability of failure does not exceed that required by Section 2.

6.4.2 Soil Strength

Drained (effective strength) and undrained (total strength) parameters should be used as appropriate. For seismic analysis with stiff or dense materials an increase in the undrained strength may be allowed for the rapid rate of strain in such cases. In seismic analyses involving sensitive cohesive soils, loose liquefiable soils, collapsible soils, or brittle highly overconsolidated soils allowance should be made for possible generation of high pore pressures or strength degradation.

6.4.3 Slope Stability Analyses

The stability of slopes supporting tanks may be analysed for seismic conditions by computing a factor of safety for the slope with an applied lateral 'pseudo static' force representing the lateral inertia of the slope mass and tank. Alternatively the "yield acceleration" may be computed. This being the horizontal acceleration necessary to reduce the factor of safety of the slope to unity.

6.4.4 Bearing Capacity Analyses

Bearing capacity checks should be carried out for the tank as a whole and locally for the compression loads under the wall.

The bearing capacity of uniform ground beneath tanks under combined vertical dead and lateral earthquake loading may be assessed using bearing capacity factors. For sites with distinctly non-homogenous properties, or with layers of low strength materials, special considerations may be necessary.

6.4.5 Settlement Analyses

The settlement of tanks and tank foundations under static loading is usually calculated using conventional one-dimensional settlement theory allowing for immediate settlement and secondary consolidation as appropriate. Under seismic loading, movements should generally be elastic except where liquefaction, soil structure collapse, or bearing failures occur. The elastic movements may be estimated, if required, using elastic theory with moduli derived from insitu testing (shear wave velocity), or assessed from soil properties.

6.5 Foundation Details

Where existing site soils are inadequate to support the tank under the design seismic loading, the soil shall be either removed and replaced with material of adequate strength, improved insitu, or avoided by piling to a lower stratum of adequate strength.

6.6 References

Building Industry Authority (BIA) ‘now DBH’, 1992. New Zealand Building Code. BIA, New Zealand.

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Section 7 - Design Details

7.1 Scope

This section identifies the importance of consideration of design details to the satisfactory performance of a tank under seismic effects.

7.2 Influence of Details

Details of, and connections to the tank shall be designed such that the performance criteria of Section 2 are satisfied.

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Section 2 - Commentary

General Design Principles

C2.1 Scope

The design earthquake and the desired performance of the tank and contents under seismic loading are defined in this section. Subsequent sections provide recommendations on the means to satisfy these general requirements.

C2.2 Performance Based Determination of Earthquake Actions

To ensure an acceptably low probability of exceeding the design level forces, tanks should have adequate strength to provide safety against rupture and that stresses and displacements should be within limits to generally ensure adequate behaviour.

Additional serviceability limit state performance (SLS) criteria may be required by owners for particular tanks and connecting services. AS/NZS 1170 (SNZ, 2004) provides recommended SLS earthquake forces.

In meeting the performance criteria the designer is to rationalise the tank design in such a way that a hierarchy of potential failure mechanisms is established, avoiding brittle failures (such as valve fracture) to minimise damage and loss of contents in very large earthquakes. At higher levels of ground shaking than that represented by the design level earthquake, it is accepted that the tank may suffer damage and no longer be serviceable immediately after the earthquake.

Under higher levels of ground shaking than the design level earthquake, loss of tank contents has an increased probability of occurring. Under such circumstances some form of secondary containment may be required depending on the nature of the product and the environment in which it is stored. Where a bund is provided to contain the spilt liquid, it should be capable of performing its function. It is thus necessary to design the bund for more intense ground shaking than the tank is designed for. In most cases it will not be necessary to design the bund for the more intense ground shaking plus the hydrodynamic forces from the spilt contents. The bund and spilt liquid combination subject to earthquake loading may be designed for the same level of importance as the tank and contents.

Since leakage of toxic chemicals into aquifers could present a significant health and environmental hazard, the design of the bund should also be taken to include the bund floor as well as the containing wall.

It is noted that current practice for bunding multiple tanks within a single bund requires design of the bund to cope with a volume less than the failure and loss of contents of all bunded tanks.

This approach implies that there is a potential risk of spillage from the bunded area in a large earthquake that might cause rupture of several tanks. The consequences of this should be considered in design of the bund system in accordance with the same overall performance objectives as the tank design.

Regulations relating to secondary containment of hazardous substances exist, some of which are referenced here (ERMA New Zealand, 2001), (ERMA New Zealand, 2004).

C2.3 References

Environmental Risk Management Authority (ERMA) New Zealand, 2001. Hazardous Substances (Emergency Management) Regulations 2001, Regulation 39. ERMA New Zealand, New Zealand.

Environmental Risk Management Authority (ERMA) New Zealand, 2004. Hazardous Substances (Dangerous Goods and Scheduled Toxic Substances) Transfer Notice 2004. ERMA New Zealand, New Zealand.

Standards New Zealand (SNZ), 2004. AS/NZS 1170, Structural Design Actions. SNZ, New Zealand.

Section 3 - Commentary

Design Loading

C3.1 Scope

Both shell and foundation flexibility will increase the impulsive mode periods and this may alter the earthquake response depending on where the periods fall on the response spectrum.

Although concrete cylindrical and spherical shaped tanks are likely to be more rigid than steel tanks, the influence of shell flexibility on the earthquake response may still be significant.

The response of relatively tall tanks on firm foundations is unlikely to be significantly influenced by foundation flexibility but generally soil-structure interaction effects should be checked.

C3.2 Earthquake Actions

The earthquake design spectra, zone factors, and return periods are obtained from AS/NZS 1170 (SNZ, 2004). Equations 3.4 to 3.7 allow calculation of the spectral shape factor, $C_h(T_i)$, for long periods greater than 3 seconds assuming constant displacement response.

The Structural Performance Factor Sp has been conservatively taken as 1.0 for tanks, rather than the value which is given in AS/NZS 1170 for buildings and other structures. This is because there is insufficient tank performance data available to justify a lesser figure for tanks, and it is considered that tanks do not have the inherent redundancy that many other structures possess.

In other countries earthquake design spectra complying with local regulatory requirements could be substituted to determine values for the horizontal design action coefficient, $C_d(T)$. The structural performance factor used in AS/NZS 1170 is implicit in the force reduction factors of some international codes. Thus the horizontal design action coefficient, $C_d(T)$, should be determined at elastic spectra load level and the spectral coefficient correction factor used to adjust for ductility. The spectral coefficient correction factor for damping and ductility in Table 3.2 may be used where applicable to adjust local spectra defined at 5% damping. Return period factors used should be consistent with appropriate national seismic safety levels.

Annual probabilities of exceedance and return period factors from AS/NZS 1170 are given in Table C3.1 for comparison.

Table C3.1: Consequences of Failure, Recommended Importance Factors and Return Period Factors from AS/NZS 1170

Consequence of Failure	Importance Level(1)	Annual Probability of Exceedence for Design Event(2)	Return Period Factor, R_u
Negligible	1	1/100	0.5
Slight	1	1/100	0.5
Moderate	2	1/500	1.0
Serious	3	1/1000	1.3
Extreme	4	1/2500	1.8

Notes: 1. Importance levels for use in accordance with AS/NZS 1170.
2. Annual probability of exceedance for design event on tank with default design life (50 years) in accordance with AS/NZS 1170.

The minimum design return period for a moderate consequence of failure has been set at a level which would be considered appropriate for a typical building under the New Zealand Building Code (NZBC, 1992). The levels of risk represented by the serious and extreme consequences of failure are also comparable with the level of risk generally accepted in the petrochemical industry for these situations.

For design purposes it is recommended that the vertical response is taken as 70% of the maximum horizontal response.

The damping values in Figure 3.1(d) were derived for t/R = 0.01 but may be used over the range t/R = 0.008 to 0.013. Outside this range the values can be adjusted by interpolation based on taking the damping for tanks with t/R = 0.005 as 40% less than those in Figure 3.1(d) and for tanks with t/R = 0.02 as 40% greater than those in Figure 3.1(d). This approach applies also to the damping values in Figure 3.2(d).

For tanks supported on the ground the consideration of the vertical earthquake forces will mainly affect the hydrostatic pressures and the design of the foundation. However, for elevated tanks the support structure design may be affected significantly by the requirement to consider simultaneous horizontal and vertical earthquake effects.

For a secondary containment system to be considered effective during earthquake shaking it is necessary to design the bund for more intense ground shaking than the tank is designed for. This would require a bund to be designed to contain the spilled product when subjected to a Peak Ground Acceleration of not less than:

$$ä_0 = C_h(0)R_u Z S_p \qquad \text{(C3.1)}$$

where

S_p = 1.0

R_u = Return Period Factor corresponding to an Importance Level one level higher than for the tank when designing for maximum excitation of the bund alone and,

= Return Period Factor corresponding to an Importance Level the same as for the tank when designing for excitation of the bund containing the spilt contents.

Guidance to Tables 3.3 and 3.7

Where the consequences of failure are Moderate or lower, it is considered acceptable to consider a reduced design life and corresponding lower return periods. For example, where a tank is required only for temporary storage, or where an existing tank is being assessed for continued use during a limited and specified remaining period of life, it is reasonable to allow lower environmental design loadings to be used.

Some of the more common stored substances and their United Nation Packing Group, UNPG, are listed in Table C3.2.

Table C3.2: UNPG Designations for Common Substances

Substance	UNPG#[(1)]
Aviation gasoline and racing gasoline (Avgas 100 and Avgas 100LL)	PG I
Crude oils (i) Extremely flammable	PG I
(ii) Highly flammable	PG II
(iii) Flammable	PG III
Diesel fuel	PG III
Ethanol	PG II
Hydrochloric acid >25% aqueous solution	PG II
Kerosene (including Jet A-1, aviation turbine fuels)	PG III
Nitric acid >10-70% aqueous solution	PG II
Petrol (Unleaded)	PG I
Sulphuric acid >10% aqueous solution	PG II

Notes: 1. As defined by hazard classification by ERMA (2004).

Guidance to Table 3.5: Environmental Vulnerability

Low vulnerability: low ecological sensitivity, rural land, no significant waterways, low sensitivity of groundwater systems, tanks or compounds having a secondary containment system designed to a standard at least that of the tank.

Moderate vulnerability: somewhere between low and high vulnerability.

High vulnerability: high ecological sensitivity, major waterways, potable groundwater resources, highly populated areas.

Guidance to Table 3.6: Community Significance

Negligible: Of commercial interest only

Slight: Mainly of private or commercial interest, but of some public value

Moderate: e.g. commercially owned bulk fuel installations,

Serious: e.g. major public utilities and infrastructure facilities, fire fighting supplies

Extreme: e.g. storage for public emergency services, lifeline facilities, critical national defence facilities

C3.3 Hydrodynamic Pressures and Forces

Information presented in this section on circular cylindrical tanks is mainly based on the work of Veletsos in the ASCE publication "Guidelines for the seismic design of oil and gas pipeline systems" (Veletsos, 1984).

The spring-mass analogy generally provides a satisfactory method of calculating the critical shear and axial in-plane stresses that occur in the wall just above the base of a tank and the stresses in the foundation. To evaluate the earthquake induced circumferential and bending stresses in the shell, it is necessary to know the magnitude of the hydrodynamic pressures. Information on hydrodynamic pressures is also required if a detailed shell analysis is to be undertaken.

Equations for calculation of hydrodynamic pressures and the spring-mass analogy parameters for common types of tanks are given below.

C3.3.1 Rigid Tanks

Rigid Vertical Circular Cylindrical Tank

The horizontal earthquake induced hydrodynamic pressures on the cylindrical walls of a rigid tank with its axis vertical, as shown in Figure C3.1, are given by:

$$p(z,\theta,t) = p_i(z,\theta,t) + p_c(z,\theta,t) \quad \text{(C3.2)}$$

where $p_i(z,\theta,t)$ is the impulsive pressure component, and $p_c(z,\theta,t)$ the convective pressure component. Both components are functions of the cylindrical coordinates z and θ, and time t.

Although the time history of total pressure is the sum of the individual components it should be noted that direct addition of individual peak responses will produce a conservative upper bound to the total peak value. The SRSS method is appropriate when combining individual peak responses.

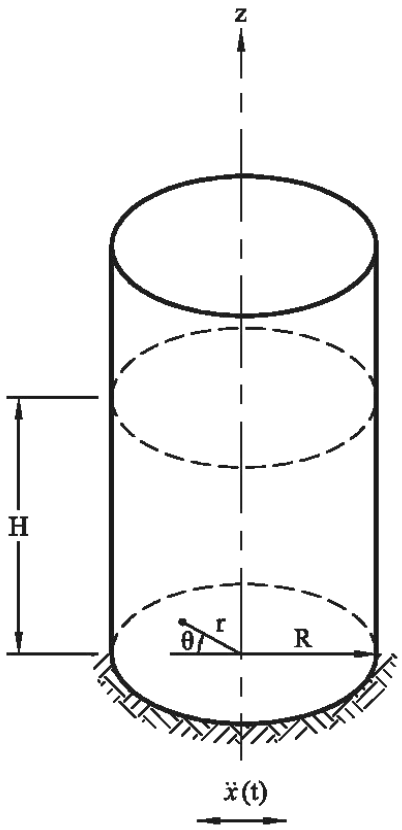


Figure C3.1: Nomenclature for Vertical Axis Cylindrical Tank

Impulsive Pressure Component On Walls

The maximum impulsive pressure component on the walls is given by:

$$p_i(z, \theta) = q_0(z)\ C_d(\check{T}_0)\ \gamma_\ell\ R \cos\theta \quad \text{(C3.3)}$$

or

$$p_i(z, \theta) = q'_0(z)\ C_d(\check{T}_0)\ \gamma_\ell\ H \cos\theta \quad \text{(C3.4)}$$

where

$q_o(z)$, $q'_o(z)$ = dimensionless impulsive pressure functions of depth z

$C_d(\check{T}_0)$ = horizontal design action coefficient for period $\check{T}_0$.

$\check{T}_0$ = period of impulsive mode of vibration including soil-structure interaction effects.

γ_ℓ = unit weight of liquid

The shape of the dimensionless impulsive pressure function given by $q_0(z)/ q_0(0)$ is plotted in Figure C3.2 for different H/R ratios. The values of the maximum dimensionless pressures at the base of the wall, $q_0(0)$ and $q'_0(0)$, are given in Figure C3.3 as a function of the H/R ratio.

It should be noted that Equations C3.3 and C3.4 produce identical results. The alternative formulations are provided for convenience when analysing tanks with high H/R and low H/R ratios respectively.

Convective Pressure Component on Walls

The convective pressure component on the walls is given by:

$$p_c(z, \theta, t) = \sum_{j=1}^{\infty} [q_j(z) a_j(t)] \gamma_\ell R \cos\theta \qquad \text{(C3.5)}$$

where

$q_j(z)$ = dimensionless pressure function for the jth sloshing mode of vibration.

$a_j(t)$ = coefficient of horizontal acceleration response in the jth sloshing mode of vibration.

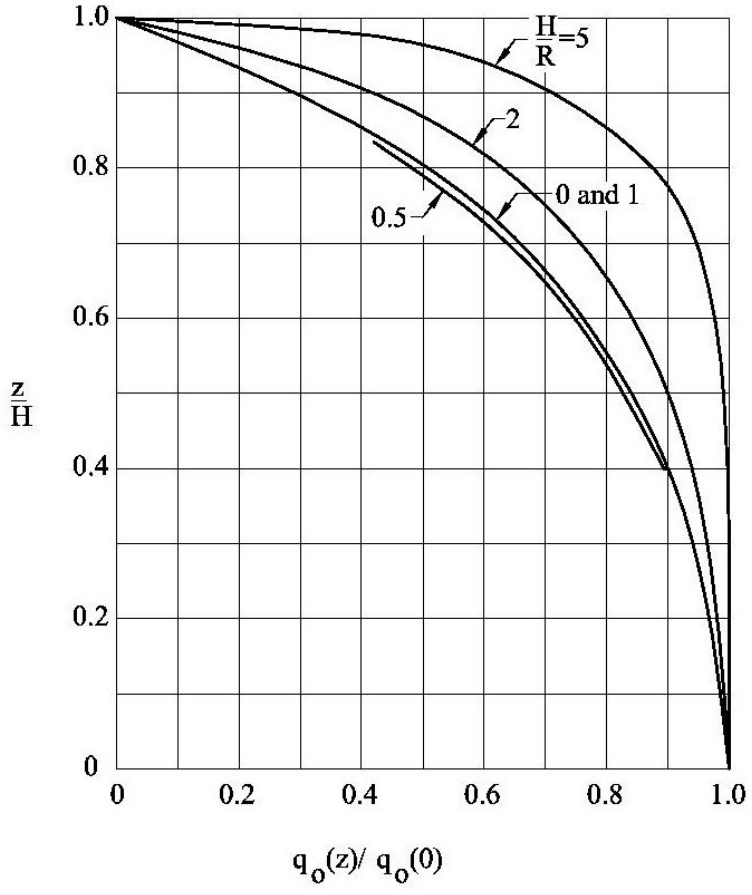


Figure C3.2: Vertical Distribution of Impulsive Pressures, (Veletsos, 1984)

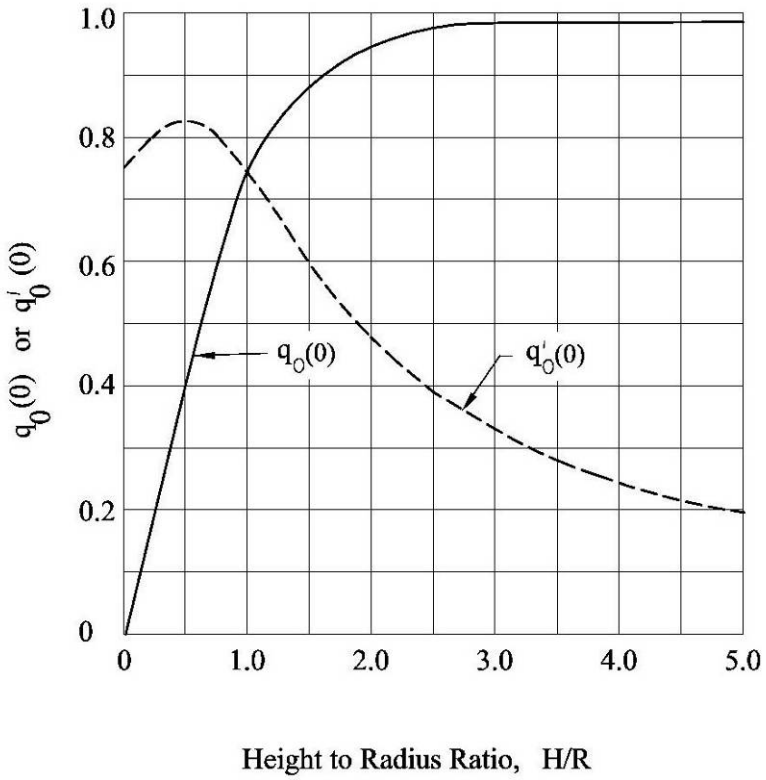


Figure C3.3: Peak Value of Dimensionless Impulsive Pressure at Tank Base, (Veletsos, 1984)

For the first sloshing mode of vibration, Equation C3.5 reduces to

$$p_{c1}(z, \theta) = q_l(z)\ C_d(T_1)\ \gamma_\ell\ R \cos\theta \qquad \text{(C3.6)}$$

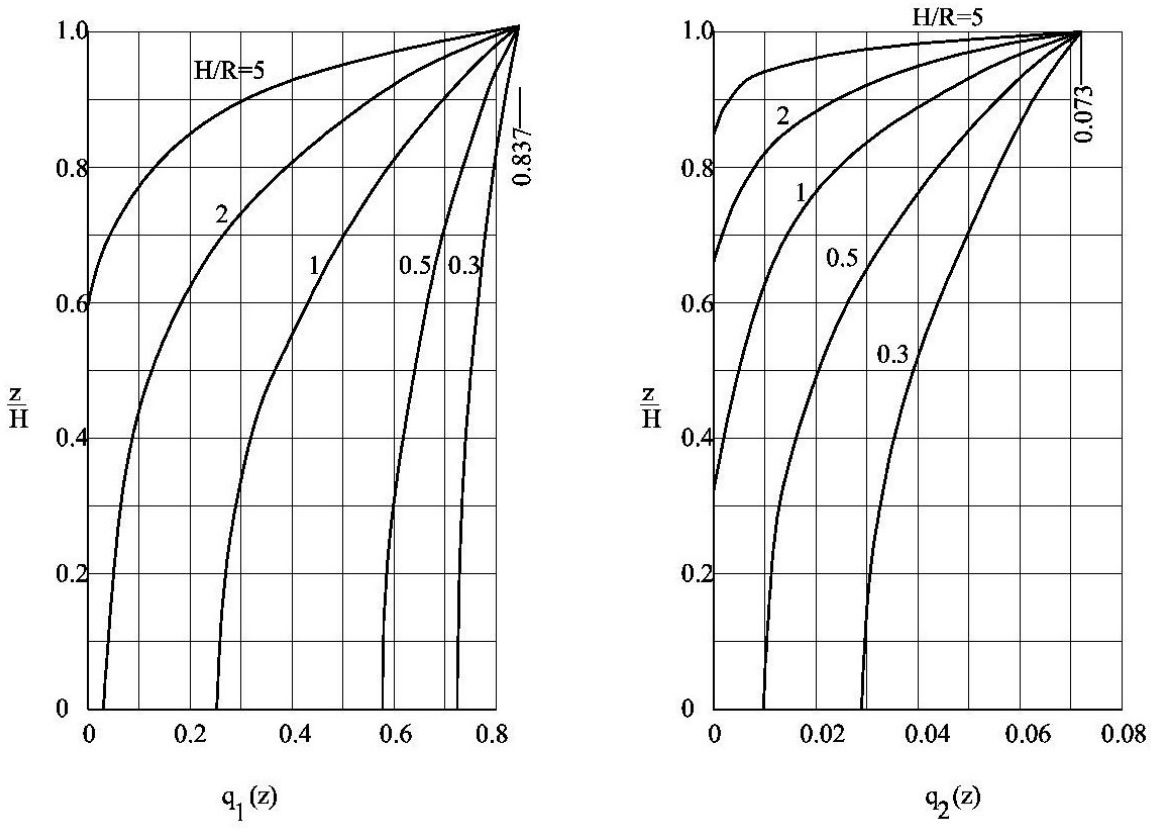
where

$p_{c1}(z, \theta)$ = maximum first convective mode pressure component on tank wall.

$q_l(z)$ = dimensionless pressure function for the first sloshing mode.

$C_d(T_1)$ = horizontal design action coefficient corresponding to period T_1 of the first convective mode (see Equation 3.2).

The dimensionless convective pressure functions for the first two sloshing modes are shown in Figure C3.4. In contrast to the impulsive pressure distributions, the shape of the convective pressures varies significantly with the H/R ratio. Because of this variation it is convenient to show the shape and magnitude in a single plot. Note that the maximum convective pressures occur at the liquid surface and are independent of the H/R ratio.



(a) First Mode **(b) Second Mode**

Figure C3.4: Vertical Distribution of Dimensionless Convective Pressures, (Veletsos, 1984)

Pressure On Circular Base

Both the impulsive and convective pressures on the tank base are non-linear in the radial direction.

The radial distribution of the impulsive pressure component is shown in Figure C3.5 and the distribution for the first convective mode in Figure C3.6. The distributions of the impulsive pressures are dependent on the H/R ratio, whereas the convective pressure distribution is independent of the tank geometry.

Pressures From Vertical Accelerations

The vertical component of ground acceleration induces a hydrodynamic wall pressure in addition to that induced by the horizontal ground accelerations. The pressure on the walls, $p_v(z)$, from the vertical acceleration component is uniformly distributed in the circumferential direction and is given approximately by:

$$p_v(z) = (1 - z/H)\ C_d(\check{T}_v)\ \gamma_\ell\ H \qquad \text{(C3.7)}$$

where

$C_d(\check{T}_v)$ = vertical design action coefficient at period $\check{T}_v$ (see Equation 3.2) including scaling factor of 0.7

$\check{T}_v$ = period of vertical mode of vibration including soil structure interaction effects

For evaluating $C_d(\check{T}_v)$

μ = ductility factor for vertical mode = 1.0

ξ_v = damping level for the vertical mode (see Figure 3.2)

Spring-Mass Analogy

To evaluate the maximum shear stresses in the shell it is necessary to have the total applied earthquake shear at the base. The maximum axial stresses in the shell can be found from the total applied earthquake moment just above the base (see Section 3.5).

To calculate the overall overturning moment on the foundation or supporting structure it is also necessary to have the base moment produced by the hydrodynamic impulsive and convective pressures acting on the circular base of the tank. The total overturning moment is the sum of the moments arising from the pressures on both the walls and the base.

It is convenient to present design data for the base shear and the two overturning moment components at the base (wall and base moments) by reference to the rigid tank mechanical spring-mass analogy shown in Figure C3.7. The mass m_0 is rigidly attached to the shell walls and represents the effects of the impulsive pressures on the shell. Masses m_1, m_2, m_3 ... represent the effect of the convective or sloshing modes. Both the impulsive and convective mass components are assumed to act at heights above the base equal to the heights of the respective centres of pressures.

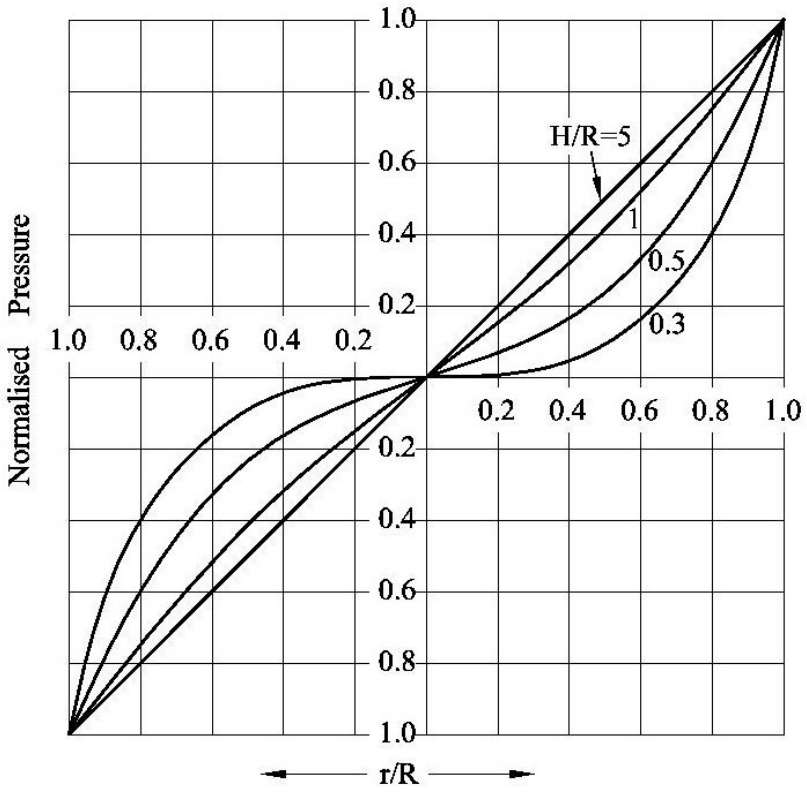


Figure C3.5: Radial Distribution of Impulsive Pressures on Tank Base, (Veletsos, 1984)

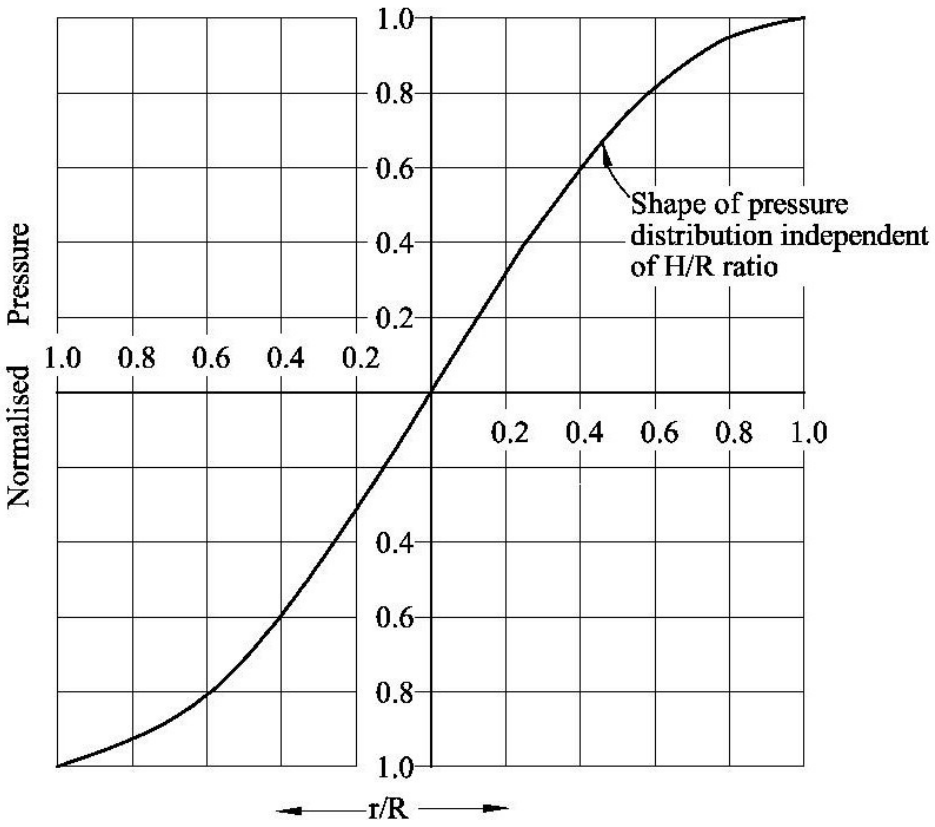


Figure C3.6: Radial Distribution of First Mode Convective Pressures on Tank Base

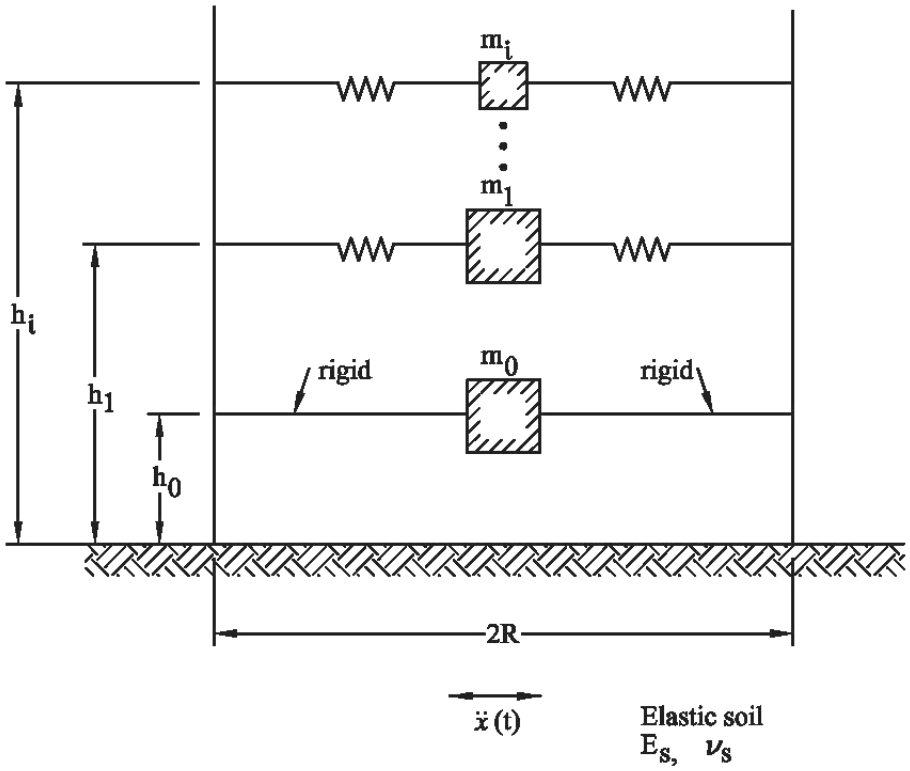


Figure C3.7: Spring-Mass Analogy for Rigid Tank

The sum of the masses is equal to the total mass of the liquid, m_l, that is:

$$m_\ell = m_0 + \sum_{i=1}^{\infty} m_i \quad \text{(C3.8)}$$

The impulsive base shear, V_0, and the impulsive moment, M_0, just above the base are given by:

$$V_0 = C_d(\check{T}_0)\, m_0\, g \quad \text{(C3.9)}$$

$$M_0 = C_d(\check{T}_0)\, m_0\, g\, h_0 \quad \text{(C3.10)}$$

The base shear and moment from the inertia forces on the tank wall and roof masses are obtained from similar expressions to Equations C3.9 and C3.10, with m_0 and h_0 replaced with appropriate values for the shell masses.

The total impulsive moment at the base, including the base pressure component is given by:

$$M'_0 = C_d(\check{T}_0)\, m_0\, g\, h'_0 \quad \text{(C3.11)}$$

where

h'_0 = the equivalent height at which the impulsive mass is placed to give the total overturning moment arising from pressures on both the walls and base.

The maximum base shear, V_1, and moments M_1, M_1', arising from the first convective or sloshing mode are given by:

$$V_1 = C_d(T_1)\, m_l\, g \quad \text{(C3.12)}$$

$$M_1 = C_d(T_1)\, m_1\, g\, h_l \quad \text{(C3.13)}$$

$$M'_l = C_d(T_1)\, m_l\, g\, h'_l \quad \text{(C3.14)}$$

where

M_1 = first mode convective moment just above the base.

M'_1 = first mode convective moment at the base including the base pressure component

h'_1 = the equivalent height at which the first mode convective mass is placed to give the total overturning moment arising from pressures on both the walls and base.

The impulsive mass and the masses for the first two convective modes can be evaluated using Figure C3.8, which shows the relationship between the mass ratios, obtained by dividing the masses by the liquid mass m_ℓ, and the H/R ratio. The equivalent heights of the masses for evaluating the moment just above the base and the total moment including the base pressure component can be evaluated from Figure C3.9. The impulsive mass heights are shown as h_0 and h'_0 and the first mode convective mass heights as h_l and h'_l.

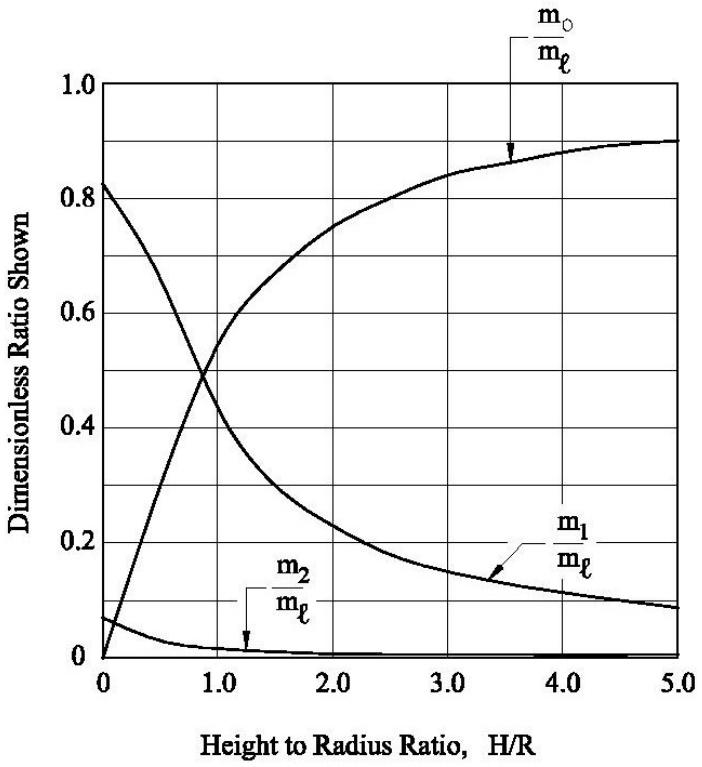


Figure C3.8: Impulsive and Convective Mass Components, (Veletsos, 1984)

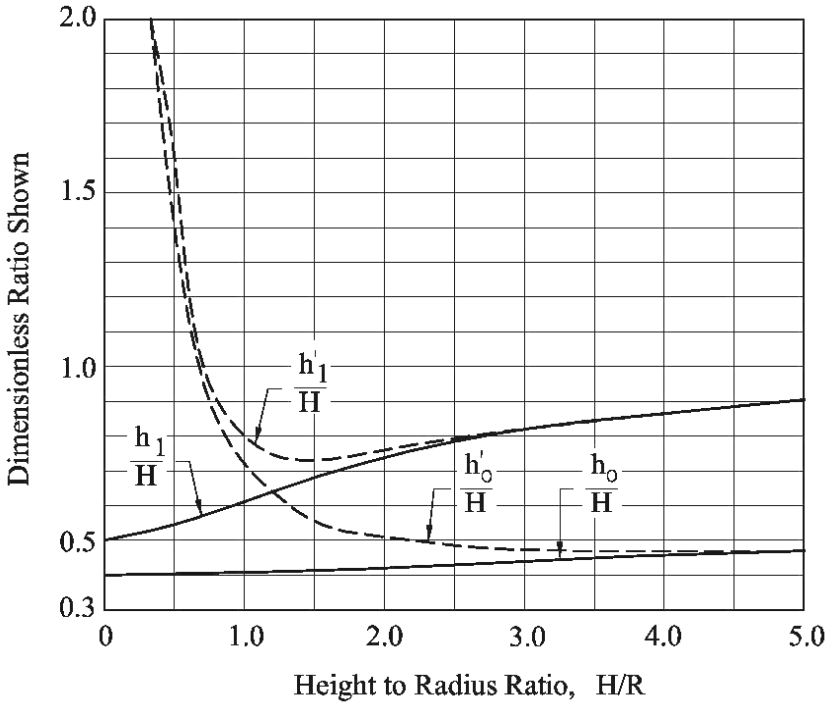


Figure C3.9: Heights of Impulsive and Convective Masses, (Veletsos, 1984)

Rigid Rectangular Tank

The procedure for the analysis of the earthquake pressures on a rigid rectangular tank is similar to that described previously for circular cylindrical tanks with vertical axes. The total pressure on the walls perpendicular to the direction of ground motion, of the rectangular tank shown in Figure C3.10, is the sum of the impulsive and convective pressures as given by:

$$p(z,t) = p_i(z, t) + p_c(z, t) \quad \text{(C3.15)}$$

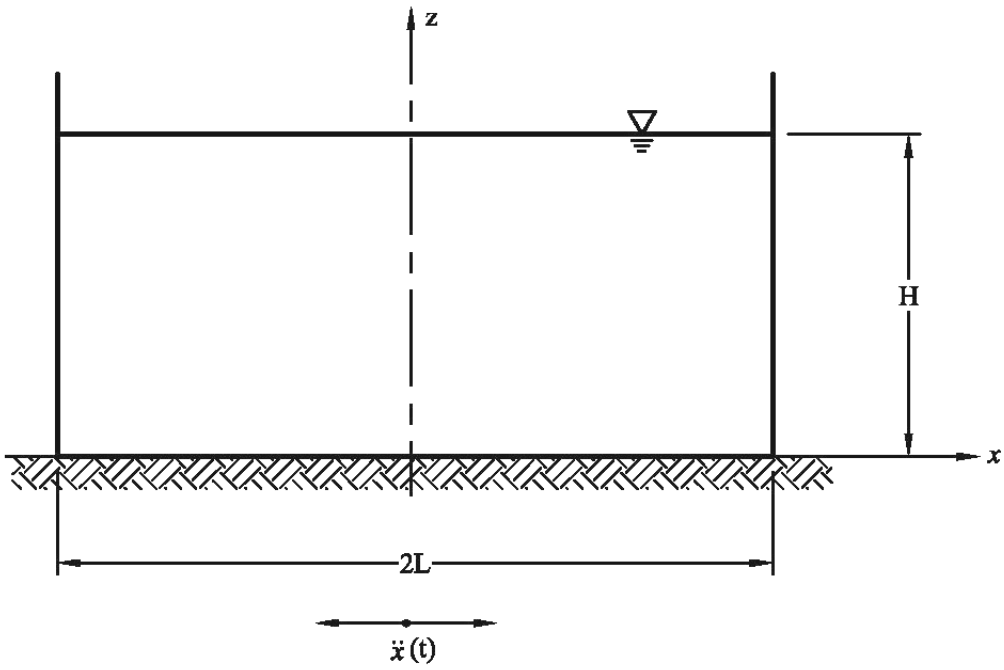


Figure C3.10: Nomenclature for Rectangular Tank

Generally the walls of rectangular tanks will be sufficiently flexible to significantly increase the period of the impulsive mode of vibration. For this case, reference should be made to the modifications given in the section on flexible tanks below.

Impulsive Pressure Component on Walls

The impulsive pressure component on the walls perpendicular to the direction of ground motion is given by:

$$p_i(z) = q_0(z)\ C_d(\check{T}_0)\ \gamma_\ell\ L \quad \text{(C3.16)}$$

The shape and magnitude of the dimensionless pressure function $q_0(z)$, varies with the geometric ratio H/L, as shown in Figures C3.11 and C3.12.

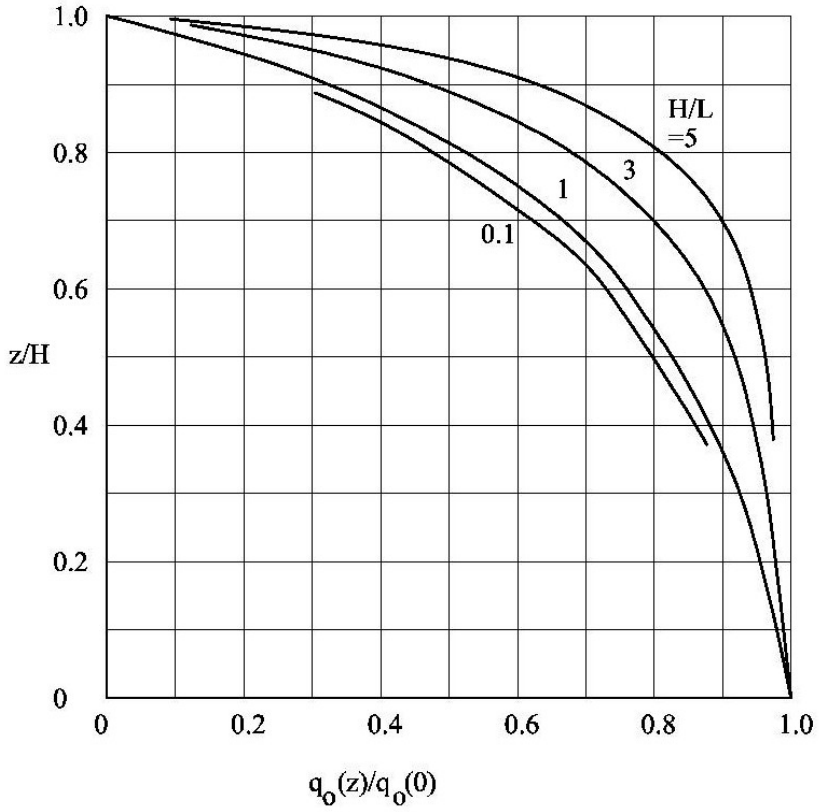


Figure C3.11: Dimensionless Impulsive Pressures on Rectangular Tank Wall Perpendicular to Direction of Earthquake

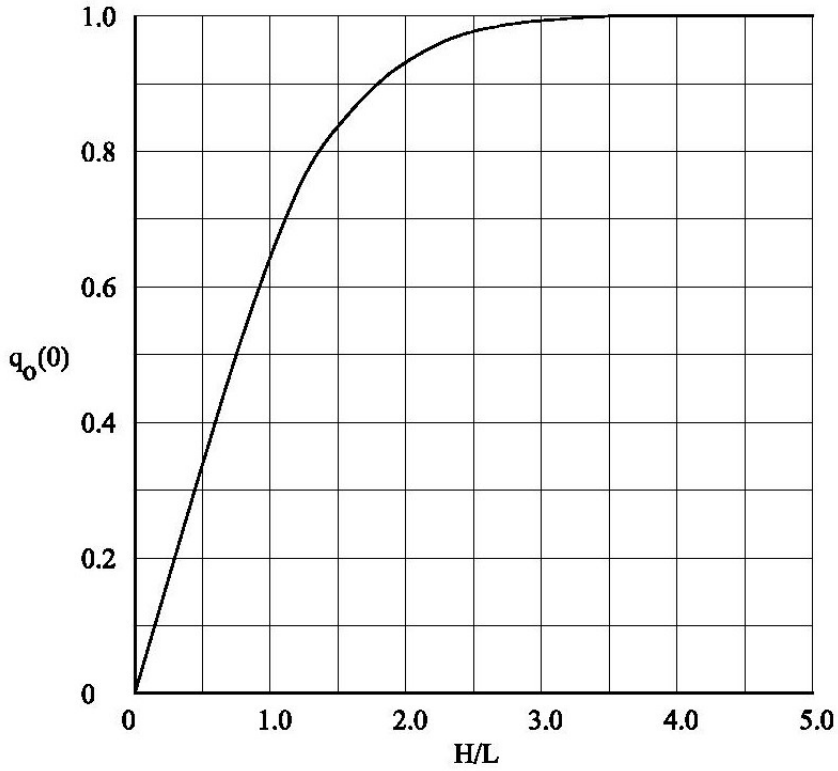


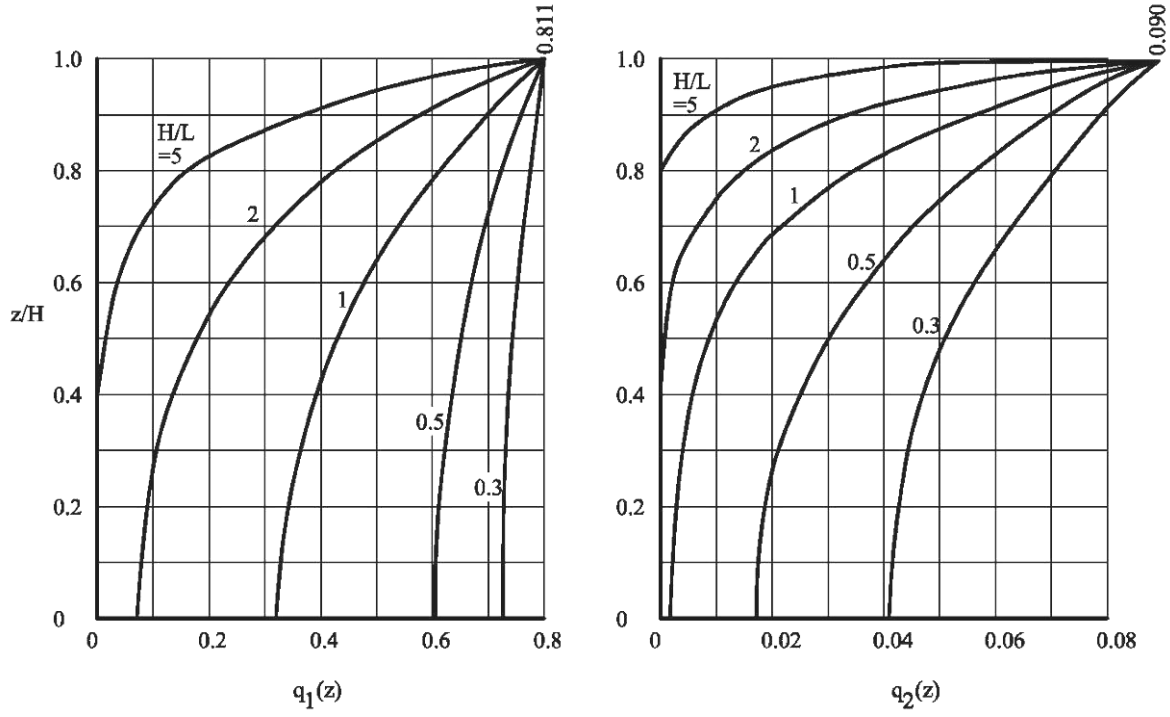
Figure C3.12: Peak Value of Dimensionless Impulsive Pressures on Rectangular Wall Perpendicular to Direction of Earthquake

Convective Pressure Component on Walls

The maximum convective pressures on the walls perpendicular to the direction of ground motion in the first sloshing mode of vibration are given by:

$$p_{c1}(z) = q_1(z)\ C_d(T_1)\ \gamma_\ell\ L \qquad \text{(C3.17)}$$

The dimensionless convective pressure functions for the first two sloshing modes are shown in Figure C3.13.



(a) First Mode **(b) Second Mode**

Figure C3.13: Dimensionless Convective Pressures on Rectangular Tank Wall Perpendicular to Direction of Earthquake

Spring-Mass Analogy

The spring mass analogy defined previously for the vertical circular cylindrical tank can also be used for rectangular tanks.

For design purposes, the values of the equivalent masses, m_0, m_1 m_2 … and their heights above the base h_0, h_1 … and h'_0, h'_1 … may be assumed to be the same as for the circular cylindrical tank with the half length, L, of the rectangular tank used instead of the circular radius, R. In most cases this approximation is expected to give base shears and overturning moments within 15% of values from more exact theory.

Horizontal Circular Cylindrical Tank

In the analysis of horizontal cylindrical tanks it is convenient to consider motion either along the direction of the axis or transverse to the axis as shown in Figure C3.14.

Approximate values for earthquake induced hydrodynamic pressures and forces 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 gives sufficiently accurate solutions for

design purposes over the range of H/R between 0.5 and 1.6. Where H/R exceeds 1.6 the tank should be assumed to behave as if it was completely full of liquid. That is, the total mass of the contents acts as an impulsive mass at mid height of the tank.

A more exact analysis is available (Werner and Sundquist, 1949), for a partially full tank with motion transverse to the axis. The maximum impulsive pressure distribution for motion in this direction can be expressed as:

$$p_i(\phi) = q_0(\phi)\ C_d(\check{T}_0)\ \gamma_\ell\ R \qquad \text{(C3.18)}$$

For the case of the tank half full of liquid (H = R), the dimensionless pressure function is given by:

$$q_0(\phi) = \frac{4}{\pi}\sum_{n=1}^{\infty}\frac{(-1)^{n-1}}{(2n)^2-1}\sin(2n\phi) \qquad \text{(C3.19)}$$

This pressure function is plotted in Figure C3.15. By integrating the pressure distribution the impulsive mass is evaluated to be:

$$m_0 = 0.4\ m_\ell \qquad \text{(C3.20)}$$

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 convective masses should be assumed to act at this point.

Solutions for the convective pressures are not available in convenient form for design. When the tank is approximately half full, (H ≈ R), the first sloshing mode mass, m_1, can be evaluated by:

$$m_1 = 0.6\ m_\ell \qquad \text{(C3.21)}$$

Equations C3.20 and C3.21 are expected to be reasonable approximations for liquid levels given by H/R ranging from 0.8 to 1.2.

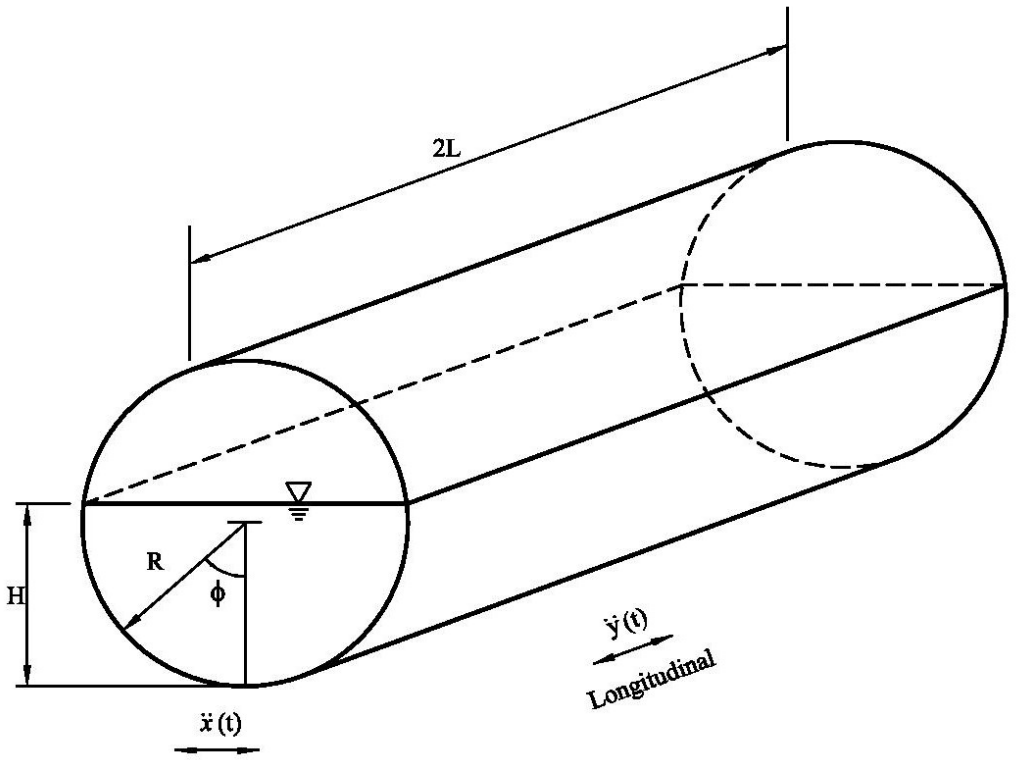


Figure C3.14: Nomenclature for Horizontal Axis Cylindrical Tank

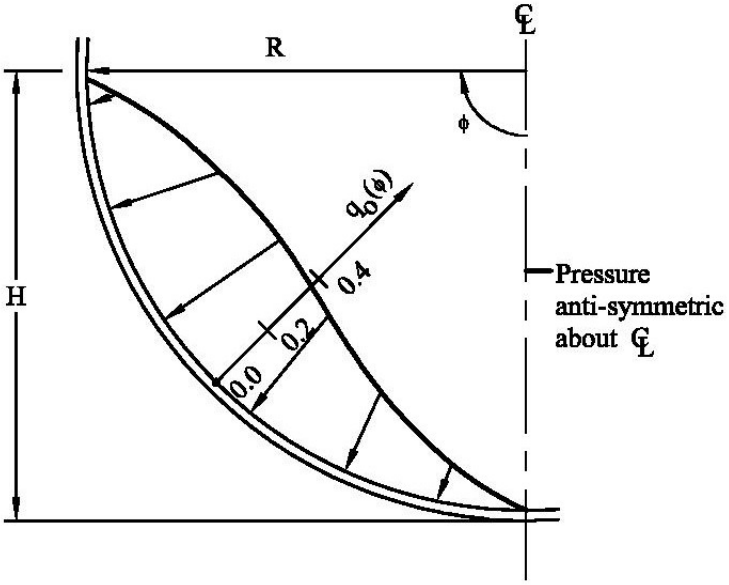


Figure C3.15: Impulsive Pressures on Horizontal Cylinder with H=R. Transverse Excitation (Werner and Sundquist, 1949)

Non-Cylindrical Rigid Axisymmetric Tank with Vertical Axis

Exact solutions are not available for other than special cases (e.g. spheres) in this category. It is believed that a reasonably conservative approximation to the seismic forces of the comparatively common class of elevated tanks whose shape approximates part of an inverted cone may be obtained from the analysis of an equivalent cylindrical tank containing the same volume of liquid as the real tank, and whose diameter is the same as the real tank at the free water level. Convective and impulsive base shears and heights of action for this equivalent cylinder should be applied to the real structural shape, when calculating stress levels.

C3.3.2 Flexible Tanks

Flexible Vertical Circular Cylindrical Tank

The flexibility of the tank wall increases the impulsive mode period. The impulsive coefficient at low periods should be determined using the plateau of the hazard spectrum.

Although there are an infinite number of tank-liquid modes of vibration related to the wall flexibility, it is generally only the lowest of these modes that contributes significantly to the earthquake stresses in the shell. In the lowest horizontal mode of vibration tall tanks deform in an analogous manner to a deep vertical cantilever beam, deforming in bending and shear under the action of horizontal loading. Radial deformations are small with the cross section generally remaining circular. The lowest tank-liquid mode for broad tanks is dominated by the radial deformation of the shell resulting in significant out-of-roundness of the cross section.

Tank wall flexibility also increases the period of the vertical modes of vibration. The lowest vertical mode is often referred to as a “breathing” mode and has a deformation pattern dominated by axially symmetric radial displacements.

The results given below for pressures, spring-mass analogy parameters and periods of vibration are based on the assumption that the top of the tank wall is unstiffened by ring girders or a roof structure. This assumption is expected to give satisfactory approximations for most practical cases where the top of the wall is stiffened.

Convective Pressure Components

In the analysis of flexible tanks, it is found that the tank flexibility has only a small effect on the convective components of the hydrodynamic pressures, and for design purposes they can be assumed to be the same as for rigid tanks.

Impulsive Pressure Components

The impulsive components may be significantly increased by tank wall or foundation flexibility, and it is important to allow for these effects in design.

For flexible tanks, with H/R ratios in the range of 0.25 to 1.5, the maximum impulsive component of hydrodynamic pressure on the tank wall can be approximated by:

$$p_i(z, \theta) = q_0(z)\ C_d(\check{T}_f)\ \gamma_\ell\ R_m \cos\theta \qquad \text{(C3.22)}$$

where

$q_0(z)$ = dimensionless pressure function for the equivalent rigid tank

$C_d(\check{T}_f)$ = horizontal design action coefficient for the first mode of the tank-liquid system

$\check{T}_f$ = period of vibration of first tank-liquid mode including the effects of soil structure interaction

R_m = mean radius of tank shell

There is no simple expression for impulsive pressures where H/R is greater than about 1.5. In these instances, Equation C3.22 gives conservative values for the pressures below mid-height and under estimates the pressures above mid-height. Typical impulsive pressure distributions for the first mode of a flexible tank are compared with rigid tank impulsive pressures in Figure C3.16 for a range of H/R ratios.

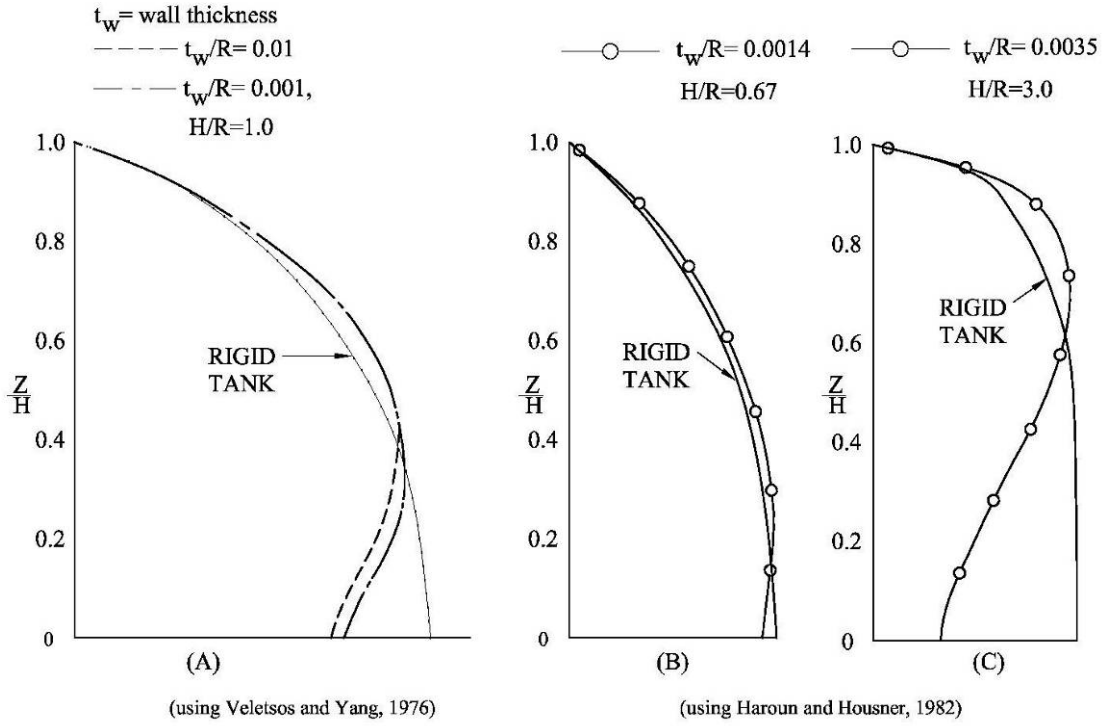


Figure C3.16: First Mode Impulsive Pressure Distributions for Flexible Tanks

Pressures arising from the higher modes of the flexible tank will reduce the difference between the first mode pressures and the rigid tank impulsive pressures. Thus, for most practical purposes, the rigid tank impulsive pressure distribution can be used to provide a conservative approximation for the flexible tank case.

The spring-mass analogy described below provides a good approximation for the practical range of H/R ratios (including values exceeding 1.5) and generally provides sufficient information for design purposes.

Spring-Mass Analogy

The influence of tank flexibility on the base shears and overturning moments is conveniently analysed by using the spring-mass analogy (Figure 3.5) proposed by Haroun and Housner (1981).

The mass ratios, m_f/m_ℓ and m_0/m_ℓ, may be obtained from the results of Haroun and Housner (1981) and Haroun and Ellaithy (1985). These ratios are plotted in Figure C3.17 as functions of the geometric ratio H/R_m.

The impulsive mass components are dependent on the ratio of the wall thickness to radius ratio and the ratio of the wall and liquid unit weights. However, these parameters are of secondary importance and for practical design purposes their influence may be neglected. Note that the impulsive mass component, m_0 shown in Figure C3.17 is identical to the rigid tank impulsive mass shown in Figure C3.8.

The effective heights at which the two masses act, for the case excluding base pressure, are plotted in Figure C3.18. Heights for the case including base pressure are given in Figure C3.19. As for the case of the mass components the effective heights are functions of other parameters that may be neglected for practical design purposes.

The base shears from the three mass components are given by Equations 3.9, 3.10 and 3.11 in Section 3.5.

Pressure From Vertical Accelerations

The pressure from the vertical acceleration component, $P_v(z)$, is uniformly distributed in the circumferential direction and is given approximately by:

$$p_v(z) = (1 - z/H)\ C_d(\check{T}_b)\ \gamma_\ell\ H \qquad \text{(C3.23)}$$

where

$C_d(\check{T}_b)$ = vertical design action coefficient for the first vertical (breathing) mode of vibration including scale factor of 0.7.

$\check{T}_b$ = period of vibration of first vertical (breathing) mode of vibration including soil-structure interaction.

For evaluating $C_d(\check{T}_b)$

μ = ductility factor for vertical modes = 1.0

ξ_v = damping level for the vertical mode (see Figure 3.2)

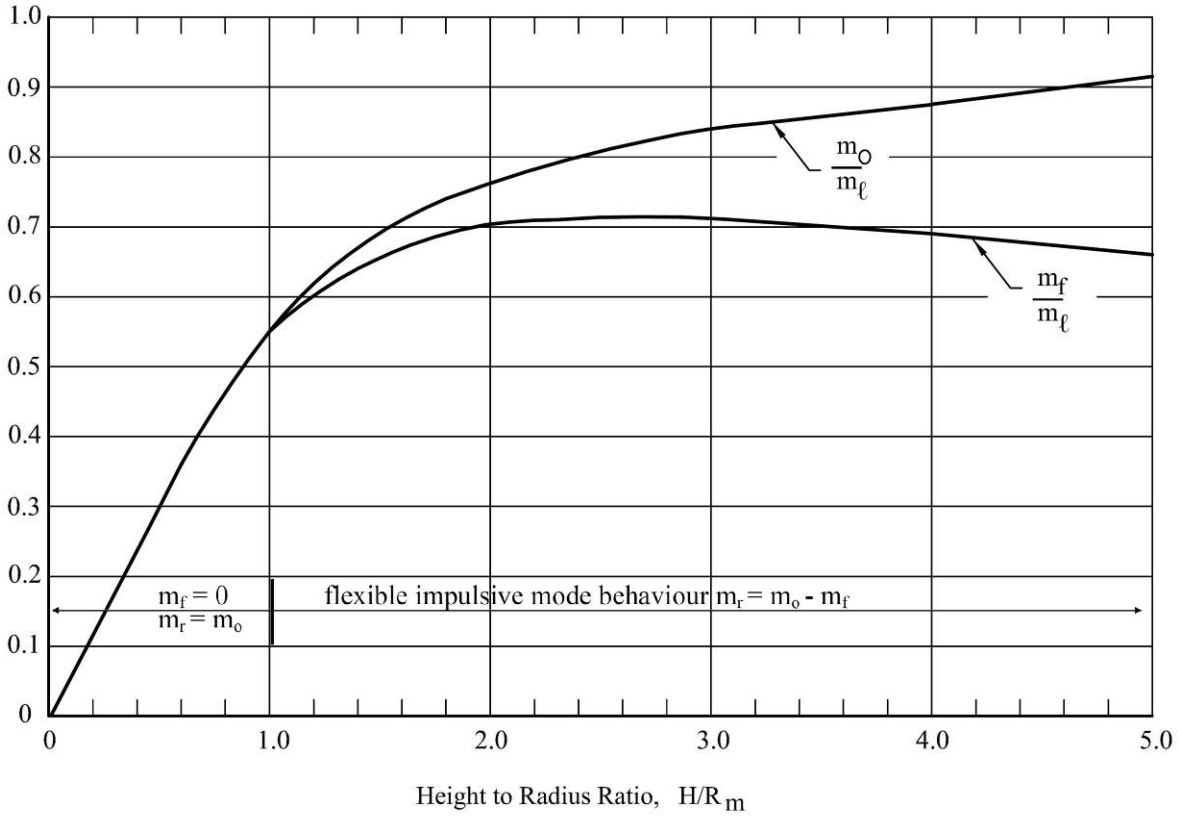


Figure C3.17: Impulsive Mass Components for Flexible Tanks

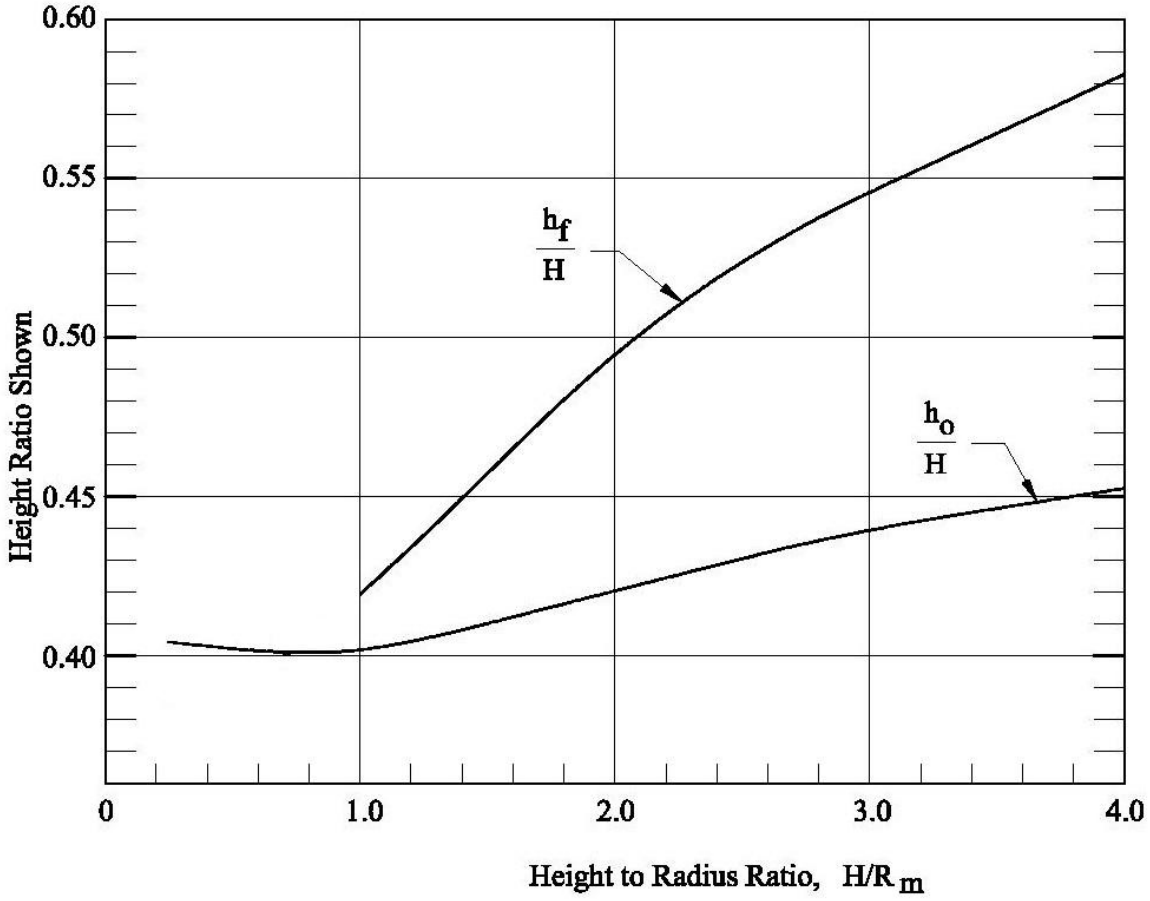


Figure C3.18: Heights of Impulsive Masses for Walls of Flexible Tanks, (Haroun and Housner, 1981)

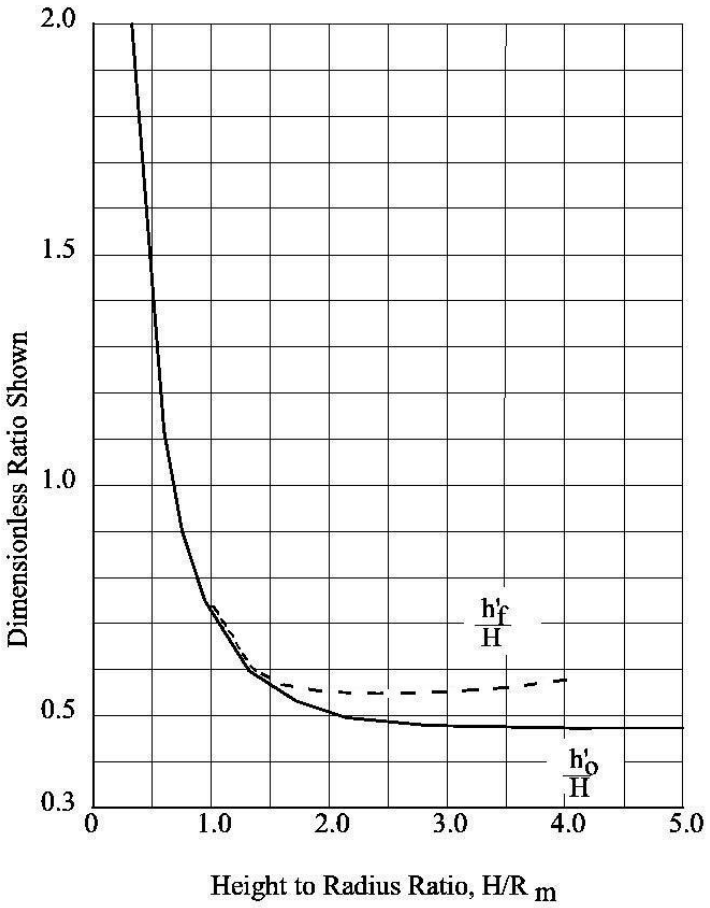


Figure C3.19: Heights of Impulsive Masses for Flexible Tanks (Including Pressure on Base), (Haroun and Ellaithy, 1985)

Flexible Rectangular Tank

Convective Pressure Components

The tank flexibility has no significant influence on the convective pressure component and thus the rigid tank pressures can be used for design.

Impulsive Pressure Component

The impulsive pressure component is influenced by the wall flexibility but because there are no analytical solutions for this case it is recommended that the rigid tank distributions be used. The order of error in this approximation is unknown but it is thought that it is a reasonable approximation for relatively rigid walls where H/L is less than 2.0.

Spring-Mass Analogy

The spring-mass analogy described previously for the flexible cylindrical tank may be used for the rectangular case. However, because of the lack of analytical solutions it is not possible to accurately define the two impulsive masses m_f and m_r. To obtain an approximate solution it is recommended that m_f be taken as equal to m_0, giving $m_r = 0$. Approximate values of the equivalent masses and their height above the base can be found from the plotted values for the rigid vertical cylindrical tank, with the half length, L, of the rectangular tank used instead of the circular radius R.

C3.3.5 Granular Contents or Full and Sealed Tanks

The assumption of the total mass of the contents being rigidly connected to the tank shell gives a reasonably good approximation to the total horizontal earthquake force when the H/R ratio is greater than 1.0. It gives conservative results for H/R less than 1.0. A more exact approximation for the force, and details of the pressures from granular materials are given by Trahair et al (1983). Tanks that are full with no air void and sealed such that contents cannot spill over or out of openings are unable to form a convective mode response and the total liquid mass acts in the impulsive mode.

C3.5 Base Shears and Overturning Moments

Equations 3.9, 3.10, 3.11, and 3.12, 3.13, 3.14, give the base shears and moments respectively from the three mass components of the spring-mass analogy for a flexible tank.

Only the first mode impulsive mass components are used in the flexible tank spring-mass analogy described in Section C3.3 above. The contributions of higher modes to the base moments are generally small for most practical cases. For relatively slender tanks, the higher modes may add a significant contribution to the base shear. However for H/R ratios up to 3.0 the higher impulsive mode contributions to the total base shear are unlikely to exceed 10%.

As a consequence of neglecting the higher mode effects, a conservative approach should be adopted for combining the components to estimate the maximum total base shear and moments. It is recommended that the impulsive mass components be combined by taking the absolute sum and that the square root of the sum of the squares rule be used for adding the total impulsive component to the convective component (see Section 4).

Equations C3.9 to C3.14 give the base shears and moments from the two mass components (impulsive and convective) of the spring-mass analogy for a rigid tank.

As for a flexible tank, an estimate of the maximum base shear and moments from the combined impulsive and convective pressures for a rigid tank can be obtained by using the square root of the sum of the squares rule as given in Section 4.

C3.6 Natural Periods of Vibration

Rigid Vertical Circular Cylindrical Tank

The period of vibration of the ith convective or sloshing mode is given by:

$$T_i = \frac{2\pi\sqrt{\frac{R}{g}}}{\sqrt{\lambda_i \tanh\left(\lambda_i \frac{H}{R}\right)}} \qquad \text{(C3.24)}$$

where

g = acceleration of gravity

λ_i = 1.841, 5.331, 8.536, … for i = 1, 2, 3, … respectively

The dimensionless period, $T\sqrt{g/R}$, is plotted for the first two modes in Figure C3.20.

Rigid Rectangular Tank

The period of vibration of the ith convective or sloshing mode is given by:

$$T_i = \frac{2\pi\sqrt{\frac{L}{g}}}{\sqrt{\alpha_i \tanh\left(\alpha_i \frac{H}{L}\right)}} \qquad \text{(C3.25)}$$

where

α_i = $\pi/2, 3\pi/2, 5\pi/2, \ldots$ For i = 1,2,3…. respectively

The dimensionless period, $T\sqrt{g/L}$, for the first two modes of vibration is plotted in Figure C3.20.

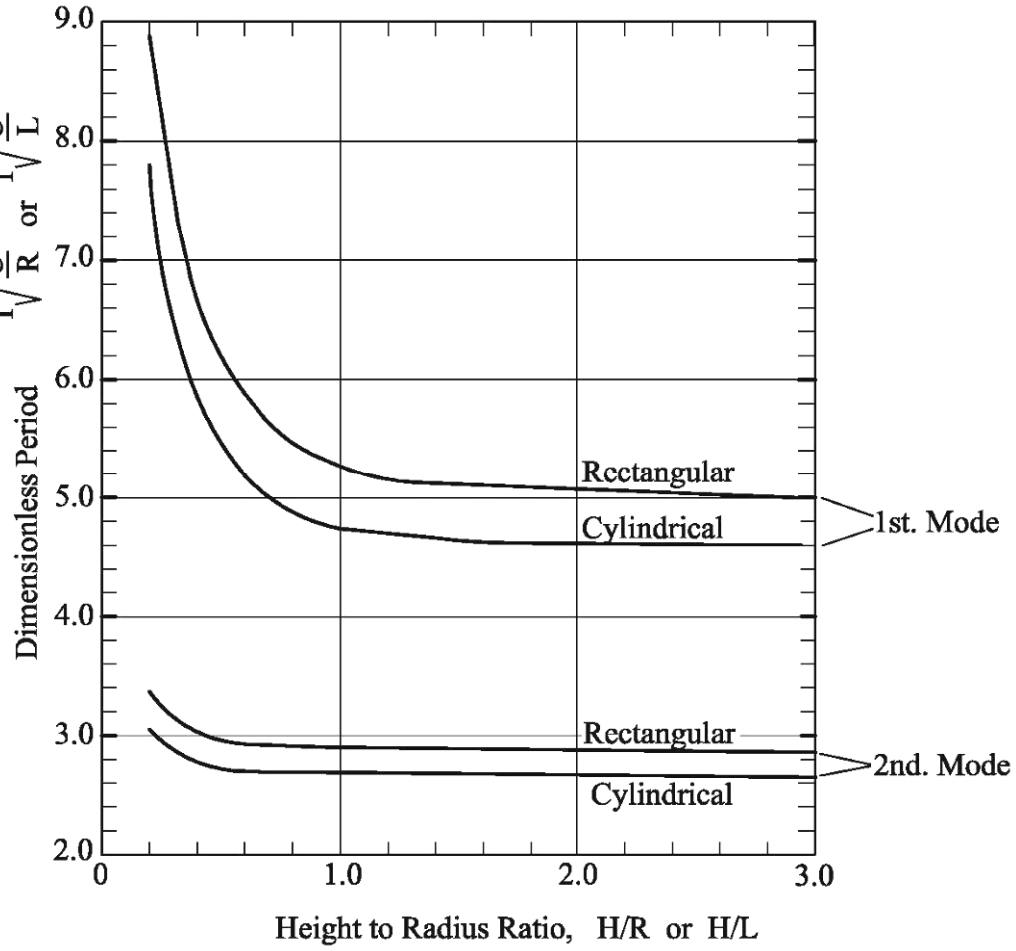


Figure C3.20: Dimensionless Periods for Convective Modes

Other Rigid Tanks

Earthquake induced hydrodynamic pressure distributions for tanks of shapes other than vertical circular cylindrical and rectangular have not been published in suitable form for design. Approximate analyses for tanks of other shapes can be undertaken using the results given previously for the vertical circular cylindrical and rectangular tanks. It is helpful in these analyses to have an approximate estimate of the first sloshing mode frequency or period. In Figure C3.21 the first sloshing mode dimensionless frequency, $f_1\sqrt{R/g}$ or $f_1\sqrt{L/g}$, is compared for the following shapes:

- Vertical circular cylinder
- Vertical circular cylinder with spherical bottom
- Horizontal circular cylinder, motion along axis

- Horizontal circular cylinder, motion transverse to axis
- Rectangular
- Spherical

Most of the information plotted in Figure C3.21 has been obtained from Abramson (1966). Results of analytical and experimental studies for more complex shapes are also given in this reference.

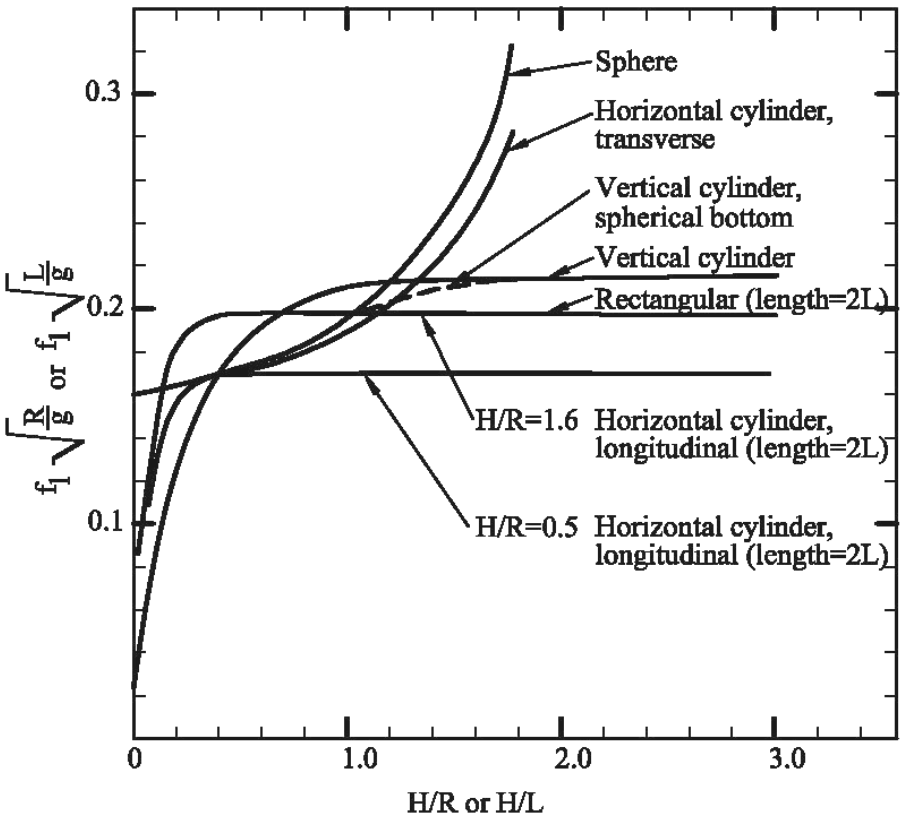


Figure C3.21: Dimensionless First Convective Mode Frequency for Rigid Tanks of Various Shapes

Rigid Vertical Circular Cylindrical and Rectangular Tanks

The horizontal and vertical periods of vibration for rigid tanks are derived from the flexible tank periods of vibration including the soil-structure interaction. Equations C3.30 and C3.33 are used to determine the rigid tank periods. In the use of Equation C3.30 to give the impulsive mode period m_r is replaced with $m_0 + m_w + m_t$ and h_r with h_0.

Flexible Vertical Circular Cylindrical Tank

Horizontal Mode

The period of vibration of the first impulsive tank-liquid horizontal mode is given by:

$$T_f = \frac{5.61\pi H}{k_h}\sqrt{\frac{\gamma_\ell}{Eg}} \qquad \text{(C3.26)}$$

where

k_h = period coefficient as given in Figure C3.22

γ_ℓ = unit weight of contained liquid

E = Young's modulus for tank material

Equation C3.26 gives the first tank-liquid horizontal mode period for a roofless tank with uniform wall thickness, t_w, Poisson's ratio of 0.3, and full with liquid. The equation was developed for the case of a steel tank filled with water but may be used for other tank materials and liquids when the mass of the tank is relatively small in comparison to the mass of the liquid. It is likely to give a good approximation for most practical cases.

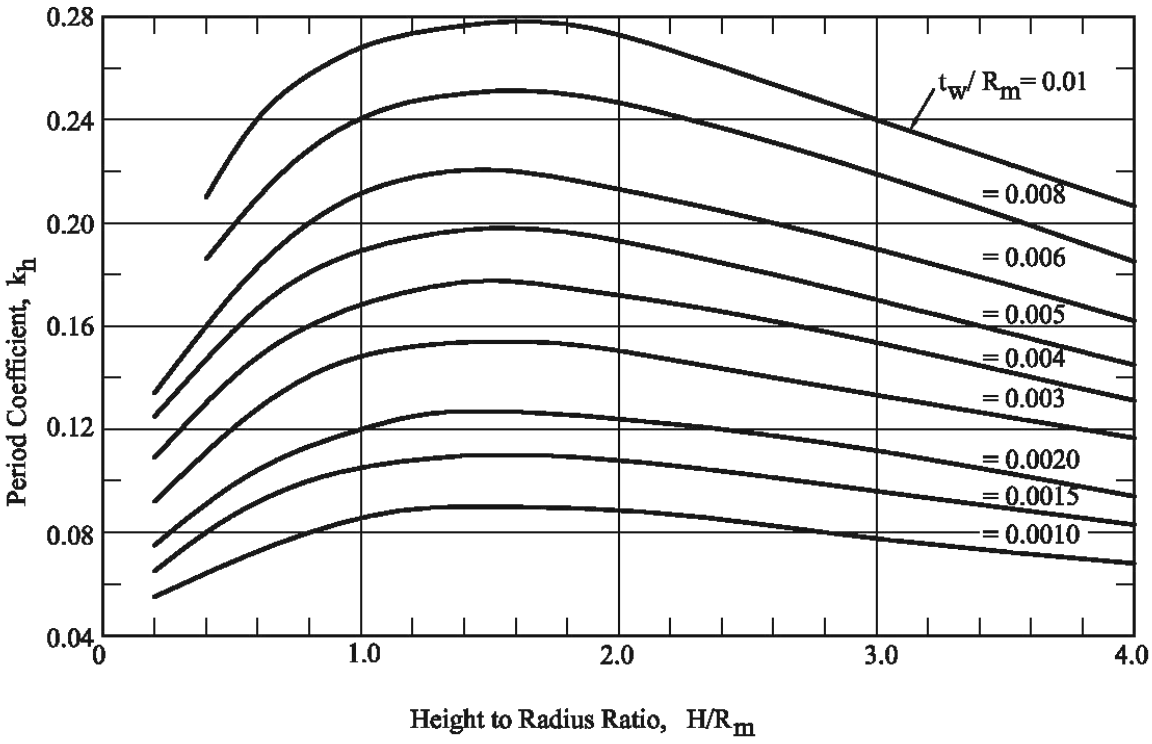


Figure C3.22: Period Coefficient k_h for First Horizontal Tank - Liquid Mode, (Haroun and Housner, 1981)

The first mode period of tanks of non-uniform wall thickness may be determined by using the average thickness. This will give a reasonably good approximation up to H/R ratios of about 4.0. Above this value the flexural deformation dominates the response and a more exact period may be obtained by using an average of the wall thickness over the lower half of the tank.

The influence of the roof mass on the period may be obtained using Dunkerley's method as outlined by Veletsos (1984). In most practical applications the roof mass may be neglected.

Vertical Mode

The period of vibration of the first tank-liquid vertical (breathing) mode of vibration is given by:

$$T_b = \frac{5.61\pi H}{k_v}\sqrt{\frac{\gamma_\ell}{Eg}} \qquad \text{(C3.27)}$$

where

k_v = period coefficient defined in Figure C3.23

γ_ℓ = unit weight of contained liquid

E = Young's modulus for tank material

Equation C3.27 gives the first tank-liquid vertical mode period for a roofless tank with uniform wall thickness, Poisson's ratio of 0.3, and full with liquid. As for the case of equation C3.26 for the horizontal period, it has been developed for a steel tank filled with water but gives good approximations for other materials and liquids when the mass of the tank is relatively small in comparison to the mass of the liquid.

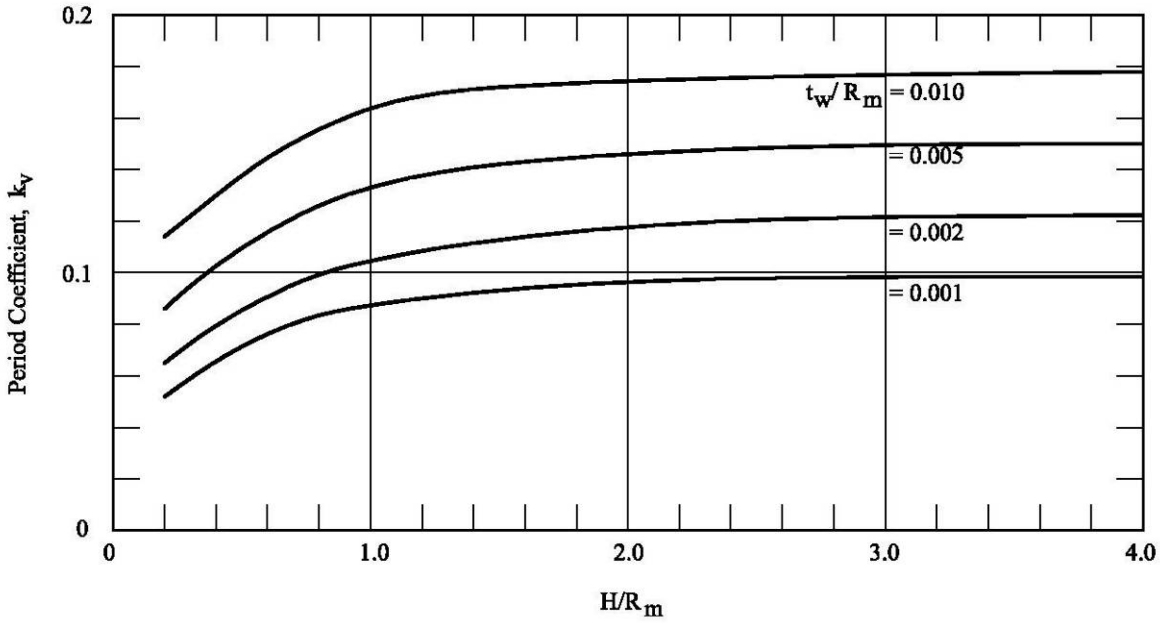


Figure C3.23: Period Coefficient k_v for First Vertical Tank - Liquid Mode, (Veletsos, 1984)

Flexible Rectangular Tank

Horizontal Mode

The period of vibration of the first impulsive tank-liquid horizontal mode is given approximately by:

$$T_f = 2\pi\sqrt{\frac{d_f}{g}} \qquad \text{(C3.28)}$$

where

d_f = deflection of the tank wall on the vertical centre-line at the height of the impulsive mass, m_f, when loaded by a uniformly distributed load in the direction of ground motion of magnitude $m_f g/4BH$

2B = tank width perpendicular to direction of loading.

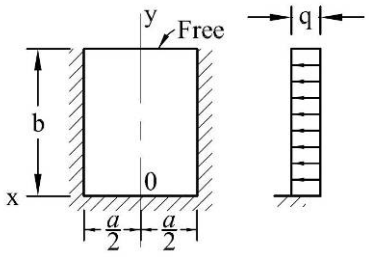
The impulsive mass, m_f, can be obtained from the equivalent cylindrical tank results and should include the wall mass.

For tanks without roofs the deflection, d_f, may be calculated assuming the wall to be free at the top and fixed on the other three sides. Table C3.3, reproduced from Timoshenko and Woinowsky-Krieger (1959), can be used to calculate d_f for this case.

Table C3.3: Deflections, Bending Moments, and Reactions of Uniformly Loaded Rectangular Plate With Three Edges Built in and Fourth Edge Free, (Timoshenko and Woinowsky-Krieger, 1959)

$\nu = \frac{1}{6}$

$\frac{b}{a}$	$x=0, y=b$		$x=0, y=b/2$			$x=a/2, y=b$		$x=a/2, y=b/2$		$x=0, y=0$	
	$w = \alpha_1 \frac{qa^4}{D}$ α_1	$M_x = \beta_1 qa^2$ β_1	$w = \alpha_2 \frac{qa^4}{D}$ α_2	$M_x = \beta_2 qa^2$ β_2	$M_y = \beta^-_2 qa^2$ β^-_2	$M_x = \beta_3 qa^2$ β_3	$V_x = \gamma_3 qa$ γ_3	$M_x = \beta_4 qa^2$ β_4	$V_x = \gamma_4 qa$ γ_4	$M_y = \beta_5 qa^2$ β_5	$V_y = \gamma_5 qa$ γ_5
0.6	0.00271	0.0336	0.00129	0.0168	0.0074	-0.0745	0.750	-0.0365	0.297	-0.0554	0.416
0.7	0.00292	0.0371	0.00159	0.0212	0.0097	-0.0782	0.717	-0.0439	0.346	-0.0545	0.413
0.8	0.00308	0.0401	0.00185	0.0252	0.0116	-0.0812	0.685	-0.0505	0.385	-0.0535	0.410
0.9	0.00323	0.0425	0.00209	0.0287	0.0129	-0.0836	0.656	-0.0563	0.414	-0.0523	0.406
1.0	0.00333	0.0444	0.00230	0.0317	0.0138	-0.0853	0.628	-0.0614	0.435	-0.0510	0.401
1.25	0.00345	0.0467	0.00269	0.0374	0.0142	-0.0867	0.570	-0.0708	0.475	-0.0470	0.388
1.5	0.00335	0.0454	0.00290	0.0402	0.0118	-0.0842	0.527	-0.0755	0.491	-0.0418	0.373



$$D = \frac{E\, t_w^3}{12(1-\nu^2)}$$

w = plate deflection

Vertical Mode

There is no simple method for calculating the periods of the vertical modes of vibration for a flexible rectangular tank. For this case, use should be made of the results for the equivalent cylindrical tank.

Soil Structure Interaction

Because of soil-structure interaction effects, a stiff tank resting on soft soil may have an earthquake response significantly different from the case of a tank on a rigid foundation. Soil-structure interaction may lengthen the period of vibration of the impulsive modes and increase the damping. Generally an increase in the impulsive mode periods will result in an increase in response, but the additional damping will tend to counteract this effect.

Strictly speaking soil-structure interaction will change both the impulsive and convective periods but the effects on the convective modes are small and can be neglected.

Jennings and Bielak (1973) showed that for single-storey buildings, soil-structure interaction under earthquake loads could be conveniently considered by defining an equivalent single degree-of-freedom system. The method can be readily extended to the case of tanks directly supported on the ground. Although the original work was for the case of a structure on a rigid foundation base resting on elastic soil, the method was further developed and extended to include the effects of foundation embedment and material damping in the soil (Bielak, 1975 and 1976).

The theory formulated by Jennings and Bielak has been simplified by Veletsos (1984) and presented with a notation more suitable for design applications. A summary of this simplified approach is given below.

Guidance on appropriate values of shear wave velocities is given in the commentary to NZS 1170.5 (SNZ, 2004) for the general site subsoil classes.

Horizontal Periods of Tank-Foundation System

The periods of vibration for the first two impulsive modes of the tank-foundation system are given by:

$$\breve{T}_f = T_f\sqrt{1+\frac{K_f}{K_x}\left[1+\frac{K_x h_f^2}{K_\theta}\right]} \qquad \text{(C3.29)}$$

$$\breve{T}_0 = 2\pi\sqrt{\frac{m_r+m_b}{K_x}+\frac{m_r h_r^2}{K_\theta}} \qquad \text{(C3.30)}$$

where

$K_f = \dfrac{4\pi^2(m_f+m_w+m_t)}{T_f^2}$ the effective stiffness of the tank-liquid system

$K_x = \dfrac{8}{2-\upsilon_s}G_s R_b \alpha_x$ the horizontal translational stiffness of the foundation

$K_\theta = \dfrac{8}{3(1-\upsilon_s)}G_s R_b^3 \alpha_\theta$ the rocking stiffness of the foundation

υ_s = Poisson's ratio for the soil

G_s = shear modulus for the soil

R_b = radius of the foundation

α_x, α_θ = dimensionless factors that convert the static stiffness values to dynamic values

h_f = height of impulsive mass, m_f, above the tank foundation

h_r = height of impulsive mass, m_r, above the tank foundation, using the value of h_0

m_b = mass of base

Equation C3.29 gives the period of the first tank-liquid soil-structure interaction mode of the spring-mass analogy. Equation C3.30 gives an approximation to the second mode period for the flexible tank on a flexible foundation. In most practical cases m_r and m_b are small and $\breve{T}_0$ can be taken as zero.

For a rigid tank $T_f = 0$ and K_f becomes infinite. For this case, Equation C3.30 may be used to give the first mode period by replacing m_r with $m_0 + m_w + m_t$ and h_r with h_0.

The dimensionless factors α_x and α_θ are plotted in Figure C3.24 as a function of the frequency parameter, a defined by:

$$a = \frac{2\pi R_b}{\breve{T}_i V_s} \qquad \text{(C3.31)}$$

where

$V_s = \sqrt{\dfrac{G_s}{\rho_s}}$ the shear wave velocity of the soil

ρ_s = the soil mass density

$\breve{T}_i$ = $\breve{T}_f$ or $\breve{T}_0$ from Equations C3.29 and C3.30

For most design applications the factors α_x and α_θ may be taken as 1.0, that is the foundation stiffnesses may be taken as the static values. However, for large tanks on soft soils α_θ may be significantly less than 1.0 for the impulsive modes of vibration.

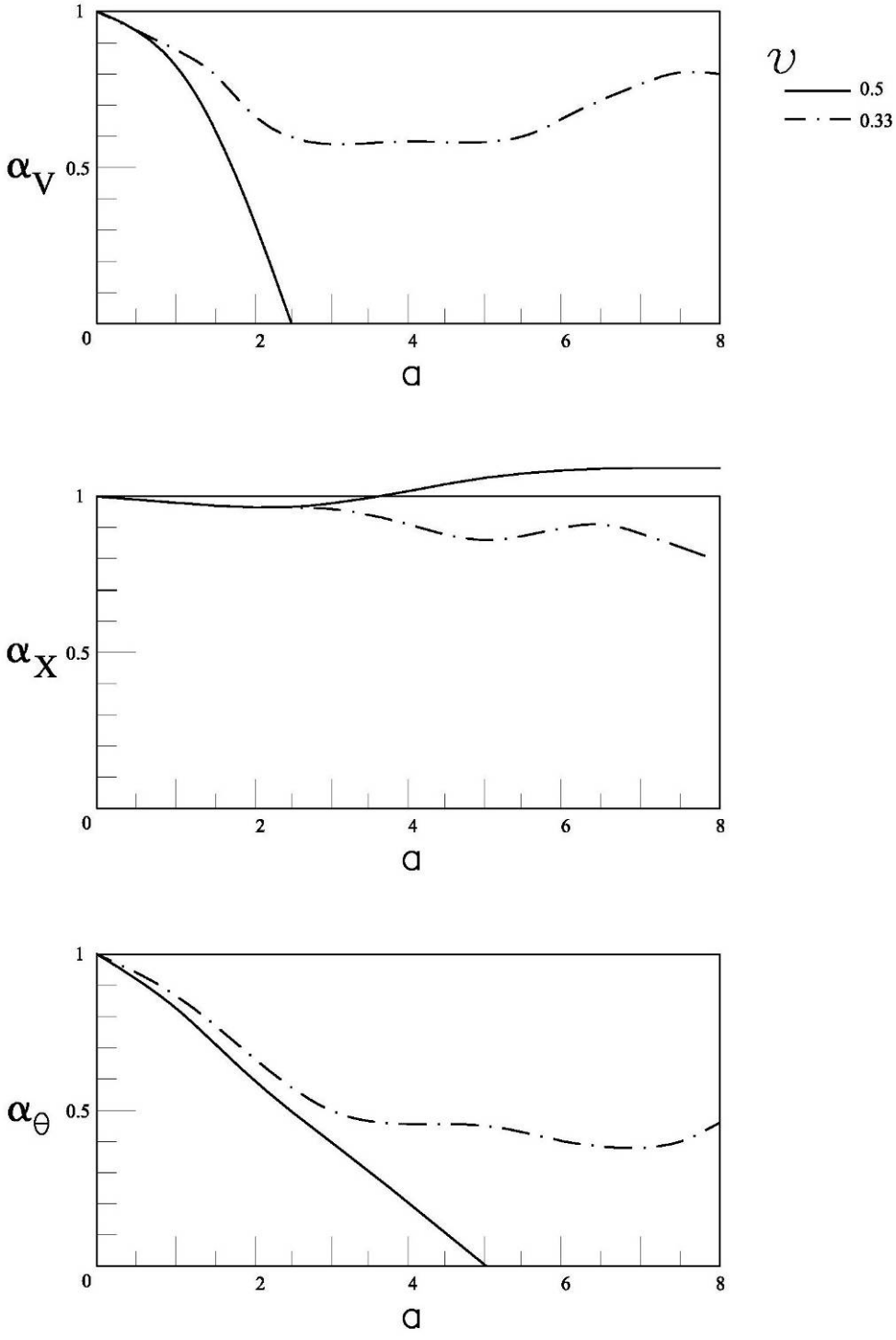


Figure C3.24: Period Coefficients for Response of Tank - Foundation System, (Gazetas, 1983)

Earthquake Response

The earthquake response of the equivalent single degree-of-freedom oscillators, with periods of vibration T_f and T_0, representing the impulsive modes of tank-foundation system can be obtained from the previously given acceleration response spectra.

Vertical Periods of Tank-Foundation System

The vertical period of vibration of a flexible tank including soil-structure interaction is given by:

$$\check{T}_b = T_b\sqrt{1+\frac{K_b}{K_v}} \qquad \text{(C3.32)}$$

where

$$K_b = \frac{4\pi^2 m_\ell}{T_b^2}$$

$$K_v = \frac{4}{1-\upsilon_s} G_s R_b \alpha_v$$

α_v = dimensionless factor to convert the static stiffness value to the dynamic value

The vertical period of vibration of a rigid tank including soil-structure interaction is given by:

$$\check{T}_b = 2\pi\sqrt{\frac{m_v}{K_v}} \qquad \text{(C3.33)}$$

where

$m_v = m_\ell + m_s$

m_s = total mass of tank including base, support system and structural foundation

The dimensionless factor α_v is plotted in Figure C3.24.

The equation for the period of the flexible tank, (C3.32), has been simplified by neglecting the mass of the tank base and any structural foundation. To include these masses requires a two mass spring-mass analogy which adds complexity that is unnecessary for many design applications. If the base and foundation are unusually heavy a more rigorous analysis may be justified.

An alternative formulation for the horizontal and vertical periods including the soil-structure interaction is presented by Veletsos and Shivakumar (1997).

C3.7 Damping

The recommended damping values for impulsive and vertical modes are given in Figures 3.1 and 3.2. The horizontal mode damping values were calculated using a procedure first developed by Veletsos and Tang (1990) for steel storage tanks. In their simplified method an equivalent (fixed base) single degree of freedom (SDOF) oscillator is used to represent the impulsive mode of a flexible tank structure interacting with a flexible soil foundation. The period of vibration and damping of the equivalent SDOF system is derived by firstly computing the response of the tank and flexible foundation system to harmonic forcing. The SDOF properties (period and damping) are then obtained by finding the best fit to the transfer function of the flexible base system.

The harmonic forcing transfer function for the soil-structure system was evaluated using the solutions given by Jennings and Bielak (1973) for a single storey structure.

Soil-structure interaction has negligible effect on the convective mode response. This response component can be calculated in the conventional manner for a fixed base tank.

The structural foundation of the tank was assumed to be a massless rigid disk resting on a homogenous visco-elastic half-space. Impedance functions were calculated using the approximate closed-form solutions of Veletsos and Verbic (1973).

The hydrodynamic interaction of the fluid on the tank base was incorporated in the analysis using the equivalent rotary inertia coefficients given by Veletsos and Tang (1990).

The elastic foundation soil was assumed to have the following material constants:

Hysteretic damping coefficient	0.05
Poisson's ratio	0.333
Density, ρ_s	2.0 t/m^3

The equivalent SDOF properties were derived for a range of ratios of the structure to soil stiffness. The soil stiffness was defined using the elastic shear modulus, G_s. The shear modulus is related to the shear wave velocity, V_s, by $G_s = \rho_s V_s^2$.

The tank wall was assumed to be elastic with the following material properties:

Viscous damping coefficient	0.02
Poisson's ratio	0.2 for concrete
	0.3 for steel
Density, ρ_w	2.45 t/m^3 for concrete
	7.85 t/m^3 for steel

Wall structural stiffness parameters used in the analysis were Young's modulus, E_w, and the wall thickness to radius ratio, t/R_w.

The liquid in the tank was assumed to be incompressible, inviscid and free at its upper surface. The liquid density, ρ, was taken as 1.0 t/m^3.

The vertical mode damping values in Figure 3.2 have been taken directly from Veletsos and Tang (1986). They used an analysis procedure similar to that described above for the horizontal mode damping values. No hysteretic soil damping was included by Veletsos and Tang in their analyses so values greater than the plotted values are likely.

Damping values can also be calculated using the following approximate equations from work of Bielak (1976) and Veletsos (1984).

The effective damping ratio of the tank-foundation system in the flexible mass impulsive mode is given by:

$$\xi_f = \xi_s + \frac{\xi_m}{\left(\frac{\check{T}_f}{T_f}\right)^3} \qquad \text{(C3.34)}$$

where

ξ_s = foundation damping including soil material and radiation damping

ξ_m = material damping in the tank system above the foundation for the impulsive mode

The foundation damping factor is given by:

$$\xi_s = \frac{2\pi^2 m_f a}{K_x \check{T}_f^2}\left[\frac{\beta_x}{\alpha_x} + \frac{K_x h_f^2 \beta_\theta}{K_\theta \alpha_\theta}\right] \qquad \text{(C3.35)}$$

where

β_x, β_θ = dimensionless factors for translation and rocking damping respectively

Values of β_x and β_θ are plotted as a function of the frequency ratio, a, in Figure C3.25.

These damping factors only include the effects of radiation damping and not the soil material damping. Damping factors that include both radiation and soil material damping can be obtained from Veletsos and Verbic (1973) and Veletsos and Nair (1975).

The soil damping for the case of a rigid rank can also be obtained from Equation C3.35 by putting $m_f = m_0$ and $h_f = h_0$.

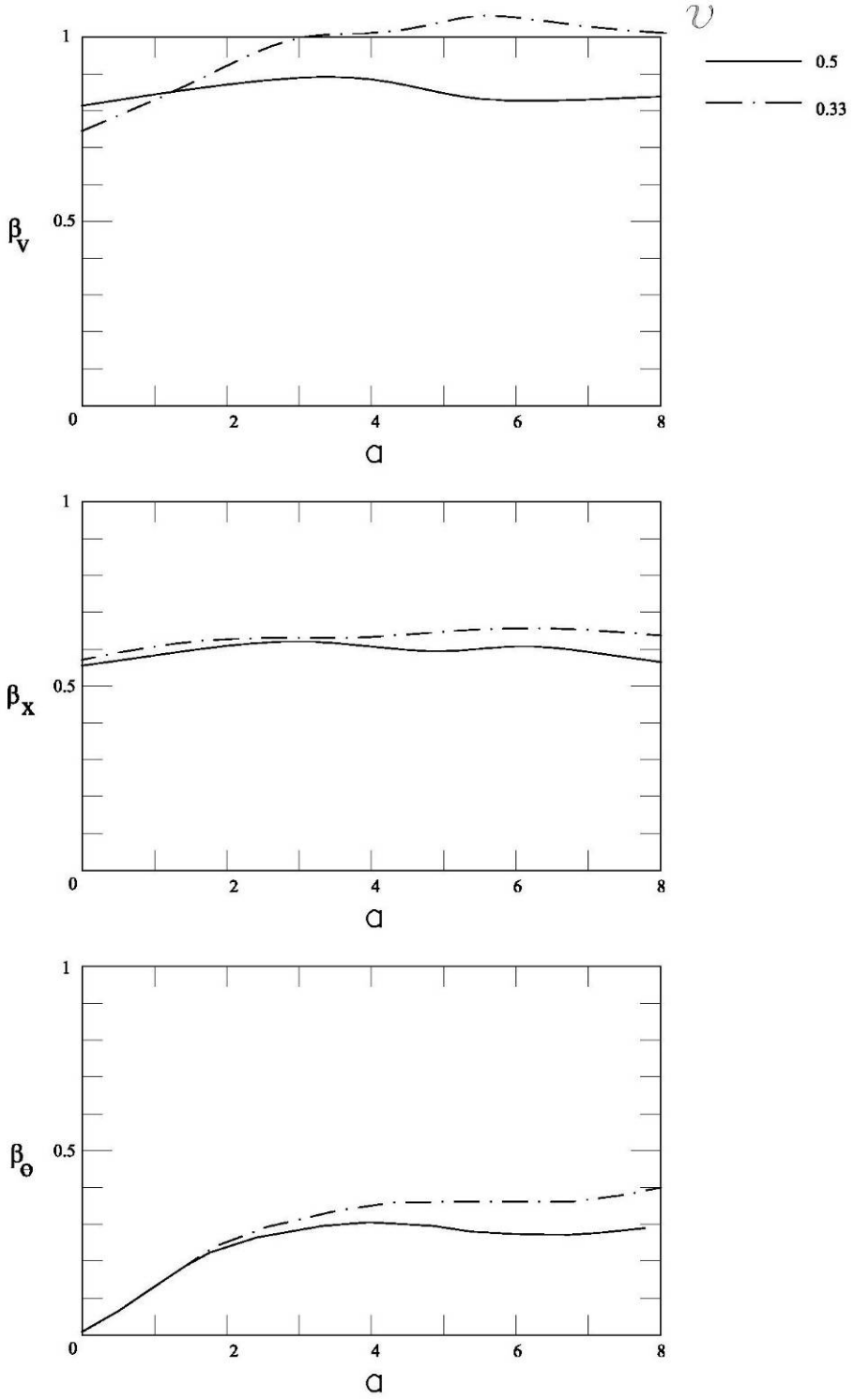


Figure C3.25: Damping Coefficients for Response of Tank - Foundation System, (Gazetas, 1983)

C3.9 Convective Wave Height

Vertical Circular Cylindrical Tanks

The maximum vertical displacement of the convective sloshing wave from the at-rest level of the liquid is given by:

$$d_{max} = R\sqrt{[0.84C_d(T_1)]^2 + [0.07C_d(T_2)]^2} \quad \text{(C3.36)}$$

where

R = Radius of tank

$C_d(T_1)$ = First convective mode horizontal design action coefficient calculated using a ductility factor $\mu = 1$ and damping $\xi = 0.5\%$

$C_d(T_2)$ = Second convective mode horizontal design action coefficient calculated using a ductility factor $\mu = 1$ and damping $\xi = 0.5\%$

The above equation is based on the assumption that the vertical liquid displacement can be calculated approximately by adding the first two sloshing mode components using the square root of the sum of the squares rule.

Rectangular Tanks

The maximum vertical displacement of the convective sloshing wave from the at-rest level of the liquid for rectangular tanks is given by:

$$d_{max} = L\ C_d(T_1) \quad \text{(C3.37)}$$

This equation only includes the first mode displacement but generally higher mode effects can be neglected.

Roof Pressures

If the freeboard is less than the height of the convective (sloshing) waves, then hydrodynamic pressures will be generated on the roof. The estimation of these pressures requires the solution of a complex non-linear wave problem that involves geometric parameters such as the slope of the roof and the height of the roof above the liquid surface.

There is little published information that provides useful design information on roof pressure loading. Some guidance can be obtained from information in Kobayashi (1980), Leonard, Garrison and Hudspeth (1981), and Malhotra (2006).

The total pressure on the roof will be the result of both wave impact loading and a varying buoyancy loading that results from the wave peak running up the roof slope. The total pressure is given by:

$$p_r = p_b + c_s \bar{u}^2 \frac{\gamma_\ell}{2g} \quad \text{(C3.38)}$$

where

p_b = buoyancy pressure

c_s = wave impact coefficient

$\bar{u}$ = wave particle velocity

An approximate graphical method of evaluating p_b is outlined below and in Figure C3.26.

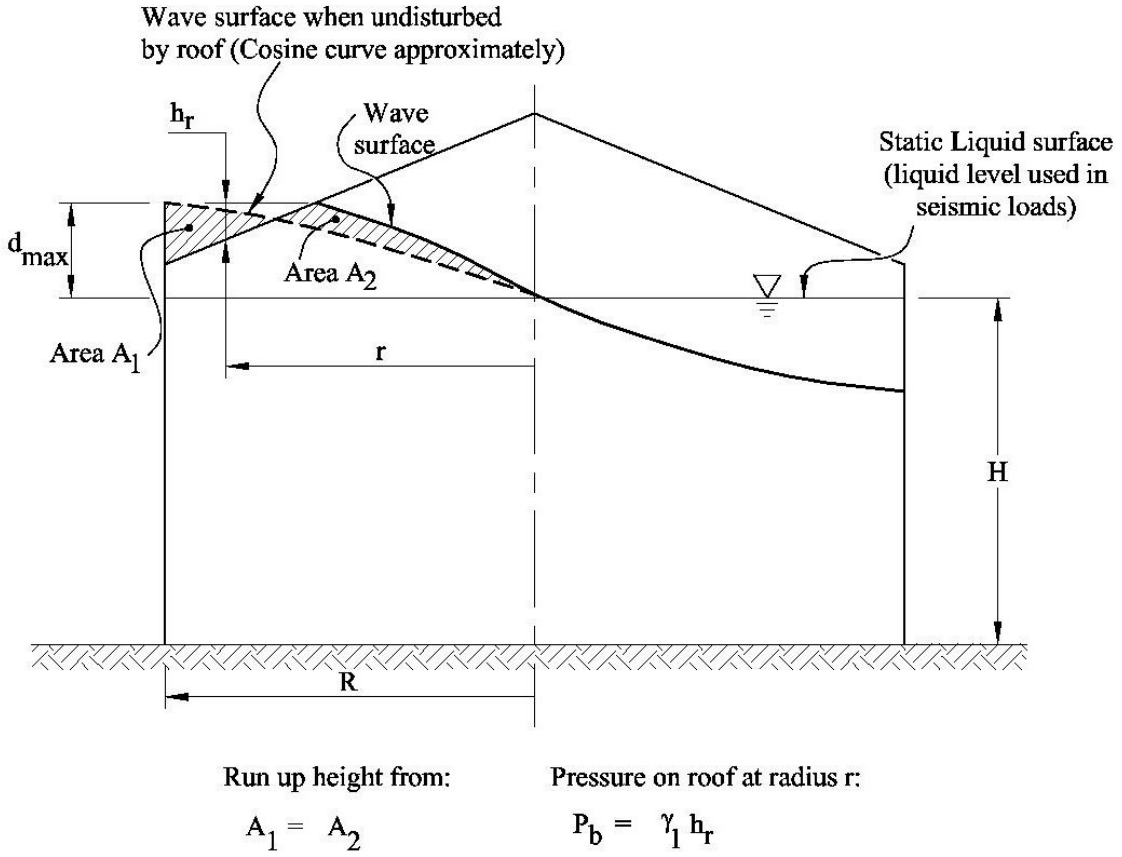


Figure C3.26: Approximate Method for Estimating Buoyancy Pressure on Tank Roof

Step 1: Determine d_{max} from equation C3.36 or C3.37 as appropriate. This is the vertical distance between the top of the theoretical cosine wave curve and the static liquid surface level used in determining seismic loads.

Step 2: Determine Area A_1 assuming it is approximately triangular.

Step 3: Modify the cosine curve such that Area A_2, is formed which is equal to Area A_1. This is a graphical process.

Step 4: Draw a line connecting the highest vertical point of A_1 and A_2.

Step 5: Determine h_r, the head of liquid at any radius r, from the vertical distance between the line just drawn and the roof level at that point.

Step 6: Determine p_b, from:

The total pressure at any given radius is given by:

$$p_b = \gamma_l h_r \quad \text{(C3.39)}$$

The value of p_b is a maximum when h_r is largest which occurs when r=R. An upper bound to c_s is about 5.0 and the approximate maximum value of $\bar{u}$ can be obtained from:

$$\bar{u} = \frac{2\pi}{T_1} d_{max} \quad \text{(C3.40)}$$

Impact of the convective wave on the tank roof can change the distribution of the liquid mass between the impulsive and convective masses. In the limiting case of the full and sealed tank with no air void the convective mass would be zero and all contents should be

assumed to be impulsive. Malhotra (2006) provides a method for adjusting the masses when the maximum vertical displacement of the sloshing wave, d_{max}, is greater than the provided freeboard, d_{free}, plus one third the roof height, h_r.

C3.10 Embedded Tanks

Soil-structure interaction effects for embedded tanks are generally complex and require special studies. Earthquake induced soil pressures can be found from rigid wall solutions given by Wood (1973). The earthquake induced pressure distribution shown in Figure C3.27 will provide a conservative estimate for most embedded tanks. The pressures shown do not include pressures arising from gravity forces in the soil.

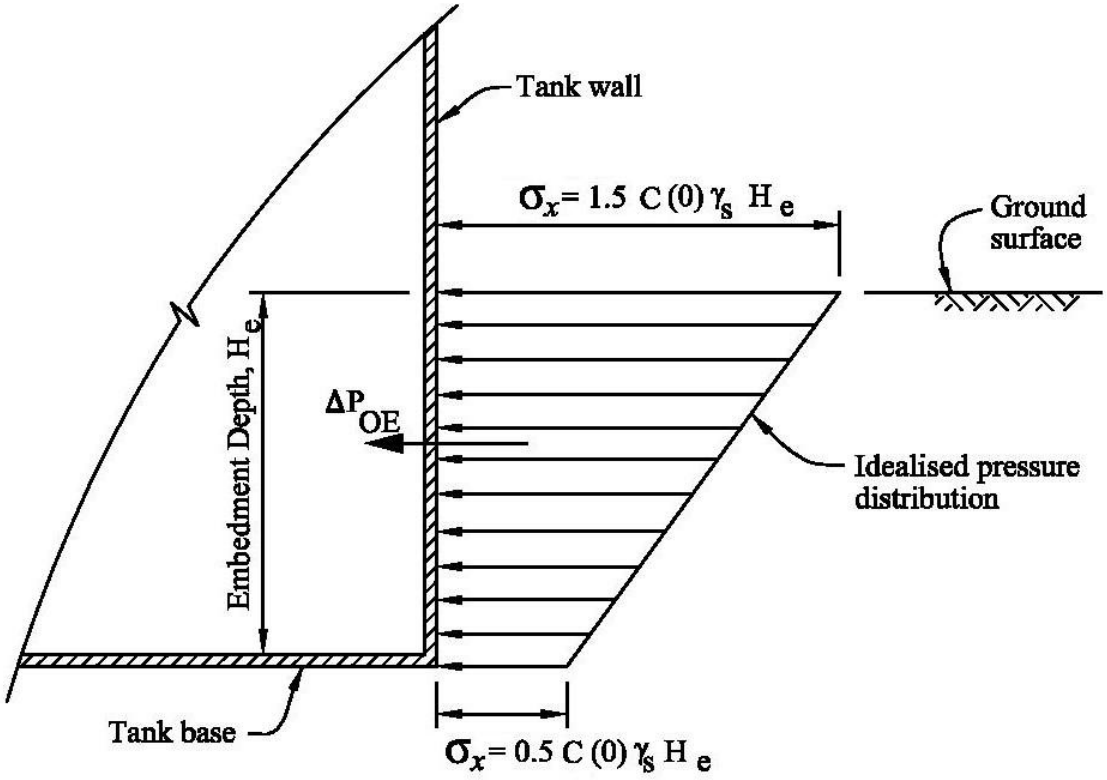


Figure C3.27: Component of Earthquake Soil Pressure on Embedded Tank Wall

In Figure C3.27 the increment in at-rest earth force due to earthquake, ΔP_{OE}, is given by

$$\Delta P_{OE} = C(0)\gamma_s H_e^2 \qquad \text{(C3.41)}$$

where

γ_s = unit weight of soil

H_e = embedment depth

σ_x = increment in at-rest soil pressure due to earthquake.

$C(0)$ = the site hazard coefficient for T=0 using the modal response spectrum from NZS 1170.5

C3.11 Elevated Tanks

A satisfactory method of analysing most elevated tanks is to use the two mass model suggested by Housner (1963) and shown in Figure C3.28.

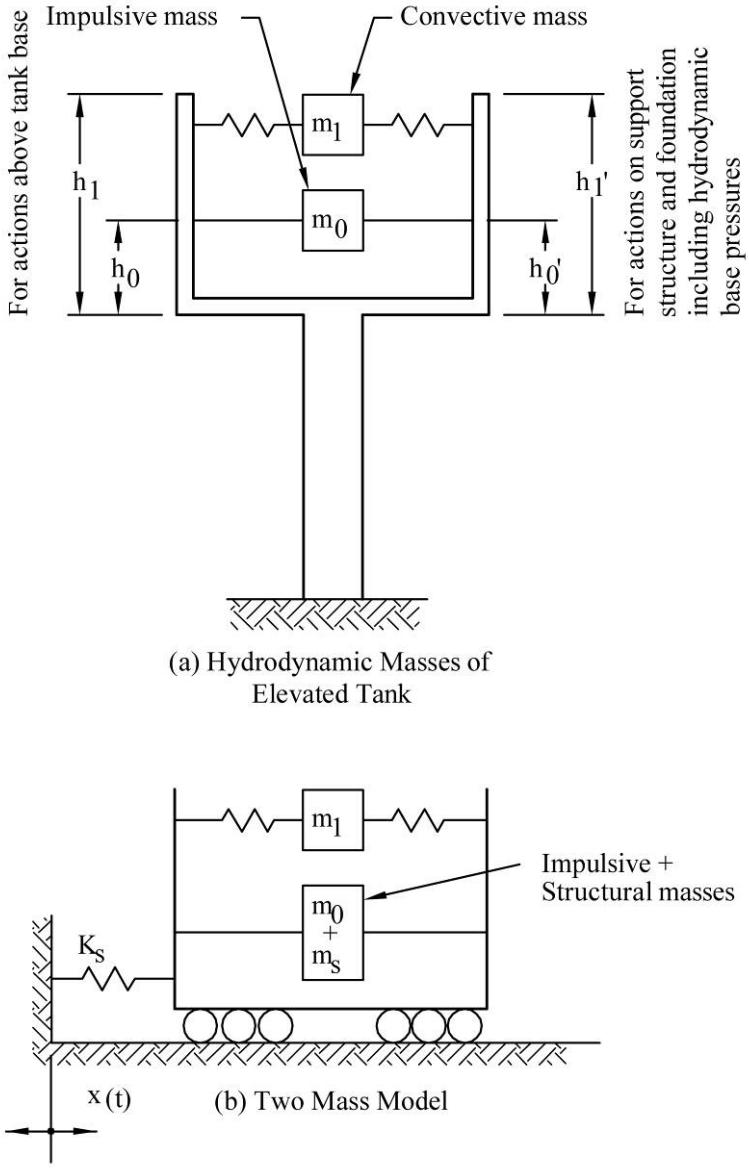


Figure C3.28: Two Mass Model for Elevated Tank

Convective and impulsive masses representing the hydrodynamic pressures may be evaluated using the methods given previously for rigid tanks. One of the masses of the model is equal to the convective mass and the other is equal to the sum of the fluid impulsive mass plus the mass of the tank and supporting structure. The point of action of this second mass is determined from the individual points of action of the two contributing masses. For loads on the support structure the impulsive mass acts at a height that includes the effects of the hydrodynamic base pressure. The stiffness of the convective mass spring may be estimated from:

$$K_1 = \frac{4\pi^2 m_1}{T_1^2} \qquad \text{(C3.42)}$$

The spring connecting the second (impulsive) mass to the ground is assigned a stiffness, K_s, equal to that of the support structure for a horizontal force applied at the same height as the mass.

The earthquake response of the two mass model can be calculated using standard structural dynamics and response spectra methods. Damping for the mode involving the convective mass should be taken as 0.5%. In the other vibration mode the damping should be evaluated from equations in Section C3.7 for foundation damping and the equations given below for equivalent damping in a ductile supporting structure. The square root of the sum of the squares rule should be used for combining the modal responses.

A simplified analysis procedure has been suggested by Housner (1963). In this approach the two masses are assumed to be uncoupled and the earthquake forces on the supports are estimated by considering two separate single degree-of-freedom systems. One representing the sloshing of the convective mass in a tank assumed to be rigidly connected to the ground. The other representing the impulsive plus structure masses behaving as an inverted pendulum with a stiffness equal to that of the support structure. If the periods of these two systems differ by more than a factor of 2.5, it is likely that the method will give a satisfactory approximation for design purposes.

If the supporting structure is detailed to have significant ductility then it may be designed to respond beyond the yield level under the design earthquake. In this case the acceleration response of the impulsive mass is reduced by the absorption of energy in the yielding system.

In designing the tank and supporting structure the method of ductile response shall be identified. All other structural elements should be designed for accelerations as to not preclude the formation of the intended ductile mechanism.

The method given below of estimating period and damping of an equivalent linear system for representing a yielding system is used as the basis for determining the spectrum correction factors in Table 3.2 for the impulsive response mode for a ductile tank.

A convenient method of allowing for ductility effects is to use the following equations given by Iwan (1980) for defining an equivalent elastic system.

$$\frac{T_{eq}}{T} = 1 + 0.12(\mu - 1)^{0.94} \qquad \text{(C3.43)}$$

$$\xi_e = \xi + 5.9(\mu - 1)^{0.37} \qquad \text{(C3.44)}$$

where

$\xi_h = 5.9(\mu - 1)^{0.37}$ = equivalent additional viscous damping to represent hysteretic energy dissipation

T_{eq} = period of vibration for equivalent linear system

T = period of vibration based on stiffness at yield

ξ_e = overall viscous damping for equivalent linear system including both pre-yield viscous damping and hysteretic damping effects, percent of critical

ξ = viscous damping estimated for response up to yield, percent of critical as defined in Figures 3.1 and 3.2

μ = displacement ductility factor

Numerical studies reported by Iwan showed that the equations above for the equivalent linear system, over at least the mid period range of 0.4 to 4.0 sec, had smaller errors than

other methods generally used to allow for ductility effects. The equivalent linear system is easy to apply as direct use can be made of the elastic response spectra given in NZS 1170.5:2004 with a correction factor applied to account for ductility and level of damping defined by Equation C3.45 (NZSEE, 2006).

$$k_f(\mu,\xi_e) = [7/(2+\xi_e)]^{0.5} \quad \text{(C3.45)}$$

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Section 4 - Commentary

Structural Analysis

C4.2 Method of Combining Actions

For a vertical cylindrical tank the stresses required for design are those resulting from:

- hoop forces and vertical bending moments (described in Section 4.3);
- axial membrane stress and base plate tension (described in Section 4.4);
- shear stress in the tank walls (described in Section 4.5).

For some tanks other than vertical cylinders, different resisting mechanisms may predominate and so not all the above stress resultants may act. For example, hoop forces would be minor in a rectangular tank but horizontal bending moments may be significant. Figure C4.1 illustrates for a vertical cylindrical tank and a rectangular tank the action of the significant stress resultants.

The horizontal impulsive components (rigid and flexible) should be added to provide the maximum horizontal impulsive response. However, because of the unlikely probability of coincident response of maximum horizontal impulsive, convective, and vertical impulsive actions, the SRSS method of combining these actions is recommended. Thus, for example, the resultant hoop force, N_θ, for a flexible vertical axis cylindrical tank would be

$$N_\theta = \sqrt{\left(N_{\theta i}^2 + N_{\theta l}^2 + N_{\theta V}^2\right)} \qquad \text{(C4.1)}$$

where

$N_{\theta i}$ = $N_{\theta r} + N_{\theta f}$ maximum horizontal impulsive hoop force

$N_{\theta r}$ = maximum horizontal rigid-impulsive hoop force

$N_{\theta f}$ = maximum horizontal flexible-impulsive hoop force (see Section 3.3.2.1)

$N_{\theta l}$ = maximum 1st-mode convective hoop force

$N_{\theta V}$ = maximum vertical impulsive hoop force.

Stresses may be found in similar fashion. For example, the components of vertical surface stresses induced at height z by seismic action will be

$$f_{zr} = \frac{N_{zr}}{t} \pm \frac{6M_{zr}}{t^2} \quad \text{(impulsive rigid)} \qquad \text{(C4.2)}$$

$$f_{zf} = \frac{N_{zf}}{t} \pm \frac{6M_{zf}}{t^2} \quad \text{(impulsive flexible)} \qquad \text{(C4.3)}$$

$$f_{zl} = \frac{N_{zl}}{t} \pm \frac{6M_{zl}}{t^2} \quad (1^{st} \text{ mode convective}) \qquad \text{(C4.4)}$$

$$f_{zv} = \frac{N_{zv}}{t} \pm \frac{6M_{zv}}{t^2} \quad \text{(vertical impulsive)} \qquad \text{(C4.5)}$$

and the resultant seismic stress would be

$$f_{ze} = \sqrt{\left(f_{zi}^2 + f_{zl}^2 + f_{zv}^2\right)} \qquad \text{(C4.6)}$$

where

f_{zi} = $f_{zr} + f_{zf}$

M_{zr} = maximum impulsive-rigid bending moment/unit length

M_{zf} = maximum impulsive-flexible bending moment/unit length

M_{zl} = maximum 1st mode convective bending moment/unit length

M_{zv} = maximum vertical impulsive bending moment/unit length

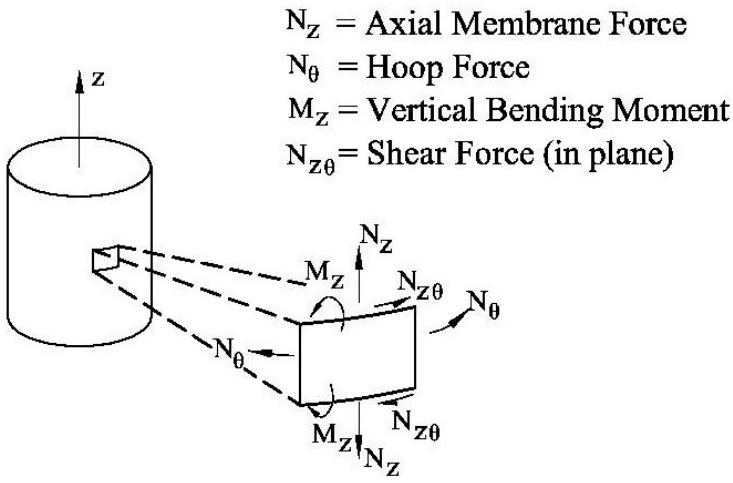
N_{zl} = maximum lst mode convective axial force/unit length

N_{zf} = maximum impulsive-flexible axial force/unit length

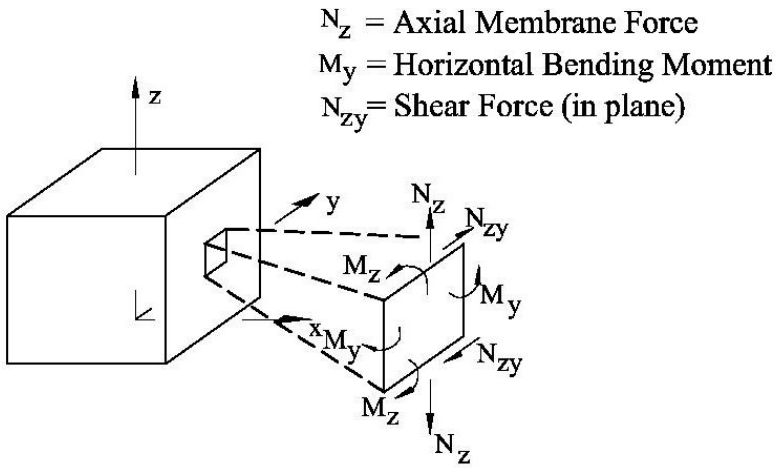
N_{zr} = maximum impulsive-rigid axial force/unit length

N_{zv} = maximum vertical impulsive axial force/unit length

For a rigid tank, $N_{\theta f}$ in Equation C4.1 and f_{zf} in Equation C4.6 are both zero, and $N_{\theta i} = N_{\theta r}$ etc.



(a) Cylindrical Tank



(b) Rectangular Tank

Figure C4.1: Stress Resultants for Design of Tanks with Vertical Axes

C4.3 Computation of Stress and Deflection in Tanks with a Vertical Axis

C4.3.1 Rigorous Analysis - Axisymmetric Tanks with a Constant Wall Thickness

Section 3.3 defines the pressure distributions for impulsive and convective modes of response. The pressure distributions vary up the vertical axis and around the circumference. However, Priestley (1985) has shown that where the pressure varies only slowly in the circumferential direction, as is the case for seismic pressure distributions, the stress distribution at the cylinder generator of maximum pressure ($\theta = 0$) can be calculated without significant loss of accuracy by assuming that the entire tank is subjected to an axisymmetric pressure distribution equal to the pressure distribution at $\theta = 0$. Consequently, axisymmetric structural simulations are possible, which greatly reduce the computational effort in calculating critical design forces. For example, the equations governing axisymmetric shell behaviour reduce to that of a beam on an elastic foundation (Hetenyi, 1961). Priestley describes a simple frame analogy of the BEF method capable of calculating stresses and deflections for both cylindrical and non-cylindrical tanks with uniform or varying wall thickness. This method has been used, in conjunction with the actual pressure distribution of Section 3.3, to produce dimensionless design charts for hoop forces and vertical bending moments, which are presented in Appendix A. Values of hoop force and bending moment are given for hydrostatic (and hence vertical acceleration), 1st mode convective, and impulsive rigid pressure distributions, for a range of H:R and R:t ratios. In using Appendix A for a flexible tank, Equation A.8 is used instead of Equation A.7. Equation A.8 singularly accounts for both the rigid and flexible impulsive modes as typically $\check{T}_0$ and $\check{T}_f$ are both on the plateau of the hazard spectrum. Should $\check{T}_f$ be on the downward slope of the hazard spectrum mass ratios of m_r/m_ℓ and m_f/m_ℓ may be applied to Equations A.7 and A.8 respectively with the resultants added together to determine $N_{\theta i}$.

C4.3.2 Approximate Analysis - Cylindrical Tanks with a Variable Wall Thickness

A reasonable approximation to behaviour where wall thickness t does not vary excessively is to use the design charts presented in Appendix A, basing stresses on the local R/t ratio. Where greater accuracy is required, the wall should be analysed under the appropriate pressure distribution for impulsive, and convective pressure, at $\theta = 0$, defined in Section C3.3.1.

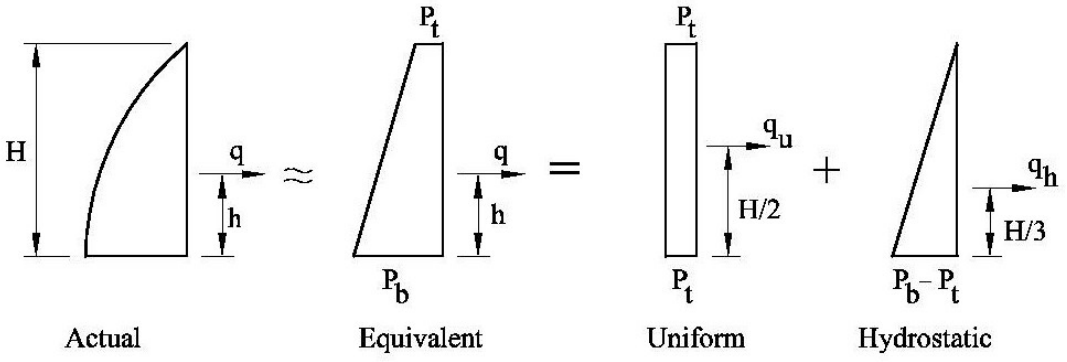


Figure C4.2: Equivalent Linear Distribution of Pressure on Tank Walls

Section 4.3.2 permits the use of an equivalent linear pressure distribution with height, rather than the curved distributions shown in Section C3.3. In calculating the equivalent linear pressure distribution, $p_{(z)}$, it is convenient to work directly from the force, V, and height of

action, h, defined from the Spring-Mass analogy of Section 3.3. For a cylindrical tank, the maximum force per unit circumferential length, q, which occurs at $\theta = 0$, is given by

$$q = \frac{V}{\pi R} \tag{C4.7}$$

The linear equivalent pressure distribution is then calculated using statics. Thus in Figure C4.2,

$$q = q_u + q_h = (p_t + p_b)\frac{H}{2} \tag{C4.8}$$

where q_u and q_h are equivalent uniform and hydrostatic pressure distributions. Taking moments about the base of the wall

$$q.h = q_u \frac{H}{2} + q_h \frac{H}{3} = (2p_t + p_b)\frac{H^2}{6} \tag{C4.9}$$

Simultaneous solution of Equations C4.8 and C4.9 yields p_t and p_b, and hence the uniform and hydrostatic pressure distributions. Computational effort can be considerably reduced if the resultant shears V_r, V_f, V_1, and the resultant moments M_r, M_f, M_1 are each combined according to the requirements of Section 4.2. Such that a resultant pressure distribution can be defined by Equations C4.8 and C4.9, thus requiring only a single structural analysis.

Non-Cylindrical Axisymmetric Tanks

Section C3.3.1, Non-Cylindrical Rigid Axisymmetric Tank with Vertical Axis, suggests an approximate method for calculating resultant forces for axisymmetric tanks whose shape approximates an inverted cone. Equations C4.7 and C4.8 can be used to calculate the equivalent linear pressure distribution on the walls of the equivalent cylinder of radius R. However, the actual horizontal pressure to be applied to the inclined tank walls should be modified to

$$p'_{(z)} = p_{(z)} \frac{R}{R_{(z)}} \tag{C4.10}$$

where $R_{(z)}$ is the tank radius at height z. This modification is needed to ensure the resultant force on the tank and its moment about the base remain correct. Structural analysis of these tanks can conveniently be carried out by the method given by Priestley (1985).

Rectangular Tanks

Haroun (1984) has calculated an exact solution for bending moments induced by an impulsive pressure distribution in the walls of vertical axis rectangular tanks. An acceptable approximation to behaviour, including convective components, is to use a linear equivalent pressure distribution as shown in Figure C4.2 defined by Equations C4.8 and C4.9. In this case the shear per unit length, q, is given by

$$q = \frac{V}{4B} \tag{C4.11}$$

where 2B is the tank width perpendicular to the ground acceleration. Analyses of rectangular plates with appropriate boundary conditions, subjected to uniform or hydrostatic pressure distributions, are available in standard texts on plates, for example (Timoshenko and Woinowsku-Krieger, 1959).

C4.4 Overturning Effects

C4.4.1 Non-Uplifting Tanks

Axial Stresses

A tank that is not securely fastened to the foundation must be initially checked to see whether it will uplift under the action of the design forces. The mechanical analogue described in Section 3 is one possible means of proceeding with this check. This gives for the overturning moment, M_{0T}, the equation:

$$M_{0T} = \sqrt{(m_1 h_1 C_d(T_1) g)^2 + [(m_f h_f + m_w h_w + m_t h_t) C_d(\check{T}_f) g + (m_0 - m_f) h_0 C_d(\check{T}_0) g]^2} \quad \text{(C4.12)}$$

The symbols in Equation C4.12 are defined in Section 3. Note that in Equation C4.12 the impulsive-rigid and impulsive-flexible moments are added directly. For a rigid tank, $\check{T}_f = \check{T}_0$, and the impulsive term reduces to $(m_w h_w + m_t h_t + m_0 h_0) C_d(\check{T}_0) g$.

Because the interaction of the vertical component of the earthquake motion with the uplifting mechanism of a tank is not understood, the usual gravitational constant g has been chosen for the calculation of tank weight. Thus, for an unanchored circular tank the restoring moment, M_R, is given by

$$M_R = (m_w + m_t) g R \quad \text{(C4.13)}$$

For non-uplifting tanks the additional stresses and displacements resulting from the overturning forces may be calculated by considering the tank to be a vertical cantilever. Thus, at the base

$$f_{zp} = \frac{M_{0T}}{Z_T} \quad \text{(C4.14)}$$

where

Z_T = section modulus for the tank shell

For a vertical cylinder:

$$Z_T = \pi R_m^2 t \quad \text{(C4.15)}$$

where

R_m = mean radius of the tank

t = wall thickness at the base

For a rectangular cylinder:

$$Z_T = 4Lt\left(B + \frac{L}{3}\right) \quad \text{(C4.16)}$$

where

$2L$ = tank length in the direction of ground acceleration

$2B$ = tank width

Displacements

For a cylindrical tank, the maximum horizontal displacement at the top of the free edge, resulting from overturning effects, (neglecting soil-structure interaction displacement component), is conservatively given by:

$$|u|_{max} \approx \beta_1 C_d(T_f) g \left(\frac{T_f}{2\pi}\right)^2 \quad \text{(C4.17)}$$

where

β_1 = modal participation factor of the fundamental mode associated with the flexibility of the shell, Figure C4.3.

$C_d(T_f)$ = elastic horizontal design action coefficient corresponding to the fundamental period T_f with a displacement ductility factor of 1.25. See Section 3.2.

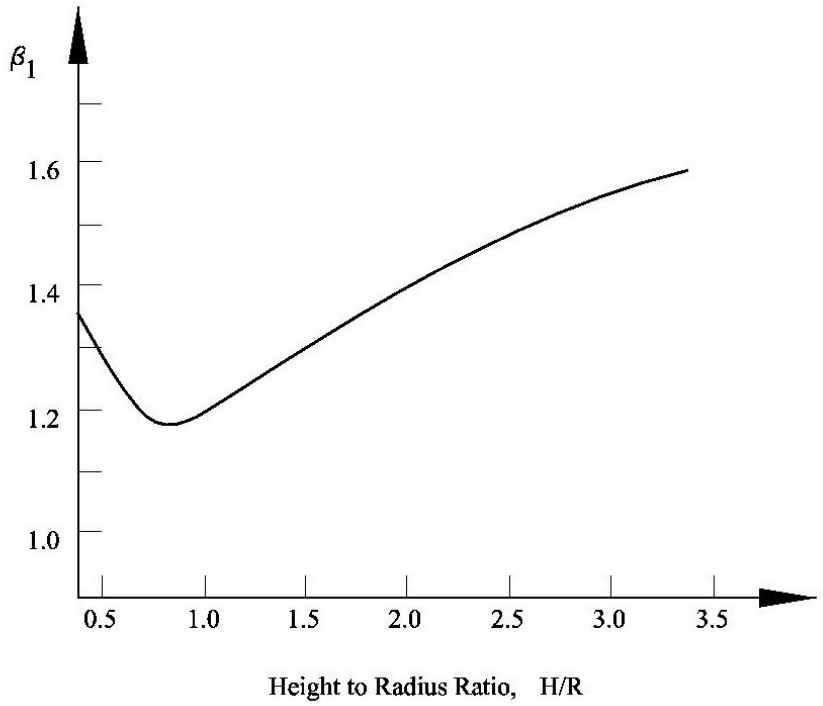


Figure-C4.3: Modal Participation Factor, β_1, (Clough and Niwa, 1979)

C4.4.2 Uplifting Tanks

If M_{0T} calculated by Equation C4.12 exceeds M_R calculated by Equation C4.13, and the tank is unanchored, uplift will occur. The primary effect of uplift is to increase the compressive axial stress in the shell. In addition, distortion of the shell and tension forces in the tank base may need to be considered (Cambra, 1982), (Choon-Foo and Shih, 1981), (Clough, 1977).

Vertical Cylindrical Tanks

The mechanism of tank uplift is complex and not completely understood. To describe it fully, the effects of large displacements of the uplifting base, yielding of the base-shell joint, membrane forces in the base, ovalling of the shell, imperfections of the shell geometry, and foundation flexibility need to be included. The calculation of design forces on a rocking tank have been by a quasi-static rather than a dynamic method (Clough, 1977), (Cambra, 1982), (Choon-Foo and Shih, 1981), (Wozniak and Mitchell, 1978). The following describes a modified version of the method presented by Clough (1977). This is recommended as a reasonable description of the rocking mechanism for a cylindrical tank.

As part of the base uplifts the weight of liquid above the uplifted portion assists in stabilising the tank. The higher M_{0T} is, the greater the extent of uplift that will be required to stabilise the tank. As shown in Figure C4.4 the overturning moment, M_{0T}, is resisted by the action of the tank weight and the reaction forces W_s and W_f forming M_R.

$$M_R = W_s\ kR + W_f\ (R - r) \quad \text{(C4.18)}$$

where

W = total weight of the fluid

W_f = weight of the fluid supported directly by the foundation over the area that does not uplift (radius r)

W_w = weight of shell and roof

W_s = $W + W_w - W_f$ = compression reaction at shell base

R = radius of the tank

θ^* = half angle which defines the arc of the shell base in contact with the foundation

kR = distance from centre of compression reaction to tank centreline.

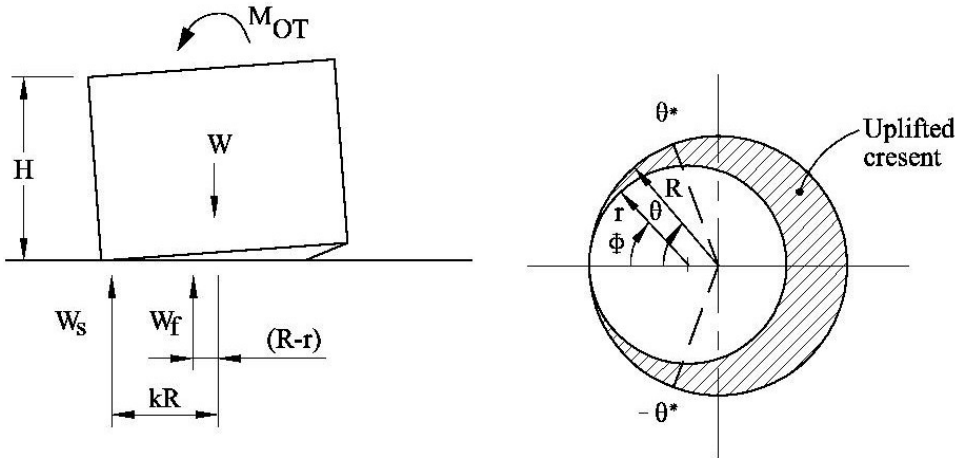


Figure C4.4: Restoring Forces on an Uplifting Circular Tank

The resisting moment, M_R, is calculated in an iterative fashion by:

(i) Guessing $\tau = r/R$

(ii) Calculating $$\theta^* = \arctan\left(\frac{\tau}{1-\tau}\right) \quad \text{(C4.19)}$$

(iii) Calculating $$k = \frac{2}{\theta^{*2}}(1 - \cos\theta^*) \quad \text{(C4.20)}$$

(iv) Calculating $$M_R = RW\left(k\left(1 + \frac{W_w}{W}\right) + (1-k)\tau^2 - \tau^3\right) \quad \text{(C4.21)}$$

(v) Adjust τ until $M_R = M_{0T}$

Alternatively, or as a useful starting point in the iterative procedure Figure C4.5 may be used.

Note that the method suggested assumes that the overturning moment M_{0T} is not affected by the degree of uplift. In fact uplift can be expected to modify the effective flexibility of the impulsive mode, thus increasing the period. More precise information is available in a research report by Peek, Jennings and Babcock (1986). Peek et al also note that stability against overturning of unanchored tanks can be greatly improved by 'pre-uplifting' the tank. This involves supporting the rim of the tank on a ring such that the base is uplifted for the outer part of the diameter right round the tank. The fluid pressure on the uplifted portion of

the base provides an additional compressive reaction on the ring. This delays the onset of uplift, and reduces stresses, relative to a tank without 'pre-uplift', after uplifting occurs. 'Pre-uplift' is further explained in Peek (1986).

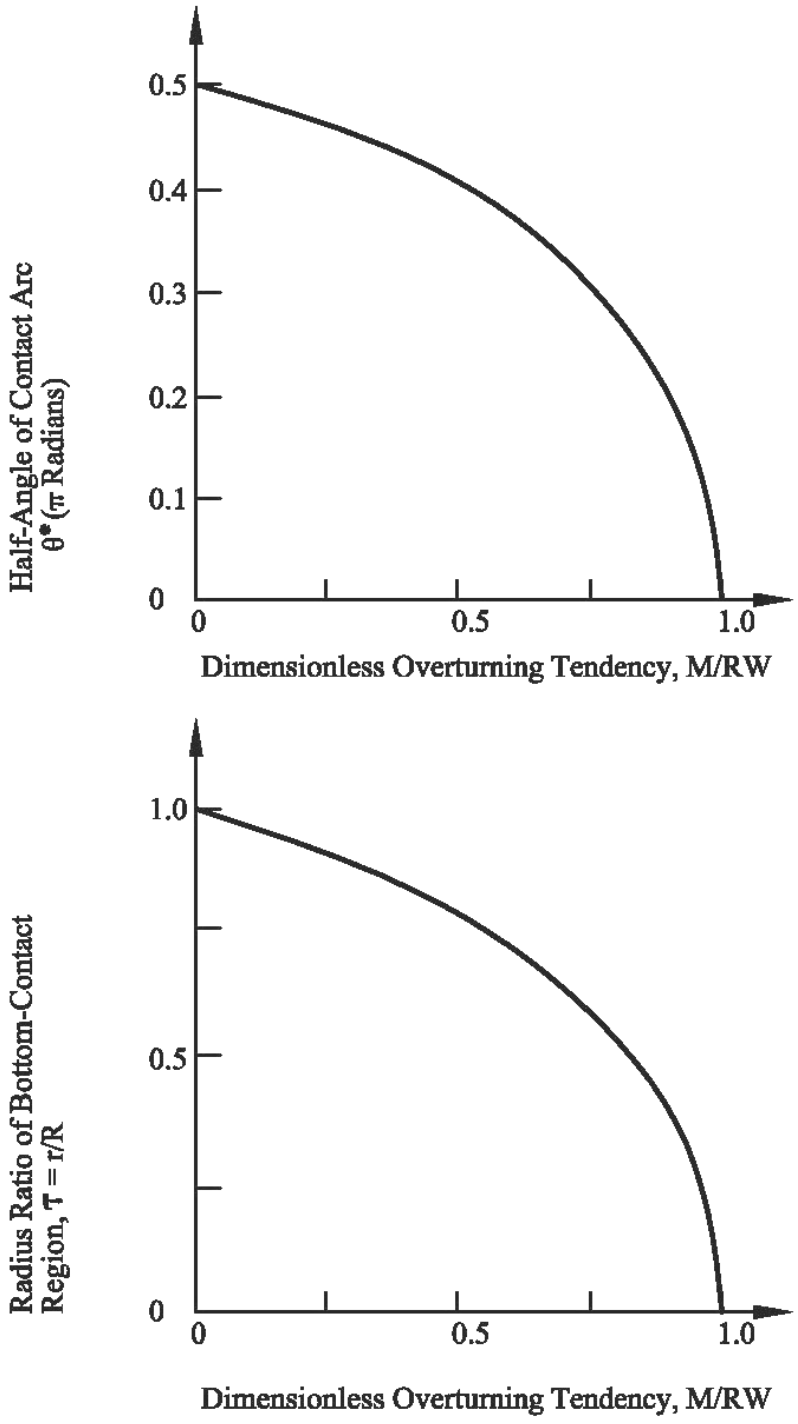


Figure C4.5: Design Aid for Overturning Tendency for Unanchored Circular Tanks

Axial Membrane Stress in Shell

The maximum axial stress in the shell is computed as

$$f_{max} = \frac{CW_s}{R\theta * t} CF \qquad \text{(C4.22)}$$

where

C = foundation stiffness factor = 1.0 for a rigid foundation
= 0.5 for a flexible foundation

$$W_s = W + W_w - W_f = W\left(1 + \frac{W_w}{W} - \tau^2\right) \qquad \text{(C4.23)}$$

CF = Calibration Factor (Suggested Value 2.5)

The method described by Clough (1977) is known to be non-conservative, (Haroun, 1983). The suggested calibration factor has been derived by comparing the results of Clough with those obtained from experiments (Clough and Niwa, 1979).

Radial Membrane Stress in Base

An estimate of the membrane stress, f_{rb}, in the base plate as a result of uplift is derived by Cambra (1982).

$$f_{rb} = \frac{1}{t_b}\left[\frac{2\overline{E}t_b p_o^2 R^2 (1-\tau)^2}{3}\right]^{\frac{1}{3}} \qquad \text{(C4.24)}$$

where

t_b = thickness of base plate in uplift region

$\overline{E} = \frac{E}{(1-\nu^2)}$

E = Young's Modulus

ν = Poisson's ratio

p_o = hydrostatic pressure on the base

Uplift of Shell

An estimate of the magnitude of the shell uplift may be obtained by using a modified version of a formula derived by Cambra (1982). The uplift shall be calculated using equations C4.19 to C4.25 for an overturning moment, M_{0T} determined at a displacement ductility of 1.25.

The uplift is,

$$v = \frac{1}{C}\left[\frac{f_{yb} t_b^2}{6N_x} + \frac{p_o L_b}{N_x}\left[\frac{L_b}{2} - \left(\frac{\overline{E} t_b^3}{12 N_x}\right)^{\frac{1}{2}}\right]\right] \qquad \text{(C4.25)}$$

where

f_{yb} = yield stress in the base material

N_x = $f_{rb}\, t_b$

L_b = $2R\ (1 - \tau)$

C = foundation stiffness factor defined in Equation C4.22

f_{rb} = as per equation C4.24 for μ=1.25

τ = as per equations C4.19-C4.21 for μ=1.25

In Equation C4.25, Cambra's formula has been modified assuming that the base material will yield at the base-shell joint.

Rectangular Tanks

There are no recorded studies of tank uplift behaviour for tanks other than that for vertical cylindrical tanks. The following is a suggested procedure for the analysis of uplifting rectangular tanks which is based upon the principles underlying the previous formulae for cylindrical tanks. In Figure C4.6 the proposed uplift mechanism is illustrated with the length of the base, $2L\ (1 - \tau)$, which has been lifted off the foundation.

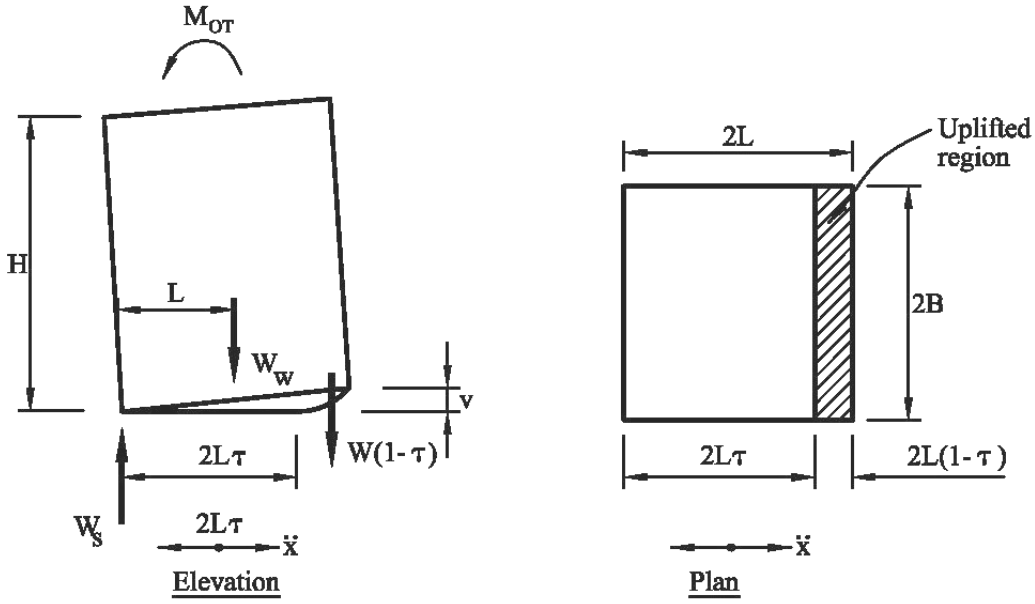


Figure C4.6: Restoring Forces on an Uplifting Rectangular Tank

With this mechanism taking moments about the compression reaction, W_s, the resisting moment, M_R, is given by:

$$M_R = W_w L + WL\left(1-\tau^2\right) \quad \text{(C4.26)}$$

Thus for a rectangular shaped tank τ can be solved for directly, as

$$\tau = \sqrt{1-\left(\frac{M_R}{WL}-\frac{W_w}{W}\right)} \quad \text{(C4.27)}$$

Axial Membrane Stress in Shell

$$f_z = \left[\frac{W_W + W\left(1-\tau\right)}{2Bt}\right]CF \quad \text{(C4.28)}$$

Longitudinal Membrane Stress in Base

$$f_{xb} = \frac{1}{t_b}\left[\frac{2\overline{E}t_b p_o L^2(1-\tau)^2}{3}\right]^{\frac{1}{3}} \quad \text{(C4.29)}$$

Uplift of Shell

This is given by Equation C4.25. In this case however, the uplifted length is given by

$$L_b = 2\ L\ (1\ -\ \tau) \quad \text{(C4.30)}$$

where

τ as per equation C4.27 for μ=1.25

All terms used in Equations C4.28 and C4.29 are defined as for their use in Equations C4.22 and C4.24.

C4.5 Shear Transfer

Roof to Wall

The total shear force to be designed for will be:

For a rigid tank (cf. Equation 3.10)

$$V = C_d(\check{T}_0)\, m_t g \quad \text{(C4.31)}$$

For a flexible tank (cf. Equation 3.11 and Figure C4.3)

$$V = \beta_1\, C_d(\check{T}_f)\, m_t g \quad \text{(C4.32)}$$

where

m_t = mass of the roof

Wall to Foundation

Sections 3.4 and 3.5 provide expressions for the seismic base shear components, and direction for the inclusion of shell and roof inertia. The shear stresses that result from these forces can be calculated in the following manner:

Vertical Cylindrical Tanks

It may be conservatively assumed that the total seismic component of base shear, V, is taken by membrane action. In this case the circumferential shear flow to be designed for is given by:

$$q_c = \frac{V_m \sin\theta}{\pi R} \quad \text{(C4.33)}$$

where V_m = V.

For broad thick-walled tanks the out of plane shear stresses resulting from vertical bending may be significant. The shear force per unit circumferential length is maximum at $\theta = 0$ and is given by:

$$q_r = \frac{Rt}{H} p(z,\theta).V_n \quad \text{(C4.34)}$$

where

$\theta = 0$

$z = 0$

V_n = the slope $\frac{\Delta M_{zn}}{\Delta z_n}$ calculated for $z' = 0$ from the charts in Appendix A.

The total base shear taken by vertical bending action is given by

$$V_b = q_r \pi R \quad \text{(C4.35)}$$

For the cases where V_b is not insignificant, the circumferential shear flow may be calculated by using Equation C4.33, where:

$$V_m = V - V_b \quad \text{(C4.36)}$$

Rectangular Tanks

In the central region of walls perpendicular to the direction of ground acceleration it may be conservatively assumed that the pressures are carried entirely through vertical bending action. For this case the shear force per unit length, perpendicular to the wall, can be calculated directly from Equation C4.11. Towards the edges of the same wall pressures will be carried by transverse bending back into the walls parallel to the direction of ground acceleration. The reactions from this action will be transmitted to the foundation by in-plane shears. The magnitude of the in-plane shear stress can be calculated but is unlikely to be critical.

C4.6 References

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Section 5 - Commentary

Design Criteria

C5.1 Scope

The intent of this section is to provide limits ensuring that tanks analysed in accordance with Sections 3 and 4 and complying with these limits, will satisfy the performance criteria of Section 2.

Storage tanks do not lend themselves to the full application of capacity design principles. In the general case, one or more of the following energy dissipating mechanisms will be available:

- Yielding anchor bolts
- Rocking tank/rocking foundations
- Yielding of tank base
- Base isolation (e.g., by sliding).
- A partial yielding non-reversible system relying on elephant's foot buckling.

If the applied loading is derived assuming no ductility, it is considered acceptable to design storage tanks without the specific application of capacity design. Where ductility has been used, a capacity design procedure should be used to ensure the desired method of energy dissipation occurs.

Designers should also give proper consideration to the likely modes of failure and satisfy themselves that none of these modes will result in sudden failure of the tank or any of its elements.

C5.2 Load Combinations including Seismic

The load combination of Equation 5.1 is based on the seismic equations for the ultimate limit state of the New Zealand Loadings Code, AS/NZS 1170 (SNZ, 2004). The load factor on Dead Load, G, and Live Load, Q, is taken as 1.0, as effects of vertical seismic accelerations are directly included in the earthquake load, E. The earthquake load combination is as defined in Section 4.2.

No Serviceability limit State (SLS) earthquake load level has been included in this document. This is because the permitted levels of ductility and associated force reductions from elastic demands are small and therefore the performance under more frequent "Service Level" earthquake shaking will generally also be satisfactory. It may be appropriate for special situations to also consider a SLS earthquake load combination.

C5.3 Stability

C5.3.1 Anchorage Limits

Except in the case of small tanks, there is generally little chance of a tank completely overturning in an earthquake. This is because the relative energy input required to cause overturning increases as the tank size increases. Ishiyama (1984) showed that for rigid bodies, overturning could only occur if the peak ground acceleration, $ä_0$, and the peak ground velocity, v_0, exceeded certain values. This is due to the ground acceleration needing

to be of a sufficient magnitude to induce rocking, and the peak velocity being great enough to establish sufficient energy input to overturn the body.

Since tanks are flexible, it is the peak response accelerations rather than the peak ground acceleration which will decide whether or not rocking is initiated. The toppling criterion should still be governed by peak ground velocity. Equations 5.2 and 5.3 have been based on Ishiyama's research. Because toppling is a problem only for small tanks, it is assumed that the base can uplift across the entire diameter without material failure. This assumption should be conservative in assessing displacements.

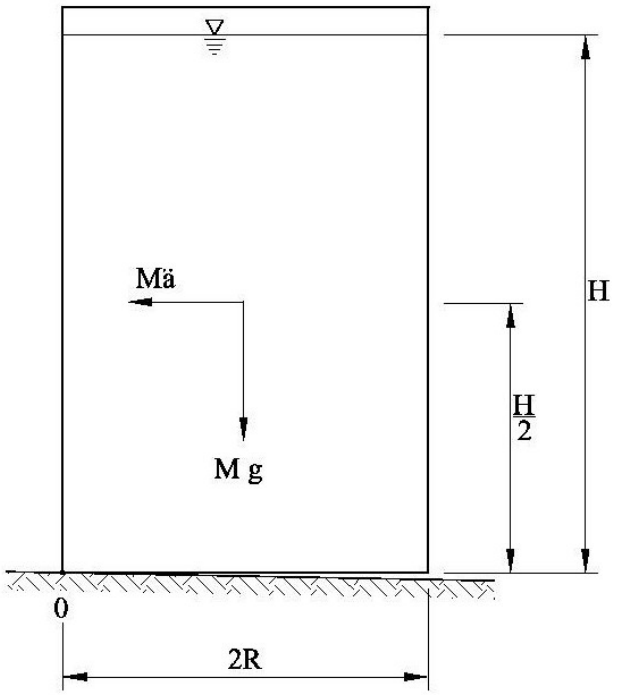


Figure C5.1: Initiation of Rocking for Rigid-Base Tank

Figure C5.1 shows forces acting at incipient rocking. Taking moments about the toe (Point 0).

$$M\ddot{a}\frac{H}{2} = MgR \qquad \text{(C5.1)}$$

i.e.

$$\frac{H}{R} = \frac{2g}{\ddot{a}}$$

where

M = mass of tank and contents

ä = inertial response acceleration

Substituting the peak response acceleration of Section 3 with a ductility μ=1.25, namely:

$$\ddot{a} = C_d(\check{T}_0)g \qquad \text{(C5.2)}$$

results in Equation 5.2 being the requirement for initiation of rocking.

Note that Equation 5.2 is conservative as it assumes that the entire fluid mass responds in the inertia mode. However, it will be tanks with high H/R ratios that are susceptible to overturning and for these tanks the approximation is reasonable.

Equation 5.3 is based on an approximation by Ishiyama (1984) that the minimum peak ground velocity, in m/s units, required to cause overturning is given by:

$$v_0^2 \geq \frac{4R^2}{H} \quad \text{(R,H in m units)} \tag{C5.3}$$

This is based on theoretical considerations modified by results of shaketable testing and dynamic analyses of rigid rocking blocks, but should be reasonably applicable to tanks.

Equation C5.3 can be expressed as:

$$R \leq \frac{v_0^2}{4}\left(\frac{H}{R}\right) \tag{C5.4}$$

Typical values relating peak ground velocity, v_0, to peak ground acceleration, $\ddot{a}_0$, from recorded large earthquakes lie in the range

$$0.07 \leq \frac{v_0}{\ddot{a}_0} \leq 0.15$$

Equation 5.3 is developed from Equation C5.4 using the conservative approximation that

$$\begin{aligned} v_0 &\leq 0.2\ddot{a}_0 \\ &= 0.2C(0)g \\ &= 0.2C_h(0)R_uZg \\ &= 2.0C_h(0)R_uZ \end{aligned} \tag{C5.5}$$

Tanks with aspect ratios greater than the limit of Equation 5.2 and with radius less than the limit of Equation 5.3 are thus potentially susceptible to overturning and should be anchored to their foundations. Other tanks may be anchored even though not required by Section 5.3.1, to reduce shell stresses or displacements.

C5.3.2 Required Anchorage

Two equations are given for the minimum required hold-down force per unit length, P, for anchored tanks, which are dependent on the ductility of the anchors. For normal anchor bolts, where fracture in a root thread can be expected. Equation 5.4 is based on an anti-symmetric distribution of anchor forces as shown in Figure C5.2(b). This equation, including allowance for vertical accelerations, is specified in API 650 (API, 2008). Where the anchors are necked over a reasonable length to a diameter less than the root thread diameter thus ensuring ductility, the distribution of tie-down force could approach the distributions of Figure C5.2(c) or C5.2(d). Equation 5.5 is based on the distribution of Figure C5.2(c), with a point compression force providing reaction to the tie-down force. In fact, the distribution will tend towards that of Figure C5.2(d) resulting in a maximum anchorage force of

$$P = \frac{2M_{0T}}{\pi D_{HD}^2} - w_t \tag{C5.6}$$

However, the distribution of Figure C5.2(d) is an extreme which will never be attained, and the assumption of a point compression reaction as assumed is also non-conservative. Thus the conservative expression of Equation 5.5 is favoured.

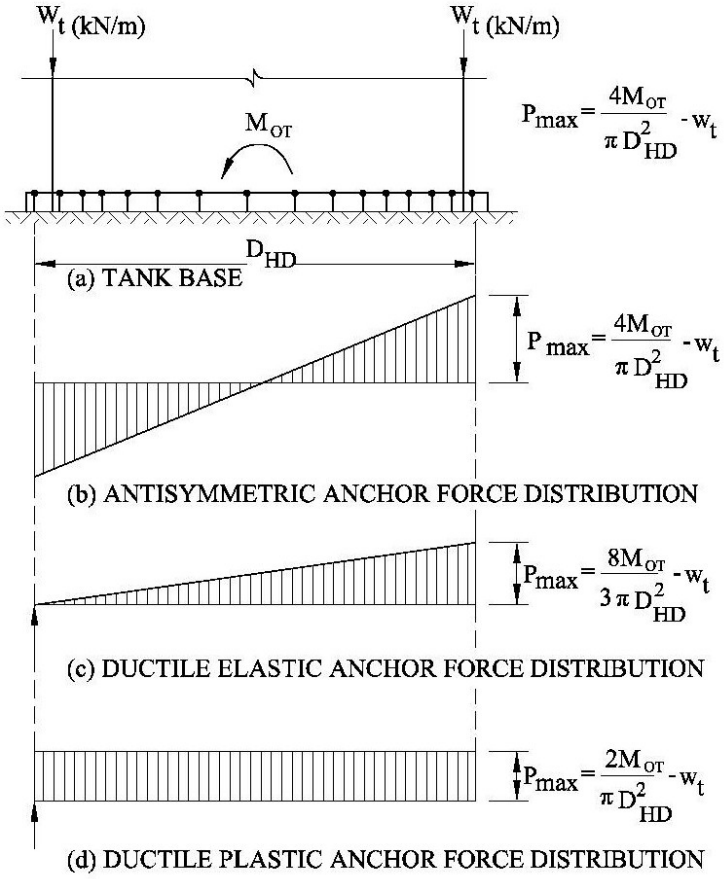


Figure C5.2: Possible Anchor Force Distribution

C5.4 Displacements

C5.4.1 Uplift Displacements

The method for predicting uplift displacements in Section C4.4 is approximate, and becomes less accurate as the displacement increases.

To ensure satisfactory performance and reasonable accuracy, the lateral displacement at the height of the centre of mass (i.e. H/2) is limited to R/4 or L/4. Simple geometric relationships then imply a limitation to the uplift displacement, v, which is given in Equation 5.6. The uplift displacement, v, for vertical cylindrical tanks can be calculated using equation C4.25. Equation C4.25 may also be used for rectangular tanks by using equation C4.30 for L_b.

C5.4.2 Radial Displacements of Top Stiffening Girder

Displacements of the stiffening girder at the top of a shell with a floating roof can be calculated using a simple frame analysis representing the geometry of the girder plane as shown in Figure C5.3. Because of symmetry and antisymmetry considerations it is only necessary to consider one quarter of the circumference. To ensure reasonable accuracy of displacement results, about 10 members are needed to model the quarter circumference. The moment of inertia and area of the members must be equal to that of the stiffening girder and adjacent shell. If the calculations indicate the possibility of the roof jamming, the consequence could be buckling of the roof under excessive uplift pressures.

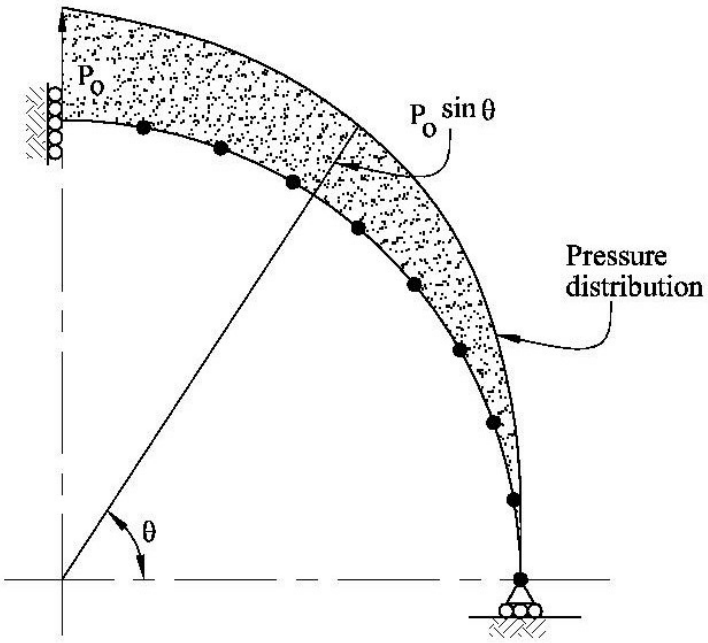


Figure C5.3: Frame Simulation for Analysis for Stiffening Girder

C5.4.4 Convective Wave Displacements

The level of wave displacement corresponding to the design-level earthquake may be assessed in accordance with Section C3.9.

C5.5 Specific Requirements for Steel Tanks

C5.5.1 Loading

The provisions of this section are based on ultimate strength, as limited by buckling or by development of yield stress, and consequently the ultimate limit state load Equation 5.1 is specified.

C5.5.2 Ultimate Limit State Stress

For cylindrical steel tanks, the maximum seismic response could be limited by a number of possible failure modes. These include elastic or inelastic buckling in membrane compression (which induces the characteristic diamond buckling), buckling in membrane shear, elastic-plastic collapse at the base of the shell wall (inducing the characteristic elephant's foot buckling), and material yield under bending moment or hoop tension. The recommendations in Section 5.5.2 are based on an extensive review of experimental results, and theories of cylindrical shell buckling by Rotter (1985a).

Buckling In Membrane Compression

The vertical membrane stress to induce buckling in a shell is a function of the internal pressure, the circumferential variation of axial stress, and importantly the relative amplitude of imperfections (δ/t) in the wall. The effect of imperfections (radial errors in wall position) is to decrease the buckling stress to a fraction of the classical "perfect shell" buckling stress, f_{cl}, given by Equation 5.9. Internal pressure decreases the effective imperfection amplitude, hence increasing the buckling stress. Circumferential variation of axial stress reduces the probability of coincidence of the maximum stress and the maximum imperfection, again increasing the buckling stress. Thus the buckling load associated with membrane compression induced by bending exceeds that where the compression is induced by axial load. However, in both cases (internal pressure, and bending) it appears that the classical buckling stress is an upper limit.

Equations 5.7 to 5.13 reflect these influences. The allowable stress is related to the classical buckling stress given by Equation 5.9. The reduction from this stress level due to initial imperfections for unpressurised tanks subjected to axial compression is given by Equations 5.11, 5.12 and 5.13 (Koiter, 1945). For buckling stresses higher than 0.5fy, (i.e. relatively thick-walled tanks) the buckling is essentially inelastic, and Equation 5.11 governs. For lower buckling stresses (relatively thin-walled tanks) Equation 5.12 governs.

Equation 5.8 expresses the increase in buckling stress due to internal pressure (Trahair et al, 1983), expressed in terms of the nominal hoop stress ratio, $\bar{p}$, given by Equation 5.10.

Equation 5.7 expresses the increase in buckling stress due to the axial stress being induced by bending, rather than axial loading (ECCS, 1983). As this is primarily the case for seismically induced membrane compression, it is appropriate to include these stress criteria. Note that the maximum allowable stress that can result from Equations 5.7 to 5.13 is the classical buckling stress, f_{cl}. Generally the level is much lower.

In the absence of better data on the expected level of imperfection, Rotter (1985a) suggests.

$$\frac{\delta}{t} = \frac{0.06}{a}\sqrt{\frac{R}{t}} \qquad \text{(C5.7)}$$

where

a = 1.0 for normal construction

= 1.5 for quality construction

= 2.5 for very high quality construction

The method for measuring imperfection level should be that recommended by ECCS (1983), using a straight rod of length $4\sqrt{Rt}$ for parts of the wall distant from welds, but of length 25t across welds. Figure C5.4 shows the method for measuring the imperfections.

Figure C5.5 compares the membrane compression buckling stresses for normal quality construction (a = 1 in Equation C5.7) with the classical buckling equation (Equation 5.9). The influence of a moderate hoop tension of 100 MPa in increasing the buckling stress is very evident, particularly at high R/t values. The influence of bending compression, compared with uniform compression is also significant, but the influence decreases as the internal pressure increases.

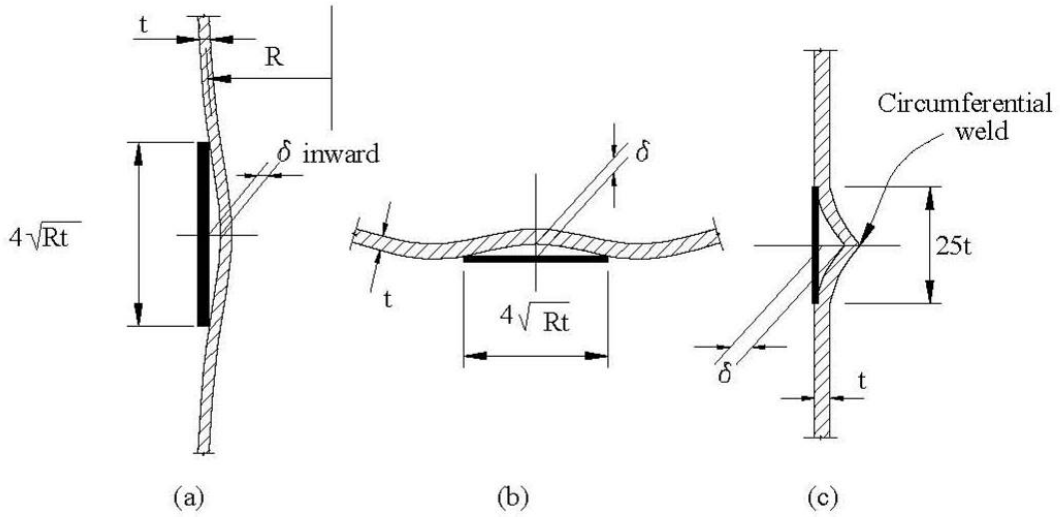


Figure C5.4: Imperfection Measurement, (ECCS, 1983)

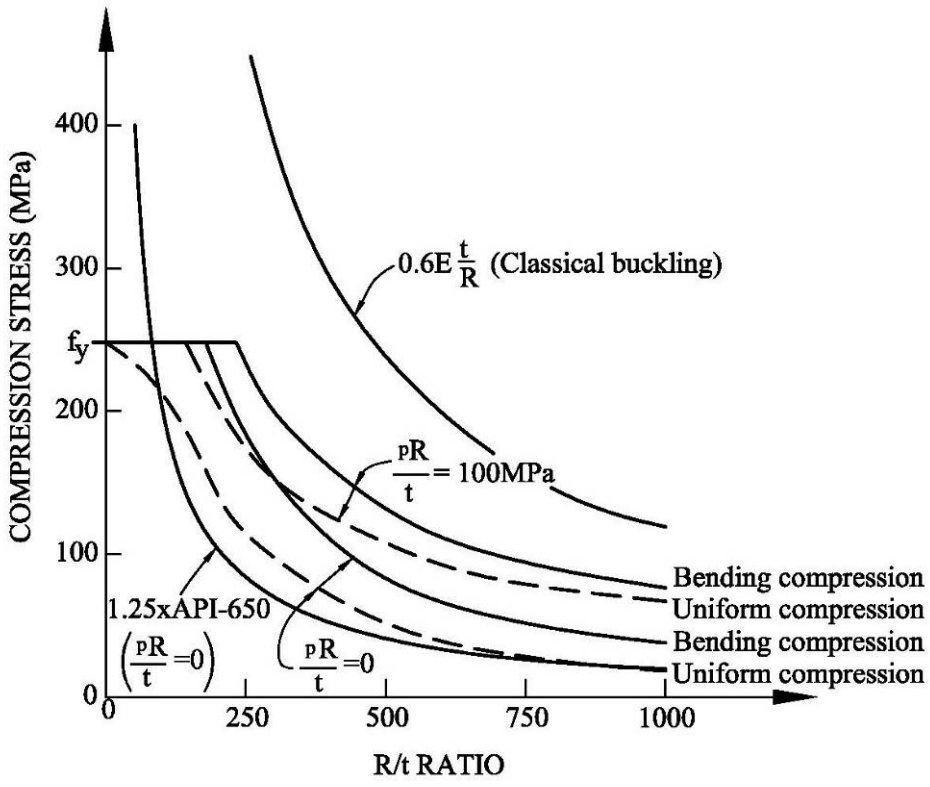


Figure C5.5: Membrane Compression Buckling Stresses for Normal Quality Construction (a=1.0, f_y=250 MPa)

For comparison, the buckling stress equation of API 650 (API, 1980) for zero internal pressure has been included. To convert from working stress to ultimate load levels, the API 650 stresses have been increased by 25%.

Elastic-Plastic Collapse

Towards the bottom of the tank, the steel is subjected to a biaxial stress state consisting of hoop tension and (in the worst case) vertical compression, as shown in Figure C5.6. Radial deformations under internal pressure create additional eccentricity, tending to induce the commonly observed "elephant's foot" buckling.

Equation 5.13 was developed by Rotter (1985b) and gives an accurate assessment of the stress required to initiate elastic-plastic collapse, or "elephant's foot" buckling as it is frequently known, in the zone adjacent to the tank base. The influence of internal pressure in Equation 5.13 is to reduce the maximum compression stress that can be sustained. Figure C5.7 compares the flexural compression stresses required to induce failure of a pressurised tank by elastic buckling or by elastic-plastic collapse. Except for thick walled tubes, elastic buckling is critical for low values of the membrane circumferential stress. As this stress increases above about 100 MPa, elastic-plastic collapse becomes the dominant failure case. It should be noted that Figure C5.7 is for normal quality construction (a = 1). Where the construction quality is poor, and the imperfection amplitude exceeds that given by Equation C5.7 for a = 1, elastic buckling will control for higher membrane tension stresses.

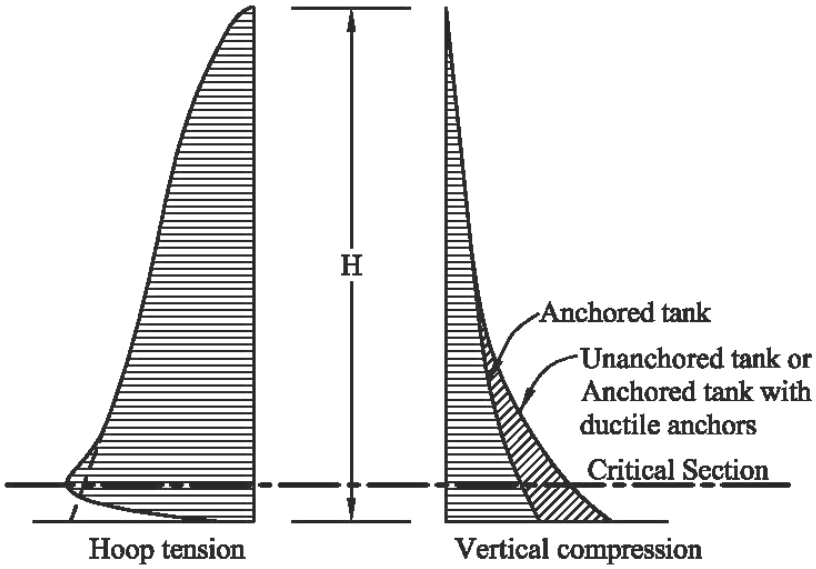


Figure C5.6: Membrane Forces in Tank Wall

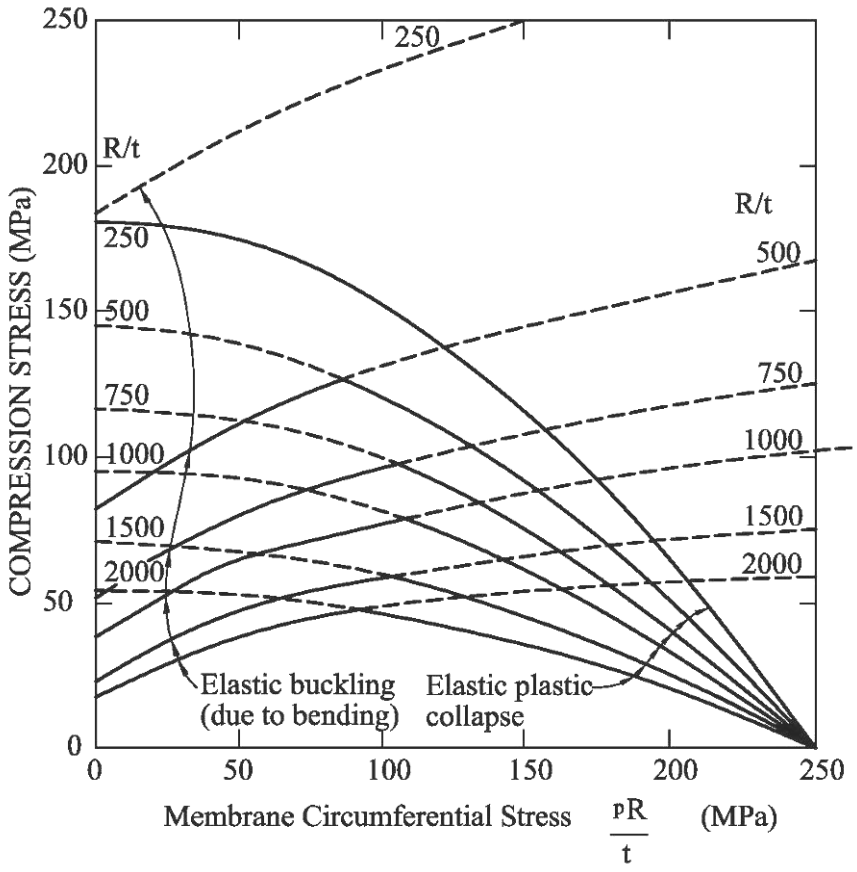


Figure C5.7: Comparison of Elastic Buckling and Elastic-Plastic Collapse Stresses (f_y=250 MPa, a=1.0)

Interaction of Stresses

The reduced yield stress, ϕf_y, provides an upper limit to allowable membrane stresses circumferentially and vertically. It is also specified for the vertical bending stresses, despite the simultaneous presence of membrane stresses.

Although interaction between hoop stress and vertical bending stress will also occur on the compression face of the shell (for vertical bending), it will result in an increase in effective yield stress. This tends to balance the reduction of effective yield stress on the opposite face of the shell. Thus, the net effect on yield moment will not be high. There will also be an interaction between the vertical axial compression in the shell and the bending moment. For plastic analysis of the shell, it can easily be shown that the ultimate moment and axial load interact according to the relationship.

$$\frac{M_u}{\phi M_p} + \left[\frac{P_u}{\phi P_p}\right]^2 = 1 \qquad \text{(C5.8)}$$

where

$M_p = \frac{t^2}{4} f_y$ plastic moment for zero axial stress per unit length

$P_p = t f_y$ plastic axial load for zero bending stress per unit length

ϕ = strength reduction factor

The reduction in M_u from the full plastic moment M_p is small for typical values of P_u. In view of this, and of the complex nature of the total interaction between hoop stress, axial vertical stress and bending moment, it is recommended that Equation 5.15 be used, with the maximum moment based on elastic analysis,

$$\phi M_u = \phi \frac{t^2}{6} f_y \qquad \text{(C5.9)}$$

C5.5.3 Base Plate Design

Clauses 5.5.3.1 and 5.5.3.2 are rewritten from the relevant API 650-E (API, 1980) clause to clarify the intended meaning. The thickness of the bottom annular ring is kept less than the wall thickness where practical to ensure that flexural yielding is not developed at the base of the wall shell.

C5.5.3.1

If the portion of tank base uplifting included regions of different thickness, the analysis method of 4.4.2 would not apply and behaviour would be extremely difficult to predict. Consequently the base plate should be of constant thickness at least over the region of potential uplift.

C5.5.3.2

Under the tank uplifting condition, the membrane tension induced in the uplifting portion of the base plate induces a circumferential ring compression reaction. The biaxial stress state reduces the effective yield stress. In the absence of more detailed analysis, it is recommended that the behaviour described in Section 4.4.2 be based on a conservative radial effective stress of 0.6 f_y.

For membrane action to develop in the tank, a plastic hinge must develop in the tank base at the junction with the wall. The rotation of this hinge must be limited to not cause fracture. Experimental information is limited about the ductility of such a connection, but failure of the weld between base plate and shell has been an identified cause of oil spillage in recent earthquakes. The assumption made in 5.5.3.4 of a maximum allowable strain of 0.05 should be conservative, but reasonable, for the base plate adjacent to a well prepared weld. The implication of the requirement is a maximum plastic rotation of

$$\theta_p = \left(\frac{0.05}{t/2}\right)2t = 0.20 \text{ Radians} \qquad \text{(C5.10)}$$

Equation C5.10 should be compared with the maximum rotation required for the specified uplift, using the method of C4.4.2. From Figure C5.8 the required plastic rotation for an uplift, v, and a base separation, L_b, is:

$$\theta_p = \left(\frac{2v}{L_b} - \frac{v}{2R}\right) \leq 0.20 \qquad \text{(C5.11)}$$

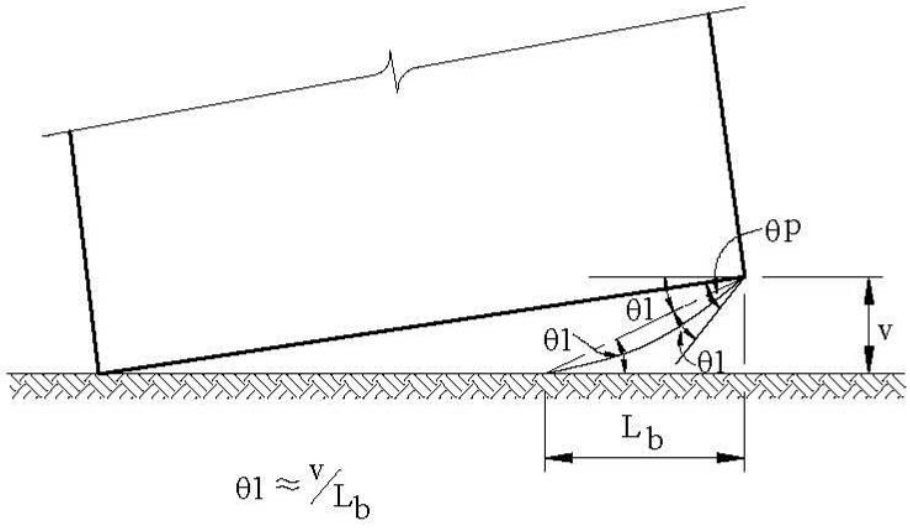


Figure C5.8: Plastic Rotation of Base Plate of Uplifting Tank

If θ_p exceeds this limit, it can be reduced by increasing the base plate thickness, which will reduce v for a given L_b. If the base plate thickness so calculated exceeds the wall thickness, it may be necessary to anchor the tank to avoid uplift.

These recommendations do not include minimum plate thicknesses for shell plates. However, the designer should be aware of typical construction requirements and code limits on plate thickness, such as those bulleted and tabulated below from API 650, (API, 2008):

- In no case shall the nominal thickness of the shell plates be less than the following (for mild steel tanks only):

Table C5.1: API 650 Tank Nominal Shell Plate Thicknesses, (API, 2008)

Nominal tank diameter (m)	Nominal shell thickness (mm)
< 15	5
15 to 36	6
36 to 60	8
> 60	10

- These nominal thickness requirements are needed for construction purposes and therefore may include any corrosion allowance provided that the shell is shown by calculation to be safe in service in the corroded condition.
- The maximum thickness of shell plates shall be 40 mm, for construction and handling purposes.

C5.6 Specific Requirements for Concrete Tanks

Fluid pressure loading in concrete tanks is carried by a combination of membrane action and vertical bending. The proportion of load carried by each of the mechanisms depends on the tank geometry and their relative stiffness. Although analysis of this behaviour is well-established for elastic response, the influence of cracking as the ultimate condition is approached will alter the proportion of load carried by the two mechanisms. The difficulty in analysis and the inappropriateness of extending elastic actions into the inelastic range, necessitate that a working stress method be used, with stresses limited to high but elastic

levels. The approach follows that recommended in NZS 3106 (SNZ, 1984). However, in some cases, notably squat rectangular concrete tanks where the load is predominantly carried by one action such as vertical bending, it may be that the ultimate strength can be predicted with accuracy. In such cases, the less conservative approach of Equation 5.1 may be used with section strength based on normal ultimate strength equations.

It is anticipated that the forthcoming revision of NZS 3106 will provide appropriate interpretation of the loadings code AS/NZS 1170. In the interim the earthquake actions given in Section 3.2 can be used for design of concrete tanks.

Prestressed Concrete

Shear stresses are unlikely to govern design. However, shear associated with bending in the vertical direction should be checked by calculating the principal tension stress existing under the combined effects of vertical prestress, and shear through the wall thickness.

Figure C5.9 illustrates the procedure for a typical tank wall subjected to axial compression force, P, moment, M_v, and shear, V, per unit length of wall. The shear force, V, may be found from the slope of the vertical bending moment diagram and will be a maximum at the wall base. The axial load and bending moment combine to give a linear distribution of direct stress given by:

$$f = \frac{P}{t} + \frac{12M_v y}{t^3} \qquad \text{(C5.12)}$$

The distribution of shear stress will be parabolic and may be expressed as:

$$\tau = \frac{1.5V}{t}\left[1-\left(\frac{2y}{t}\right)^2\right] \qquad \text{(C5.13)}$$

From Mohrs circle for stress, the principal tensile stress will be given by:

$$f_t = \frac{f}{2} - \sqrt{\frac{f^2}{4}+\tau^2} \qquad \text{(C5.14)}$$

The distribution of f_t will be unsymmetrical with the maximum occurring close to the centre, but offset to the side of reduced flexural compression stress, as shown in Figure C5.9. For most cases it will be sufficient to check f_t at the wall centre line (y = 0). Thus:

$$f_{to} = \frac{P}{t} - \sqrt{\left(\frac{P}{t}\right)^2+\left(\frac{1.5V}{t}\right)^2} \qquad \text{(C5.15)}$$

In Equations C5.13 and C5.14 the sign convention used is compression positive.

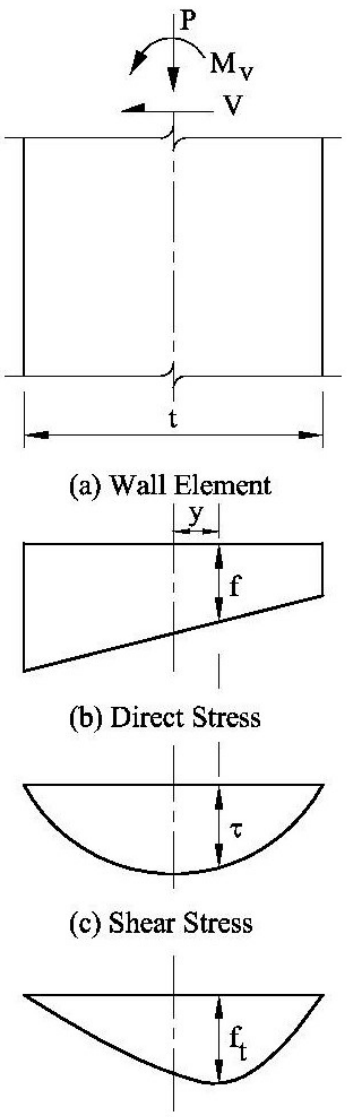


Figure C5.9: Principal Tension Stress Due to Shear

Cracking at vertical construction joints is considered acceptable, provided that the crack is controlled by mild steel reinforcement, a flexible sealant, or other acceptable means. In the flexible sealant case, a redistribution of stress occurs between the uncracked panels and the cracked construction joint, as shown in Figure C5.10. In Figure C5.10, the wall is subjected to circumferential hoop compression, P, (including prestress force) and moment, M, per unit wall height. At Section A-A, the surface stresses are given by:

$$f_{cA} = \frac{P}{t} + \frac{6M}{t^2} \quad \text{(C5.16)}$$

$$f_t = \frac{6M}{t^2} - \frac{P}{t} \quad \text{(C5.17)}$$

At Section B-B, the neutral axis position must propagate away from the tension face for the forces to balance the applied moments. Equilibrium requires that:

$$P = \frac{1}{2} f_{cB} x_B \qquad \text{(C5.18)}$$

and

$$M = Pe = P\left(\frac{t}{2} - \frac{x_B}{3}\right) \qquad \text{(C5.19)}$$

Hence

$$x_B = 3\left[\frac{t}{2} - \frac{M}{P}\right] \qquad \text{(C5.20)}$$

and

$$f_{cB} = \frac{2P}{3\left[\frac{t}{2} - \frac{M}{P}\right]} \leq 0.55 f_c' \qquad \text{(C5.21)}$$

Note Equation C5.21 applies only for:

$$\frac{t}{6} \leq \frac{M}{P} \leq \frac{t}{2}$$

For $\frac{M}{P} < \frac{t}{6}$ the entire section is in compression and stresses at A-A and B-B are identical, as:

$$\frac{M}{P} \rightarrow \frac{t}{2} \qquad f_{CB} \rightarrow \infty$$

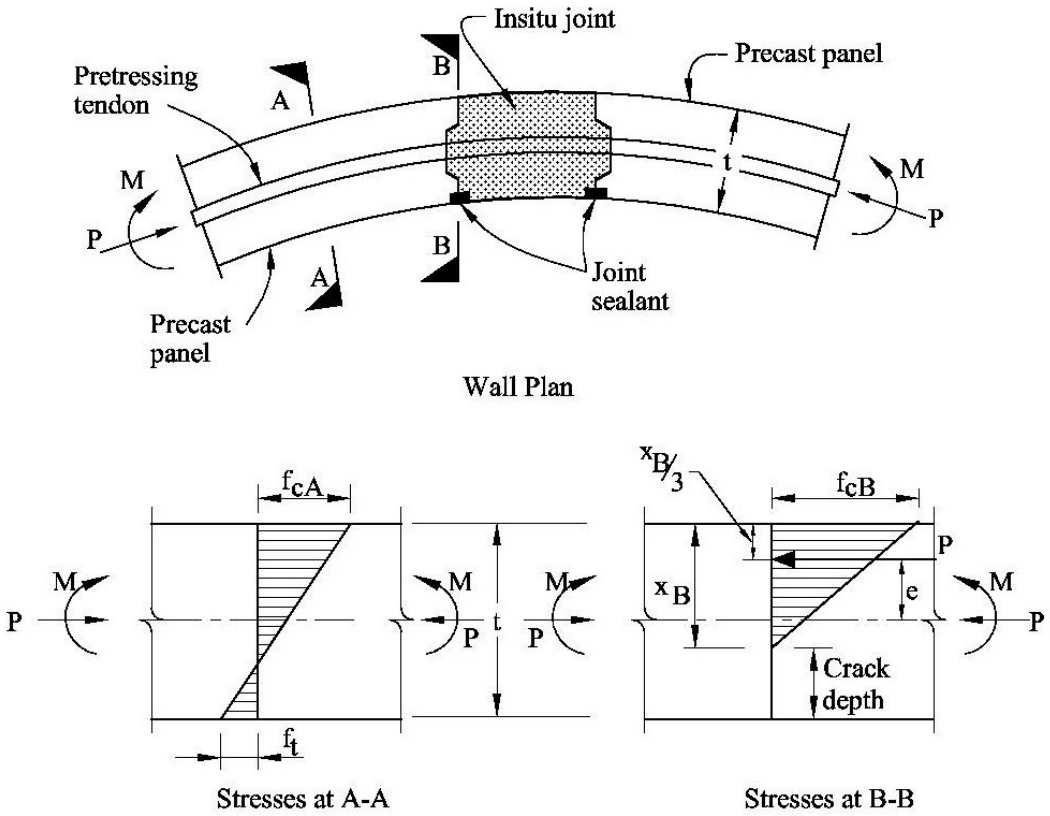


Figure C5.10: Stresses through Wall Thickness of Concrete Tank

C5.7 Tanks Constructed from Materials other than Concrete or Steel

It would be unrealistic to attempt to write code clauses for all possible tank materials, such as timber, copper, aluminium, or corrugated steel. This clause gives guidance in the principles to which such tanks should be designed.

C5.8 References

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American Petroleum Institute (API), 2008. API-650, Welded Steel Tanks for Oil Storage, Eleventh Edition, Addendum 1. API, United States.

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Section 6 - Commentary
Foundations

C6.1 Scope

A wide variety of factors will affect the behaviour of the foundation and the supported structure. These factors together with the consequences of failure will influence the degree of conservatism necessary in the design, and the extent and sophistication of the necessary investigations. For this reason it is not practicable to establish design criteria to cover all situations and it is recommended that suitably experienced geotechnical engineers should be engaged for foundation design for all major or critical tanks.

This section also sets out some specific factors for the investigation and design of storage tank foundations for New Zealand conditions and in particular for seismic loading cases. There is an absence of comprehensive published codes of practice for foundation design. Available codes do not specifically or adequately deal with the particular problems of seismic design, New Zealand soil/geological conditions, and storage tanks. For major or hazardous storage tanks the foundation investigation and design should therefore be carried out by an experienced geotechnical engineer.

The New Zealand Building Code Verification Method B1/VM4, Foundations, provides requirements to be met for the ultimate limit state design of foundations.

Recommendations on foundation investigation and design for flexible storage tanks are given in API 650 (API, 2008), BS 2654 (BSI, 1984) and unpublished industry guides. These provide good, though incomplete, guidelines for considering static loading conditions. The general soil mechanics theory, testing, and analysis methods, necessary to calculate stability and settlement, are set out in handbooks such as Winterkorn and Fang (1975). Specific papers related to static stability and settlements associated with steel storage tanks include Duncan and Orazio (1984), Penman (1978), Harris (1976), Duncan, Lucia, and Bell (1980).

C6.2 Site Investigations

The site investigations should be sufficiently extensive to identify the distribution and appropriate properties of all site soils that may influence tank performance, determine groundwater conditions and identify/describe geological defects. The adequacy of tank foundations will be largely dependent on the effectiveness of the site investigations and detection of soil materials with undesirable static or cyclic loading behaviour or settlement characteristics.

Site investigation techniques and practice are described in a number of text books and codes of practice (Hunt, 1984), (MWD, 1982). It should be noted that these references do not always adequately address the full range of soil conditions present in New Zealand and in particular the soils referred to in Section C6.3.

C6.3 Foundation Evaluation

Soils requiring special attention include the following:

Erodible Soils

Soils susceptible to internal or surface erosion/dispersion include volcanic ash, weathered ignimbite and loess materials.

While some tests have been proposed to assess erodibility (e.g. pinhole tests proposed by Sherard, Dunnigan and Deeker (1976)), specialist local engineering geological and geotechnical input is essential for assessing erodibility. For example, loess in the Christchurch area has erodibility potential which is related to soil chemistry.

Liquefiable Soils

Established techniques are available on identification of liquefiable soils Seed and Idriss (1982), Berrill and Davis (1985), Youd and Idriss (1996).

Collapsible Soils

These soils, while of considerable strength and stability under static loading, have a potential for rapid and major loss of strength with associated volume reduction when saturated or subjected to increased, dynamic, or impulse loading. Collapsible soils include loess and volcanic air fall materials. Their potential for collapse can generally be inferred from density, moisture content, liquid limit, strength sensitivity, and volume change characteristics on flooding. Clemence and Finnbar (1981) provides guidelines for identifying collapsible soils.

Sensitive Soils

Sensitive soils such as 'quick clays' and collapsible soils exhibit marked strength loss when sheared beyond their peak strength. They are likely to behave poorly as foundations during earthquakes.

The strength loss may be 'permanent' or temporary, in the latter case the soil being referred to as thixotropic. Soil composition and chemistry can be used to identify potentially sensitive soils.

C6.4 Foundation Analysis

Quantitative analysis or qualitative assessment of the foundation will generally be required for some or all of the following. The sophistication and type of analyses will depend on circumstances.

- Overall stability of the site where slope failure is possible.
- The pattern of support applied by the ground to the tank as it affects overturning stability of the tank.
- Foundation resistance to lateral movement of the tank by base sliding.
- Bearing capacity of the ground supporting the tank.
- Settlement of the tank.
- Site modification of seismic ground motions.
- Foundation stiffness parameters.

C6.4.1 Strength Reduction Factors

Strength reduction factors should be based on advice by a specialist geotechnical engineer and in accordance with the New Zealand Building Code B1/VM4 (BIA, 1992). As a guide the following strength reduction factors would generally be appropriate. Pender (2000) gives further guidance on strength reduction factors for foundation design.

(i) Slope stability: Soil strength reduction factor = 0.85 for a pseudostatic force coefficient equal to 67% of peak ground acceleration, defined in Section 3.

(ii) Bearing capacity: Soil strength reduction factor = 0.7.

(iii) Liquefaction: Soil strength reduction factor = 0.8.

(iv) Base sliding: Soil strength reduction factor = 0.85 under the maximum tank response calculated in accordance with Section 3.

Some guidance as to reasonable values of shear wave velocity are given in the commentary to NZS 1170.5 (SNZ, 2004) for the general site subsoil classes.

C6.5 Foundation Details

Foundation detailing should ensure that:

- Soils with questionable seismic behaviour (i.e. potential for loss of strength or excessive settlement) do not directly support the structure.
- Structural foundation components are capable of redistributing loads in an acceptable fashion if local failure of a foundation occurs momentarily.
- Drainage provisions, tie-down of buried tanks and other appropriate provisions are provided to prevent catastrophic consequences of minor leakage or spillage or build up of groundwater pressures associated with an earthquake.

Ground Improvement

Sites with significant depths of loose or soft, weak and compressible deposits may be improved by various techniques which densify the materials and hence strengthen them. Alternatively, weak materials may be replaced either locally or over the whole foundation. Where ground improvement does not extend over the whole site, special consideration should be given to potential differential settlements and their consequences.

Ground improvement techniques include: preloading with or without sand drains or wicks to accelerate consolidation; vibroflotation compaction; dynamic (Menard) compaction; compaction piling; grouting. Stone columns while considered as a form of ground improvement, may be more realistically considered as foundation units as the structure is supported.

For large diameter storage tanks the critical area for bearing capacity is at the edge of the tank. Two techniques are commonly used to enhance edge-bearing capacity. The material beneath the edge can be excavated and replaced by a high strength granular material (stone wall). Alternatively a concrete ringwall/footing can be used to increase the area of bearing and extend the potential bearing failure surface deeper to enhance stability. The need for, selection of, and detailing of stonewalls or concrete ring walls will depend on local conditions.

In all cases where ground improvement techniques are employed, insitu properties (usually penetration resistance) should be measured before and after the ground improvement. The results should be used to check the effectiveness of the improvement techniques.

Piled Foundations

Where storage tanks are to be founded on piles passing through weak materials to a hard bearing stratum, consideration should be given to the effects of negative skin friction on the piles and the possibility, and effect, of loss of lateral support to the piles if soft surface material liquefies or loses strength.

Foundation Drainage

Where appropriate, drainage should be provided beneath tank foundations to prevent build up of groundwater pressures beneath the tank or to intercept and discharge any leakage from the tank. All such drains should be graded to allow for generous fall (approximately 1 in 50) plus differential settlement of twice that calculated. Where the tank is founded on materials that may be adversely affected by addition of water (e.g. loess, volcanic materials, swelling clays), or where tank leakage would cause unacceptable groundwater contamination, the whole site should be covered with an impermeable blanket on which the drainage system is laid. The impermeable blanket should have an upper surface graded to ensure leakage enters the drains. To minimise distance from leakage sources to drains, it is recommended that drains be placed beneath any sealant-treated or flexible joints (e.g. between concrete slabs).

Drainage materials should be selected for scour resistance where they are not positively confined. Outlets from drainage layers should, where possible, be monitorable to enable any leaks to be located.

Mounded or Buried Tanks

Adequate provisions should be made to ensure such tanks do not float out of the ground. Either free draining backfill with a positive drainage outlet or tie-down to prevent flotation may be considered. The material used over mounded tanks should be free-draining and non-erodible. The slope stability of the mound should also be considered.

Foundation improvement and footing details specifically for steel tanks are addressed in API 650 (API, 2008) and BS 2654 (BSI, 1984).

C6.6 References

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Section 7 - Commentary

Design Details

C7.1 Scope

The aim of this section is to highlight areas of tank seismic design that can have special problems and to give designers some guidance.

C7.2 Influence of Details

General

The majority of the wide variety of tanks designed have some form of cylindrical shape to take advantage of the simple and efficient design for hoop tension under service conditions. These efficient designs imply high hoop tensile stresses.

With the seismic loadings, actions and expected displacements derived from previous sections in mind, the designer should:

- Examine the tank parts, attachments, fittings and piping connections looking for seismic behaviour that will increase hoop tension or produce actions, such as severe bending, not normally considered.
- Examine the ancilliary parts for adequate attachment and for the consequences of any restraint they may impose on the expected tank displacements.

Piping Connections

The design aim is to provide sufficient flexibility in the piping system to avoid damage to the tank shell or tank connection nozzles and valves. Valves and nozzles should be stronger than any other adjacent item to ensure that transfer of the tank contents can be controlled after the design earthquake.

The three actions illustrated in Figure C7.1 produce stresses that add to already high hoop tensile stresses.

The three actions not illustrated, circumferential and meridional shear and torsion about the radial axes are normally not significant.

Under seismic conditions the three actions illustrated are produced by relative displacements between the piping and the tank. The tank may rock or slide and the piping may move about any of the axis illustrated. An increase in the radial thrust is also produced by an increase in the pressure within the tank.

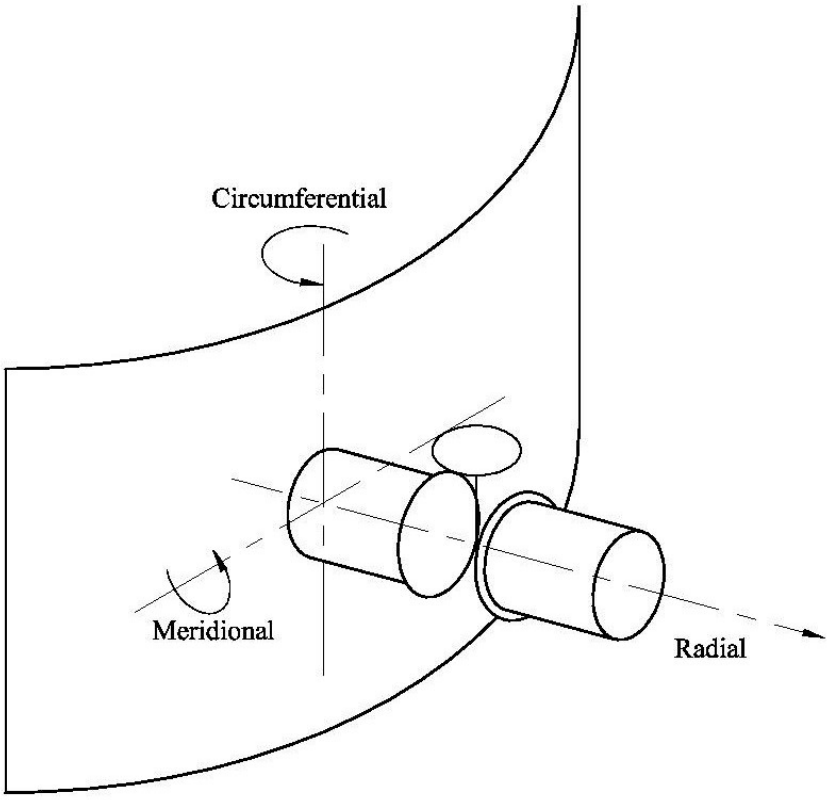


Figure C7.1: Forces Induced in Shell by Piping Connection

Thin Walled Tanks

Thin tank shells with standard shell reinforcement are often not strong enough to force ductile yielding to occur in connected piping, particularly since the piping stiffness can easily be larger than the shell stiffness.

Some guidance on the strength of thin shells at reinforced nozzle connections is available in Billimora and Hagstrom (1978) which uses limits of 110% of the design allowable stress for the maximum hoop tension and three times the allowable stress for the maximum surface stress.

Figure C7.2 illustrates three methods of providing flexibility in the piping to ensure that the tank shell is not overstressed.

In choosing a piping configuration to provide flexibility the ductility demand at the bends can be adjusted to a suitable level by an appropriate choice of the length from the valve to the first bend, the offset length, and the length from the second bend to the guide.

In the cases where any piping configuration is still too stiff, very flexible piping connections to the tank nozzles can be provided by the use of armoured hose, or special devices such as compensators, swing arm joints or ball joints, as illustrated in Figure C7.2. Of the three methods illustrated the use of special devices is the least desirable because such devices have their own normally narrow limits on displacements. Beyond these limits such devices normally have only a small ductile range, in which case the design may have an undesirable or unpredictable failure mechanism.

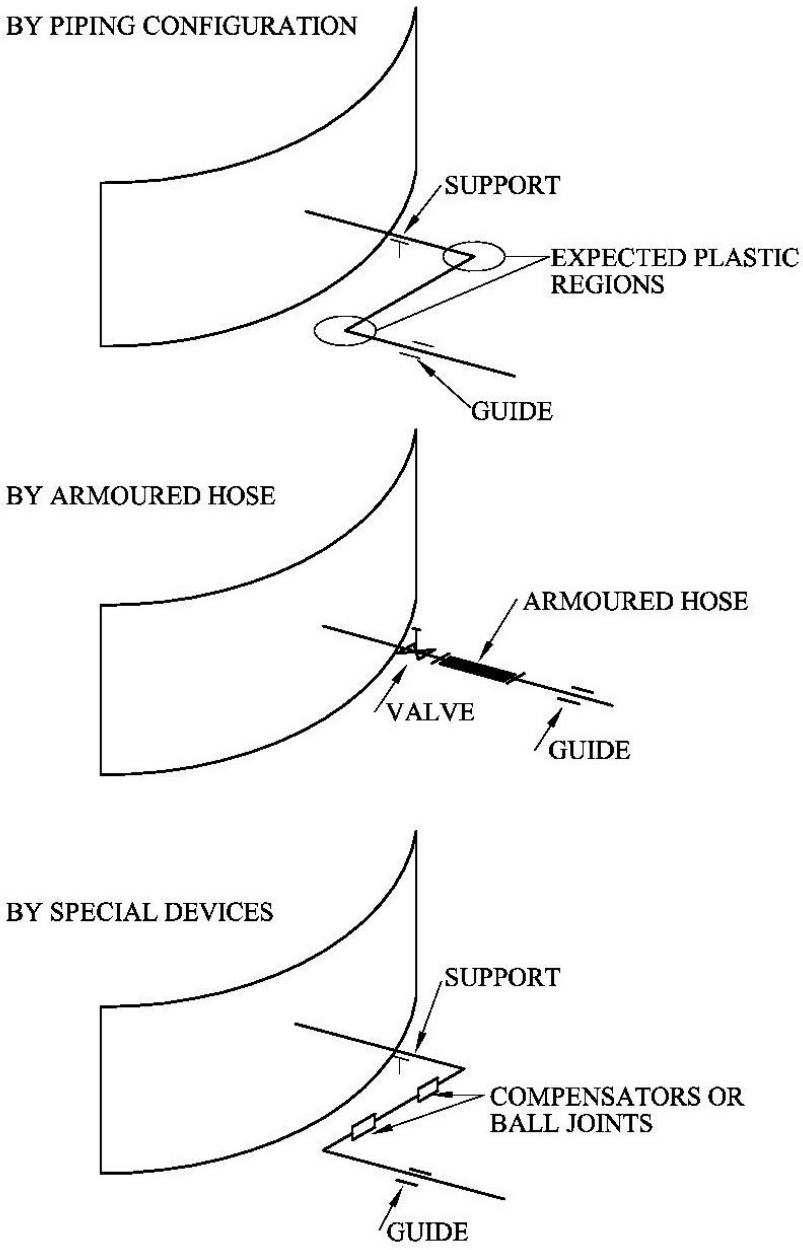


Figure C7.2: Provision of Flexibility in Connecting Piping

Thick Walled Tanks

The designer should be able to obtain ductile behaviour in the pipework by a suitable choice of piping configuration without having to use any special devices. Such ductile behaviour can be expected to occur in a suitably placed bend or elbow.

Steel Tanks

Floating Roofs

Some codes require the roof to be designed for buoyancy under a rainwater flooded condition. Seismic effects need not be considered concurrently with this condition. Roof anti-rotation posts designed to current codes are unlikely to resist roof rotation or roof displacement due to liquid sloshing under seismic attack. Such posts should be designed to be expendable with the floor attachment detailed to prevent tearing of the floor plate. The flexible hose type of roof drain is recommended.

Rolling roof access ladder details should allow large roof displacements and rotations without causing the ladder to jam and act as a strut.

Roof seal damage can be expected in the design earthquake, but should not affect the serviceability performance.

Fixed Roofs

Tanks designed for a nominal internal gas pressure have roof plates which normally sit loosely on roof support members and are attached only at the shell and centre boss with a weak frangible joint to provide an overpressure relief mechanism. Where insufficient freeboard is provided this joint should be checked using the forces generated by liquid sloshing, in accordance with Section C3.9.

Roof support members will impose concentrated loads on the shell top ring when acting to maintain circularity of the shell.

The induced forces can be estimated using the methods set out in Section 3.

Tank Shells

The shell behaviour considered here assumes that the shell maintains reasonable circularity throughout its height. Shell wind girders designed to current code requirements should be checked using the pressure distributions set out in Section 3, in accordance with Section C5.4.2.

Horizontal Cylindrical Tanks

Such tanks are often designed as pressure vessels and as such have a greater resistance to buckling and damage from piping connection forces.

Restraint of internal components should be considered together with any requirements for freedom to expand under thermal conditions.

Tank Anchorage

Past performance indicates that poorly detailed anchor bolts have a potential for tearing the tank shell. Their performance can be improved through the application of appropriate design details. There are two factors which must be considered in providing anchorage. In the event of an earthquake exceeding the design level, loads may cause

(i) Anchor bolts to be pulled out of the foundation leading to possible shell and attached piping damage; and

(ii) Failure of the anchor bolt attachments at the shell leading to possible shell failure.

The pullout problem can be overcome by providing a bearing plate at the bottom of the anchor bolt. Attachment to the shell is provided by an anchor bolt ring. These details are shown in Figure C7.3, reproduced from Miles (1977). The anchor ring also provides resistance to the moment caused by the eccentricity of the anchor bolts from the shell.

The anchor bolts should be designed so that they yield (taking into account overstrength) before there is an anchorage or shell attachment failure. In practice, to limit the degree of overstrength and to ensure yielding occurs in the desired position, it will usually be necessary to machine the bolts to the required diameter over the yielding length.

If an earthquake exceeding the design event occurs the bolts will provide energy dissipation when they stretch. Large curved washers located under the retaining nut can also provide energy absorption as they flatten out under the load.

When anchor bolts are used, the foundation must be designed to withstand the imposed loading (including torsion) corresponding to bolt overstrength.

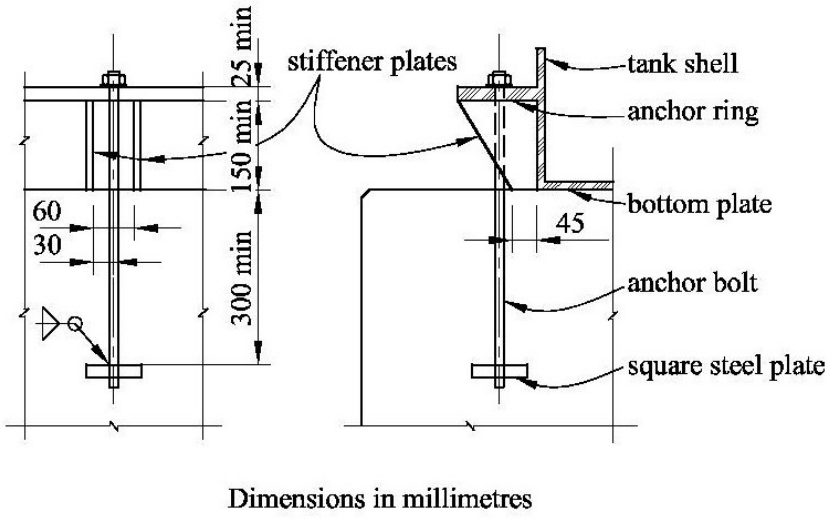


Figure C7.3: Recommended Anchor Bolt Details, (Miles, 1977)

Wall/Footing Detail for Concrete Tanks

In general the wall/footing interface of concrete tanks has sufficient reinforcement through the joint to transmit the shear force required by Section 4.5. However for precast tank construction the wall panels are often located in a preformed slot in the ring beam footing. Radial friction between the wall base and footing may be insufficient to resist the seismic shear thereby requiring some form of mechanical restraint such as radial steel dowels connecting the precast panels to the footing, or by use of a structural nib (see Figure C7.4).

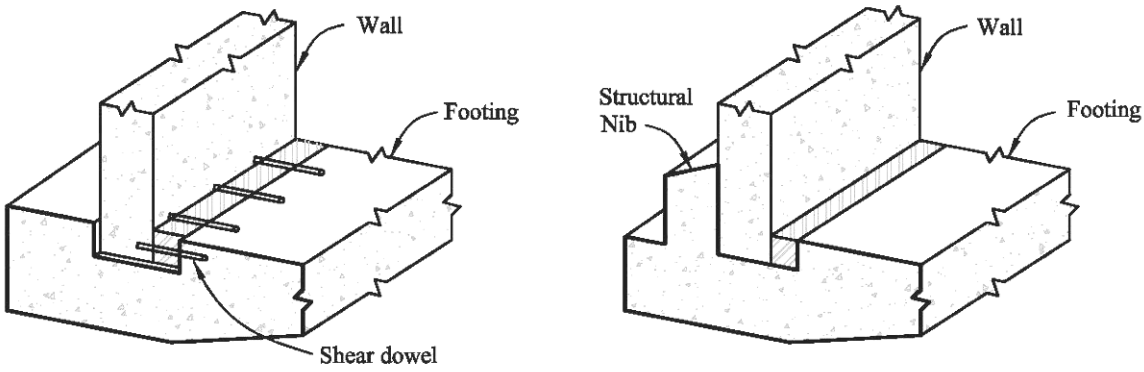


Figure C7.4: Shear Connection between Precast Panels and Footing for Concrete Tank

The roof/wall joint is also subject to seismic shear, from the horizontal acceleration of the roof only and therefore of much lesser magnitude.

For tanks with a concrete roof that engages the top of the wall the roof edge can be adequately reinforced to withstand the seismic forces. However, other roofs will require mechanical means such as shear dowels to prevent excessive movement and subsequent damage to the roof.

C7.3 References

Billimora, H.D. and Hagstrom, F., 1978. Stiffness Coefficients and Allowable Loads for Nozzles in Flat-Bottom Storage Tanks. Journal of Pressure Vessel Technology Transactions of ASME, Vol. 100, November 1978. American Society of Mechanical Engineers (ASME), United States.

Miles, R.W., 1977. Practical Design of Earthquake Resistant Steel Reservoirs, Proceedings of the ASCE Lifeline Earthquake Engineering Conference, Los Angeles. American Society of Civil Engineers (ASCE), United States.

Appendix A:

Dimensionless Design Charts for Hoop Force and Bending Moment in Walls of Cylindrical Tanks with Constant Wall Thickness

This Appendix includes non-dimensionalised method of calculation for hoop force, $N_{\theta n}$, vertical bending moment, M_{zn}, for convective, impulsive (with rigid container assumption), and hydrostatic, pressure distributions acting on a cylindrical storage tank with a vertical axis. For all charts, the mass density ρ and the appropriate acceleration g for hydrostatic pressure, spectral acceleration for convective and impulsive motion associated with flexible walls, and peak ground acceleration for impulsive response associated with rigid walls are equal to unity. Thus the form of components of the hoop force are given by:

$$N_\theta = N_{\theta n} R.P \tag{A.1}$$

and that of the vertical bending moment is:

$$M_z = M_{zn} R.P.t \tag{A.2}$$

where P is a representative pressure.

For the hydrostatic stress resultants, the representative pressure is that at the base of the tank, $\gamma_\ell H$. For convective stress resultants, the pressure used is that at the top of the tank, Z = H, and at $\theta = 0$. From Equation C3.6:

$$P = p_c(H,0) = 0.837\, C_d(T_1)\, \gamma_\ell R \tag{A.3}$$

where T_1 is given by Figure C3.20.

For the impulsive stress resultants, the representative pressure is the pressure occurring at the base of the tank, z = 0, at $\theta = 0$, for the rigid tank. This is given by Equation C3.3 as:

$$p_i(0,0) = q_o(0)\, C_d(\check{T}_o)\ \gamma_\ell R \tag{A.4}$$

where $q_o(0)$ is given by Figure C3.3 and $\check{T}_o$ is the period of the impulsive mode of vibration including soil-structure interaction effects.

Approximate solutions for flexible tanks can be obtained by using $\check{T}_f$ instead of $\check{T}_o$ in equation A.4. This approximation will provide a conservative estimate of the critical stresses in tanks with uniform walls (see Section C3.3).

The expanded form of the components of the hoop force are then:

(i) Hydrostatic

$$N_{\theta h} = N_{\theta nh} R\ \gamma_\ell\ H \tag{A.5}$$

(ii) Convective

$$N_{\theta l} = 0.837\, N_{\theta n1}\, R\, C_d(T_1)\ \gamma_\ell R \tag{A.6}$$

(iii) Impulsive, rigid tank

$$N_{\theta i} = N_{\theta ni}\, R\, q_o(0)\, C_d(\check{T}_o)\ \gamma_\ell R \tag{A.7}$$

(iv) Impulsive, flexible tank (see Section C4.3.1)

$$N_{\theta i} = N_{\theta ni}\, R\, q_o(0)\, C_d(\check{T}_f)\ \gamma_\ell R \tag{A.8}$$

(v) Vertical

$$N_{\theta v} = N_{\theta nh}\, R\, C_d(\check{T}_v)\ \gamma_\ell\ H \tag{A.9}$$

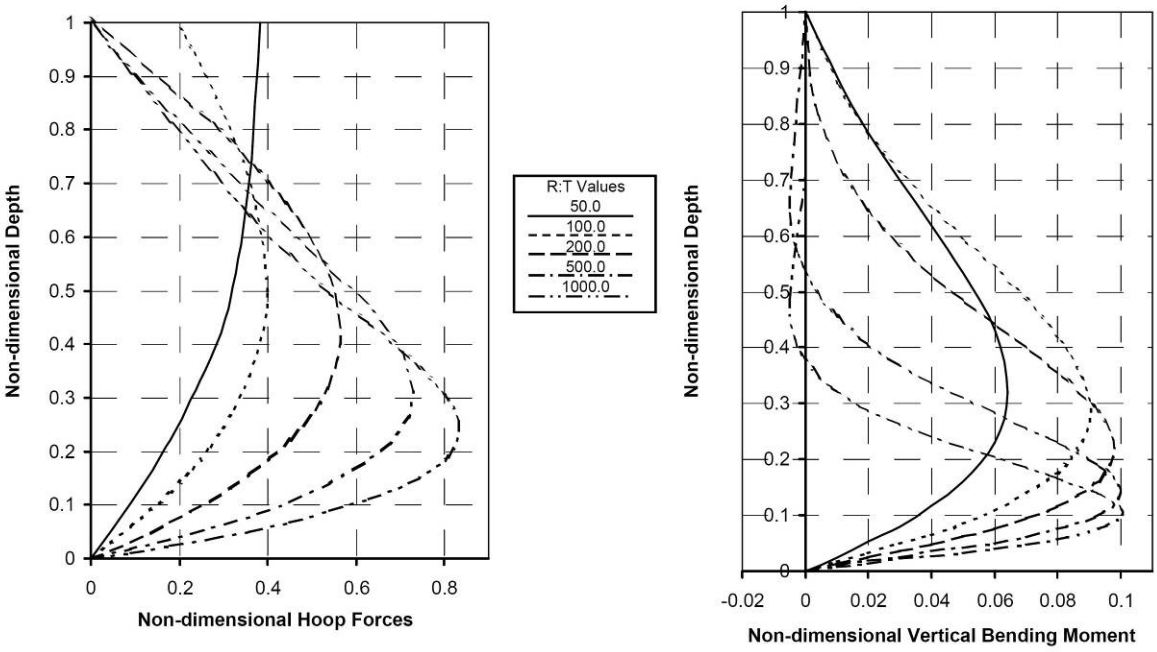
$$= N_{\theta h}\, C_d(\check{T}_v)$$

Figure A-1 to A-15 give the various non-dimensionalised hoop stress and bending moment coefficients plotted against a non-dimensionalised depth and other significant variables.

The dynamic components of any stress resultant are to be combined by the Square Root of the sum of the Squares method in accordance with Section 4.2.

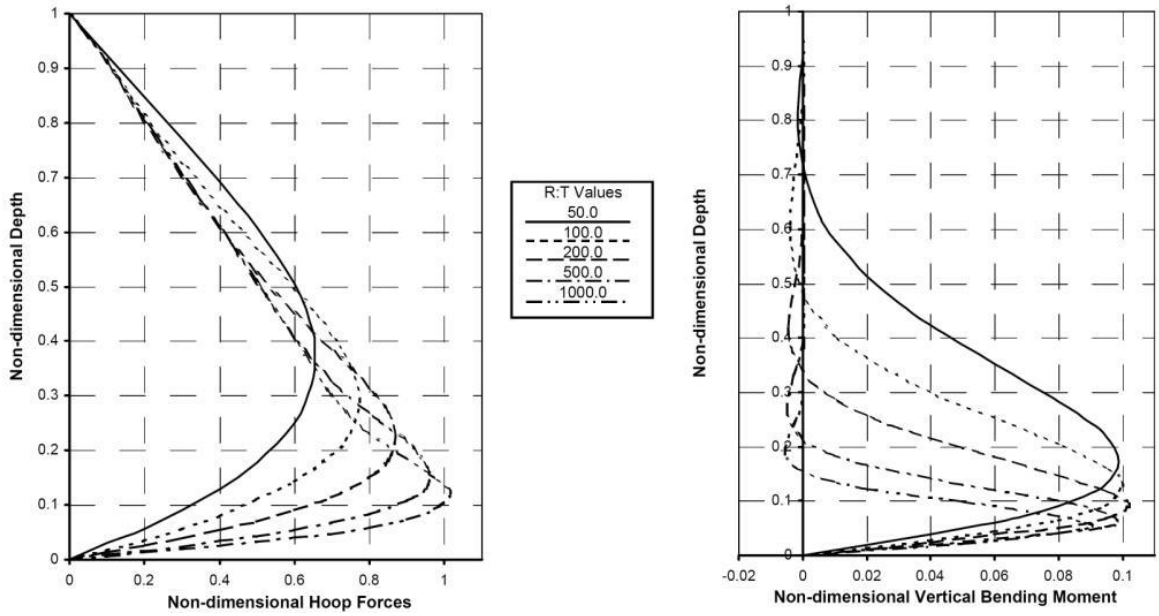
$$N_\theta = \sqrt{N_{\theta l}^2 + N_{\theta v}^2 + N_{\theta i}^2} \quad \text{(A.10)}$$

The form of the equation for combining vertical bending moment follows the form of Equation A.10, with the appropriate modifications for use of Equation A.2 rather than Equation A.1.



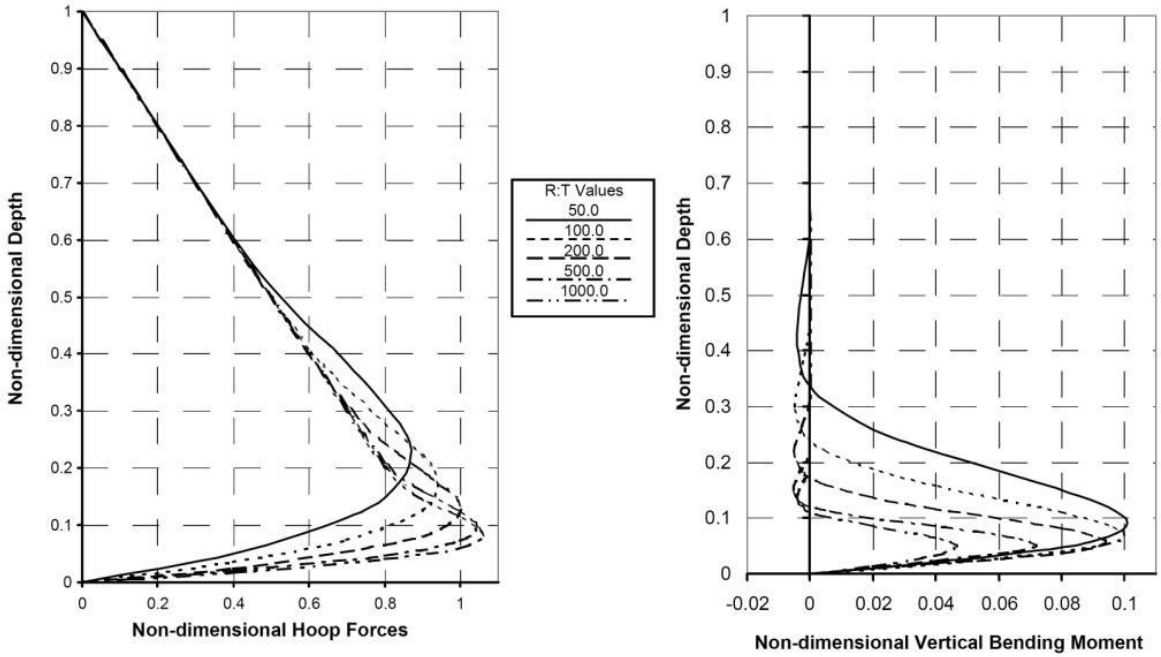
$N_{\theta nh}$ M_{znh}

Figure A-1: Hydrostatic Pressure, H:R=0.2



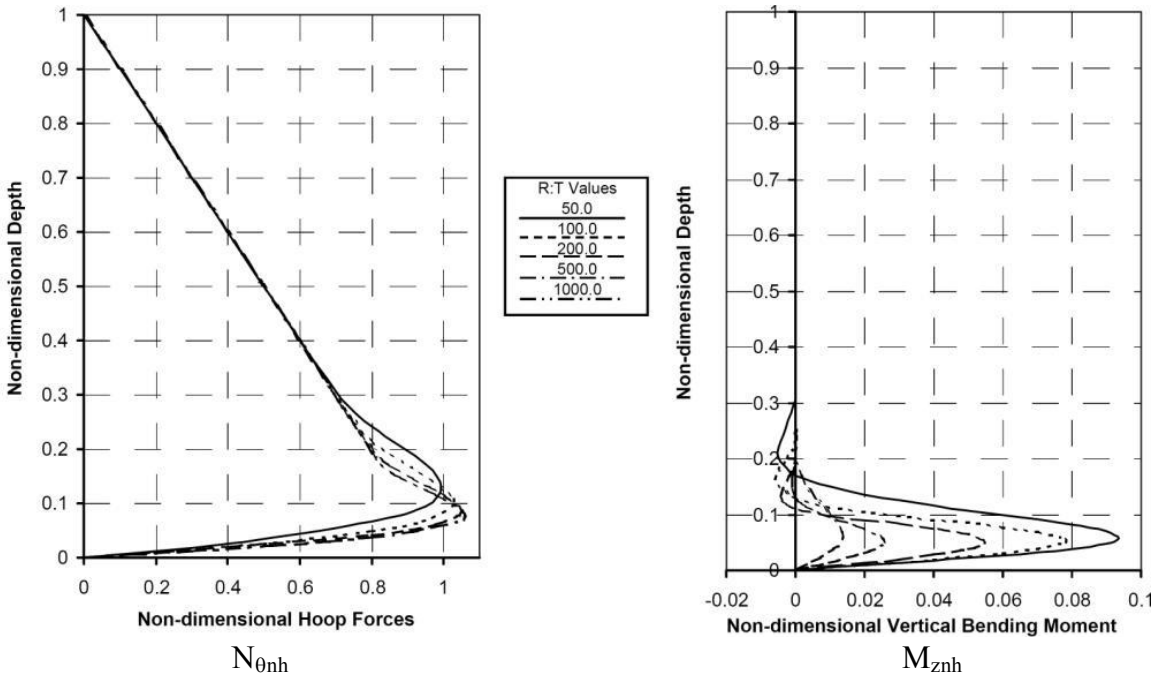
$N_{\theta nh}$ M_{znh}

Figure A-2: Hydrostatic Pressure, H:R=0.5



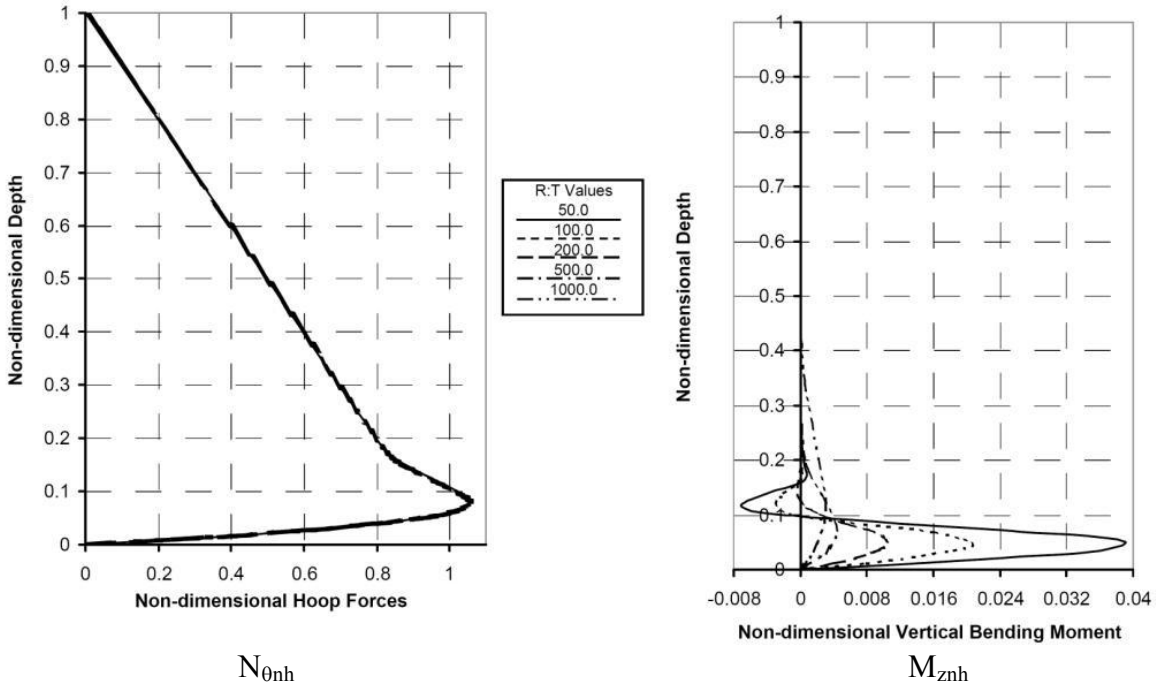
$N_{\theta nh}$ M_{znh}

Figure A-3: Hydrostatic Pressure, H:R=1.0



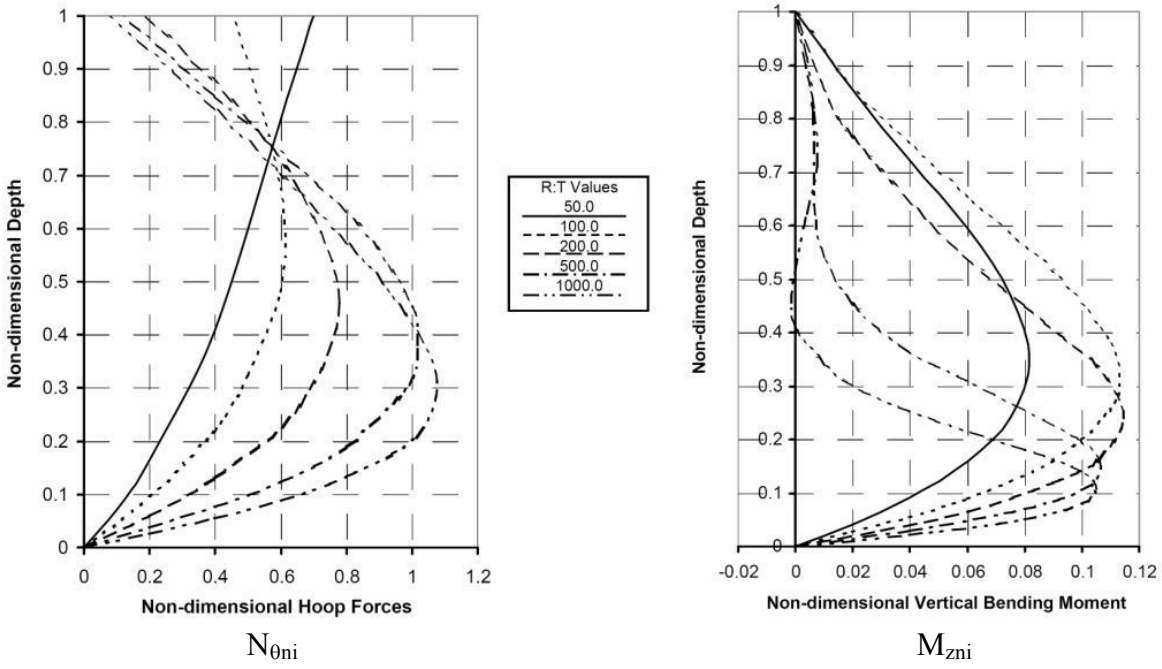
$N_{\theta nh}$ M_{znh}

Figure A-4: Hydrostatic Pressure, H:R=2.0



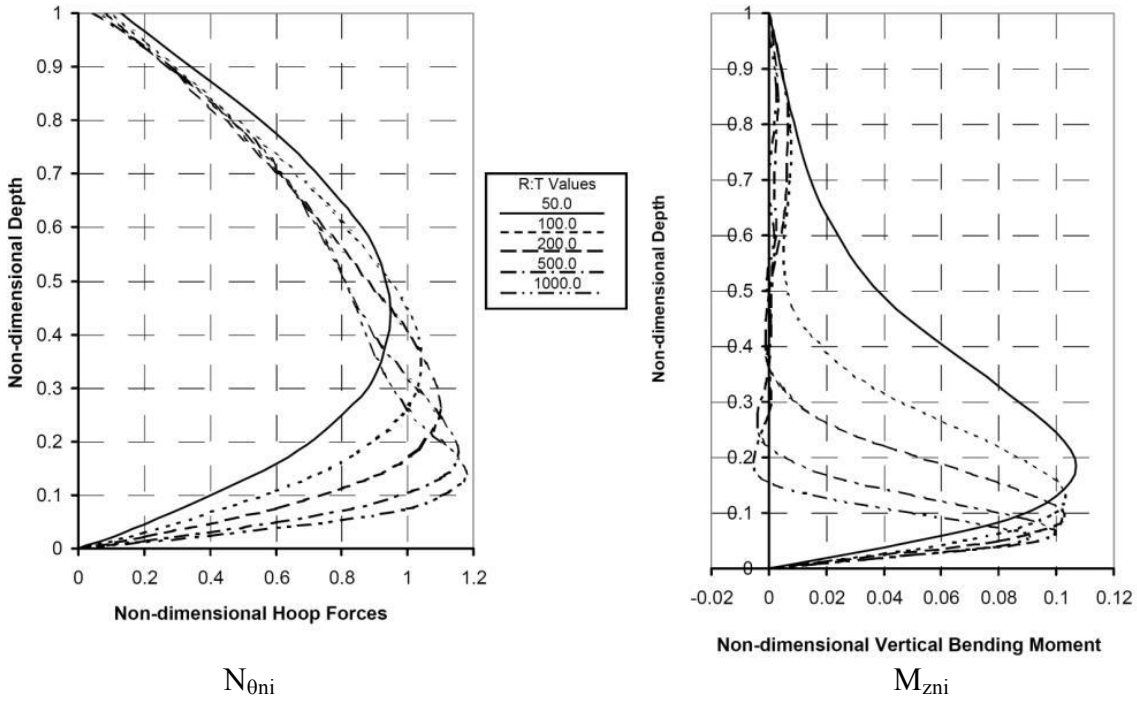
$N_{\theta nh}$ M_{znh}

Figure A-5: Hydrostatic Pressure, H:R=5.0



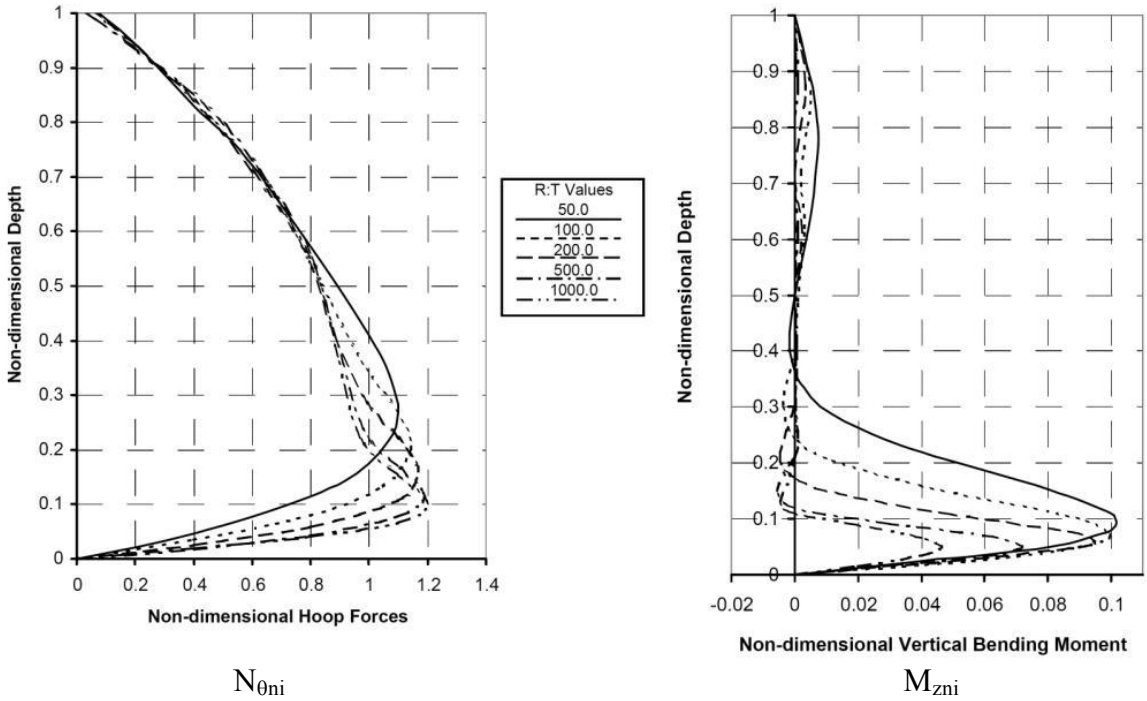
$N_{\theta ni}$ M_{zni}

Figure A-6: Impulsive-Rigid Pressure, H:R=0.2



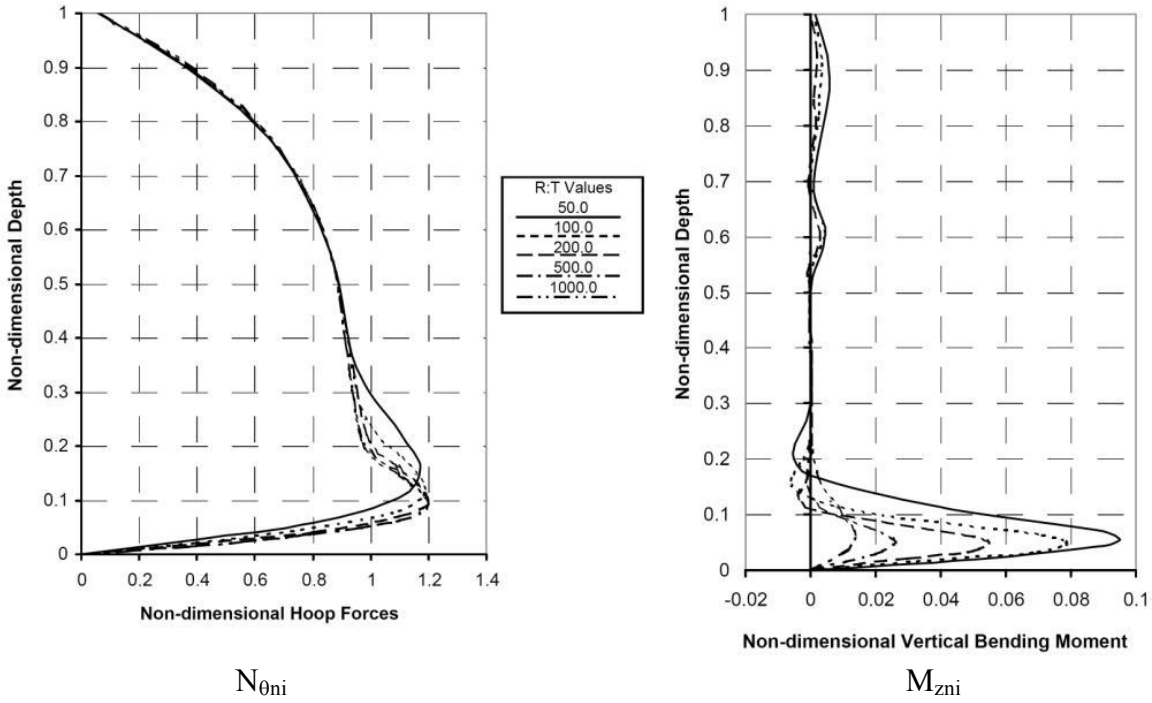
$N_{\theta ni}$ M_{zni}

Figure A-7: Impulsive-Rigid Pressure, H:R=0.5



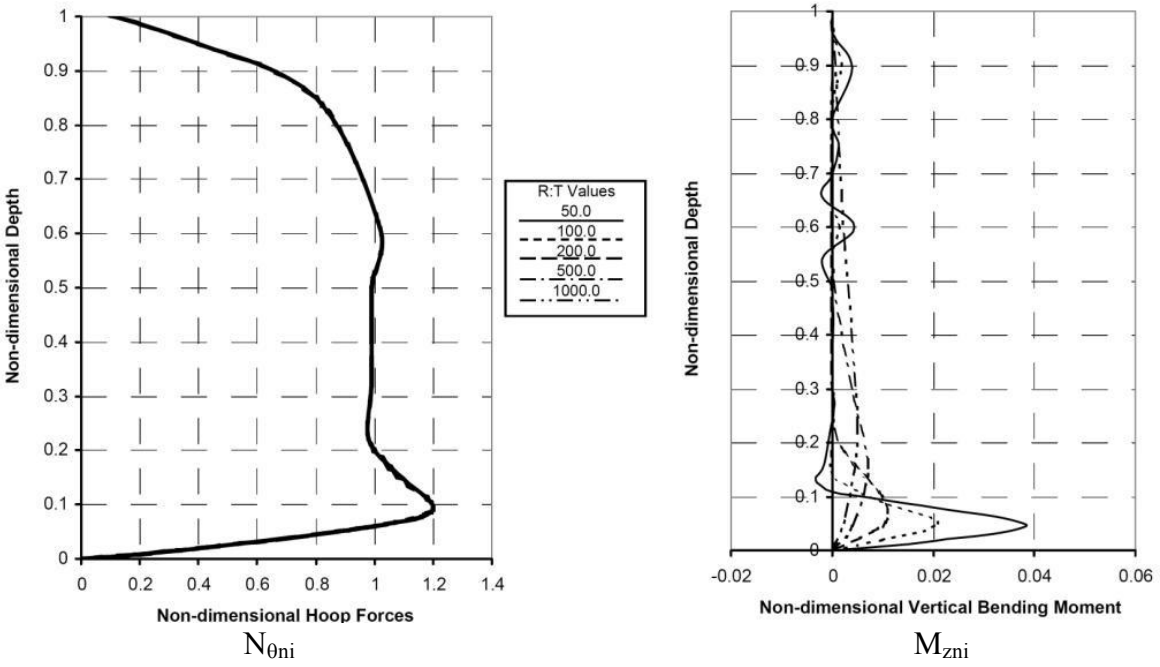
$N_{\theta ni}$ M_{zni}

Figure A-8: Impulsive-Rigid Pressure, H:R=1.0



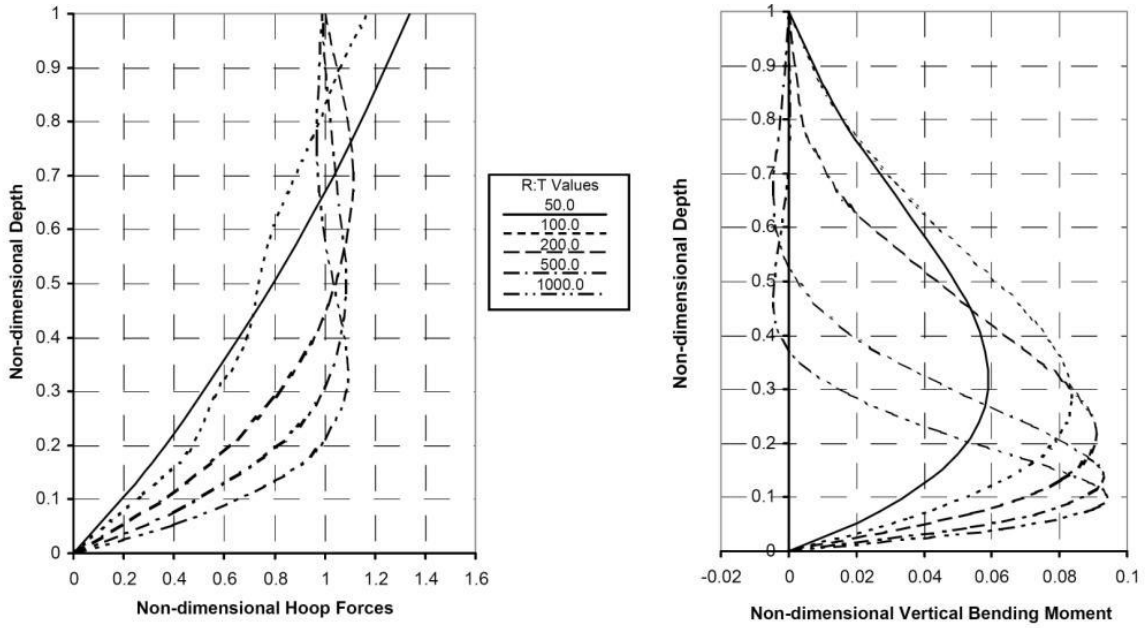
$N_{\theta ni}$ M_{zni}

Figure A-9: Impulsive-Rigid Pressure, H:R=2.0



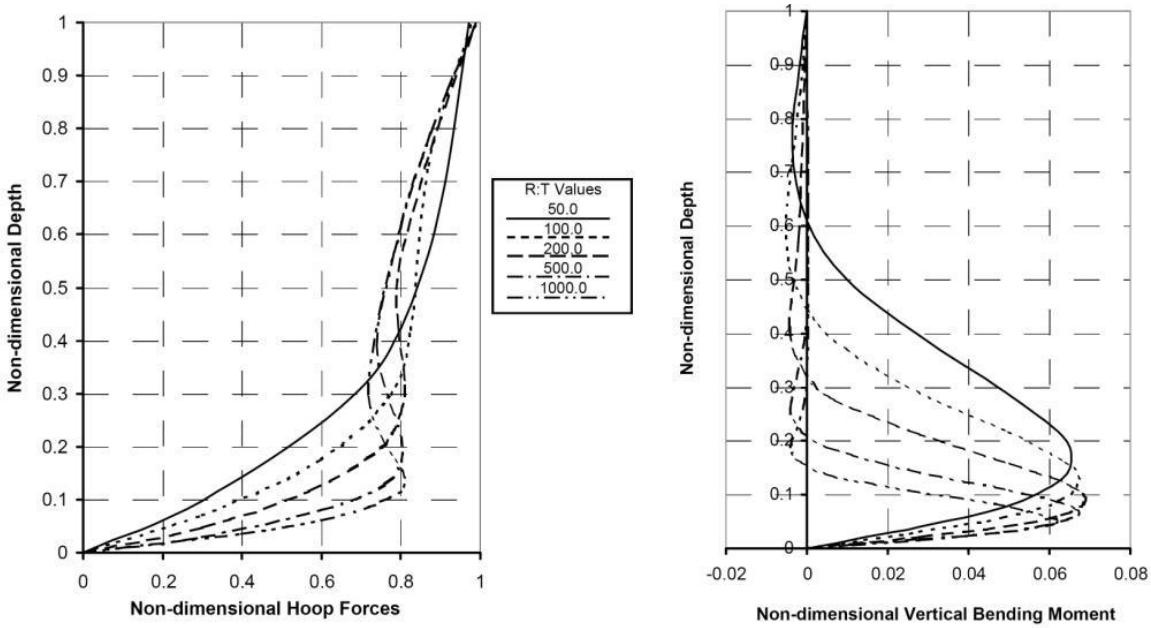
$N_{\theta ni}$ M_{zni}

Figure A-10: Impulsive-Rigid Pressure, H:R=5.0



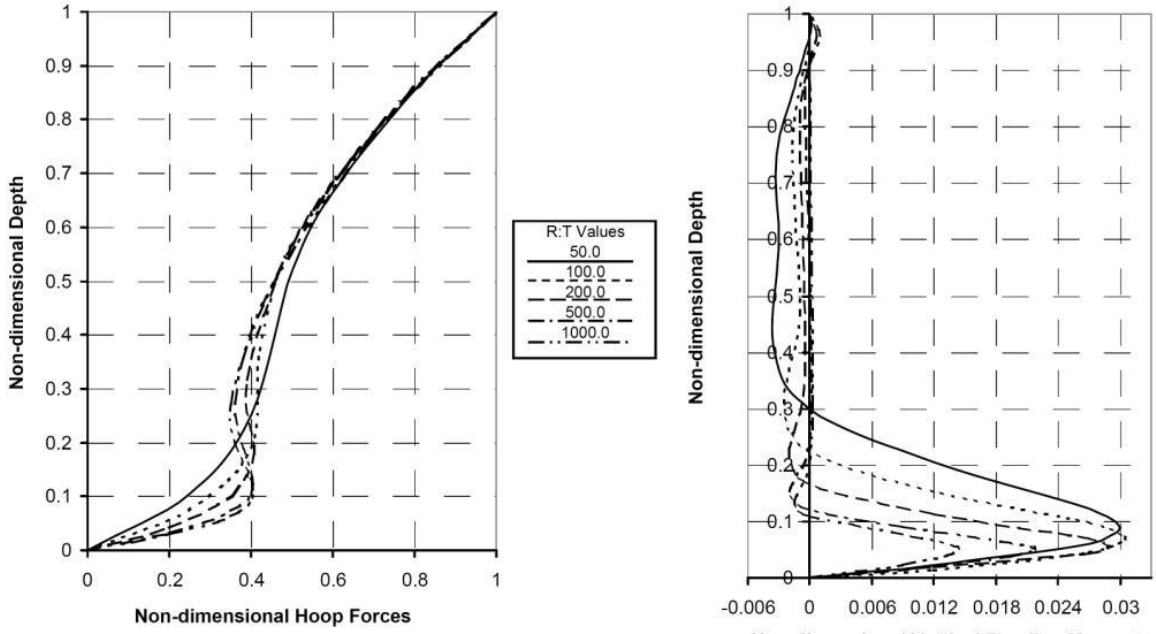
$N_{\theta n1}$ M_{zn1}

Figure A-11: Convective Pressure, H:R=0.2



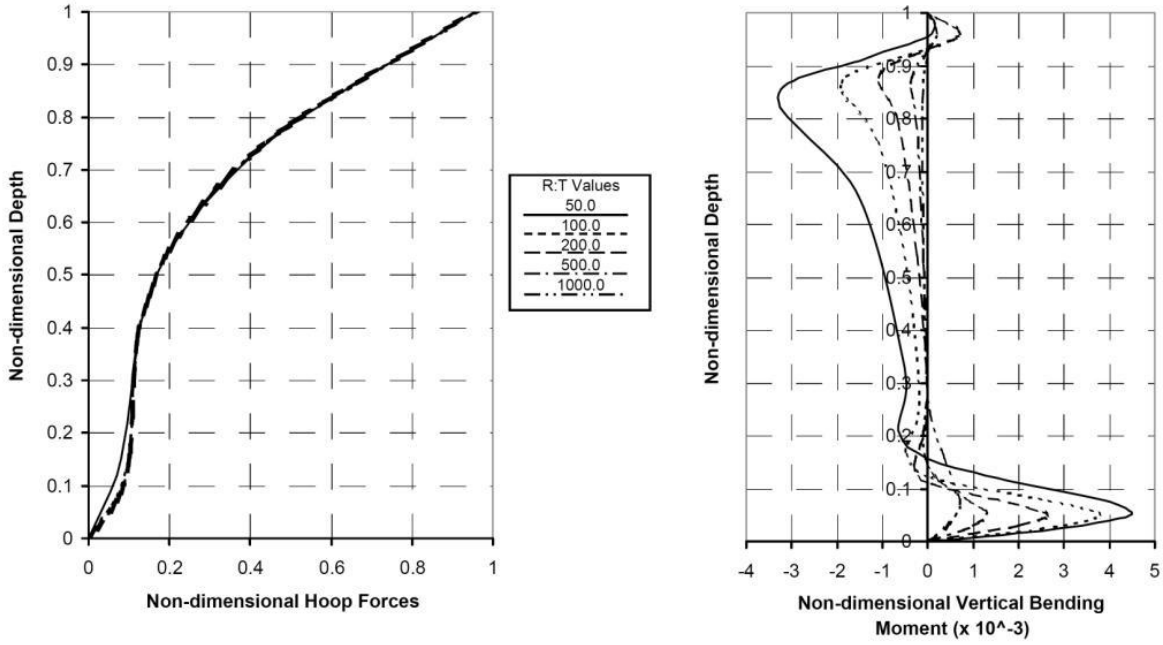
$N_{\theta n1}$ M_{zn1}

Figure A-12: Convective Pressure, H:R=0.5



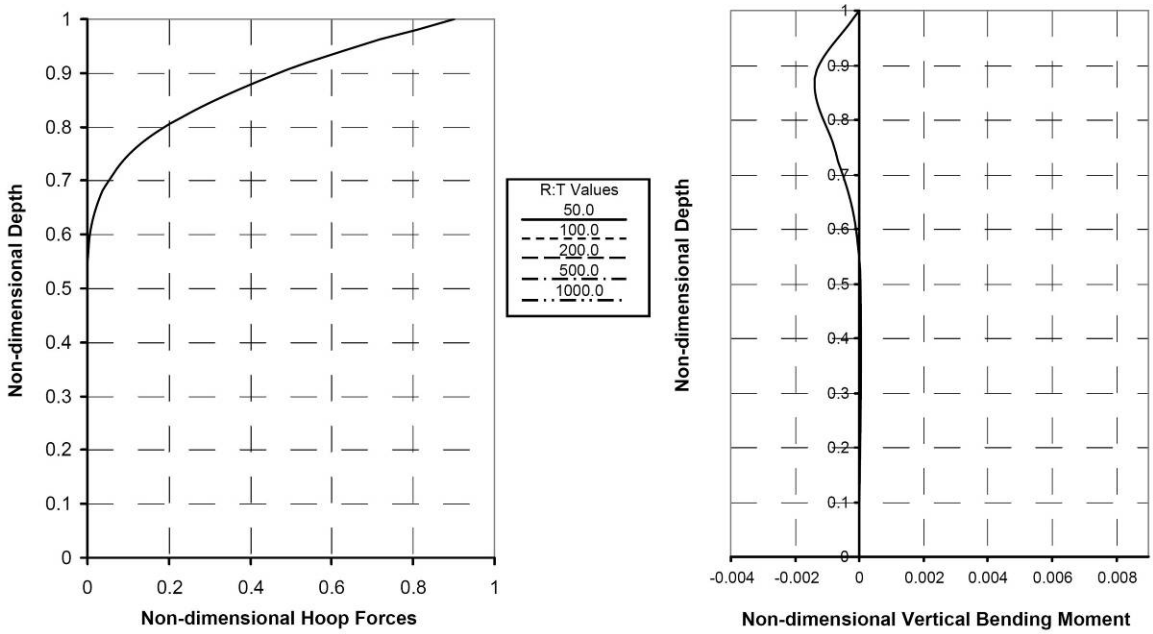
$N_{\theta n1}$ M_{zn1}

Figure A-13: Convective Pressure, H:R=1.0



$N_{\theta n1}$ M_{zn1}

Figure A-14: Convective Pressure, H:R=2.0



$N_{\theta n1}$ M_{zn1}

Figure A-15: Convective Pressure, H:R=5.0

Appendix B:
Design Examples

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DESIGN EXAMPLE 1: LARGE STEEL OIL TANK

A large steel tank is to be constructed in Wellington on a deep alluvium site. The foundation soil is a sandy gravel of medium density with an estimated Shear modulus of 55MPa. The tank is 3km from the Wellington fault. The tank is to contain aviation gasoline.

Tank Dimensions

Radius	R = 15	m
Height (required fill height)	H = 15	m
Wall thickness	t_w = 25	mm
Base thickness	t_b = 20	mm
Roof thickness	t_r = 6	mm
Roof slope	θ_r = 5	degrees
Radius of foundation	R_b = 15.5	m

Material Properties

Steel yield strength	f_y = 250	MPa
Steel tensile strength	f_u = 400	MPa
Steel Young's modulus	E = 200	GPa
Poisson's Ratio	ν = 0.25	

For this example the capacity reduction factor, ϕ, and the welded joint efficiency factor are taken as 1

Foundation Details

Shear modulus	G_s = 55	MPa
Assume Poisson's ratio	ν_s = 0.33	
Mass density of soil	ρ_s = 1800	kg/m^3
Elastic (Young's) modulus	$E_s = 2G_s(1+\nu_s)$ = **145**	**MPa**
Shear Wave Velocity	$v_s = \sqrt{\frac{G_s}{\rho_s}}$ = **174**	**m/s**

Seismic masses

Liquid Mass

Product: Av gas — Packaging Group UNPG I

Specific gravity		SG	= 0.8	
Unit weight of product		γ_l	= 7.848	kN/m^3
Unit mass of product			= 800	kg/m^3
Total product mass		m_l	= **8.48E+06**	**kg**
H/R Ratio		H/R	= 1	
From Fig C3.17	$\frac{m_o}{m_l} = 0.55 \;\&\; m_f = 0$	m_f	= **0**	**kg**
		m_o	= **4.67E+06**	**kg**
From Section 3.3.2.1	$m_r = m_o - m_f$	m_r	= **4.67E+06**	**kg**

Thus the tank can be considered rigid.

From Fig C3.8	$\frac{m_1}{m_l} = 0.45$	m_1	= **3.82E+06**	**kg**

Wall Mass

$$m_w = 2\pi RtH \,\gamma_{steel}/g$$ — m_w = **2.77E+05** **kg**

Roof Mass

$$m_t = \frac{\pi R^2 t_r}{Cos\,\theta_r}\frac{\gamma_{steel}}{g}$$ — m_t = **3.34E+04** **kg**

Base Mass

$$m_b = \pi R^2 t_b \,\gamma_{steel}/g$$ — m_b = **1.11E+05** **kg**

Total Impulsive Mass *including liquid contents, walls, and roof*

$$m_r = m_r + m_w + m_t$$ = **4.98E+0.6** **kg**

For a rigid tank the wall and roof masses may be added to the implusive rigid mass in calculating the tank period from Equation C3.30. Typically wall and roof masses are small and can be neglected in calculations of the tank periods.

Heights of Masses

From Fig C3.18	$\frac{h_o}{H} = 0.405$	h_o	= **6.1**	**m**
From Fig C3.9	$\frac{h_1}{H} = 0.62$	h_1	= **9.3**	**m**

Periods

Impulsive modes

As an effectively rigid tank the implusive period including correction for soil-structure interaction is found by Equation C3.30.

$$\breve{T}_0 = 2\pi\sqrt{\frac{m_r + m_b}{K_x} + \frac{m_r h_r^{\,2}}{K_\theta}}$$

Where

$$K_x = \frac{8}{2-\nu_s} G_s R_b \alpha_x$$

$$K_\theta = \frac{8}{3(1-\nu_s)} G_s R_b^3 \alpha_\theta$$

Equations for K_x and K_θ have a circular reference so provide initial guess for α_X and α_θ

Initially guess from Fig C3.24	α_X = 1.0		
	α_θ = 0.7		
Thus	K_x = 1.08E+10	N/m	
	K_θ = 1.50E+12	Nm	
And initial period	$\breve{T}_0$ = 0.25	seconds	

Check α_X and α_θ

Using Eqn C3.31 $a = \dfrac{2\pi R_b}{\breve{T}_i \mathrm{v}_s}$ a = 2.24

From Fig C3.24	α_X = 0.97		
	α_θ = 0.6		
For new values of α_X and α_θ	K_x = 1.04E+10	N/m	
	K_θ = 1.29E+12	Nm	
Using the new values the implusive period converges at	$\breve{T}_0$ = **0.26**	**seconds**	

Convective Mode

From Fig C3.20	$T_1\sqrt{g/R}$ = 4.75		
	$T_2\sqrt{g/R}$ = 2.70		
Thus	**T_{c1} = 5.9**	**seconds**	
	T_{c2} = 3.3	**seconds**	

Vertical Mode

Using Eqns C3.27 $$T_b = \frac{5.61\pi H}{k_v}\sqrt{\frac{\gamma_l}{Eg}}$$

H/R Ratio	H/R = 1.0		
t_w/R Ratio	t_w/R = 0.0017		
From Fig C3.23	k_v = 0.10		
Thus for Eqn C3.27	**T_b = 0.17**	**seconds**	

Correcting for soil-structure interaction using Eqn C3.32

$$\breve{T}_b = T_b\sqrt{1+\frac{K_b}{K_v}}$$

Where $$K_b = \frac{4\pi^2 m_l}{T_b^2}$$ K_b = 1.20E+10 N/m

$$K_v = \frac{4}{1-\nu_s}G_s R_b \alpha_v$$

Equation for K_v has a circular reference so provide initial guess for α_v

Initially guess from Fig C3.24	α_v = 0.8		
Thus	K_v = 1.07E+10	N/m	
And initial period	$\breve{T}_b$ = 0.24	seconds	

Check α_v

Using Eqn C3.31 $$a = \frac{2\pi R_b}{\breve{T}_i \mathrm{v}_s}$$ a = 2.30

From Fig C3.24	α_v = 0.75		
For new value of α_v	K_v = 1.01E+10	N/m	
	$\breve{T}_b$ = 0.25	seconds	
Iterating again	a = 2.26		
From Fig C3.24	α_v = 0.78		
For new value of α_v	K_v = 1.05E+10	N/m	
Using the new values the vertical period converges at	$\breve{T}_b$ = **0.24**	**seconds**	

Design Earthquake Seismic Coefficients

Using Eqns 3.2 and 3.3

$$C_d(T_i) = C(T_i)k_f(\mu, \xi_i)S_p$$

$$C(T_i) = C_h(T_i)ZR_uN(T_i, D)$$

Where		**S_p**	**= 1.0**
Constructed in Wellington,	from NZS 1170.5:2004	**Z**	**= 0.4**
	Site Subsoil Class		**D**
Using Tables 3.4 to 3.7		Table 3.4	Serious
		Table 3.5	Moderate
		Table 3.6	Moderate
		Table 3.7	Moderate
	Governing consequence of failure		**Serious**
Using Table 3.3	**Importance level for AS/NZS 1170**		**3**
Thus from AS/NZS 1170	probability of exceedance		1/1000
		R_u	**= 1.3**
Site hazard coefficient for T=0	$C(0) = C_h(0)ZR_uN(0, D)$	C(0)	= 0.58

Impulsive Coefficient

From NZS 1170.5:2004 for		$\breve{T}_0$	= 0.26	seconds
		$C_h(T_i)$	= 3	
		$N(T_i,D)$	= 1	
Thus		$C(T_i)$	= 1.56	
Using Fig 3.1(a)		ξ_i	= 23	%
And from Table 3.1	Unanchored tank design for uplift	μ	= 2	
	For capacity design against diamond buckling	μ	= 1.25	
Thus from Table 3.2		$k_f(2,23)$	= 0.48	
	For capacity design	$k_f(1.25,23)$	= 0.50	
And		**$C_d(T_i)$**	**= 0.75**	
	For capacity design	$C_d(T_i)$	= 0.78	

Convective Coefficient:

From NZS 1170.5:2004 for		T_{c1} = 5.9	seconds
		T_{c2} = 3.3	seconds
		$C_h(T_{c1})$ = 0.18	
		$C_h(T_{c2})$ = 0.59	
		$N(T_1,3km)$ = 1.68	
		$N(T_2,3km)$ = 1.41	
		$C(T_{c1})$ = 0.16	
		$C(T_{c2})$ = 0.43	
From Table 3.2		$k_f(1,0.5)$ = 1.67	
Thus		**$C_d(T_{c1})$ = 0.26**	
		$C_d(T_{c2})$ = 0.72	

Vertical Coefficient

From NZS 1170.5:2004 for		$\breve{T}_b$ = 0.24	seconds
	$0.7C_h(T)$	$C_h(T_b)$ = 2.1	
		$N(T_b,D)$ = 1	
Thus		$C(T_b)$ = 1.09	
Using Fig 3.2(a)		ξ_v = 25	%
Thus from Table 3.2		$k_f(1,25)$ = 0.51	
And		**$C_d(T_b)$ = 0.56**	

Base Shear

Impulsive

Using Eqns 3.10 and 3.11 $V_r = C_d\left(\breve{T}_o\right)\left(m_o - m_f\right)g$

$V_f = C_d\left(\breve{T}_f\right)\left[m_f + m_w + m_t\right]g$

with $\breve{T}_f = \breve{T}_0$

$m_f = 0$

V_r = 34325 kN

V_f = 2287 kN

$V_i = V_r + V_f$ **V_i = 36612 kN**

Convective

Using Eqn 3.9 $V_1 = C_d(T_1)m_1 g$ **V_1 = 9615 kN**

Total Shear

$V = \sqrt{V_i^2 + V_1^2}$ **V = 37853 kN**

Base Moment in Wall

Impulsive

Using Eqns 3.13 and 3.14

$$M_r = C_d(\breve{T}_o)[m_o - m_f]h_o g$$

$$M_f = C_d(\breve{T}_f)[m_f h_f + m_w h_w + m_t h_t]g$$

with

$$\breve{T}_f = \breve{T}_0$$

$$m_f = 0$$

M_r = 208524 kNm

M_f = 19294 kNm

$M_i = M_r + M_f$ **M_i = 227818 kNm**

Convective

Using Eqn 3.12 $M_1 = C_d(T_1)m_1 g h_1$ **M_1 = 89417 kNm**

Total Base Moment

$M_{OT} = \sqrt{M_i^2 + M_1^2}$ **M_{OT} = 244737 kNm**

For capacity design M_{OT} = 253242 kNm

Calculated values are based on a equivalent ductility of 2 for design. Capacity design requires that elastic shell (diamond) buckling be checked at an equivalent ductility of 1.25.

Seismic Forces in Walls

For this design it is sufficient to check the hoop stress and propensity for buckling in the region of the base of the wall, since this will have the highest forces and the wall thickness is uniform. Consequently the complete distribution of hoop force and bending moment will not be found.

Hoop Force

From Appendix A Eqn A.1, hoop forces are given in the form

$$N_\theta = N_{\theta n} RP$$

Hydrostatic

		p_h = 118	kPa
From Fig A-3 at 0.1H		$N_{\theta nh}$ = 1.04	
Using Eqn A.5		$N_{\theta h}$ = 1836	kN/m
Thus hydrostatic hoop stress	$f_{hh} = N_{\theta h} / t_w$	f_{hh} = 73	MPa

Impulsive

From Fig C3.3		q_o = 0.74	
Using Eqn A.4		p_i = 65	kPa
From Fig A-8 at 0.1H		$N_{\theta ni}$ = 1.2	
Using Eqn A.8		$N_{\theta i}$ = 1176	kN/m
Thus impulsive hoop stress	$f_{hi} = N_{\theta i} / t_w$	f_{hi} = 47	MPa

Convective

Using Eqn A.3		p_1 = 26	kPa
From Fig A-13 at 0.1H		$N_{\theta n1}$ = 0.4	
Using Eqn A.6		$N_{\theta 1}$ = 155	kN/m
Thus convective hoop stress	$f_{h1} = N_{\theta 1} / t_w$	f_{h1} = 6.2	MPa

Vertical

		p_v = 66	kPa
Using Eqn A.9		$N_{\theta v}$ = 1023	kN/m
Thus vertical hoop stress	$f_{hv} = N_{\theta v} / t_w$	f_{hv} = 41	MPa

Vertical Bending Moments

From Appendix A Eqn A.2, vertical bending moments are given in the form

$$M_z = M_{zn} RP t_w$$

Hydrostatic

From Fig A-3		M_{znh} = 0.07	
Thus		M_{zh} = 3.1	kNm/m
Wall section modulus	$Z = t_w^2/6$	Z = 1.04E-04	m^3/m
Thus hydrostatic bending stress	$f_{bh} = M_{zh}/Z$	f_{bh} = 30	MPa

Impulsive

From Fig A-8		M_{znf} = 0.07	
Thus		M_{zf} = 1.7	kNm/m
Thus impulsive bending stress	$f_{bi} = M_{zf}/Z$	f_{bi} = 16	MPa

Convective

From Fig A-13		M_{zn1} = 0.022	
Thus		M_{z1} = 0.21	kNm/m
Thus convective bending stress	$f_{b1} = M_{z1}/Z$	f_{b1} = 2	MPa

Vertical

From Fig A-3		M_{zhn} = 0.07	
Thus		M_{zv} = 1.7	kNm/m
Thus vertical bending stress	$f_{bv} = M_{zv}/Z$	f_{bv} = 17	MPa

Check Stress Levels

Firstly assume tank is prevented from uplifting. Maximum tension from combined hoop stress

$$f_{\max} = f_{hh} + \sqrt{f_{hi}^2 + f_{h1}^2 + f_{hv}^2}$$ **f_{max} = 136** **MPa**

For no yielding f_{max} < 250 MPa

Thus **No yield by maximum tensile hoop stress**

Maximum combined bending stress

$$f_{b\max} = f_{bh} + \sqrt{f_{bi}^2 + f_{b1}^2 + f_{bv}^2}$$ **f_{bmax} = 53** **MPa**

For no yielding f_{bmax} < 250 MPa

Thus **No yield by maximum bending stress**

Buckling in Membrane Compression (Elastic Diamond Buckling)

Capacity design requires that elastic diamond buckling be checked at an equivalent ductility of 1.25

Axial stress due to self weight

$$\sigma_{axialsw} = \frac{(m_w + m_t)g}{2\pi R t_w}$$ $\sigma_{axialSW}$ = 1.3 MPa

Axial stress due to M_{OT}, μ = 1.25 $\sigma_{axialM_{OT}} = M_{OT}/Z$

Z = 18 m^3

Thus $\sigma_{axialM_{OT}}$ = 14 MPa

And Total Stress **f_m = 16** **MPa**

Classical buckling stress from Eqn 5.9

f_{cl} = 200 **MPa**

Assume normal construction

Thus a = 1

Using Eqn C5.7 δ/τ = 1.5

Using Eqn 5.12 $\overline{\sigma}$ = 0.18

By Eqn 5.11 λ^2 = 6.8

λ^2 > 2.00

Thus $f_o = \overline{\sigma} f_{cl}$ **f_o = 36.6** **MPa**

Correction for Internal Pressure

The limiting condition will be when the internal pressure is a minimum. This occurs when the vertical acceleration is a maximum in the upward direction, thus reducing the hydrostatic pressure. All pressures should be calculated at the base of the tank and for μ = 1.25.

		p_h = 118	kPa
Using Eqn A.4 with μ = 1.25		p_i = 68	kPa
Using Eqn C3.6		p_1 = 8	kPa
		p_v = 66	kPa
Thus	$p_{\min} \approx p_h + \sqrt{p_i^2 + p_1^2} - p_v$	p_{min} = 121	kPa

Negative pressures develop over part of the tank and thus conventional hydrodynamic theory is only an approximation to behaviour.

Using Eqn 5.10		$\overline{p}$ = 0.36	
		< 5	Ok
Thus using Eqn 5.8		f_p = 82	MPa

Correction for Bending Compression

Using Eqn 5.7		**$f_m \leq$ 104**	**MPa**
If tank is prevented from uplifting then compressive stress			
		f_m = 16	**MPa**
		Therefore no elastic diamond buckling	

Elastic-Plastic Collapse (Elephants Foot Buckling)

The limiting condition will be when the internal pressure is a maximum. All pressures should be calculated at the base of the tank and for μ = 2.

		p_h = 118	kPa
Using Eqn A.4 with μ = 2		p_i = 65	kPa
Using Eqn 3.6		p_1 = 8	kPa
		p_v = 66	kPa
Thus	$p_{\max} = p_h + \sqrt{p_i^2 + p_1^2 + p_v^2}$	p_{max} = 211	kPa
Using Eqn 5.14		s = 1.50	
Thus using Eqn 5.13		**$f_m \leq$ 99**	**MPa**

For an equivalent ductility of 2 and no uplift

Axial stress due to self weight

$$\sigma_{axialsw}=\frac{(m_w+m_t)g}{2\pi Rt_w}$$ $\sigma_{axialSW}$ = 1.3 MPa

Axial stress due to M_{OT} $\sigma_{axialM_{OT}} = M_{OT}/Z$

Z = 18 m^3

Thus $\sigma_{axialM_{OT}}$ = 14 MPa

And Total Stress **f_m = 15 MPa**

$f_m \leq$ 99 MPa

Therefore no elephant's foot buckling (with no uplift)

Uplift response

Check for uplift using Eqn C4.13 **M_R = 45743 kNm**

$M_R < M_{OT}$

Uplift Occurs

Vertical axis cyclindrical tanks shall be anchored when either Eqn 5.2 or Eqn 5.3 are satisfied at nominally elastic load levels, i.e $C_d(\tilde{T}_f)$ calculated at μ = 1.25

Using Eqn 5.2 $\frac{H}{R} > \frac{2}{C_d(\tilde{T}_f)}$

1 > 2.6

Eqn 5.2 not satisfied

Using Eqn 5.3 $R < [C(0)]^2 \frac{H}{R}$

15 < 0.34

Eqn 5.3 not satisfied

Therefore mechancial anchorage not required by limits

Axial compressive stresses will be increased by uplift. Check uplifting in accordance with Section C4.4.2

Using Eqn C4.21 for M_R an iterative approach is required

Guess r = 12 m

Thus $\tau = r/R$ τ = 0.80

Using Eqn C4.19 θ^* = 1.33

Using Eqn C4.20 k = 0.86

Thus using C4.21 M_R = 586448 kNm

> M_{OT}

Increase τ as M_R is greater than M_{OT}

Guess $\tau = 0.9$

Using Eqn C4.19 $\theta^* = 1.46$

Using Eqn C4.20 $k = 0.83$

Thus using C4.21 $M_R = 337175$ kNm

$> M_{OT}$

Increase τ as M_R is greater than M_{OT}

Guess $\tau = 0.933$

Using Eqn C4.19 $\theta^* = 1.50$

Using Eqn C4.20 $k = 0.83$

Thus using C4.21 $M_R = 244147$ kNm

$\approx M_{OT}$

Thus tank uplifts but does not require mechanical anchorage

Axial Membrane Stress in Shell

Using Eqn C4.23 $W_s = 13826$ kN

Using Eqn C4.22

Where Flexible foundation $C = 0.5$

Calibration factor $CF = 2.5$

Thus from Eqn C4.22 $\mathbf{f_{max} = 31}$ **MPa**

$\leq$ **99** **MPa**

Therefore no elephant's foot buckling

Radial Membrane Stress in Base Plate

Capacity design requires that base plate be checked at an equivalent ductility of 1.25

At nominally elastic load levels $\tau = 0.93$

$\theta^* = 1.50$

$k = 0.83$

$M_R = 252840$ kNm

Assume thickness of baseplate in uplift region is t_b = 20mm.

Using Eqn C4.24

Where $\overline{E} = \dfrac{E}{1-\nu^2}$ $\overline{E} = 213$ GPa

Thus from Eqn C4.24 **f_{rb} = 176 MPa**

< f_y = 250 MPa

Yielding of base plate does not occur during uplift

$t_b < t_w$

Yielding will not occur in base of tank wall due to uplift

Uplift Displacement

Following from capacity design of the base plate, uplift is at nominally elastic load levels

Using Eqn C4.25

Where $N_x = f_{rb} t_b$ N_x = 3516 kN/m

$L_b = 2R(1-\tau)$ L_b = 2.1 m

Thus from Eqn C4.25 **v = 0.129 m**

Check maximum allowable uplift using Eqn 5.6

0.98 ≤ 113

Uplift limit satisfied

Plastic Rotation of Base Plate due to Uplift

Following from capacity design of the base plate, plastic rotation due to uplift is at nominally elastic load levels

Using Eqn C5.11 **θ_p = 0.12 radians**

≤ 0.2 radians

Plastic rotation limit is satisfied

Thus, tank with wall of 25 mm and base ring of 20 mm thickness is satisfactory for an unanchored tank under the design earthquake. Reduction in the tank wall thickness may be achieved by anchoring the tank but economic and construction considerations may govern.

Convective Wave Height

Using Eqn C3.36 **d_{max} = 3.4 m**

This wave height value shall be used in determining the provided freeboard and associated pressures of the roof of the tank.

Roof Pressure

Assume only 2.0m of freeboard available and check pressure on roof.
From graphical construction in Fig. C3.26 maximum bouyancy pressure on roof at r = R is

Using Fig C3.26	h_r = 1.4	m
	c_s = 5.0	
Thus using Eqn C3.40	$\overline{u}$ = 3.6	m/s
Using Eqn C3.39	p_b = 10.9	kPa
Thus using Eqn C3.38	**p_r = 37.3**	**kPa**

The pressure distribution will be approximately triangular with the maximum value at the wall and willl extend inwards about 9.85 m. Hence the maximum shear force on roof to wall connection will be:

$$V_{rw} = 9.85 p_r / 2$$ **V_{rw} = 184** **kN/m**

The roof to wall connection needs to be designed for this force and the roof structure designed for the pressure on the roof, p_r, over the area loaded by the convective wave impacting on the roof.

Foundation Design

The foundation should be designed considering the calculated maximum shear at the base of the tank and the overturning moments at the base including the base pressure component as determined by Eqns 3.15, 3.16, and 3.17 combined in accordance with Section 4.2.

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DESIGN EXAMPLE 2: STAINLESS STEEL CHEMICAL TANK

A stainless steel (304) chemical tank is to be constructed in Nelson on a shallow alluvium site. The foundation soil is a sandy gravel of medium density with an estimated shear modulus of 66MPa. The tank is anchored to a concrete slab. There are no near-field faults.

Tank Dimensions

Radius	$R = 1.5$	m
Height (required fill height)	$H = 6$	m
Wall thickness	$t_w = 1.5$	mm
Base thickness	$t_b = 1.5$	mm
Roof thickness	$t_r = 1.5$	mm
Roof slope	$\theta_r = 10$	degrees
Radius of foundation	$R_b = 1.5$	m
(anchored to concrete slab set R_b=R)		
Roof pipe fixtures	$m_{fix} = 80.0$	kg

Material Properties

Steel yield strength	$f_y = 254$	MPa
Steel tensile strength	$f_u = 578$	MPa
Steel Young's modulus	$E = 190$	GPa
Poisson's Ratio	$\nu = 0.25$	

For this example the capacity reduction factor, ϕ, and the welded joint efficiency factor are taken as 1

Foundation Details

Shear modulus	$G_s = 66$	MPa
Assume Poisson's ratio	$\nu_s = 0.33$	
Mass density of soil	$\rho_s = 1800$	kg/m^3
Elastic (Young's) modulus	$E_s = 2G_s(1+\nu_s) = \mathbf{175}$	**MPa**
Shear Wave Velocity	$v_s = \sqrt{\frac{G_s}{\rho_s}} = \mathbf{191}$	**m/s**

Seismic masses

Liquid Mass

Product: Industrial Chemical — Packaging Group Schedule 2

Specific gravity		SG	= 1	
Unit weight of product		γ_l	= 9.81	kN/m^3
Unit mass of product			= 1000	kg/m^3
Total product mass		**m_l**	**= 4.24E+04**	**kg**
H/R Ratio		H/R	= 4	
From Fig C3.17	$\frac{m_f}{m_l} = 0.69$	**m_f**	**= 2.93E+04**	**kg**
And	$\frac{m_o}{m_l} = 0.88$	**m_o**	**= 3.73E+04**	**kg**
From Section 3.3.2.1	$m_r = m_o - m_f$	**m_r**	**= 8.06E+03**	**kg**
From Fig C3.8	$\frac{m_1}{m_l} = 0.12$	**m_1**	**= 5.09E+03**	**kg**

Wall Mass

$$m_w = 2\pi RtH \gamma_{steel} / g$$ — **m_w = 672** **kg**

Roof Mass

$$m_t = \frac{\pi R^2 t_r}{Cos\theta_r} \frac{\gamma_{steel}}{g}$$ — **m_t+m_{fix} = 165** **kg**

Base Mass

The base mass m_b should include both the tank base and concrete foundation slab.

m_b = 3477 **kg**

Heights of Masses

From Fig C3.18	$\frac{h_f}{H} = 0.58$	**h_f**	**= 3.5**	**m**
	$\frac{h_o}{H} = 0.455$	**h_o**	**= 2.7**	**m**
From Fig C3.9	$\frac{h_1}{H} = 0.88$	**h_1**	**= 5.3**	**m**

Periods

Impulsive modes

The tank is flexible and both the rigid and flexible impulsive modes require consideration. First calculate the period for a flexible tank on rigid foundation and then correct for foundation flexibility

Using Eqn C3.26 $$T_f = \frac{5.61\pi H}{k_h}\sqrt{\frac{\gamma_l}{Eg}}$$

H/R Ratio H/R = 4.0

t_w/R Ratio t_w/R = 0.001

From Fig C3.22 k_h = 0.068

Thus for Eqn C3.26 **T_f = 0.11** seconds

Correcting for soil-structure interaction using Eqn C3.29

$$\breve{T}_f = T_f\sqrt{1+\frac{K_f}{K_x}\left(1+\frac{K_x h_f^2}{K_\theta}\right)}$$

Where $$K_f = \frac{4\pi^2\left(m_f + m_w + m_t\right)}{T_f^2}$$ K_f = 9.34E+07 N/m

$$K_x = \frac{8}{2-\nu_s}G_s R_b \alpha_x$$

$$K_\theta = \frac{8}{3\left(1-\nu_s\right)}G_s R_b^3 \alpha_\theta$$

Typically wall and roof masses are small and can be neglected in calculations of the tank periods but have been included here for completeness.

Equations for K_x and K_θ have a circular reference so provide initial guess for α_x and α_θ

Initially guess from Fig C3.24 α_x = 1.0

α_θ = 1.0

Thus K_x = 1.26E+09 N/m

K_θ = 8.84E+08 Nm

And initial period $\breve{T}_f$ = 0.17 seconds

Check α_x and α_θ

Using Eqn C3.31 $$a = \frac{2\pi R_b}{\breve{T}_i \mathrm{v}_s}$$ a = 0.29

From Fig C3.24 α_x = 0.99

α_θ = 0.96

For new values of α_x and α_θ K_x = 1.24E+09 N/m

K_θ = 2.26E+09 Nm

Using the new values the flexible implusive mode period converges at $\breve{T}_f$ = **0.14** **seconds**

The concrete slab and its interaction with the underlying basecourse is considered to be flexible in terms of the soil-structure interaction and thus the second implusive mode period should be found.

Using Eqn C3.30

$$\breve{T}_o = 2\pi\sqrt{\frac{m_r + m_b}{K_x} + \frac{m_r h_r^2}{K_\theta}}$$

Equations for K_x and K_θ have a circular reference so provide initial guess for α_x and α_θ

Initially guess from Fig C3.24 α_x = 1.0

α_θ = 1.0

Thus K_x = 1.26E+09 N/m

K_θ = 2.30E+09 Nm

And initial period $\breve{T}_o$ = 0.04 seconds

Check α_x and α_θ

Using Eqn C3.31 a = 0.81

From Fig C3.24 α_x = 0.98

α_θ = 0.86

For new values of α_x and α_θ K_x = 1.23E+09 N/m

K_θ = 2.02E+09 Nm

Using the new values the implusive period converges at $\breve{T}_o$ = 0.06 **seconds**

The rigid mass component can be considered for T=0 seconds

Convective Mode

From Fig C3.20 $T_1\sqrt{g/R}$ = 4.60

$T_2\sqrt{g/R}$ = 2.65

Thus $\mathbf{T_{c1}}$ = **1.8** **seconds**

$\mathbf{T_{c2}}$ = **1.0** **seconds**

Vertical Mode

Using Eqns C3.27 $$T_b = \frac{5.61\pi H}{k_v}\sqrt{\frac{\gamma_l}{Eg}}$$

H/R Ratio		H/R = 4.0	
t_w/R Ratio		t_w/R = 0.001	
From Fig C3.23		k_v = 0.10	
Thus for Eqn C3.27		**T_b = 0.08**	**seconds**

Correcting for soil-structure interaction using Eqn C3.32

$$\breve{T}_b = T_b\sqrt{1+\frac{K_b}{K_v}}$$

Where $$K_b = \frac{4\pi^2 m_l}{T_b^2}$$ K_b = 2.84E+08 N/m

$$K_v = \frac{4}{1-\nu_s}G_s R_b \alpha_v$$

Equation for K_v has a circular reference so provide initial guess for α_v

Initially guess from Fig C3.24		α_v = 1	
Thus		K_v = 1.57E+09	N/m
And initial period		$\breve{T}_b$ = 0.08	seconds

Check α_v

Using Eqn C3.31 $$a = \frac{2\pi R_b}{\breve{T}_i v_s}$$ a = 0.59

From Fig C3.24		α_v = 0.94	
For new value of α_v		K_v = 1.47E+09	N/m
Using the new values the vertical period converges at		$\breve{T}_b$ = **0.08**	**seconds**

Design Earthquake Seismic Coefficients

Using Eqns 3.2 and 3.3 $C_d(T_i) = C(T_i)k_f(\mu, \xi_i)S_p$

$C(T_i) = C_h(T_i)ZR_uN(T_i, D)$

Where		**S_p**	**= 1.0**
Constructed in Nelson,	from NZS 1170.5:2004	**Z**	**= 0.27**
	Site Subsoil Class		**C**
Using Tables 3.4 to 3.7		Table 3.4	Slight
		Table 3.5	Slight
		Table 3.6	Negligible
		Table 3.7	Negligible
	Governing consequence of failure		**Slight**
Using Table 3.3	**Importance level for AS/NZS 1170**		**1**
Thus from AS/NZS 1170	probability of exceedance		1/100
		R_u	**= 0.5**
Site hazard coefficient for T=0	$C(0) = C_h(0)ZR_uN(0, D)$	C(0)	= 0.18

Impulsive Coefficient

From NZS 1170.5:2004 for		$\breve{T}_f$	= 0.14	seconds
		$C_h(T_i)$	= 2.36	
		$N(T_i,D)$	= 1	
Thus		$C(T_i)$	= 0.32	
Using Fig 3.1(b)		ξ_i	= 4	%
And from Table 3.1	Anchored with non-ductile holding down bolts	μ	= 1.25	
Thus from Table 3.2		$k_f(1.25,4)$	= 0.87	
And		**$C_d(T_f)$**	**= 0.28**	
		$C_d(T_0)$	**= 0.28**	

Convective Coefficient:

From NZS 1170.5:2004 for		T_{c1} = 1.8	seconds
		T_{c2} = 1.0	seconds
		$C_h(T_{c1})$ = 0.73	
		$C_h(T_{c2})$ = 1.19	
		$N(T_1,D)$ = 1.0	
		$N(T_2,D)$ = 1.0	
		$C(T_{c1})$ = 0.10	
		$C(T_{c2})$ = 0.16	
From Table 3.2		$k_f(1,0.5)$ = 1.67	
Thus		**$C_d(T_{c1})$ = 0.16**	
		$C_d(T_{c2})$ = 0.27	

Vertical Coefficient

From NZS 1170.5:2004 for		$\breve{T}_b$ = 0.08	seconds
	$0.7C_h(T)$	$C_h(T_b)$ = 1.65	
		$N(T_b,D)$ = 1	
Thus		$C(T_b)$ = 0.22	
Using Fig 3.2 (a)		ξ_v = 8	%
Thus from Table 3.2		$k_f(1,8)$ = 0.86	
And		**$C_d(T_b)$ = 0.19**	

Base Shear

Impulsive

Using Eqns 3.10 and 3.11	$V_r = C_d(\breve{T}_o)(m_o - m_f)g$	**V_r = 22**	**kN**
	$V_f = C_d(\breve{T}_f)[m_f + m_w + m_t]g$	**V_f = 82**	**kN**

Convective

Using Eqn 3.9	$V_1 = C_d(T_1)m_1 g$	**V_1 = 8**	**kN**

Total Shear

	$V = \sqrt{(V_f + V_r)^2 + V_1^2}$	**V = 104**	**kN**

Base Moment in Wall

Impulsive

Using Eqns 3.13 and 3.14 $M_r = C_d(\breve{T}_o)[m_o - m_f]h_o g$ **M_r = 60** **kNm**

Ignore M_r as negative

$$M_f = C_d(\breve{T}_f)[m_f h_f + m_w h_w + m_t h_t]g$$

M_f = 285 **kNm**

Convective

Using Eqn 3.12 $M_1 = C_d(T_1)m_1 g h_1$ **M_1 = 43** **kNm**

Total Base Moment

$$M_{OT} = \sqrt{(M_f + M_r)^2 + M_1^{\,2}}$$ **M_{OT} = 348** **kNm**

Calculated values are based on a equivalent ductility of 1.25 for design.

Seismic Forces in Walls

For this design it is sufficient to check the hoop stress and propensity for buckling in the region of the base of the wall, since this will have the highest forces and the wall thickness is uniform. Consequently the complete distribution of hoop force and bending moment will not be found.

Hoop Force

From Appendix A Eqn A.1, hoop forces are given in the form

$$N_\theta = N_{\theta h} RP$$

Hydrostatic

p_h = 59 kPa

From Fig A-5 at 0.1H $N_{\theta nh}$ = 1.06

Using Eqn A.5 $N_{\theta h}$ = 94 kN/m

Thus hydrostatic hoop stress $f_{hh} = N_{\theta h} / t_w$ f_{hh} = 62 MPa

Impulsive

From Fig C3.3		q_o = 0.98	
Using Eqn A.4		p_i = 4	kPa
From Fig A-10 at 0.1H		$N_{\theta ni}$ = 1.22	
Using Eqn A.8 as	$C_d(T_f) = C_d(T_0)$	$N_{\theta i}$ = 7	kN/m
Thus impulsive hoop stress	$f_{hi} = N_{\theta i} / t_w$	f_{hi} = 5	MPa

Convective

Using Eqn A.3		p_1 = 2	kPa
From Fig A-15 at 0.1H		$N_{\theta n1}$ = 0.0	
Using Eqn A.6		$N_{\theta 1}$ = 0.00	kN/m
Thus convective hoop stress	$f_{h1} = N_{\theta 1} / t_w$	f_{h1} = 0.00	MPa

Vertical

		p_v = 11	kPa
Using Eqn A.9		N_{qv} = 18	kN/m
Thus vertical hoop stress	$f_{hv} = N_{\theta v} / t_w$	f_{hv} = 12	MPa

Vertical Bending Moments

From Appendix A Eqn A.2, vertical bending moments are given in the form

$$M_z = M_{zn} R P t_w$$

Hydrostatic

From Fig A-5		M_{znh} = 0.0032	
Thus		M_{zh} = 0.0004	kNm/m
Wall section modulus	$Z = t_w^2 / 6$	Z = 3.75E-07	m^3/m
Thus hydrostatic bending stress			
	$f_{bh} = M_{zh} / Z$	f_{bh} = 1.13	MPa

Impulsive

From Fig A-10 M_{znf} = 0.0044

Thus M_{zf} = 0.00004 kNm/m

Thus impulsive bending stress

$$f_{bi} = M_{zf}/Z$$ f_{bi} = 0.11 MPa

Convective

From Fig A-15 M_{zn1} = 0.0

Thus M_{z1} = 0.0 kNm/m

Thus convective bending stress

$$f_{b1} = M_{z1}/Z$$ f_{b1} = 0.0 MPa

Vertical

From Fig A-5 M_{znh} = 0.0006

Thus M_{zv} = 0.00002 kNm/m

Thus vertical bending stress

$$f_{bv} = M_{zv}/Z$$ f_{bv} = 0.04 MPa

Vertical bending moments are small and could be neglected

Check Stress Levels

Tank is prevented from uplifting. Maximum tension from combined hoop stress

$$f_{\max} = f_{hh} + \sqrt{f_{hi}^2 + f_{h1}^2 + f_{hv}^2}$$ **f_{max} = 72 MPa**

For no yielding f_{max} < 254 MPa

Thus **No yield by maximum tensile hoop stress**

Maximum combined bending stress

$$f_{b\max} = f_{bh} + \sqrt{f_{bi}^2 + f_{b1}^2 + f_{bv}^2}$$ **f_{bmax} = 1.2 MPa**

For no yielding f_{bmax} < 254 MPa

Thus **No yield by maximum bending stress**

Buckling in Membrane Compression (Elastic Diamond Buckling)

Axial stress due to self weight

$$\sigma_{axialsw}=\frac{(m_w+m_t)g}{2\pi Rt_w}$$ $\sigma_{axialSW}$ = 0.6 MPa

Axial stress due to M_{OT} $\sigma_{axialM_{OT}} = M_{OT}/Z$

Z = 0.01 m^3

Thus $\sigma_{axialM_{OT}}$ = 33 MPa

And Total Stress **f_m = 33 MPa**

Classical buckling stress from Eqn 5.9

f_{cl} = 114 MPa

Assume normal construction

Thus a = 1

Using Eqn C5.7 δ/τ = 1.9

Using Eqn 5.12 $\overline{\sigma}$ = 0.15

By Eqn 5.11 λ^2 = 14.6

λ^2 > 2.00

Thus $f_o = \overline{\sigma} f_{cl}$ **f_o = 17.4 MPa**

Correction for Internal Pressure

The limiting condition will be when the internal pressure is a minimum. This occurs when the vertical acceleration is a maximum in the upward direction, thus reducing the hydrostatic pressure. All pressures should be calculated at the base of the tank and for μ = 1.25.

p_h = 59 kPa

Using Eqn A.4 with μ = 1.25 p_i = 4 kPa

Using Eqn C3.6 p_1 = 0 kPa

p_v = 11 kPa

Thus $p_{\min} \approx p_h + \sqrt{p_i^2 + p_1^2} - p_v$ p_{min} = 52 kPa

Negative pressures develop over part of the tank and thus conventional hydrodynamic theory is only an approximation to behaviour.

Using Eqn 5.10 $\overline{p}$ = 0.45

< 5 Ok

Thus using Eqn 5.8 f_p = 50 MPa

Correction for Bending Compression

Using Eqn 5.7 $f_m \leq$ **62** **MPa**

If tank is prevented from uplifting then compressive stress

f_m **= 33** **MPa**

Therefore no elastic diamond buckling

Elastic-Plastic Collapse (Elephants Foot Buckling)

The limiting condition will be when the internal pressure is a maximum. All pressures should be calculated at the base of the tank and for μ = 1.25.

p_h = 59 kPa

Using Eqn A.4 with μ = 1.25 p_i = 4 kPa

Using Eqn 3.6 p_1 = 0 kPa

p_v = 11 kPa

Thus $p_{\max} = p_h + \sqrt{p_i^2 + p_1^2 + p_v^2}$ p_{max} = 71 kPa

Using Eqn 5.14 s = 2.50

Thus using Eqn 5.13 $f_m \leq$ **85** **MPa**

For an equivalent ductility of 1.25 and no uplift

Axial stress due to self weight $\sigma_{axialsw} = \frac{(m_w + m_t)g}{2\pi R t_w}$ $\sigma_{axialSW}$ = 0.6 MPa

Axial stress due to M_{OT} $\sigma_{axialM_{OT}} = M_{OT} / Z$

Z = 0.01 m^3

Thus $\sigma_{axialM_{OT}}$ = 33 MPa

And Total Stress f_m **= 33** **MPa**

$f_m \leq$ **85** **MPa**

Therefore no elephant's foot buckling (with no uplift)

Uplift response *To check whether anchorage is required as specified.*

Check for uplift using Eqn C4.1: M_R **= 15** **kNm**

$M_R < M_{OT}$

Uplift Occurs

Vertical axis cyclindrical tanks shall be anchored when either Eqn 5.2 or Eqn 5.3 are satisfied at nominally elastic load levels

Using Eqn 5.2 $\frac{H}{R} > \frac{2}{C_d\left(\bar{T}_f\right)}$

4 > 7.1

Eqn 5.2 not satisfied

Using Eqn 5.3 $R < \left[C(0)\right]^2 \frac{H}{R}$

1.5 < 0.13

Eqn 5.3 not satisfied

Therefore mechancial anchorage not required by limits

Axial compressive stresses will be increased by uplift. Check uplifting in accordance with Section C4.4.2

Using Eqn C4.21 for M_R an iterative approach is required

Guess		r = 1.05	m
Thus	$\tau = r/R$	τ = 0.70	
Using Eqn C4.19		θ^* = 1.17	
Using Eqn C4.20		k = 0.89	
Thus using C4.21		M_R = 386	kNm
		> M_{OT}	
Increase τ as M_R is greater than M_{OT}			
Guess		τ = 0.75	
Using Eqn C4.19		θ^* = 1.25	
Using Eqn C4.20		k = 0.88	
Thus using C4.21		M_R = 338	kNm
		< M_{OT}	
Increase τ as M_R is greater than M_{OT}			
Guess		τ = 0.74	
Using Eqn C4.19		θ^* = 1.23	
Using Eqn C4.20		k = 0.88	
Thus using C4.21		M_R = 348	kNm
		= M_{OT}	

Thus tank uplifts but does not require mechanical anchorage

Axial Membrane Stress in Shell

Using Eqn C4.23		W_s	= 196	kN
Using Eqn C4.22				
Where	Rigid foundation	C	= 1	
	Calibration factor	CF	= 2.5	
Thus from Eqn C4.22		**f_{max}**	**= 177**	**MPa**
			> 62	**MPa**

Therefore elastic diamond buckling occurs if unanchored. Thus anchorage is required

Convective Wave Height

Using Eqn C3.36 **d_{max} = 0.2 m**

This wave height value shall be used in determining the provided freeboard and associated pressures of the roof of the tank.

Roof Pressure

Assume only 0.1m of freeboard available and check pressure on roof.
From graphical construction in Fig. C3.26 maximum bouyancy pressure on roof at r = R is

Using Fig C3.26	h_r	= 0.1	m
	c_s	= 5.0	
Thus using Eqn C3.40	$\overline{u}$	= 0.7	m/s
Using Eqn C3.39	p_b	= 1.0	kPa
Thus using Eqn C3.38	**p_r**	**= 2.2**	**kPa**

The pressure distribution will be approximately triangular with the maximum value at the wall and will extend inwards about 640mm. Hence the maximum shear force on roof to wall connection will be:

$$V_{rw} = 0.64 p_r / 2$$

V_{rw} = 0.70 kN/m

The roof to wall connection needs to be designed for this force and the roof structure designed for the pressure on the roof, p_r, over the area loaded by the convective wave impacting on the roof.

Foundation Design

The foundation should be designed considering the calculated maximum shear at the base of the tank and the overturning moments at the base including the base pressure component as determined by Eqns 3.15, 3.16, and 3.17 combined in accordance with Section 4.2.



DESIGN EXAMPLE 3: PRESTRESSED CONCRETE CIRCULAR RESERVOIR

A prestressed reservoir is to be constructed in Christchurch on a soft deep alluvium site. The foundation soil is a dense sand with an estimated shear modulus of 19MPa. The tank is to be functional for post-diasaster water supply. There are no near-field faults.

Tank Dimensions

Radius (inside)	R = 15	m
Height (fill height)	H = 7.5	m
Wall thickness	t_w = 175	mm
Base thickness	t_b = 200	mm
Roof thickness	t_r = 100	mm
Radius of foundation	R_b = 16.0	m

Material Properties

Concrete strength	f'_c = 40	MPa
Concrete density	ρ_c = 2400	kg/m^3
Young's modulus	E = 28	GPa

Foundation Details

Shear modulus	G_s = 19	MPa
Assume Poisson's ratio	ν_s = 0.33	
Mass density of soil	ρ_s = 2000	kg/m^3
Elastic (Young's) modulus	$E_s = 2G_s(1+\nu_s)$ = **50**	**MPa**
Shear Wave Velocity	$v_s = \sqrt{\frac{G_s}{\rho_s}}$ = **97**	**m/s**

Seismic masses

Liquid Mass

Product: Water	Packaging Group Schedule 4	
Specific gravity	SG = 1	
Unit weight of product	γ_l = 9.81	kN/m^3
Unit mass of product	= 1000	kg/m^3
Total product mass	**m_l = 5.30E+06**	**kg**

Assume tank is rigid on a flexible foundation.

H/R Ratio H/R = 0.5

From Fig C3.8 $\frac{m_o}{m_l} = 0.3$ **$m_{r(liquid)} = m_o$ = 1.59E+06 kg**

$\frac{m_1}{m_l} = 0.66$ **m_1 = 3.50E+06 kg**

Wall Mass

$m_w = 2\pi RtH\rho_c$ **m_w = 2.97E+05 kg**

Roof Mass $m_t \approx \pi R^2 t_r \rho_c$ **m_t = 1.70E+05 kg**

Base Mass $m_b = \pi R^2 t_b \rho_c$ **m_b = 3.39E+05 kg**

Heights of Masses

From Fig C3.9 $\frac{h_o}{H} = 0.4$ **h_o = 3.0 m**

$\frac{h_1}{H} = 0.55$ **h_1 = 4.1 m**

Periods

Impulsive mode

The impulsive period for a rigid tank on a flexible foundation can be calculated in accordance with Section C3.6. The mass m_r includes m_o, m_w and m_t.

$m_r = m_o + m_w + m_t$ **= 2.06E+06 kg**

Using Eqn C3.30 $\breve{T}_o = 2\pi\sqrt{\frac{m_r + m_b}{K_x} + \frac{m_r h_r^2}{K_\theta}}$

Where $K_x = \frac{8}{2-\nu_s} G_s R_b \alpha_x$

$K_\theta = \frac{8}{3(1-\nu_s)} G_s R_b^3 \alpha_\theta$

$m_r = m_o$

$h_r = h_o$

Equations for K_x and K_θ have a circular reference so provide initial guess for α_x and α_θ

Initially guess from Fig C3.24	α_x	= 1.0	
	α_θ	= 0.5	
Thus	K_x	= 3.83E+09	N/m
	K_θ	= 4.08E+11	Nm
And initial period	$\breve{T}_o$	= 0.16	seconds

Check α_x and α_θ

Using Eqn C3.31	a	= 6.37	
From Fig C3.24	α_x	= 0.91	
	α_θ	= 0.45	
For new values of α_x and α_θ	K_x	= 3.49E+09	N/m
	K_θ	= 3.67E+11	Nm
Using the new values the implusive period converges at	$\breve{T}_o$	= **0.17**	**seconds**

Convective Mode

From Fig C3.20	$T_1\sqrt{g/R}$	= 5.5	
	$T_2\sqrt{g/R}$	= 2.7	
Thus	T_{c1}	= **6.8**	**seconds**
	T_{c2}	= **3.3**	**seconds**

Vertical Mode

Using Eqn C3.33 $$\breve{T}_b = 2\pi\sqrt{\frac{m_v}{K_v}}$$

where $$m_v = m_\ell + m_s$$ m_v = 6.11E+06 kg

$$K_v = \frac{4}{1-\nu_s} G_s R_b \alpha_v$$

Equation for K_v has a circular reference so provide initial guess for α_v

Initially guess from Fig C3.24	α_v	= 0.75	
Thus	K_v	= 3.58E+09	N/m
And initial period	$\breve{T}_b$	= 0.26	seconds

Check α_v

Using Eqn C3.31 $$a = \frac{2\pi R_b}{\breve{T}_i v_s}$$ a = 3.99

From Fig C3.24 α_v = 0.58

For new value of α_v K_v = 2.77E+09 N/m

Using the new values the vertical period converges at $\breve{T}_b$ = 0.30 seconds

Check α_v again

Using Eqn C3.31 a = 3.51

From Fig C3.24 α_v = 0.61

For new value of α_v K_v = 2.91E+09 N/m

Using the new values the vertical period converges at $\breve{T}_b$ = 0.29 **seconds**

Design Earthquake Seismic Coefficients

Using Eqns 3.2 and 3.3 $$C_d(T_i) = C(T_i) k_f(\mu, \xi_i) S_p$$
$$C(T_i) = C_h(T_i) Z R_u N(T_i, D)$$

Where **S_p = 1.0**

Constructed in Christchurch, from NZS 1170.5:2004 **Z = 0.22**

Site Subsoil Class **E**

Using Tables 3.4 to 3.7 Table 3.4 Negligible

Table 3.5 Negligible

Table 3.6 Serious

Table 3.7 Slight

Governing consequence of failure **Serious**

Using Table 3.3 **Importance level for AS/NZS 1170** **3**

Thus from AS/NZS 1170 probability of exceedance 1/1000

R_u = 1.3

Impulsive Coefficient

From NZS 1170.5:2004 for		$\breve{T}_o$ = 0.17	seconds
		$C_h(T_i)$ = 3.0	
		$N(T_i,D)$ = 1	
Thus		$C(T_i)$ = 0.86	
Using Fig 3.1(d)		ξ_i = 28	%
And from Table 3.1	Concrete tank on grade	μ = 2.0	
Thus from Table 3.2		$k_f(2,28)$ = 0.44	
And		**$C_d(T_i)$ = 0.38**	

Convective Coefficient:

From NZS 1170.5:2004 for		T_{c1} = 6.8	seconds
		T_{c2} = 3.3	seconds
		$C_h(T_{c1})$ = 0.22	
		$C_h(T_{c2})$ = 0.91	
		$N(T_1,D)$ = 1.0	
		$N(T_2,D)$ = 1.0	
		$C(T_{c1})$ = 0.06	
		$C(T_{c2})$ = 0.26	
From Table 3.2		$k_f(1,0.5)$ = 1.67	
Thus		**$C_d(T_{c1})$ = 0.10**	
		$C_d(T_{c2})$ = 0.44	

Vertical Coefficient

From NZS 1170.5:2004 for		$\breve{T}_b$ = 0.29	seconds
	0.7C_h(T)	$C_h(T_b)$ = 2.10	
		$N(T_b,D)$ = 1	
Thus		$C(T_b)$ = 0.60	
Using Fig 3.2(c)		ξ_v = 20	%
Thus from Table 3.2		$k_f(1,20)$ = 0.56	
And		**$C_d(T_b)$ = 0.34**	

Calculations from this point are similar to those in the previous examples except it is likely that the prestress and mild steel reinforcement will vary over the height of the wall. Thus the entire distribution of hoop force and bending moment is required. These can be obtained using the figures of Appendix A.