NZS 3106:2009

New Zealand Standard

Design of concrete structures for the storage of liquids

Superseding NZS 3106:1986

NZS 3106:2009

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COMMITTEE REPRESENTATION

This Standard was prepared under the supervision of the P 3106 Committee the Standards Council established under the Standards Act 1988.

The committee consisted of representatives from the following organisations:

Cement and Concrete Association of New Zealand
Department of Building and Housing
GNS Science
Institute of Professional Engineers New Zealand
New Zealand Concrete Association
New Zealand Society for Earthquake Engineering

ACKNOWLEDGEMENT

Standards New Zealand gratefully acknowledges the contribution of time and expertise from all those involved in developing this Standard.

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AMENDMENTS

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REFERENCED DOCUMENTS

Reference is made in this document to the following:

NEW ZEALAND STANDARDS

NZS 1170.5:2004	Structural design actions – Earthquake actions – New Zealand
NZS 3101.1 & 2:2006	Concrete structures
NZS 3109:1997	Concrete construction
NZS 3112.2:1986	Methods of test for concrete – Tests relating to the determination of strength of concrete

JOINT AUSTRALIAN/NEW ZEALAND STANDARDS

AS/NZS 1170 :- - - -	Structural design actions
Part 0:2002	General principles
Part 1:2002	Permanent, imposed and other actions
AS/NZS 4671:2001	Steel reinforcing materials

AUSTRALIAN STANDARDS

AS 3735:2001	Concrete structures retaining liquids
AS 3735:2001 Supp 1	Concrete structures retaining liquids – Commentary

BRITISH STANDARDS

BS EN 1992-1-1:2004	UK National Annex to Eurocode 2: Design of concrete structures. General rules and rules for buildings
BS EN 1992-3:2006	UK National Annex to Eurocode 2. Design of concrete structures. Liquid retaining and containment structures.
BS 8007:1987	Code of practice for design of concrete structures for retaining aqueous liquids

OTHER PUBLICATIONS

American Concrete Institute (ACI), 344R-70, Analysis and design of reinforced concrete structures.

Comite Euro-International du Beton (CEB) and the Federation Internationale de la Precontrainte (FIP).CEB-FIP Model code, shrinkage and creep effects on prestressed concrete structures 1990.

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NEW ZEALAND LEGISLATION

Building Act 2004

New Zealand Building Code (NZBC)

Hazardous Substances (Dangerous Goods and Scheduled Toxic Substances) Transfer Notice 2004

WEBSITES

http://www.legislation.govt.nz/

http://www.dbh.govt.nz

LATEST REVISIONS

The users of this Standard should ensure that their copies of the above-mentioned New Zealand Standards are the latest revisions. Amendments to referenced New Zealand and Joint Australian/New Zealand Standards can be found on **www.standards.co.nz**.

REVIEW OF STANDARDS

Suggestions for improvement of this Standard will be welcomed. They should be sent to the Chief Executive, Standards New Zealand, Private Bag 2439, Wellington 6140.

FOREWORD

The aim of this Standard is to provide design information for users to meet the requirements of the New Zealand Building Code for concrete structures that will store liquid. NZS 3106 is intended to be used by engineers and organisations about to embark on a new storage tank project.

The committee has reviewed a number of Standards and other publications to identify 'best practice' and included provisions from these where appropriate.

This revision of NZS 3106:1986 aims to supplement NZS 3101:2006 and the AS/NZS 1170 suite of standards. Provisions that are adequately covered by the other Standards have not been included in this Standard.

Some of the the most significant changes are:

(a) In addition to the serviceability limit state loads, this Standard now requires users to also take into account the strength considerations required for ultimate limit state loads. This makes NZS 3106 consistent with the design procedures used for other structures.

(b) A new system to classify the liquid tightness of a structure has been adopted from BS EN 1992-3:2006. The designer can now decide what degree (if any) of leakage is acceptable from concrete cracks. The designer may then choose the appropriate crack control provisions to achieve liquid tightness.

(c) A new Appendix B provides procedures on how to calculate crack widths in reinforced concrete. These cracks may be caused by early-age thermal strains or from imposed loads and actions and given the critical importance of crack control in liquid retaining structures, the new procedures have been based on the worst case 'fully restrained' condition.

(d) The earthquake provisions have been reviewed in line with the New Zealand Society for Earthquake Engineering Study Group on the Seismic Design of Storage Tanks 2009 draft recommendations. Seismic force coefficients are based on NZS 1170.5, with allowance made for increased damping from soil-structure interaction which is particularly significant for liquid storage tanks.

OUTCOME STATEMENT

This Standard provides a basis for designing concrete structures for the storage of liquids so that they will require only limited periodic maintenance to remain serviceable for their design life, and will not allow an uncontrolled, rapid loss of the liquid contents in extreme events such as a major earthquake. This Standard supports public safety through designs that are safe and serviceable.

NEW ZEALAND STANDARD

DESIGN OF CONCRETE STRUCTURES FOR THE STORAGE OF LIQUIDS

1 GENERAL

1.1 SCOPE

This Standard is intended to provide a means of compliance with the requirements of the New Zealand Building Code for concrete structures that will store liquid, and would be used regularly by engineers and organisations about to embark on a new storage tank project.

The Standard covers:

(a) Design requirements and guidance;

(b) Loading requirements;

(c) Seismic considerations and allowances.

1.1.1 Inclusions

This Standard applies to containment structures for use with water of normal temperature (approximately 17 ˚C) and pH (approximately 5.5 to 8.5) commonly found in drinking water supplies (fresh water) sewage, wastewater, and sea water.

The use of this Standard in the design of the following types of containment structures requires special consideration of the effects on the structure:

(a) Highly aggressive waters (for example, corrosive);

(b) High temperature waters (> 35 ˚C);

(c) Chemicals;

(d) Oils (mineral and non-mineral) and fuels;

(e) Slurries.

Tanks used for the storage of hazardous substances are required to comply with a Standard specified in Schedule 8, Clause 8 of the Hazardous Substances (Dangerous Goods and Scheduled Toxic Substances) Transfer Notice 2004 (as amended) or a code of practice approved under this clause. As at the time of publication, this Standard was not approved under this clause.

1.1.2 Exclusions

This Standard does not apply to the design of:

(a) Dams;

(b) Hydraulic tunnels;

(c) Precast concrete pipes (pressure or non-pressure);

(d) Fibre-impregnated concrete that does not comply with the design requirements and procedures of NZS 3101;

(e) Bins or silos for storage of dry bulk materials;

(f) Pressure vessels.

1.2 NEW ZEALAND BUILDING LEGISLATION

1.2.1 Building Act and Code

Liquid retaining structures are buildings in terms of the Building Act 2004 and must therefore meet all the relevant performance requirements of the New Zealand Building Act and Code.

1.2.2 Verification method

It is intended that this Standard will be cited as part of a Verification Method to be used in conjunction with other cited Standards to achieve compliance with the Building Code.

C1.2.2 Non-specific terms

Provisions in this Standard that are in non-specific or unquantified terms will not form part of the Verification Method. Non-specific terms include, but are not limited to, special studies, manufacturer's advice and references to methods that are appropriate, adequate, suitable, relevant, satisfactory, acceptable, applicable, or the like.

1.3 INTERPRETATION

1.3.1 'Shall' and 'should'

In this Standard the word 'shall' indicates a requirement that is to be adopted in order to comply with the Standard. The word 'should' indicates practices which are advised or recommended.

1.3.2 Clause cross-references

Cross-references to other clauses or clause subdivisions within this Standard quote the number only, for example: '… is given by 6.2 (a)'.

1.3.3 Commentary

Clauses prefixed by *C*, printed in italic type and shaded are intended as comments on the corresponding mandatory clauses. The commentary to this Standard, NZS 3106, does not contain requirements essential for compliance with this Standard but summarises technical background, offers explanations, and suggests approaches which satisfy the intent of the Standard.

1.4 APPENDICES

The terms 'Normative' and 'Informative' have been used in this Standard to define the application of the Appendix to which they apply. A 'Normative' Appendix is an integral part of a Standard while an 'Informative' Appendix is only for information and guidance.

1.5 DESIGN AND CONSTRUCTION

Unless otherwise specified in this Standard, all concrete design and construction shall be in accordance with NZS 3101 and NZS 3109.

C1.5 Design data

It is recommended that the following design data should be shown in the drawings:

(a) Reference number and date of issue of applicable design Standards;

(b) Tightness class;

(c) Live loads used in design;

(d) Exposure classification for durability;

(e) Class and, where appropriate, grade designation of concrete;

(f) Grade and type of reinforcement and tendons.

2 DEFINITIONS AND ABBREVIATIONS

2.1 DEFINITIONS

For the purposes of this Standard the following definitions shall apply:

Ambient temperature Nominal temperature of the liquid to be stored

Anchorage The means by which prestress force is permanently transferred to the concrete. Also, the method of ensuring that reinforcing bars and fixings acting in tension or compression are tied into a concrete member

Beam A member subjected primarily to loads and forces producing flexure

Capacity design In the capacity design of structures subjected to earthquake forces, regions of members of the primary lateral force-resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural members are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained

Column An element subjected primarily to compressive axial loads

Component A physically distinguishable part of a structure such as a wall, beam, column, slab or connection

Concrete A mixture of Portland cement or any other hydraulic cement, sand, coarse aggregate, and water

Construction joint An intentional joint in concrete work detailed to ensure monolithic behavior at both the serviceability and ultimate limit states

Convective mode The liquid in the upper portion of the tank vibrates with a long period sloshing motion and is referred to as the convective mode

Creep strain Relates to the deformation over a period of time under constant load

Ductility The ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake

End anchorage	Length of reinforcement, or a mechanical anchor, or a hook, or combination thereof, required to develop stress in the reinforcement; mechanical device to transmit prestressing force to concrete in a post-tensioned member
Fatigue load	Load representing repeated stress cycles on the component
Force, earthquake	Forces assumed to simulate earthquake effects
Hydrostatic pressure	The lateral pressure exerted by liquid when it is at rest
Impulsive mode	A portion of the liquid in the tank moves rigidly with the tank and is referred to as the impulsive mode
Load dead	The weight of all permanent components of a structure, including permanently fixed plant and fittings
Load, design	Combinations of loads and forces used in design as set out in AS/NZS 1170.0 and NZS 1170.5 or other referenced loading Standard for the applicable limit state. In seismic design for the ultimate limit state, the design load may be either the ultimate limit state forces or the forces resulting from the capacity design procedure depending on the case being considered
Monolithic concrete	Reinforced concrete cast with no joints other than construction joints
Overstrength	The overstrength value takes into account factors that may contribute to strength, such as higher than specified strengths of the steel and concrete, steel strain hardening, confinement of concrete, and additional reinforcement placed for construction and otherwise unaccounted for in calculations
Precast concrete	A concrete element cast-in other than its final position in the structure
Prestressed concrete	Concrete in which internal stresses of such magnitude and distribution have been introduced that the stresses resulting from loads are counteracted to some extent to ensure the required strength and serviceability are maintained
Pre-tensioning	A method of prestressing in which the tendons are tensioned before the concrete is placed
Progressive failure	Failure progressing from initial stage of damage, such as cracking, to final state of reaching the ultimate strength of any component within the system, which often occurs in interactive fashion

Reinforced concrete	Concrete containing steel reinforcement, and designed and detailed so that the two materials act together in resisting loads and forces
Restrained concrete	Concrete with restrained boundary conditions not allowing free shrinkage
Serviceability limit state	The state that corresponds to conditions beyond which specified service criteria for a structure or structural element are no longer met
Settlement	Permanent downward displacement of the foundation
Slenderness ratio	Ratio of the effective length of column to its least radius of gyration of its cross section
Stability	The ability of a member to maintain its structural function when deformed
Strength, compressive of concrete	The crushing resistance of cylindrical specimens of concrete, prepared, cured and tested in accordance with the Standard procedures prescribed in sections 3, 4 and 6 of NZS 3112: Part 2. This is normally denoted by the general symbol f'_c
Strength, design	The nominal strength multiplied by the appropriate strength reduction factor
Strength, lower characteristic yield of non-prestressed reinforcement	That yield stress below which fewer than 5% of results fall when obtained in a properly conducted test programme. Refer to AS/NZS 4671
Strength, nominal	The theoretical strength of a member section, calculated using the section dimensions as detailed and the lower characteristic reinforcement strengths as defined in this Standard and the specified compressive strength of concrete
Strength, over	See Overstrength
Strength, probable	The theoretical strength of a member section calculated using the expected mean material strengths as defined in this Standard.
Strength reduction factor	A factor used to multiply the nominal strength to obtain the design strength

Strength, specified compressive of concrete	A singular value of strength, normally at age 28 days unless stated otherwise, denoted by the symbol f'_c, which classifies a concrete as to its strength class for purposes of design and construction. It is that level of compressive strength which meets the production Standards required by section 6 of NZS 3109
Strength, upper characteristic breaking strength of non-prestressed reinforcement	That maximum tensile strength below which greater than 95% of the results fall when obtained in a properly conducted test programme
Structural	A term used to denote an element or elements which provide resistance to loads and forces acting on a structure
Structural adequacy	The ability of a member to maintain its structural function when exposed to fire
Structural ductility factor	A numerical assessment of the ability of a structure to sustain cyclic inelastic displacements
Structural performance factor	A factor which is used in the derivation of design earthquake forces in accordance with AS/NZS 1170.0 and NZS 1170.5 or other referenced loading Standard and 2.6.2.2 of NZS 3101 – Part 1
Surcharge loads	Loads on the surface of the backfill of the tank wall which induces additional pressure on the wall
Tendon	A steel element such as wire, cable, bar, rod, or strand which when tensioned imparts a prestress to a concrete member
Transfer	Act of transferring stress in prestressing tendons from jacks or pre-tensioning bed to a concrete member
Ultimate limit state	The state at which the design strength or ductility capacity of the structure is exceeded, when it cannot maintain equilibrium and becomes unstable
Wall	Means a structural wall, a vertical thin member, usually planar, which because of its position, strength, shape, and stiffness, contributes to the rigidity and strength of a structure

C2.1 Definitions

NZS 3101 provides further guidance on definitions.

2.2 ABBREVIATIONS

For the purposes of this Standard the following abbreviations shall apply:

NZBC	New Zealand Building Code
NZSEE	New Zealand Society for Earthquake Engineering
SLS	Serviceability limit state
ULS	Ultimate limit state

2.3 NOTATION

2.3.1 Quantity symbol definition

The following notations are used in this Standard.

NOTE – Appendix B uses separate notations that are not included in 2.3.1.

$A_{c,eff}$	effective concrete area
A_s	cross-sectional area of reinforcement
a	radius of a circular tank
b	one half of the width of a rectangular tank, perpendicular to the direction being considered
C	thermal stress coefficient from appropriate table
$C(T_i)$	elastic site hazard spectrum for horizontal loading for the site subsoil type, and the relevant mode
$C_d(T_c)$	horizontal design action for the first convective mode
$C_d(T_i)$	horizontal design action coefficient for mode i
$C_d(T_I)$	horizontal design action coefficient for the first impulsive mode
$C_h(T_i)$	spectral shape factor for the site subsoil type and the relevant mode, from NZS 1170.5
C_t	creep factor
c_s	wave impact coefficient
D	overall depth of a concrete cross section
d	distance from the extreme compression fibre to the centroid of tension reinforcement
d_b	nominal diameter of a bar, wire, or tendon
d_{max}	maximum vertical displacement of the convective sloshing wave from the at-rest level of the liquid
E	modulus of elasticity
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of reinforcing steel MPa
E_{s1}	earthquake load for serviceability limit state 1 (SLS1)
E_{s2}	earthquake load for serviceability limit state 2 (SLS2)
E_u	ultimate earthquake load
e	base of natural logarithms (approximately 2.71828)
F_e	earth pressure load
F_{gw}	ground water load
F_i	the equivalent static horizontal seismic force at a wall level

F_{lp}	liquid pressure load
F_P	forces or stresses resulting from prestress
F_R	horizontal earthquake force from tank roof
F_{sh}	forces or stresses resulting from shrinkage
F_{sw}	forces or stresses resulting from swelling
F_T	forces or stresses resulting from temperature variation
f'_c	characteristic compressive cylinder strength of concrete at 28 days
f_{cp}	compressive strength of concrete at transfer
$f_{ct.3}$	direct tensile strength of the concrete at three days
f_i	equivalent static horizontal force per unit length at a wall level
f_{in}	thermal stress on the inside face of wall
f_{ot}	thermal stress on the outside face of wall
f_s	tensile stress in non-tensioned reinforcing steel
f_{sr}	residual compression stress in reinforcement
f_t	principal tension stress
f_{t0}	principal tension stress at wall centreline
f_y	yield strength of the reinforcement
G	permanent action (self-weight or 'dead action')
H	height of the tank wall to the surface level of the liquid
H_e	height of the tank wall to the surface level of the soil
h_c	height of the centre of gravity of the convective weight
h'_c	equivalent height at which the convective weight is placed to give the total overturning moment arising from pressures on both the walls and base
h_D	hydrostatic head
h_I	height of the centre of gravity of the impulsive weight
h'_I	equivalent height from at which the impulsive weight is placed to give the total overturning moment arising from pressures on both walls and base
h_R	height of the centre of gravity of the roof
h_W	height of the centre of gravity of the wall
I_{cr}	cracked moment of inertia
I_g	uncracked moment of inertia
$k_f(\mu,\xi_i)$	correction factor for NZS 1170.5 elastic site hazard spectrum to account for ductility and level of damping
L_I	Langelier Saturation Index
ℓ	one half of the length of a rectangular tank in the direction being considered
M	design moment action for ULS
M_T	moment due to thermal stress
M_W	total overturning moment acting at the level of the base of the wall immediately above the floor slab, from the hydrodynamic loads on the walls

M'_W	total overturning moment acting on the floor slab, from the hydrodynamic loads on the floor and the walls
N_T	axial force due to thermal stress
$N(T_i,D)$	near fault factor, from NZS 1170.5
p_b	bouyancy pressure
p_r	pressure on roof due to convective wave
Q	imposed load (due to occupancy and use, 'live' load) in accordance with AS/NZS 1170.1
Q_v	membrane shear force
q_v	shear force flow
R	return period factor for the relevant limit state and tank importance level and design life, from NZS 1170.5
R_F	reduction factor that accounts for the reduced section rigidity that accompanies concrete cracking
S_{FACT}	shape factor used for determining thermal stress coefficients (Appendix D)
S_p	structural performance factor, to be taken as 1.0
S_u	ultimate value of various loads appropriate for particular combinations in accordance with AS/NZS 1170.0
t	thickness of the wall, roof or floor
T_C	period of vibration of the fundamental convective mode
T_i	period of vibration of appropriate mode of response
T_I	period of vibration of the fundamental impulsive mode
u	wave particle velocity
V	shear force
V_H	total horizontal shear force at base of tank
V_{Hi}	horizontal seismic base shear associated with mode i (impulsive, convective, and so on)
V_R	horizontal earthquake force from roof
v	shear stress
W_C	equivalent convective weight of the liquid contents
W_i	equivalent weight of tank and liquid contents for the particular mode of vibration considered
W_I	equivalent impulsive weight of the liquid contents
W_L	total weight of the liquid contents
W_R	weight of the tank roof
W_W	weight of the tank walls
w_{bc}	equivalent convective unit weight at base of the wall
w_{bi}	equivalent unit weight (impulsive or convective) at the base of the wall
w_{bI}	equivalent impulsive unit weight at the base of the wall
w_i	equivalent unit weight on the tank wall of liquid contents for the impulsive or convective mode of vibration
w_k	mean crack width
w_{tc}	equivalent convective unit weight at the surface of the liquid

w_{ti}	equivalent unit weight (impulsive or convective) at the surface of the liquid
w_{tI}	equivalent impulsive unit weight at the surface of the liquid
x_{min}	concrete compression zone depth
Z	seismic zone hazard factor, from NZS 1170.5
α_1, α_2	effective concrete area parameters
α_C	linear coefficient of thermal expansion of the concrete
$\Delta_{\sigma s}$	increase in tendon stress once decompression occurs in a partially prestressed member
ε_s	shrinkage or swelling strain
ε_{sh}	shrinkage strain
ε_{sw}	swelling strain
μ	displacement ductility factor for impulsive modes
π	ratio of the circumference of a circle to its diameter
δ	angle to the earthquake direction
γ_e	unit weight of soil
γ_ℓ	unit weight of liquid
ψ_c	combination factor for imposed loads in accordance with AS/NZS 1170.0
ψ_ℓ	factor for determining quasi-permanent values (long-term) of loads in accordance with AS/NZS 1170.0
ξ_i	damping level appropriate to mode of response
θ	temperature change in tank wall
θ_A	average temperature change in tank wall
θ_D	differential temperature change in tank wall
θ_T	total temperature change in tank wall
ρ	ratio of tension reinforcement area to concrete area
ρ_{min}	minimum ratio of tension reinforcement area to concrete area
ρ_p	ratio of total reinforcement area to effective concrete area
ρ_{pmin}	minimum ratio of total reinforcement area to effective concrete area

3 GENERAL DESIGN REQUIREMENTS

3.1 DESIGN METHODS

Tank design loads and load combinations are specified in section 4 and design methods for the serviceability limit state, the ultimate limit state and durability are specified in sections 5, 6, and 7 respectively.

The design should take account of the conditions of edge restraint at wall junctions with floor and roof.

C3.1

Serviceability rather than ultimate limit state considerations will commonly govern the design of liquid retaining structures.

All connections of wall with floor and roof exert some measure of restraint that affects wall design. Particular attention should be given to the translation and rotation restraint, the extent of which varies depending on the type of joint: fixed, hinged or free. Actual details may exhibit properties of one or more types at different stages of construction. Design calculations are generally based on the assumption that joints are either fully fixed or completely unrestrained against rotation and/or displacement. In reality such things as friction, soil movement and foundation deformation result in an intermediate degree of the implications of which may need to be assessed.

3.2 SUPPORT STRUCTURES

Support structures for elevated tanks shall be designed to the requirements of AS/NZS 1170.0 together with the appropriate design code for the material to be used.

3.3 TANKS STORING POTABLE WATER

Roofs and screens shall be provided to potable water tanks to prevent contamination and the entry of vermin.

3.4 PIPEWORK

Pipework in the close vicinity of, attached to or located adjacent to the liquid retaining structure shall be designed to withstand applied actions including settlement of the tank, earthquake induced ground deformations, seismic forces and hydrodynamic seismic forces.

3.5 FREEBOARD

Provisions shall be made to accommodate the maximum wave oscillation generated by earthquake acceleration.

C3.5

The amount of freeboard required for design will vary. Where overtopping is tolerable no freeboard provision is necessary. Where loss of liquid must be prevented (for example, tanks for the storage of toxic liquids), or where overtopping may result in scouring of the foundation materials or cause damage to pipes and/or roof, then provisions should be made by:

(a) Freeboard allowance; and/or

(b) Designing the roof structure to resist the resulting uplift pressures.

The consequences of overtopping of the walls or damage to the roof under uplift pressure are unlikely to be as significant as damage to the walls and are unlikely to result in significant loss of liquid. Consideration may be given to using a lower seismic return period factor when checking overtopping or roof pressure than used for the design of the tank walls.

4 LOADS AND LOAD COMBINATIONS

4.1 GENERAL

The design of structures and members for stability, strength and serviceability shall take account of the load and load combinations for strength in accordance with AS/NZS 1170.0 and of the action effects directly arising from the loads and other actions included in this section.

> ***C4.1***
>
> *There are two broad types of loading: that resulting from the application of forces and that resulting from the application of deformation (strain). In a tank the force loading is exemplified by contained fluid pressure; the strain induced loads are temperature, shrinkage, and swelling.*

4.2 LOADS AND OTHER ACTIONS

4.2.1 Liquid pressure

The pressure from liquid at maximum overflow level based on the liquid specific gravity at the design temperature plus any internal vapour pressure above the liquid level.

> ***C4.2.1***
>
> *Overflow systems usually require a surcharge to initiate operation. In most cases the surcharge is small and can safely be ignored, that is, the overflow level is taken as the inlet level of the overflow pipe. Where the surcharge is likely to be large, the additional head should be included as a design load.*

4.2.2 Earth pressure

The lateral pressure from earth backfill, symmetrical or asymmetrical. Net lateral loads, including those caused by unequal backfill, shall be determined by rational methods of soil mechanics based on foundation and soils investigations. Surcharge loads on backfilled surfaces shall be considered.

4.2.3 Temperature

The walls and roofs of tanks shall be designed for the action effect arising from differential temperature gradients through the member.

For tanks containing liquids at ambient temperature and subject to direct solar radiation, the design temperature gradients considered shall include the following cases:

(a) For roofs:

  (i) A ±20 °C variation from the mean temperature, and

  (ii) The temperature criteria given in table 1.

> ***C4.2.3(a)***
>
> *Temperature stresses on the roof are in general small and of little significance unless the roof is cast monolithically with the wall. However roof support details should make provision for temperature-induced displacements.*

(b) For walls:

(i) When the tank is filled with liquid, by a +30, –20 °C variation, and

(ii) When the tank is empty, by a +20, –12 °C variation; from the internal wall temperature as shown in figure 1.

NOTE – Temperature effects for liquids at other than ambient temperatures are not specified in this Standard.

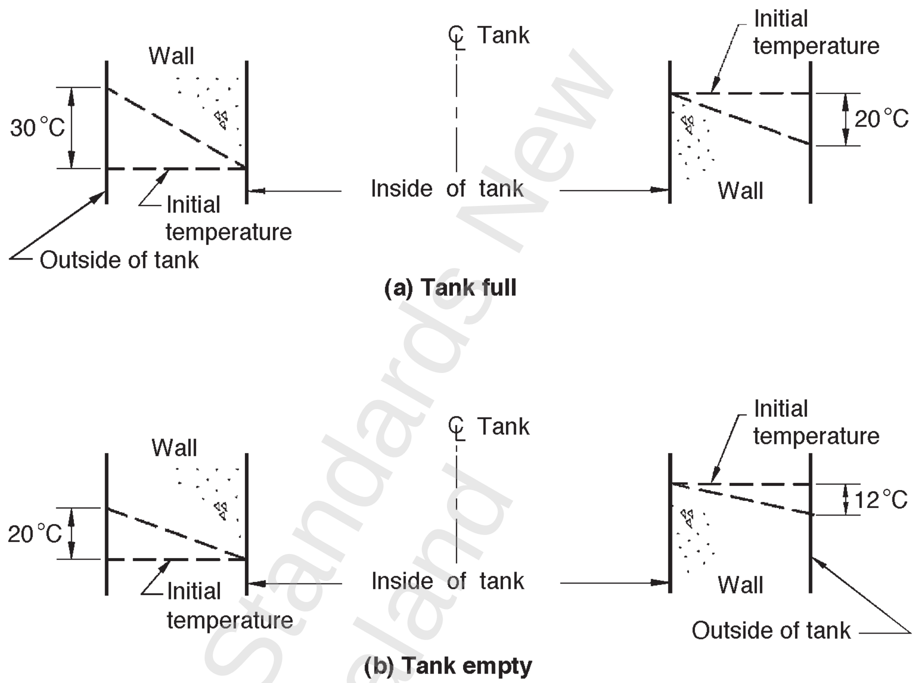


Figure 1 – Temperature distributions in tank walls

C4.2.3

The temperature distributions of figure 1 are appropriate for tanks subject to direct solar radiation. Under normal circumstances a tank will be considered to be stress free at the initial temperature. Special consideration should be given to shielded or buried tanks for which lower design gradients will generally be applicable. The walls and roof of buried tanks may be exposed during testing, consideration should be given to the appropriate temperature gradient to be used during this period of exposure, which may be lower than specified in figure 1.

Design tables suitable for the common range of circular tanks are included in Appendix D. Thermal stresses are presented in the form:

$$f = CE_c \alpha_c \theta$$

where C is a coefficient dependent on the shape factor ($H^2/2at$), vertical location on the wall, the degree of base restraint, and type of temperature effect (average, differential or combined). The tables are in terms of surface stresses assuming uncracked section stiffness. If the tank is designed on the basis of a cracked section, then these stresses should be modified by a reduction factor R_F to account for the reduced section rigidity that accompanies cracking.

A method of evaluating R_F reinforced concrete design is described in section 5 and the implications for partially prestressed design discussed in the same section. There is no reduction ($R_F = 1.0$) for fully prestressed design as the concrete is assumed to remain uncracked.

Thermal moments and axial forces are given by:

$$M_T = R_F \frac{(f_{in} - f_{ot})}{2} \cdot \frac{t^2}{6}$$

$$N_T = R_F \frac{(f_{in} + f_{ot})t}{2}$$

where f_{in} and f_{ot} are the total thermal stresses on the inside and outside surfaces respectively for an uncracked wall as shown in figure.1. A tank designed on the basis of a cracked section should be checked under the stresses resulting from the application of the moment M_T and axial force N_T.

The evaluation of the thermal response of concrete tanks subjected to unusual temperature conditions may require sophisticated heat flow analysis. A description of the procedures involved in such analysis is given by Priestley (1976). In addition to final temperature gradients, transient thermal conditions may also be an important design consideration. For example, the temporary thermal gradients resulting from the rapid filling of a tank with a relatively hot liquid may be more severe than the eventual equilibrium condition.

Where a thermal condition is maintained for a long period, creep reduces the stress experience by the member. The removal of the thermal condition can produce reversals of the stress system of similar magnitude but opposite sign to initial elastic.

Stresses induced in storage tanks by thermal loading and creep and shrinkage effects are directly proportional to the value of the modulus E, pertaining at the time the action is applied. Consequently it is important that realistic values should be adopted. Wherever possible, locally based test data should be used to assist in assessing the design value of E_c and recognition made of the fact that the in situ compressive strength will normally exceed the specified strength by 20% or more where E_c is calculated as a function of compressive strength.

Table C4.2.3 – Typical coefficients of thermal expansion for water-cured concrete made from different aggregate types

Aggregate	Coefficient of thermal expansion x 10^{-6}/°C
Andesite	6.5
Basalt	9.5
Dolerite	8.5
Foamed slag	9
Granite	9
Greywacke	11
Limestone	6
Pumice	7
Quartzite	13
Sandstone	10

In the absence of information on the aggregate type to be used in construction of the tank, a reasonably conservative value of $\alpha_c = 11 \times 10^{-6}$ should be used.

Table 1 – Roof temperature criteria

Region	Linear temperature gradient, degrees Celsius per 100 mm of roof thickness
Snow (outside colder than inside)	10
Other (outside hotter than inside)	5
NOTE – Thermal forces calculated in accordance with NZS 3106 can govern the design of reinforcement in some cases.	

4.2.4 Moisture variation

In the absence of a rational analysis of moisture variation, appropriate to the expected construction/loading history for the structure, the minimum effects due to moisture variation, either shrinkage or swelling, for both roofs and walls, shall be determined for the strains as given in table 2.

Table 2 – Moisture variation – Shrinkage and swelling strains

Thickness (mm)	Mean shrinkage and swelling strain (creep adjusted) x 10^{-6}			
	Shrinkage (ε_{sh})		Swelling (ε_{sw})	
	Precast	Cast in situ	Precast	Cast in situ
100	70	120	300	250
150	50	85	205	170
200	45	70	160	135
≥250	35	60	135	110

C4.2.4

The shrinkage and swelling strains given in table 2 were derived using the CEB-FIP (1978) Model Code, except that predicted shrinkages were doubled in accordance with the recommendations of Vadhanavikkit and Bryant (1984).

The following assumptions were made:

(a) Shrinkage commences immediately after casting;

(b) Shrinkage regain is 100 % and occurs immediately the tank is filled;

(c) Precast wall panels are subject to free shrinkage until they are erected, 50 days after casting. Shrinkage continues until the tank is filled, a further 50 days after erection;

(d) For tanks cast in situ, filling occurs 100 days after the walls are cast;

(e) Shrinkage strains are reduced by a creep reduction factor given by:

$$\frac{1-e^{-C_t}}{C_t}$$

Where C_t is the creep factor for the concrete between the time shrinkage stresses commence (that is, when shrinkage movement is restrained) to the time the tank is filled;

(f) The creep reduction factor used to assess long term (500 days after filling) swelling (implied by the load combinations) is given by $e^{-C_{t\infty}}$ where $C_{t\infty}$ is the long term creep factor.

For a tank cast in situ, shrinkage stresses develop between the time the walls are cast until the tank is filled. Precast panels on the other hand do not develop shrinkage stresses until the panels are locked into position by which time a significant amount of shrinkage has already occurred.

On filling, there is a rapid shrinkage regain or swelling of the concrete. Swelling strains generally exceed creep-reduced shrinkage strains because of the swelling rate and hence in the short term is less affected by creep relaxation. (The swelling strains given in table 2 are net strains, that is, counteracting shrinkage strains have been deducted). Although swelling strains are generally similar for both types of construction, because the counteracting shrinkage strains are lower for precast panels than those for cast in situ construction, the net swelling strains are correspondingly higher.

The initial swelling strains caused by the shrinkage regain that occurs when the tank is filled are in time reduced by creep relaxation. This reduction is taken into account in the value given to the load factor used in the load combinations in section 4.

Exposure of a tank to wind and sun causes the outside surface to dry out resulting in a shrinkage gradient through the wall. Few data are available relating to the extent of the differential and to the distribution through the wall thickness. It appears however that the gradient is low for much of the wall thickness with most of the differential occurring in the outer 15%. This results in crazing of the outer surface which, while relieving the shrinkage stresses, has negligible effect on the serviceability of the tank. Consequently, differential shrinkage gradients occur between the inside and outside faces of sections of the wall so they do not require specific design.

The stresses caused by volumetric changes in concrete are characteristically similar to those caused by thermal effects. Shrinkage is directly analogous to an average temperature decrease while swelling corresponds to an average temperature increase.

The similarity of thermal and shrinkage effects means that the method of analysis developed for temperature stresses can also be used for calculating shrinkage stresses. The thermal equivalent is derived by dividing the shrinkage (or swelling) strain by the coefficient of thermal expansion for concrete:

$$T = \frac{\varepsilon_s}{\alpha_c}$$

In many practical cases it will be found that shrinkage induced stresses in the combinations specified in section 4 will not control design.

Shrinkage (or swelling) of the roof will not produce significant stresses unless the shrinkage (or swelling) movement is restrained, for example, where the roof is cast monolithically with the walls.

4.2.5 Earthquake

Loads due to earthquakes shall be determined in accordance with Appendix A.

4.2.6 Other actions

Any action that may significantly affect the stability, strength and serviceability of structures and members, including but not limited to the following, shall be taken into account:

(a) Fatigue;

(b) Progressive failure;

(c) Ground movements;

(d) Construction loads;

(e) Wind;

(f) Snow and ice;

(g) Prestress load.

4.3 LOAD CONTROL

Positive means such as an overflow pipe of adequate size shall be provided to prevent overfilling tanks. Overflow pipes, including their inlet and outlet details, should be capable of discharging the liquid at a rate equal to the maximum fill rate when the liquid level is at its highest acceptable level.

Unless the tank is designed for internal pressures, one or more vents shall be provided for tanks with roofs. The vents shall limit the internal pressure to an acceptable level when the tank is being filled or emptied at its maximum rate. The maximum emptying rate may be taken as the rate caused by the largest pipe being broken immediately outside of the tank.

Provision shall be made to accommodate the wave oscillation generated by earthquake acceleration as outlined in Appendix A.

4.4 NON-SYMMETRIC LOADS FOR CIRCULAR TANKS

For thin walled circular tanks ($t/a < 0.03$) non-symmetrical loads (such as temperature loads and hydrodynamic seismic pressures) may be considered to be rotationally symmetric and equal to the value at the section under consideration.

C4.4

Ambient thermal loads and hydrodynamic seismic pressures are not rotationally symmetric, but vary continually around the tank's perimeter. Analyses have shown that this variation is generally low enough for stresses at any given section to depend only on the local temperature or pressure distribution.

4.5 LOAD COMBINATIONS FOR SERVICEABILITY LIMIT STATE

The following combinations of actions for the serviceability limit states shall be considered:

(a) Group A (long term) loads:

Wall: $G + F_e + F_{gw} + F_P + (F_{sh} \text{ or } 0.5F_{sw})$ ....(1)

$G + F_{lp} + F_P + 0.5F_{sw}$ ....(2)

$G + F_{lp} + F_e + F_{gw} + F_P + 0.5F_{sw}$ ....(3)

Roof: $G + F_P + F_{sh}$ ....(4)

(b) Group B (short term) loads:

Wall, tank full: $G + F_{lp} + F_e + F_{gw} + F_P + F_T + 0.7F_{sw}$ ....(5)

$G + F_{lp} + F_e + F_{gw} + F_P + (E_{s1} \text{ or } E_{s2})$ ....(6)

Wall, tank empty: $G + F_e + F_{gw} + F_P + F_T + (0.7F_{sh} \text{ or } 0.35F_{sw})$ ....(7)

Roof: $G + Q + F_P + F_T$ ....(8)

$G + F_p + F_T + (0.7F_{sh} \text{ or } 0.7F_{sw})$ ....(9)

If a worse effect is obtained by the omission of one or more of the transient actions then this effect shall be taken into account.

C4.5

Transient loads that should be omitted, if beneficial, are shrinkage (F_{sh}), swelling (F_{sw}), temperature (F_T) and groundwater (F_{gw}).

The prestress force may vary between F_{Pmax} and F_{Pmin}, the maximum and minimum due to in-time losses respectively. To ensure that the more adverse condition is incorporated in design, both F_{Pmax} and F_{Pmin} should be considered in the load combinations.

(a) Group A loads. Group A load cases are permanent loads plus variable loads of long duration; or permanent loads plus frequently repetitive loads. Shrinkage is a long duration load. Swelling can be either short or long duration; this is accounted for in the load factor. Load case (1) equally applies for shrinkage and swelling; shrinkage applies when the tank is empty prior to filling, swelling applies when the tank is emptied for maintenance. For buried tanks it is necessary to consider the possibility of test loading prior to backfilling (load case (2)).

(b) Group B loads. Group B load cases are permanent loads plus infrequent combinations of transient loads. Load case (7) applies equally to shrinkage and swelling; shrinkage – tank empty prior to filling, swelling – tank empty for maintenance. The earthquake load for AS/NZS 1170.0 serviceability limit state 1 (E_{s1}) or limit state 2 (E_{s2}) should be applied depending on the importance level of the structure. It is considered to be overly conservative to combine the very short duration earthquake serviceability loads with other transient loads.

4.6 LOAD COMBINATIONS FOR ULTIMATE LIMIT STATE

The following combinations for the ultimate limit states shall be considered:

Wall $1.2G + 1.2F_{lp}$ ....(10)

$1.2G + 1.5\,F_e + 1.5\,F_{gw}$ ....(11)

$1.2G + 1.2\,F_{lp} + 1.5F_e + 1.5\,F_{gw}$ ....(12)

$G + E_u + F_{lp}$ ....(13)

$G + E_u + F_{lp} + F_e + F_{gw}$ ....(14)

Roof $1.35G$ ....(15)

$1.2G + 1.5Q$ ....(16)

$1.2G + 1.5\Psi_{\ell}Q$ ....(17)

$G + E_u + \Psi_c Q$ ....(18)

$1.2G + S_u + \Psi_c Q$ ....(19)

Wind loads shall be considered for lightweight roofs.

C4.6

The strain induced loads: temperature, shrinkage, swelling, and prestress, will not significantly affect the ultimate strength of a structure and hence do not need to be considered at the ultimate limit state.

5 DESIGN FOR SERVICEABILITY

5.1 GENERAL CONSIDERATIONS

5.1.1 Liquid tightness

The components (walls and floor) of liquid-retaining structures shall be classified for the required standard of tightness as prescribed in table 3, taking into account the required function of the structure. Appropriate limits to cracking depending on the classification of the component considered shall be selected. In the absence of more specific requirements, the provisions of table 3 may be adopted.

Table 3 – Liquid tightness class and leakage control provisions for Group A loads

Tightness class	Tightness standard	Provisions for achieving tightness standard
0	Leakage acceptable, or leakage of liquids irrelevant	The crack control provisions of NZS 3101 may be adopted.
1	Leakage to be limited to a small amount. Some surface staining or damp patches acceptable Sustained leakage of contents will not cause corrosion of reinforcement (for example, fresh water)	Any cracks which can be expected to pass through the full thickness of the section should be limited to w_k. The crack control provisions of NZS 3101 apply where the full thickness of the section is not cracked and where the conditions in 5.1.2 are fulfilled.
2	Leakage to be minimal. Appearance not to be impaired by staining Sustained leakage of contents may promote corrosion of reinforcement (for example, sea water)	Cracks which may be expected to pass through the full thickness of the section should be prevented unless appropriate measures (for example, liners, coatings or water bars) have been incorporated. The crack control provisions of NZS 3101 apply where the full thickness of the section is not cracked and where the conditions in 5.1.2 are fulfilled.
3	No leakage permitted	Generally, special measures (for example liners or prestress) will be required to ensure water tightness.

For tightness class 1 components, the full thickness crack widths shall be limited as follows:

for $h_D/t \leq 20$, $w_k < 0.2$ mm

for $h_D/t \geq 50$, $w_k < 0.05$ mm

where, h_D is the hydrostatic head, and t is the wall or floor thickness.

For intermediate values of h_D/t, linear interpolation between 0.2 mm and 0.05 mm may be used.

The extent of cracking shall be controlled by limiting the tensile stress in the reinforcing steel in accordance with 5.2 of this Standard.

C5.1.1

It should be noted that all concrete will permit the passage of small quantities of liquids and gases by diffusion.

The tightness classes of the walls and floor in the same tank may be different.

The permissible crack width for tightness class 1 is related to the pressure gradient across the section. Limitation of the crack widths to these values should result in the effective sealing of the cracks by autogenous healing within a relatively short time. It should be noted that concrete containing supplementary cementitious materials, for example fly ash, may take longer to heal due to slower rate of hydration and a smaller crack width may therefore be required to achieve the required tightness class.

5.1.2 Minimum compression zone depth

To provide adequate assurance for structures of tightness classes 2 or 3 that cracks do not pass through the full width of a section, the design value of the depth of the compression zone should be at least x_{min} calculated for the Group A (long term) combination of actions. Where a section is subjected to alternate actions, cracks should be considered to pass through the full thickness of the section unless it can be shown that some part of the section thickness will always remain in compression. This thickness of concrete in compression should normally be at least x_{min} under all appropriate combinations of actions. The action effects may be calculated on the assumption of linear elastic material behaviour. The resulting stresses in a section should be calculated assuming that concrete in tension is neglected.

The recommended value for x_{min} is the lesser of 50 mm or 0.2t where t is the element thickness.

5.1.3 Movement joints

Movement joints shall be provided as necessary to ensure that design assumptions are realised. Provision shall be made for displacement and rotation without loss of contents.

C5.1.3

A movement joint is a specially formed joint intended to accommodate relative movement between adjoining parts of a structure, special provision being made for maintaining the watertightness of the joint. Movement joints may be of the following types:

(a) Contraction joint. This is a movement joint which has a deliberate discontinuity but no initial gap between the concrete on both sides of the joint. The joint is intended to permit contraction of the concrete.

A distinction should be made between a complete contraction joint, in which both the concrete and reinforcement are interrupted, and a partial contraction joint, in which only the concrete is interrupted while the reinforcement is continued through the joint.

A water stop and/or sealing compound should be provided at contraction joints.

(b) *Expansion joint. This is a movement joint which has complete discontinuity in both reinforcement and concrete and is intended to accommodate either expansion or contraction of the structure. Water stops, a joint sealing compound and joint filler are essential at expansion joints.*

(c) *Sliding joint. This is a movement joint which has complete discontinuity in both reinforcement and concrete. Special provision is made to facilitate relative movement in the plane of the joint.*

(d) *Sliding layer. A sliding layer is a special category of sliding joint intended to permit sliding over a considerable area, for example, between a floor and a blinding layer.*

The effectiveness of movement joints in controlling cracking depends on their correct location, which may be characterised as the place where cracks would otherwise develop, for example, at changes of section. Movement joints in walls should preferably align with joints in the floor or wall footing. All movement joints should be designed and constructed so that the watertightness will be maintained during the subsequent movement of the joint. The location of all movement joints should be decided by the engineer and be indicated on the drawings.

Joints in floors. *The floor of a reservoir may be designed to permit thermal and shrinkage contraction by minimising restraints to movement. A separating layer should be provided between the floor slab and blinding concrete, for example, a layer of thick polyethylene sheeting. Panels may be cast consecutively or with a gap between adjacent panels. Joints should be complete contraction joints except where allowance is made for expansion and an expansion joint provided. Guidelines for the design of joints are provided in CIRIA C660.*

Cracking can by controlled by strip casting and cutting to form contraction joints, provided the concrete is cut before initial thermal cracking occurs. It should be noted that this may be within three days of placing.

Alternatively, the floor may be designed as fully restrained against thermal contraction. Joints to allow for contraction should not be necessary but, if provided, should be of the partial contraction type.

Joints in walls. *Careful consideration should be given to the probable contraction behaviour of the walls. Design and construction practice for restrained and unrestrained walls is described in BS 8007.*

Pipes through walls and floors. *When it is necessary for a pipe to pass through a wall or floor it is preferable to cast the pipes into the panel when it is concreted. If this is not practicable it will be necessary to box out. In either case it is desirable that the position of the pipe should not coincide with a joint. When an opening has been boxed out the sides of the opening should be treated as construction joints.*

Jointing material and water stops. *The performance of joints is very dependent on the performance of the jointing materials used and care should be taken in selecting appropriate materials for particular conditions.*

5.2 REINFORCED CONCRETE

5.2.1 Control of cracking

The structure shall be designed so that maximum calculated crack widths from early-age thermal strains or the imposed loads and actions specified in section 4, meet the requirements for leak control and durability.

C5.2.1

It has not been normal practice to add early-age thermal crack widths to those arising from structural loading, with no reported detriment to performance (CIRIA C660).

The early-age crack control provisions in this section are for fully restrained concrete with normal mix designs. The extent of cracking and amount of crack-control reinforcement can be reduced by removing or reducing restraints, limiting the temperature rise due to hydration of the cement and other methods. Guidelines are given in CIRIA C660.

5.2.2 Effective reinforcement ratio

The effective reinforcement ratio shall be calculated using the following equation:

$$\rho_p = A_s / A_{c,eff} \qquad \text{(Eq.5.1)}$$

where $A_{c,eff}$ is the effective concrete area as shown in figure 2 (α_2= 0.5D, but not greater than 250 mm, except for the surface of the slab that is in contact with the ground, then not greater than 100 mm.)

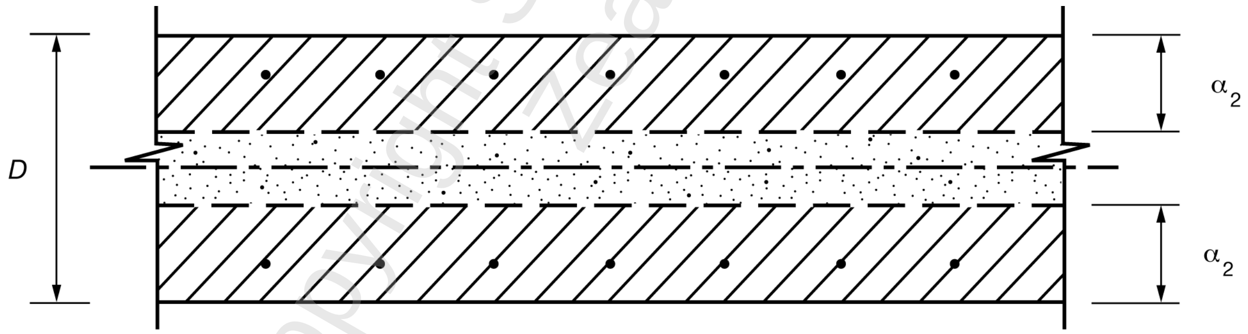


Figure 2 – Effective concrete area

5.2.3 Minimum reinforcement ratio for control of early-age thermal cracks

The minimum reinforcement ratio shall be determined from:

$$\rho_{pmin} = \frac{f_{ct.3}}{f_y} \qquad \text{(Eq.5.2)}$$

Where

ρ_{pmin} = minimum reinforcement ratio

$f_{ct.3}$ = direct tensile strength of concrete at three days

f_y = yield strength of the reinforcement

_C5.2.3_

This minimum reinforcement is required to control crack spacing and hence crack widths, by preventing the yielding of the reinforcement when early-age thermal cracks form.

$f_{ct.3}$ is the mean value of the tensile strength of the concrete at the time when the cracks may first be expected to occur. The values are given by the relationship from 3.1.2 of BS EN1992-1-1. Estimated values are set out in table C5.2.3:

Table C5.2.3 – Mean tensile strength of concrete at three days

Concrete strength *f'_c (MPa)*	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*
$f_{ct.3}$ (MPa)	*1.32*	*1.53*	*1.73*	*1.92*	*2.10*	*2.27*	*2.44*	*2.52*	*2.61*

5.2.4 Reinforcement ratios for control of early-age thermal cracks

The reinforcement ratio for 0.05 mm, 0.1 mm, 0.15 mm, and 0.2 mm mean crack widths are given in table 4.

Table 4 – Reinforcement ratio for fully restrained concrete

d_b (mm)	ρ_{pmin} (%)			
	0.05 mm	0.10 mm	0.15 mm	0.20 mm
<12	2.0	0.78	0.48	0.35
16	2.7	1.0	0.64	0.47
20	3.4	1.3	0.80	0.58
25	4.2	1.6	1.0	0.73
32	5.4	2.1	1.3	0.93

_C5.2.4_

The reinforcement ratios in table 4 are for fully restrained concrete calculated in accordance with the method given in Appendix B. Refer to CIRIA C660 for other restraint conditions.

5.2.5 Cracking from imposed loads and actions

The size of cracks shall be controlled by limiting the tensile stress in the reinforcing steel (f_s) under the most severe combination of service loads as follows:

For Group A (long term) loads f_s shall be limited so that calculated crack widths meets the liquid tightness class requirements, and the requirements for durability (see section 7).

For Group B (short term) loads except combinations including E_{S2} ....f_s < 240 MPa

For Group B (short term) load combinations including E_{S2} ....$f_s < 0.9f_y$

Provisions for the design of reinforcement to control cracking are given in Appendix B.

Limiting stresses in steel reinforcement calculated in accordance with Appendix B for a range of mean crack widths are given in table 5 and table 6.

Table 5 – Limiting stresses in steel reinforcement where compression zone depth complies with 5.1.1

d_b (mm)	f_s (MPa)		
	0.2 mm	0.3 mm	0.4 mm
<12	180	270	340
16	150	230	305
20	130	200	265
25	115	170	225
32	95	140	190
NOTE – The steel stress should not exceed 0.8 f_y			

Table 6 – Limiting stresses in steel reinforcement where compression zone depth is less than 5.1.1

d_b (mm)	f_s (MPa)			
	0.05 mm	0.10 mm	0.15 mm	0.20 mm
<12	27	53	80	105
16	21	43	64	85
20	18	35	53	70
25	15	30	45	60
32	12	24	36	48

C5.2.5

The limiting steel stresses given in table 5 and table 6 have been calculated in accordance with the method given in Appendix B. The reinforcement was assumed to be in two equal layers and at 50 mm cover. The values in the tables should be conservative for reasonable section thicknesses and reinforcement contents, but may be unconservative for high reinforcement contents or cover exceeding 50 mm. As an alternative to limiting the steel stress to the value given in the tables, crack widths may be calculated directly using the method in Appendix B.

Larger crack widths are allowed for the Group B (short term) loads.

5.2.6 Stiffness of cracked section

Strain induced forces (such as thermal and shrinkage) in the walls of circular reservoirs may be calculated on the basis of an uncracked section and reduced by a factor R_F representing the local reduction in stiffness resulting from cracking in each direction.

A more rigorous approach, including an analysis of the cracked section stiffness and/or finite element analysis may be used and may be less conservative than the use of R_F.

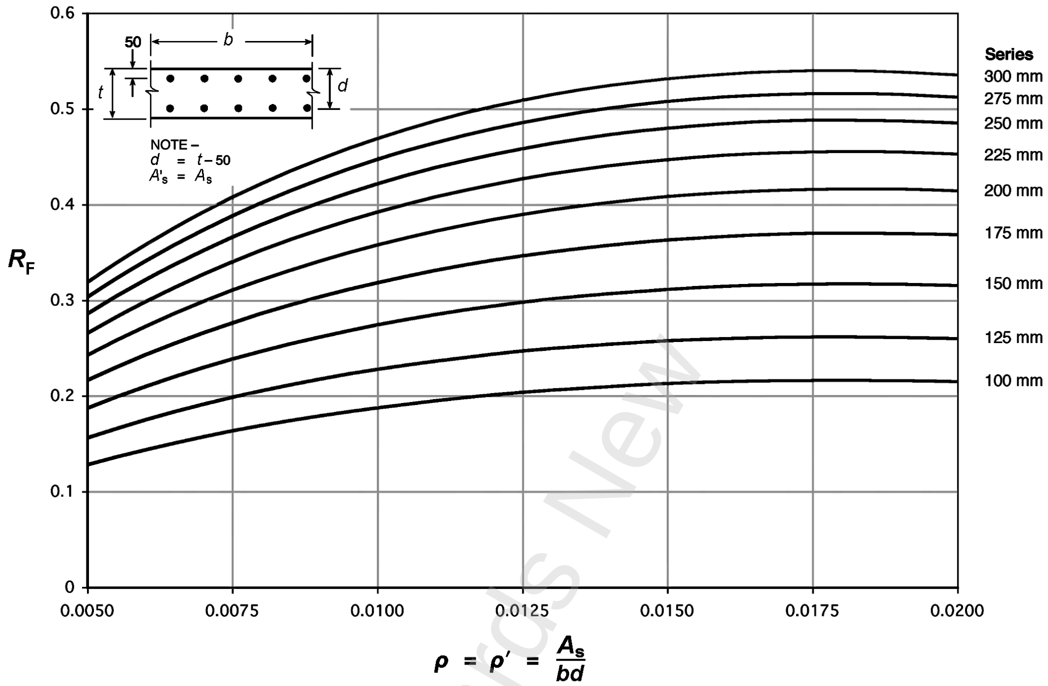


Figure 3 – Reduction of stiffness of doubly reinforced wall on cracking (including tension stiffening effect)

C5.2.6

Although curvatures caused by strain induced loads (temperature and shrinkage) are insignificant as a proportion of the ultimate deformation, they may be large compared to elastic loadings at design load level. Hence, although temperature or shrinkage will not significantly affect the ultimate capacity of the tank, they may severely bear on the serviceability of the structure.

Increasing temperature or shrinkage subject the concrete to increasing stresses until the cracking strength of the section is reached. Further increase in stress is accompanied by a decrease in section rigidity as the crack propagates. A point is eventually reached where the crack propagation stops because the section reaches a rigidity capable of resisting the stress without further deformation. This stress is somewhat less than that calculated assuming an uncracked section and can be assessed simply, and with sufficient accuracy, by factoring the uncracked section moments and axial forces by a reduction coefficient R_F *representing the reduction in stiffness with cracking. This reduction factor is given by the ratio of the cracked moment of inertia* I_{cr} *to the uncracked moment of inertia* I_g *adjusted for tension stiffening in the concrete. Values of* R_F *are plotted in figure 3 for a range of wall thickness and steel ratios.*

The I_{cr} *values used in tabulating* R_F *ignore the presence of axial forces. This omission is necessary to maintain simplicity, however the resulting errors are expected to be small and on the conservative side. Specifically, axial tension would further reduce section stiffness with a corresponding decrease in thermal and/or shrinkage stresses. Axial compression on the other hand increases section stiffness with a corresponding increase in stresses. However, unless allowable compressive stresses are exceeded in the concrete, this load case is unlikely to result in an adverse service condition. Refer to Vitharana and Priestley (1998 and 1999) for further discussion on the effect of cracking on thermal loading.*

The ratio I_{cr}/I_g depends on wall thickness and reinforcement content. Because these parameters, the latter in particular, may vary with wall height, it may be necessary to calculate R_F values for each critical section of the wall, that is, where there is a change of wall thickness or steel content.

Tension stiffening significantly increases the stiffness above that calculated at a crack. Limited experimental and theoretical evidence for slabs indicates that the stiffening effect decreases with increasing reinforcement content ρ, and with increasing moment level (after cracking). Approximate maximum figures are 100 % increase at $\rho = 0.005$ and 30% increase at $\rho = 0.02$. Because of its significance, tension stiffening has been included in the derivation of the R_F values.

5.3 PRESTRESSED CONCRETE

5.3.1 General

All prestressing tendons shall be bonded.

C5.3.1

In the case of circular prestressed tanks, care needs to be taken to avoid the possibility of local failures due to the tendons breaking out through the inside cover. In general, this will be avoided if the theoretical centroid of the horizontal cables lies in the outer half of the wall.

The diameter of a duct within a wall should generally not exceed 0.25. times the wall thickness. The prestressing force on a wall should be distributed as evenly as possible. Anchorages or buttresses should be so arranged as to reduce the possibilities of uneven force distribution unless specific measures are taken to take the effects into account.

5.3.2 Analysis

In addition to the requirements of section 4, the analysis shall take account of the full effects of prestressing including secondary effects and time-dependent creep effects. Analysis shall be carried out for the following load conditions and combinations:

(a) Conditions at any stage of prestress;

(b) Group A load combination that comprises predominantly long-term loads;

(c) Group B load combination that includes the short-term transient loads.

5.3.3 Limiting concrete stresses for serviceability

Except as permitted by 5.3.5 and for load combinations that include E_{S2}, stresses shall be calculated on the basis of uncracked sections, and shall remain within the limits specified in table 7.

For Group B (short term) load combinations including E_{S2}, $f_s < 0.9f_y$

5.3.4 Non-tensioned reinforcement

Non-tensioned reinforcement shall be provided in prestressed elements in:

(a) End anchorage zones; and

(b) Between end anchorages, where prestress is calculated to be inadequate to sustain applied forces.

Table 7 – Limiting stresses in prestressed concrete

Type of stress	Load combination stress, MPa (Compressive stress positive)		
	Transfer	Group A (long-term loads)	Group B (short-term loads)
(a) Maximum compression	$0.50\ f_{cp}$	$0.40\ f'_c$	$0.55\ f'_c$
(b) Minimum stress at tendon location	—	0.70	$-0.5\sqrt{f'_c}$
(c) Minimum stress at construction joints	—	0.70	0 (1)
(d) Minimum extreme fibre stress in monolithic concrete			
(i) at water-retaining face	$-0.17\sqrt{f_{cp}}$	-0.70	$-0.5\sqrt{f'_c}$
(ii) at non-water-retaining face(2)	$-0.17\sqrt{f_{cp}}$	$-0.25\sqrt{f'_c}$	$-0.5\sqrt{f'_c}$
(e) Minimum principal stress resulting from shear	$-0.17\sqrt{f_{cp}}$	$-0.30\sqrt{f'_c}$	$-0.5\sqrt{f'_c}$

NOTE –

(1) Cracking is permitted in joints under Group B combinations when non-tensioned reinforcement is provided to carry the entire tension force across the joint. The force shall be calculated on the basis of an uncracked section and reinforcement stresses shall comply with the limits specified in 5.2. Alternatively, non-tensioned reinforcement may be omitted if the construction joint is protected with a suitable joint sealant.

(2) For members less than 225 mm thick, the face remote from the liquid shall be considered as though it was in contact with the liquid.

C5.3.4

Table 7 requires residual compression under long duration loads, but allows significant tension stresses under strain-induced load combinations or seismic loading. No distinction is made between inside and outside surfaces, as moisture levels in the concrete will be similar, except for very thick walls. 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.

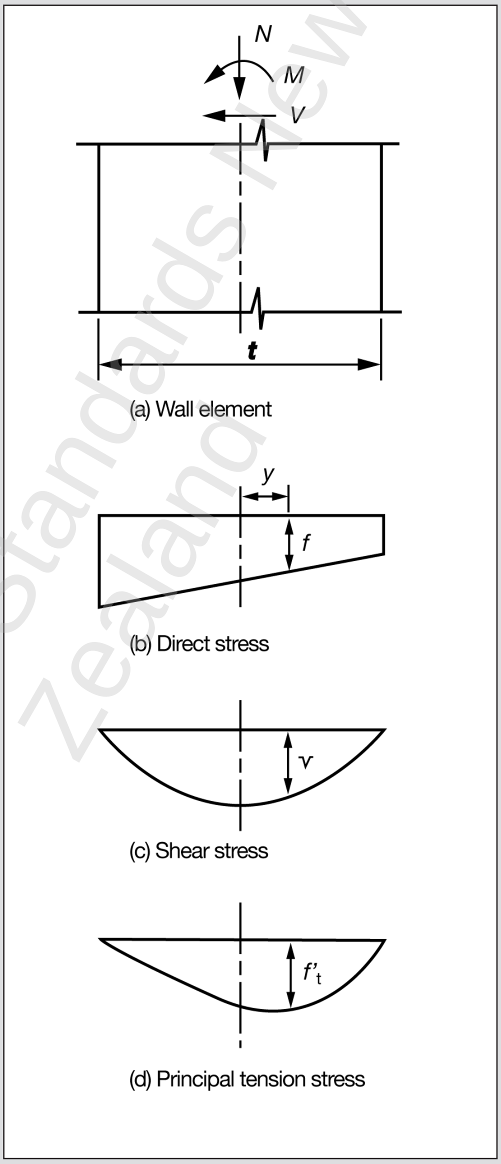


Figure C5.1 – Principal tension stress due to shear

Figure C5.1 illustrates the procedure for a typical tank wall subjected to axial compression force P, moment M, 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, (figure C5.1(b)) given by:

$$f = \frac{N}{t} + \frac{12My}{t^3}$$

The distribution of shear stress will be parabolic, (figure C5.1(c)) and may be expressed as:

$$v = \frac{1.5V}{t}\left[1 - \left(\frac{2y}{t}\right)^2\right]$$

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

$$f_t = \frac{f}{2} - \sqrt{\frac{f^2}{4} + v^2}$$

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 C5.1(d) For most cases it will be sufficient to check f_t at the wall centre line (y = 0). Thus

$$f_{t0} = \frac{N}{t} - \sqrt{\left(\frac{N}{t}\right)^2 + \left(\frac{1.5V}{t}\right)^2}$$

In the equations above, the sign convention used is compression positive.

5.3.5 Partial prestressing

A partially prestressed design approach, permitting cracking of concrete, may be used provided the tensile stress in the non-tensioned reinforcement, and the increase in tendon stress once decompression occurs ($\Delta_{\sigma s}$), taking full account of shrinkage and creep effects, satisfy the following requirements:

(a) the tensile stress in the reinforcement is less than the limiting stress given in table 5 and table 6;

(b) $\Delta_{\sigma s} \leq$ 100 MPa, for Group A load combinations; or

(c) $\Delta_{\sigma s} \leq$ 125 MPa, for Group B load combinations.

C5.3.5

This clause permits the use of partial prestressing as an alternative to full prestressing for the design of concrete elements, providing stresses in non-tensioned reinforcement and crack widths satisfy limiting values. Concrete compression and shear stresses must still satisfy table 7.

In adopting a partially prestressed approach however, considerable care is needed in calculating crack widths because of the effects of creep and shrinkage. Prior to cracking, the non-tensioned reinforcement in a partially prestressed section is subject *to an initial compression stress which gradually increases due to creep of the concrete under the prestress force and also due to shrinkage. On the application of a load sufficient to reduce concrete stresses at the level of the reinforcement to zero the strain in the reinforcement is reduced by an amount equal to the initial elastic strain resulting from prestress.*

Thus though the surrounding concrete is at zero stress, the reinforcement is still subject to compression stress, which may be of considerable magnitude. As the load is increased to a level where cracking results, and concrete tension force is transferred to the reinforcement, the final reinforcement tension stress is effectively dictated by requirements of equilibrium of forces. The result is that the stress change in the reinforcement associated with cracking (from a compression stress at zero concrete tension, to a tension stress after crack initiation) is larger than if the effect of creep on the initial stress distribution has been ignored. Consequently the crack width will be proportionally larger. It is this change in reinforcement stress which must not exceed the allowable stress levels given in tables 5 and 6. The residual compression stress in the reinforcement at zero concrete stress may be calculated from the expression:

$$f_{sr} = E_s C_t \cdot \frac{f_c}{E_c} \qquad \text{.......... (Eq.C5.1)}$$

Where fc is the average compression stress in the concrete immediately adjacent to the reinforcement, prior to decompression, and C_t is the appropriate creep factor. The influence of shrinkage is not included in equation C5.1, since the normal operating condition for the tank will be with tank full, and thus swelling will compensate for previous shrinkage.

It should be noted that cracking of a partially prestressed tank will cause reduction of stiffness and hence a reduction of strain-induction forces, such as those resulting from thermal load. However the reduction factor given in figure 3 applies only to non-prestressed elements, and rational analysis, taking tension stiffening effects into account, must be adopted for partially prestressed elements. At the time of drafting this Standard, specific information on the tension stiffening characteristics of partially prestressed walls was not available, but it is felt that tension stiffening would be considerable. In this absence of such specific information the reduction factor R_F should be taken as 1.0.

5.3.6 Wound tanks

Cylindrical tanks constructed using wound pre-stressing tendons shall be constructed using equipment capable of stressing and measuring the applied tendon force to within 7.5% of the nominal force required.

Systems not consistently meeting this requirement may be used if a possible variation of initial prestress force of 20% is treated as an additional load case in the design.

Protection of the tendons shall include the proper application and curing of a shotcrete cover of sufficient thickness and density to meet durability requirements.

C5.3.6

Tanks constructed with wound tendons which are stressed by drawing the tendon through a die are subject to variable prestress due to variations in wire diameter as supplied and to wear of the die. Such a method will not consistently meet the requirement for uniform prestress.

The protection of the prestressing wires is crucial to the satisfactory long term performance of the tank. Poorly placed shotcrete coatings can have voids along the wires due to a shadow effect. Such voids can collect and transmit water leading to corrosion and early failure of the tank.

Good construction practice for the application of shotcrete is described in ACI 344R-70.

6 DESIGN FOR STRENGTH AND STABILITY AT THE ULTIMATE LIMIT STATE

6.1 GENERAL

Although design for serviceability will generally dominate, design for strength shall be considered to ensure that the load capacity for members of the structure are within acceptable limits.

The structure and its component members shall be designed for the ultimate limit state by providing stiffness, strength and ductility and ensuring stability, as appropriate, in accordance with the relevant principles and procedures for design as set out in AS/NZS 1170.0 and NZS 3101.

6.2 STABILITY DESIGN

In addition to the mass of the empty structure, the design resistance against uplift and/or buoyancy may take account of:

(a) Anchoring systems;

(b) Drainage systems;

(c) Pressure relief valves; or

(d) Any combination of items (a), (b) and (c).

The minimum safety factor against uplift and/or buoyancy shall be determined in accordance with AS/NZS 1170.1. The designer shall assess the effectiveness of the devices in items (a) to (d).

6.3 SHEAR TRANSFER

Earthquake shear forces shall be considered in the design of wall to footing, footing to floor slab and wall to roof joints.

C6.3

The horizontal earthquake force V_H generates shear forces between the wall and footing and the wall and roof. In rectangular tanks, the earthquake shear is transmitted directly by reaction to vertical bending. In circular tanks, the earthquake shear is transmitted partly by membrane shear and the rest by reaction to vertical bending. For a tank with a height to diameter ratio of 1:4 approximately 20% of the earthquake shear force is transmitted by the radial base reaction to vertical bending. The remaining 80% is resisted by membrane shear transfer Q_V:

$$Q_V = 0.8V_H$$

This ratio may vary depending on the aspect ratio of the tank and the tank wall thicness. Specific calculation may be required. Maximum $Q_V = V_H$.

To transmit this shear Q_V, a shear flow q_V is required at the wall/footing interface

$$q_V = \frac{Q_V \sin\delta}{\pi a} \quad \text{(Eq.C6.1)}$$

This distribution is illustrated in figure C6.1.

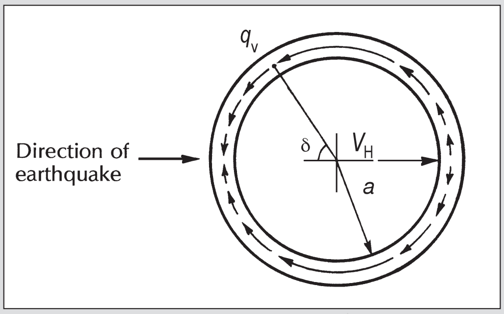


Figure C6.1 – Membrane shear distribution

The maximum shear occurs at 90 degrees to the earthquake direction and is given by:

$$q_{v\,max} = \frac{Q_v}{\pi a} = \frac{0.8V_H}{\pi a} \qquad \text{(Eq.C6.2)}$$

In general the wall/footing interface has sufficient reinforcement through the joint to transmit this shear. However, for precast tank construction the wall panels may be located in a preformed slot in the ring beam footing. Friction between the wall base and footing will generally be insufficient to resist the earthquake shear, thereby requiring some form of mechanical restraint such as galvanised steel dowels.

Failure to provide a means for shear transfer around the circumference will cause circumferential sliding of the wall. The shear resistance is transferred to the principal diagonal, inducing high membrane stresses at the wall junction, balanced by high radial reactions as shown in figure C6.2.

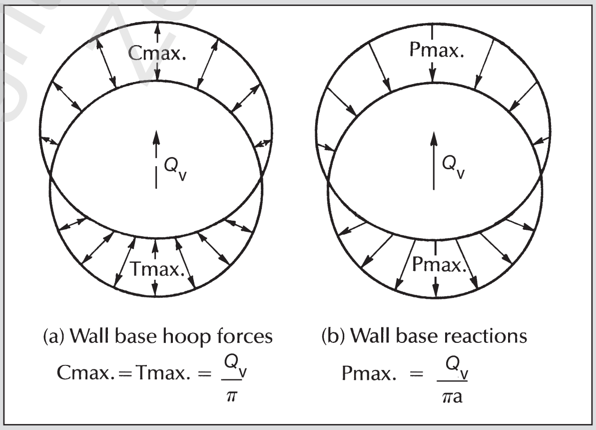


Figure C6.2 – Forces induced by base shear transfer for a wall free to slide circumferentially, but restrained radially

The roof to wall joint is subject to earthquake shear from the horizontal acceleration of the roof. Where dowels are provided to transfer this shear, the distribution will be the same as shown in Figure C6.1 with maximum shear given by:

$$q_{v\,max} = \frac{0.8F_R}{\pi a} \qquad \text{(Eq.C6.3)}$$

where F_R is the force from the horizontal acceleration of the roof.

For tanks with roof overhangs, the concrete nib can be designed to withstand the earthquake force. Because the roof is free to slide on top of the wall, the shear transfer will be reacted over that portion of the circumference where the nib overhang comes into contact with the wall. Typically, the distribution of forces and wall reactions will be similar to that shown in figure C6.2, but reacting on only half of the circumference. The maximum reaction force will be given by:

$$P_{max} = \frac{2F_R}{\pi a} \quad \text{...(Eq.C6.4)}$$

6.4 MINIMUM REINFORCEMENT

The minimum reinforcement to meet the ultimate limit state requirements shall be as follows:

6.4.1 Hoop reinforcement

The minimum hoop reinforcement ratio at any wall section shall be:

$$\rho_{min} = 0.05f'_c / f_y \quad \text{...(Eq.6.1)}$$

C6.4.1

Sufficient reinforcement is required providing a ductile hoop response by ensuring that the cracked strength is greater than the uncracked strength so that reinforcement yielding will spread around the tank perimeter rather than be concentrated at a few crack locations.

6.4.2 Flexural reinforcement

The minimum tension reinforcement ratio in potential plastic hinge zones of walls and floor slabs shall be:

$$\rho_{pmin} = \sqrt{f'_c}/4f_y \quad \text{...(Eq.6.2)}$$

C6.4.2

Sufficient tension reinforcement is required in the potential plastic hinge regions of walls and floors to ensure that the cracked strength is greater than the uncracked strength so that reinforcement yielding will be distributed rather than be concentrated at a single crack location, with the potential for large strains and fracture of the steel.

7 DESIGN FOR DURABILITY

Durability shall be allowed for in design by determining the exposure classification and, for that exposure classification, complying with the appropriate requirements for:

(a) Concrete quality and curing;

(b) Cementitious binder composition;

(c) Cover;

(d) Chemical content restrictions;

(e) Alkali silica reaction precautions;

(f) Protection of fixings;

all in accordance with NZS 3101 for a 50-year or 100-year design life.

Exposure classification shall be determined from table 8 and NZS 3101.

C7

For liquids having a detrimental effect on concrete, appropriate special precautions may include the provision of linings impervious to the liquid to be contained. Particular attention should be paid to any joints in the lining which must remain impervious for the life of the structure, for much damage can occur before a leak is detected.

The air space below roof soffits is often at 100% humidity and the slab is exposed to severely corrosive conditions. Thus the roof slab should be designed with the same crack control philosophy as for the rest of the structure.

Table 8 – Exposure classifications

Item	Characteristic of liquid in contact with concrete surface	Exposure classification (1)		
		Predominantly submerged		Alternate wet and dry (condensation splashing or washing)
		Generally quiescent	Agitated or flowing	
1	Freshwater: (2, 3, 4) (a) L_I positive or pH >7.5 (b) L_I negative & pH 5.5 to 7.5	B1 B2	B2 U	B1 B2
2	Sewage and waste water: (5) (a) Fresh and no exposure to H_2S (b) Stale or exposure to H_2S (c) Anaerobic sludge	B2 C C	B2 U U	B2 U U
3	Sea water	B2(6)	C	C
4	Other corrosive liquids, vapours or gases (7) Severity: (a) Slight/mild (b) Moderate (8) (c) Severe/extreme (8)	B1 B2 U	B2 C U	B2 C U
5	Other liquids: (9) (a) Water containing chloride, sulfate, magnesium or ammonium (b) Wine, non-corrosive vegetable oils, mineral oils and coal tar products	B1-U B1-U	B1-U B1-U	B1-U B1-U
6	Ground water (inground) (10)	B1-XA3	–	–

NOTE –

(1) For exposure classification U the use of sacrificial surface layers or appropriate protective coatings should be considered.

(2) An approximate value of Langelier Saturation Index (L_I) may be obtained from the equation:
L_I = pH of water – pH when in equilibrium with calcium carbonate
= pH –12.0 + log10[2.5 × Ca^{2+} (mg/L) × total alkalinity (as $CaCO_3$ mg/L)].
(A negative value for L_I means the water has a demand for $CaCO_3$).

(3) For lower pH values see Item 4 in table 8.

(4) For water containing significant quantities of aggressive dissolved materials see Item 5 in table 8.

(5) Industrial sewage and waste water may contain aggressive chemicals. The designer shall refer to other liquids as given in Item 5 of table 8.

(6) Only applicable for permanent submergence, that is, no more than two years total of dry periods exceeding two weeks in the life of the tank.

(7) Typical examples of severities are given in AS 3735 Supp1.

(8) The use of calcareous aggregate should be considered. Details are specified in AS 3735 Supp1.

(9) Guidance on the selection of an appropriate exposure classification from within the range indicated is specified in AS 3735 Supp1.

(10) Guidance on the selection of an appropriate exposure classification from within the range indicated is specified in NZS 3101.

APPENDIX A – EARTHQUAKE ACTIONS

(Normative)

A1 GENERAL

The structure shall be designed for the forces resulting from earthquake accelerations of equivalent hydrodynamic mass, structure mass and external mass responding with the structure.

The equivalent hydrodynamic mass comprises two components:

(1) The impulsive mass representing the portion of the contents accelerating with the tank, and

(2) The convective mass representing the portion of the contents oscillating in the tank.

CA1

The forces on a tank under earthquake motion are a combination of hydrostatic water pressure, tank wall and roof inertia forces, and hydrodynamic forces resulting from the interaction of the tank structure with the liquid contents.

The hydrodynamic effects can most conveniently be included by adding to the structure mass, 'equivalent masses' to represent the effects of the contents on the dynamic response of the structure. This equivalent mass approach has been adopted in this Standard. Reference should be made to the New Zealand Society for Earthquake Engineering (NZSEE), Seismic design of storage tanks *for more detail on the seismic response of tanks and the derivation of the procedures specified in this Standard.*

A2 EARTHQUAKE ACTIONS

A2.1 Base shear

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

$$V_{Hi} = C_d(T_i)W_i$$

where

$$C_d(T_i) = C(T_i)k_f(\mu,\xi_i)\mathrm{Sp}$$

$$C(T_i) = C_h(T_i)\,ZRN(T_i,D)$$

and

V_{Hi} = horizontal base shear associated with mode i (impulsive, convective, and so on).

W_i = equivalent weight of tank and contents responding in particular mode of vibration considered. (NOTE – weight is used rather than mass for consistency with NZS 1170.5.)

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

$C(T_i)$ = elastic site hazard spectrum for horizontal loading for the site subsoil type, and the relevant mode.

$k_f(\mu,\xi_i)$ = correction factor for NZS 1170.5 elastic site hazard spectra to account for ductility and level of damping, see table A2.

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

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

Z = seismic zone hazard factor, from NZS 1170.5.

R = return period factor for the relevant limit state and tank importance level and design life, from NZS 1170.5.

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

μ = displacement ductility factor for impulsive modes, see table A1.

T_i = period of vibration of appropriate mode of response, see A2.2.

ξ_i = damping ratio appropriate to mode of response, see figures A1 and A2 for impulsive mode. For convective mode, shall not be taken greater than 0.5%.

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

Table A1 – Displacement ductility factor, μ

Response mode	Ductility factor μ
Reinforced or prestressed concrete tanks on grade or embedded	
Horizontal impulsive modes, ULS loads	1.25
Horizontal impulsive modes, SLS loads	1.0
Vertical impulsive mode, ULS loads	1.25
Vertical impulsive mode, SLS loads	1.0
Convective mode, all	1.0
Elevated tanks (1)	As appropriate for support structure

NOTE –
(1) A capacity design approach shall be used to protect these tanks against failure while yielding occurs in the chosen support system.

CA2.1

Adoption of a 1.25 ductility factor recognises that some response modes (for example, hoop tension and wall flexure) are potentially ductile. However higher ductility factors have not been recommended because:

(1) As the magnitude of the concurrent hydrostatic pressures will be significant compared to the hydrodynamic pressures, the inelastic deformation is likely to occur only when the earthquake response is in the same direction as the hydrostatic pressure. Consequently inelastic displacements will be uni-directional and the actual displacement ductility demand may be significantly greater than the design ductility. Also the inelastic displacement (and crack widths) cannot be calculated from the elastic displacement by normal methods.

(2) A capacity design approach would be required to ensure that an undesirable response mode (for example, wall rocking) does not occur before the desired, ductile response mode (for example, hoop yielding) occurs, and that the out of plane shear and flexure does not result in premature failure of the tank walls.

For guidance on the selection of the appropriate return period factor also refer to NZSEE (2009).

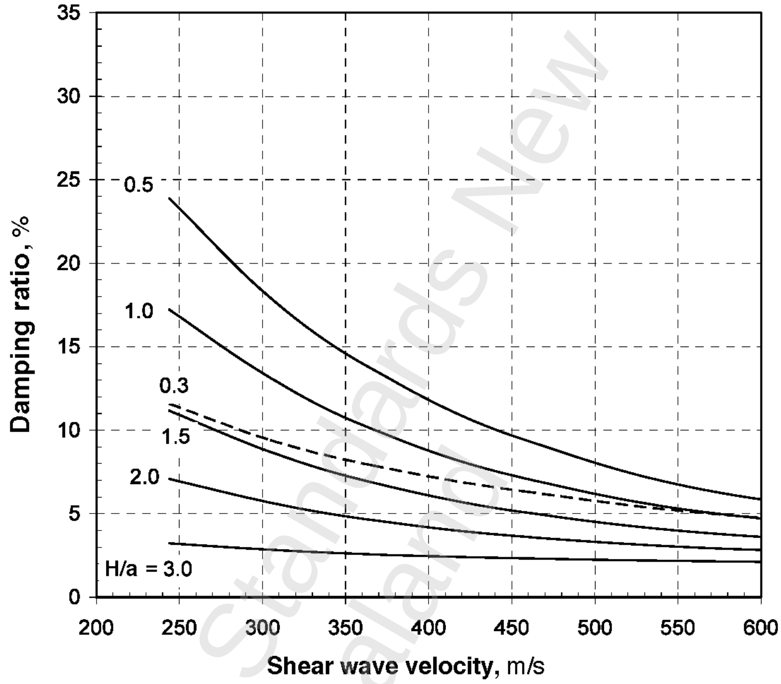


Figure A1 – Damping for horizontal impulsive mode

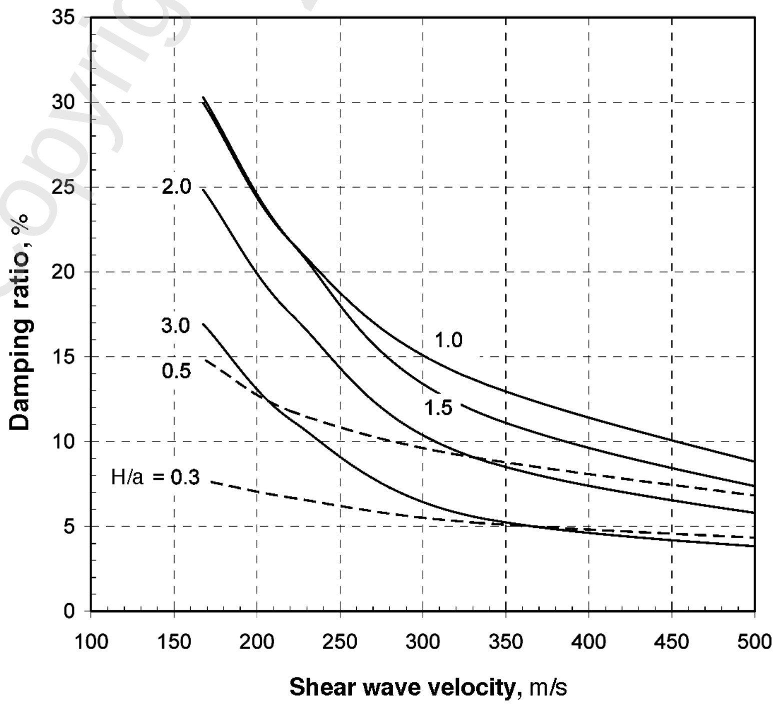


Figure A2 – Damping for vertical impulsive mode

***CA2.1** (continued)*

Figures A1 and A2 are for circular concrete tanks and were derived from Veletsos and Shivakumar (1997) and Veletsos and Tang (1986) respectively. Damping ratio for rectangular concrete tanks can be derived from these figures by using an equivalent radius (a) for a circular tank with the same plan area as the rectangular tank (refer to Gazetas and Tassoulas (1987)).

The correction factor applied to NZS 1170.5 elastic site hazard spectrum for varying damping and ductility is provided in Table A2.

Table A2 – Correction factor k_f (μ,ξ_i)

Ductility μ	k_f (μ, ξ_i)							
	$\xi^{(1)}$ = 0.5%	ξ = 1%	ξ = 2%	ξ = 5%	ξ = 10%	ξ = 15%	ξ = 20%	ξ = 30%
1.0	1.67	1.53	1.32	1.00	0.76	0.64	0.56	0.47
1.25	1.08	1.04	0.96	0.82	0.67	0.58	0.52	0.44
2.0	0.91	0.89	0.84	0.75	0.63	0.55	0.50	0.43

NOTE –
(1) ξ = damping ratio of elastic system.

A2.2 Periods of vibration

A2.2.1 Impulsive modes

Unless calculated on the basis of a more rigorous analysis, the period of the fundamental impulsive modes shall be as follows:

(a) Ground supported circular or rectangular tanks horizontal mode, assume T_I = 0.1 sec;

(b) Ground supported circular or rectangular tanks vertical mode, assume T_I = 0.1 sec;

(c) Fully ground-embedded circular or rectangular tank: T_I =0 sec, use $C_h(T_I)=C_h(0)$ (that is, peak ground acceleration);

(d) Elevated tanks: The natural period of the tank and supporting structure shall be calculated in accordance with NZS 1170.5. The flexibility of the tank walls may be neglected.

CA2.2.1

The first horizontal impulsive mode periods of 10 m diameter x 6 m high and 30 m diameter x 6 m high circular concrete tanks on rigid foundations are approximately 0.04 sec and 0.1 sec respectively. Soil-structure interaction on less rigid soils, concrete cracking and non-linear response would increase these periods, but generally periods will be less than 0.3 sec. This puts the response within or near to the peak spectral response. Consequently the peak response spectrum ordinates are required to be used, unless it can be shown that other ordinates are appropriate.

A2.2.2 Convective modes

The period of the convective modes shall be determined from figure A3.

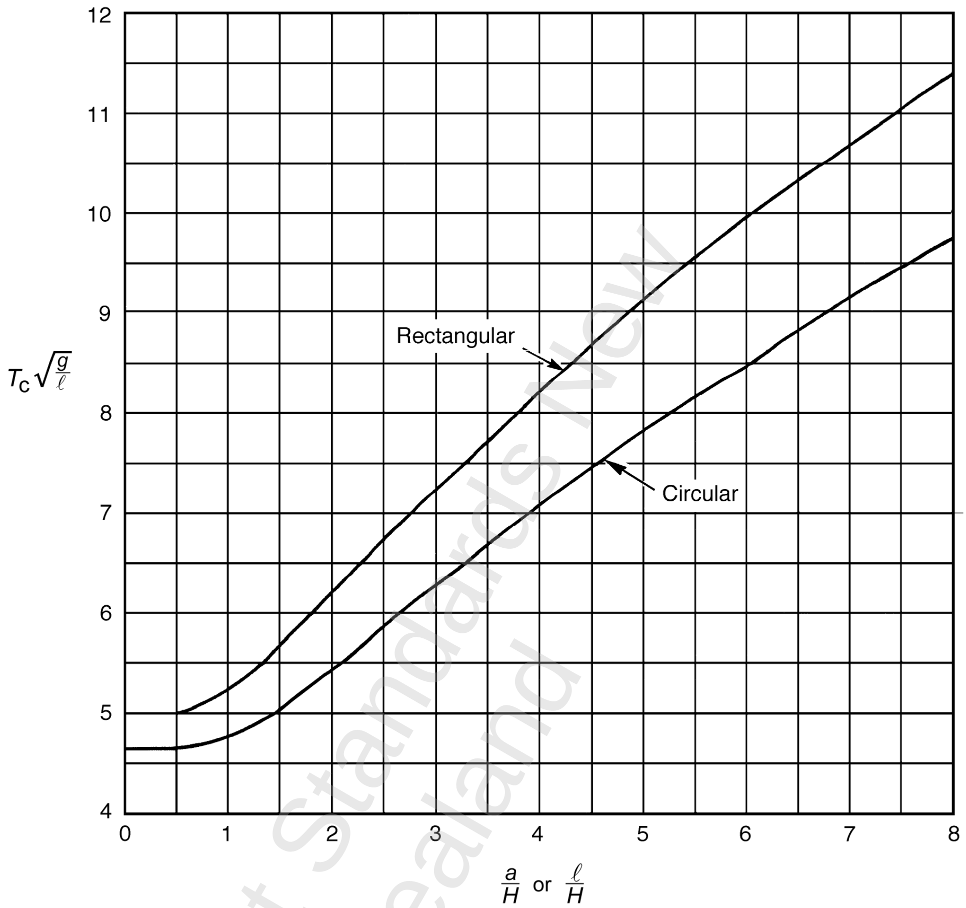


Figure A3 – Dimensionless period for fundamental sloshing model

A2.3 Equivalent hydrodynamic weights

The total equivalent impulsive weight, W_I and the total equivalent convective weight W_C, and their respective heights to centres of gravity h_I and h_C shall be calculated from figure A4.

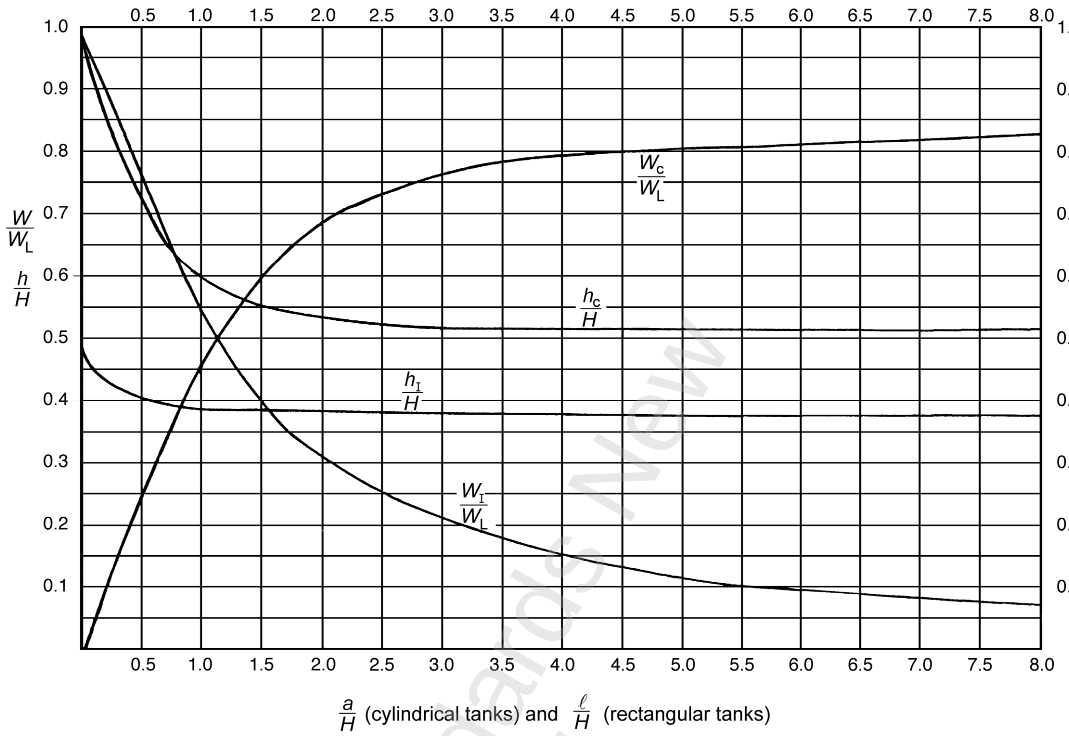


Figure A4 – Equivalent weights of impulsive and convective contents (circular and rectangular tanks)

A2.4 Vertical distribution of equivalent weights

Analysis of the earthquake response of the tank shall be based on distributions of the impulsive and convective equivalent weight that vary linearly from w_{ti} at the surface of the liquid to w_{bi} at the base,

where

$$w_{ti} = \frac{W_i(6h_i - 2H)}{2H^2}$$

and

$$w_{bi} = \frac{W_i(4H - 6h_i)}{2H^2}$$

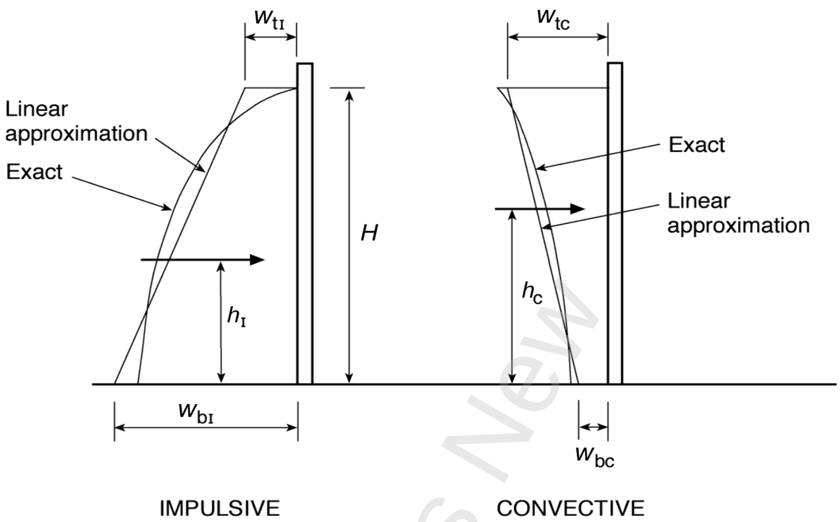


Figure A5 – Equivalent weight distribution

> ***CA2.4***
>
> *Vertical equivalent weight distributions are of the form shown by the 'exact' curves in figure A5. The equivalent linear distributions, although a simplification of the actual distribution, are sufficiently accurate for design purposes, and form the basis of the equations for w_{ti} and w_{bi}. Note that 50% of the total weight W_i is assigned to each side of the tank.*

A2.5 Convective wave height

The maximum vertical displacement d_{max} of the convective sloshing wave from the at-rest level of the liquid shall be given by:

$$d_{max} = 0.84aC_d(T_C) \text{ for circular tanks}$$

and $$d_{max} = \ell\, C_d(T_C) \text{ for rectangular tanks}$$

where a = radius of tank

ℓ = one half of length of rectangular tank in direction being considered

$C_d(T_C)$ = the convective mode C_d as defined in A2 and based on:

μ = 1.0

ξ = 0.5%

A2.6 Embedded tanks

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

CA2.6

Consideration should be given to the horizontal acceleration of surrounding backfill. The effects of this can be significant, having been the apparent cause of a number of tank failures. For example, a large underground reinforced concrete tank (part of the Balboa water treatment plant) suffered severe damage in the 1977 Fernando earthquake, the damage apparently caused by movement of the surrounding ground.

The horizontal earthquake pressures on a smooth, perfectly rigid wall from horizontal inertia forces on the soil can be approximated by the distribution shown figure CA.1:

Where $f_{\Delta e}$ = *increment of pressure due to earthquake*

$C(0)$ = *peak ground acceleration*

γ_e = *unit weight of soil*

H_e = *soil depth*

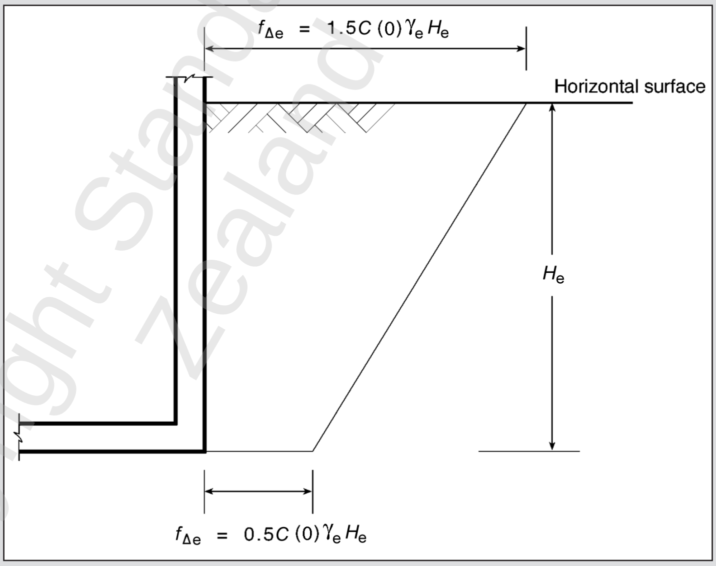


Figure CA.1 – Approximation of the earthquake increment of earth pressure on a rigid wall

This pressure distribution will be conservative for most embedded tanks with horizontal backfill.

The pressures shown do not include the pressures arising from gravity forces in the soil, i.e. they are the increment of earth pressures due to earthquake.

A3 STRUCTURAL ANALYSIS

A3.1 Methods of analysis

A structural analysis to determine the action effects shall be carried out in accordance with one of the methods specified in NZS 1170.5.

CA3.1

Normally the equivalent static method would be used, but the modal response spectrum and the numerical integration time history methods could also be used.

The formulation of the equivalent impulsive weights in Appendix A is based on a rigid wall tank. However analyses have shown (for example, Veletsos and Shivakumar (1997)) that the equivalent mass representing the interaction of a rigid structure can be used to account for the hydrodynamic effects for all modes of non-rigid structures to an acceptable degree of accuracy.

A3.2 Combined actions

The combined impulsive and convective action effects shall be taken as the square root of the sum of the squares of the separate effects.

CA3.2

The periods of the inertia and convective responses are generally widely separated, the impulsive period being much shorter than the convective period. When responses are widely separated, near-simultaneous occurrence of peak values could occur. However the convective response takes much longer to build up than the impulsive response, consequently the impulsive component is likely to be subsiding by the time the convective component reaches its peak. It is thus recommended that the combined impulsive and convective responses be taken as the square root of the sum of the squares of the separate components.

A3.3 Equivalent static method: horizontal actions

A3.3.1 Horizontal seismic shear

The total horizontal seismic shear V_H acting at the base of the tank is:

$$V_H = \sqrt{\left[C_d(T_I) \times \left(W_I + W_W + W_R\right)\right]^2 + \left[C_d(T_c) \times W_c\right]^2} \quad \text{(Eq.A1)}$$

where $C_d(T_I)$ and $C_d(T_C)$ are the impulsive and convective seismic coefficients respectively as defined in section A2.

W_I and W_C are the total equivalent impulsive and convective weights respectively as defined in section A2.

W_W and W_R are the tank wall and roof weights respectively.

A3.3.2 Equivalent static horizontal force at each level

For all circular tanks and for rectangular tanks with walls supported either top and bottom or supported on three sides and with length to height ratio of less than 3.0, the equivalent static force F_i at each level shall be obtained as the product of the seismic coefficient $C_d(T_i)$ and the weight w_i at that level for the particular response mode being considered.

For rectangular tanks where the hydrostatic and hydrodynamic pressures on the walls are resisted largely by face-loaded cantilever action of the walls, the impulsive equivalent static force at each level shall be obtained from equation 6.2(2) of NZS 1170.5. Convective forces shall be obtained as the product of the seismic coefficient $C_d(T_C)$ and the weight w_c at that level.

CA3.3.2

As illustrated in figure CA2, the displacement response of circular concrete tanks, which tend to be relatively broad (typically H/a < 1.5), is dominated by the hoop response of the tank walls while the cantilever response of the tank barrel is negligible. Also for broad tanks, soil structure interaction is dominated by horizontal rather than rocking displacements. The accelerations can therefore be assumed constant up the height of the tank. (To keep the figure simple hydrodynamic loads on the floor and the convective weight have not been included.)

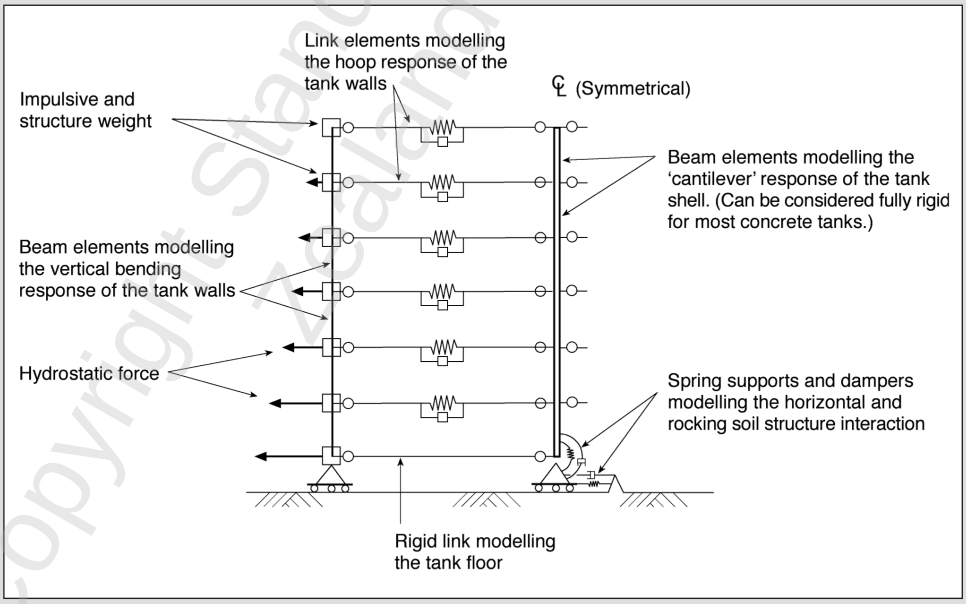


Figure CA2 – Simplified 2-D model of a circular tank under impulsive loads

Similarly for rectangular tanks with small length to height ratios the displacement response is dominated by plate-action bending of the walls, and accelerations can be considered to be uniform for design purposes. However for rectangular tank walls with larger length to height ratios and not roof supported, the hydrodynamic forces will be resisted by wall cantilever action for which the accelerations will increase with height.

A3.3.3 Horizontal distribution of hydrodynamic forces

In the absence of a more rigorous analysis which takes into account the exact and complex horizontal variations in hydrodynamic pressures, analysis of the earthquake response of the tank shall be based on circumferential/horizontal distributions of the impulsive and convective forces f_i per unit length of wall as shown in figure A6,

where

f_i = $2F_i/\pi a$ for circular tanks

f_i = $F_i/2b$ for rectangular tanks

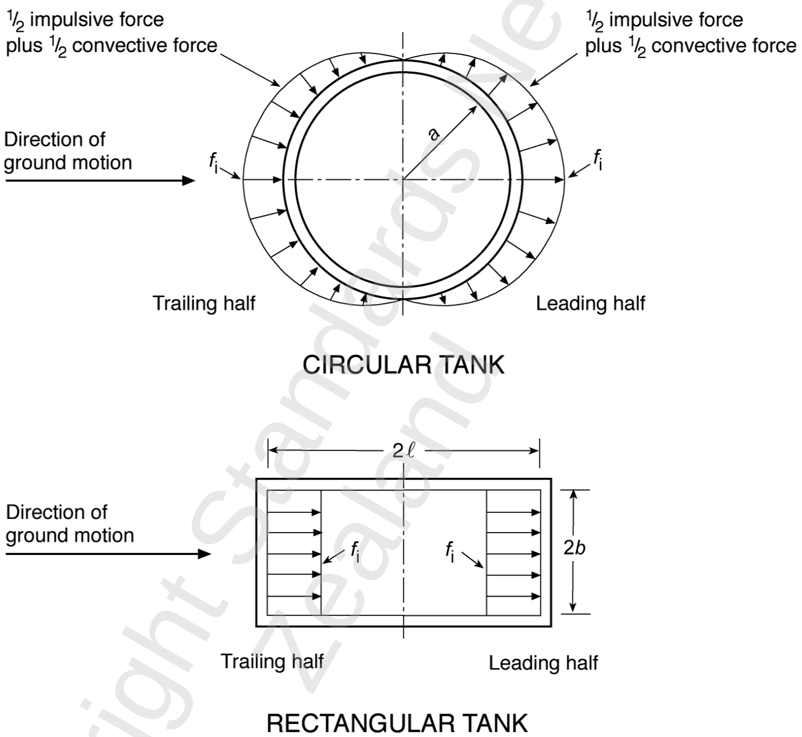


Figure A6 – Horizontal force distribution

> ***CA3.3.3***
>
> *Circumferential force distribution can be represented by a sinusoidal variation for a circular tank where f_i is the peak force per unit length.*
>
> *The horizontal force distribution for a rectangular tank is uniform.*

A3.3.4 Seismic actions on walls

The effect of the individual actions on the tank walls (hoop forces, membrane forces, bending moments and shear forces) shall be computed from structural analysis models or from Standard design charts.

The individual effects shall be combined in accordance with A3.2.

For thin walled circular tanks ($t/a < 0.03$), the non-rotationally symmetric hydrodynamic loads may be considered to be rotationally symmetric and equal to the value at the section under consideration.

CA3.3.4

Hydrodynamic forces on circular tanks are not rotationally symmetric but vary continually around the tank's perimeter (see figure A6). Analyses have shown that this variation is generally small enough for stresses at any given section to depend only on the local force distribution. This significantly simplifies the analysis of the effect of these non-symmetric forces.

A3.3.5 Overturning moment above floor slab

The total overturning moment acting at the level of the base of the wall immediately above the floor slab shall be determined by:

$$M_W = \sqrt{\left[C_d(T_I) \times \left(W_I h_I + W_w h_w + W_R h_R\right)\right]^2 + \left[C_d(T_C) \times W_C h_C\right]^2} \quad \text{.................(Eq.A2)}$$

where h_C = height to the centre of gravity of the convective weight from figure 4

h_I = height to the centre of gravity of the impulsive weight from figure 4

h_W = height to centre of gravity of the wall

h_R = height to centre of gravity of roof

CA3.3.5

This moment, with ordinary beam theory, can be used to calculate the axial force at the base of the wall and hence on the wall footing.

A3.3.6 Overturning moment on floor slab

The total overturning moment acting on the floor slab from the hydrodynamic loads on the floor and the walls shall be determined by:

$$M'_w = \sqrt{\left[C_d(T_I) \times \left(W_I h'_I + W_w h_w + W_R h_R\right)\right]^2 + \left[C_d(T_C) \times W_C h'_C\right]^2} \quad \text{......... (Eq.A3)}$$

where h'_I = the equivalent height from figure A7 at which the impulsive weight is placed to give the total overturning moment arising from pressures on both the walls and base.

h'_C = the equivalent height from figure A7 at which the convective weight is placed to give the total overturning moment arising from pressures on both the walls and base.

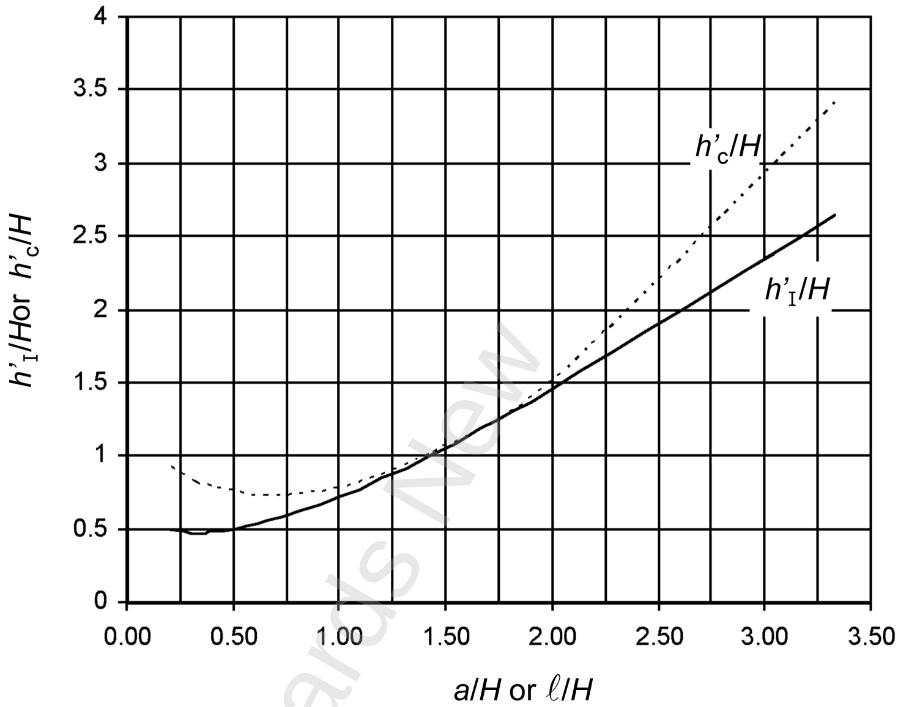


Figure A7 – Equivalent heights of impulsive and convective weights for overturning (circular and rectangular tanks)

CA3.3.6

Moment M'_w can be used to calculate the bearing pressures on the ground beneath the floor slab.

A3.4 Vertical seismic actions

Vertical seismic actions shall be considered to act concurrently with horizontal seismic actions. The combined horizontal and vertical action effects shall be taken as the square root of the sum of the squares of the separate effects.

CA3.4.

The effect of vertical ground motion is to alter the internal pressure exerted by the contained liquid; an upward acceleration of the tank will cause an increase in pressure. The incremental stresses caused by a vertical acceleration are identical in distribution to those produced by the static liquid load while their magnitudes are some proportion thereof. For example, an upward earthquake acceleration of 0.25 g produces incremental stresses whose magnitude is 25% of the static liquid containment stresses.

The peak horizontal and vertical seismic effects are combined by their root mean square to account for the reduced probability of their concurrence.

A3.5 Water pressure on roof

If the freeboard is less than the height of the convective waves, then hydrodynamic pressures will be generated on the roof.

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

$$p_r = p_b + c_s u^2 \gamma_\ell /2g$$

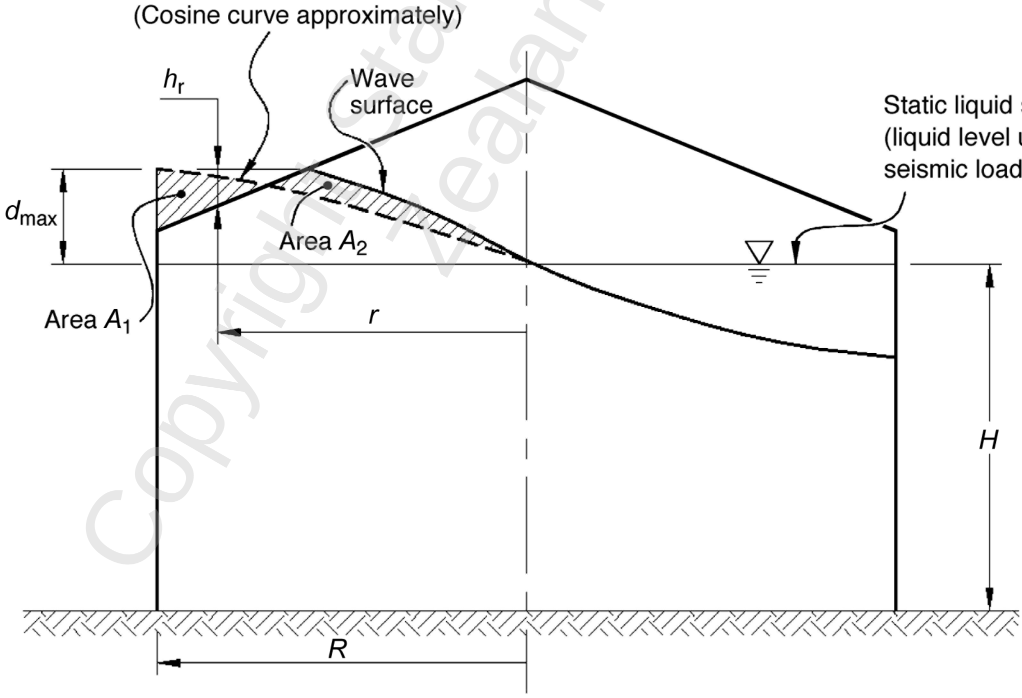
where p_b = buoyancy pressure

c_s = wave impact coefficient

u = wave particle velocity

The roof shall be designed to withstand water pressures due to the convective sloshing waves, and an approximate graphical method of evaluating p_b is shown in Figure A8. An upper bound to c_s is about 5.0 and the approximate maximum value of u can be obtained from:

$$u = 2\pi\, d_{max}/T_C$$



Run up height from:

$A_1 = A_2$

Pressure on roof at radius r:

pb = $\ell_1 h_r$

Figure A8 – Approximate method for estimating buoyancy pressure on roof

APPENDIX B – DESIGN OF REINFORCEMENT TO CONTROL CRACKING

(Normative)

NOTE – Appendix B has been adopted from BS EN 1992-1-1. All notation in Appendix B is separate from the 2.3 notation list.

B1 CALCULATION OF CRACK WIDTHS

B1.1

The crack width, w_k may be calculated from equation (B1):

$$w_k = S_{r,max}\,(\varepsilon_{sm} - \varepsilon_{cm}) \quad \text{.......... (Eq.B1)}$$

Where

$S_{r,max}$ is the maximum crack spacing

ε_{sm} is the mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformation and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered

ε_{cm} is the mean strain in the concrete between cracks

B1.2

$\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from the expression:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}(1+\alpha_e \rho_{p.eff})}{\mathrm{E_s}} \geq 0.6\frac{\sigma_s}{E_s} \quad \text{.......... (Eq.B2)}$$

where

σ_s is the stress in the tension reinforcement assuming a cracked section. For pretensioned members, σ_s may be replaced by $\Delta\,\sigma_p$ the stress variation in prestressing tendons from the state of zero strain of the concrete at the same level

α_e is the ratio E_s / E_{cm}

$f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur:

$f_{ct,eff} = f_{ctm}$ (or lower if cracking is expected earlier than 28 days) refer to table B2

$\rho_{p,eff}$ $(A_s + \xi_1^2 A_p') / A_{c,eff}$ .... (Eq.B3)

A_s is the area of reinforcing steel

A_p' is the area of pre or post-tensioned tendons within

$A_{c,eff}$ is the effective area of concrete in tension surrounding the reinforcement or prestressing tendons of depth $h_{c,ef}$, where $h_{c,ef}$ is the lesser of 2.5(h-d), (h-x/3) or h/2 (see figure B1)

ξ_1 is the adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel

$$= \sqrt{\xi \frac{\phi_s}{\phi_p}}$$

if only prestressing steel is used to control cracking, $\xi_1 = \sqrt{\xi}$

ξ ratio of bond strength of prestressing and reinforcing steel, according to table B1

ϕ_s largest bar diameter of reinforcing steel

ϕ_p equivalent diameter of tendon

ϕ_p $= 1.6 \sqrt{A_p}$ for bundles, where A_p is area of prestressing tendon or tendons

ϕ_p $= 1.75\ \phi_{wire}$ single 7 wire strands where ϕ_{wire} is the wire diameter

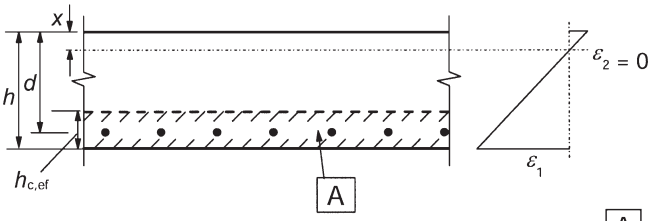
ϕ_p $= 1.20\ \phi_{wire}$ single 3 wire strands where ϕ_{wire} is the wire diameter

k_t is a factor dependent on the duration of the load

k_t = 0.6 for short term loading, and 0.4 for long term loading

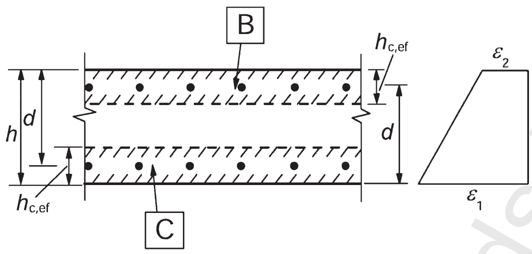
Table B1 – Ratio of bond strength

Prestressing steel	ξ		
	Pre-tensioned	Bonded, post-tensioned	
		≤ C50/60	≥ C70/85
Smooth bars and wires	Not applicable	0.3	0.15
Strands	0.6	0.5	0.25
Indented wires	0.7	0.6	0.3
Ribbed bars	0.8	0.7	0.35
NOTE – For intermediate values between C50/60 and C70/85 interpolation may be used.			



A – effective tension area, $A_{c,eff}$

(a) Bending



B – effective tension area for upper surface, $A_{ct,eff}$

C – effective tension area for lower surface, $A_{cb,eff}$

(b) Tension

Figure B1 – Effective tension area

Table B2 – Strength and deformation characteristics for concrete

f'_c	25	30	35	40	45	50	55	60	70	80
f_{cm}	33	38	43	48	53	58	63	68	78	88
f_{ctm}	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	4.8
E_{cm}	31	33	34	35	36	37	38	39	41	42

f'_c is the characteristic compressive cylinder strength of concrete at 28 days.

f_{cm} is the mean value of concrete cylinder compressive strength

f_{ctm} is the mean value of axial tensile strength

E_{cm} is the secant modulus of elasticity

B1.3

In situations where bonded reinforcement is fixed at reasonably close centres within the tension zone (spacing ≤ $5(c + \phi / 2)$, the maximum final crack spacing may be calculated from equation B4 (see figure B2).

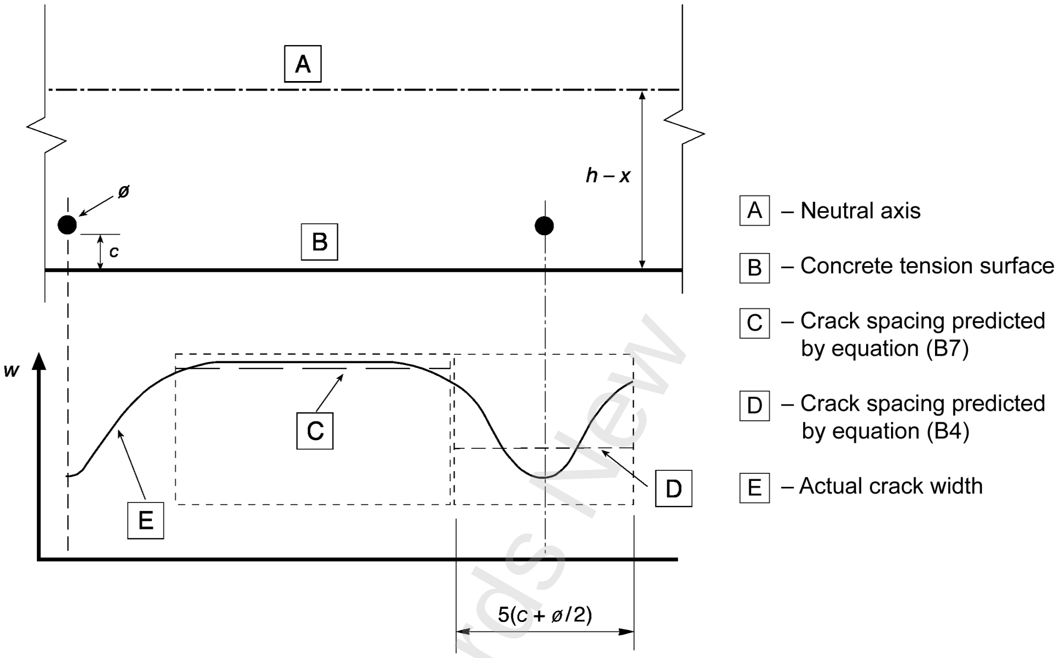


Figure B2 – Crack width, w, at concrete surface relative to distance from bar

$S_{r,max} = k_3c + k_1k_2k_4\phi / \rho_{p,eff}$ .... (Eq.B4)

where

ϕ is the bar diameter. Where a mixture of bar diameters is used in a section, an equivalent diameter, ϕ_{eq}, should be used. For a section with n_1 bars of diameter ϕ_1 and n_2 bars of diameter ϕ_2, the following expression should be used:

$$\phi_{eq} = \frac{n_1\phi_1^{\ 2} + n_2\phi_2^{\ 2}}{n_1\phi_1 + n_2\phi_2}$$ .... (Eq.B5)

c is the cover to the longitudinal reinforcement

k_1 is a coefficient which takes account of the bond properties of the bonded reinforcement:

= 0.8 for high bond bars

= 1.6 for bars with an effectively plain surface (for example, prestressing tendons)

k_2 = is a coefficient which takes account of the distribution of strain:

= 0.5 for bending

= 1.0 for pure tension

For cases of eccentric tension or for local areas, intermediate values of k_2 should be used which may be calculated from the relation:

$$k_2 = (\varepsilon_1 + \varepsilon_2) / 2\,\varepsilon_1 \quad \text{(Eq.B6)}$$

Where

ε_1 is the greater and ε_2 is the lesser tensile strain at the boundaries of the section considered, assessed on the basis of a cracked section

k_3 = 3.4

k_4 = 0.425

Where the spacing of the bonded reinforcement exceeds $\leq 5(c + \phi / 2)$ (see Figure B2) or where there is no bonded reinforcement within the tension zone, an upper bound to the crack width may be found by assuming a maximum crack spacing:

$$S_{r,max} = 1.3\,(h - x) \quad \text{(Eq.B7)}$$

B1.4

Where the angle between the axes of principal stress and the direction of the reinforcement, for members reinforced in two orthogonal directions, is significant (>15°), then the crack spacing may be calculated from the following expression:

$$S_{r,max} = \frac{1}{\dfrac{\cos\theta}{S_{r,max,y}} + \dfrac{\sin\theta}{S_{r,max,z}}} \quad \text{(Eq.B8)}$$

where

θ is the angle between the reinforcement in the y direction and the direction of the principal tensile stress

$S_{r,max,y}$ $S_{r,max,z}$ are the crack spacings calculated in the y and z directions respectively, according to B1.3.

APPENDIX C – TESTING
(Informative)

C1 GENERAL

Inspection and testing for safety, serviceability and durability should be carried out on completion of construction.

NOTE – Inspections should be carried out at regular intervals (maximum five years) during the service life of the structure.

C2 TESTING FOR LIQUID TIGHTNESS

At an appropriate time after completion of construction, the structure or section thereof as considered necessary should be tested for liquid-tightness in accordance with C3 and C4.

C3 TESTING OF LIQUID RETAINING STRUCTURES

For a test of liquid retention, a structure should be cleaned and initially filled with the specified liquid (usually water) at a uniform rate generally not greater than 2 m in 24 h.

Structures should not be backfilled unless specified.

When first filled, the liquid level should be maintained by the addition of further liquid for a stabilizing period of 7 days while absorption and autogenic healing takes place. After the stabilizing period, the level of the liquid surface shall be recorded at 24 h intervals, for a test period of 7 days. During this 7-day test period, the total permissible drop in level, after allowing for evaporation and rainfall (if the test is made for an uncovered structure) shall not exceed 1/500th of the average water depth of the full tank or 10 mm, whichever is less.

If at the end of the 7 days, the structure does not satisfy the condition of the test, and the daily drop in water level is decreasing, the period of test may be extended for a further 7 days and if the specified limit is not exceeded, the test may be considered satisfactory.

Notwithstanding the satisfactory completion of the test, any evidence of seepage of the liquid to the outside faces of the liquid-retaining walls or intensified underdrain flow should be assessed against the liquid tightness requirements of section 5. Any necessary remedial treatment of the concrete to the cracks or joints should, where practicable, be carried out from the liquid face. When a remedial lining is applied to inhibit leakage at a crack it shall have adequate flexibility and have no reaction with the stored liquid.

Where the structure fails to satisfy the 7-day test then, after completion of the remedial work, it should be refilled and a further 7-day test undertaken in accordance with this clause.

C4 TESTING OF ROOFS

Where applicable, the roofs of liquid-retaining structures should be watertight and should, where practicable, be tested on completion by flooding the roof with water to a minimum depth of 25 mm for a period of 24 h or longer if so specified. Where it is not possible, to contain 25 mm depth of water, because of roof falls or otherwise, a hose or sprinkler system should provide a sheet flow of water over the entire area of the roof for a period of not less than 6 hours. In either case, the roof shall be considered satisfactory if no leaks or damp patches show on the soffit. Where the structure fails to satisfy either of these tests, then after the completion of the remedial work it should be retested in accordance with this clause. The roof covering, if any, should be completed as soon as possible after satisfactory testing.

APPENDIX D – THERMAL STRESS COEFFICIENTS

(Normative)

D1 GENERAL

D1.1

This Appendix gives tables of values of vertical and hoop thermal stress coefficients in the wall of a cylindrical tank at different heights in the wall and for different shape factors, where the shape factor is defined by:

$S_{FACT} = H^2 / 2a.t$

D1.2

Tables are given for three thermal conditions as set out in D2, D3 and D4, and for three base conditions:

Table D1: Pinned-base condition

Table D2: Fixed-base condition

Table D3: Sliding-base condition.

D1.3

The stresses given in this Appendix apply to an uncracked wall. Where the wall is designed on the basis of a cracked-section analysis, the forces and moments implied by the stresses may be reduced to reflect the reduced stiffness resulting from cracks, in accordance with C4.2.3.

D2 AVERAGE TEMPERATURE CHANGE

Uniform temperature change of θ_A

Tables assume θ_A is a temperature increase

Reverse sign for temperature decrease.

D3 DIFFERENTIAL TEMPERATURE CHANGE

Differential temperature change of $\pm\theta_D$ (see figure D1).

The total gradient through wall = $2\theta_D$

Tables assume outside hotter than inside. Reverse sign for inside hotter than outside.

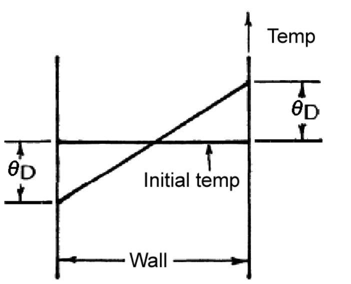


Figure D1 – Differential temperature change

D4 TOTAL TEMPERATURE CHANGE

Temperature variation on outside surface only = θ_T (see figure D2).

Tables assume outside hotter than inside.

Reverse sign for outside colder than inside.

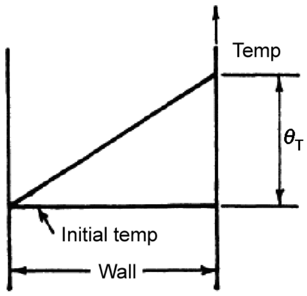


Figure D2 – Total temperature change

D5 STRESSES

Thermal stress is given by:

$f = C.E_c\, \alpha_c\, \theta$

where C = Thermal coefficient from appropriate table

E_c = Modulus of elasticity of the concrete

α_c = Linear coefficient of thermal expansion of the concrete

θ = θ_A, θ_D or θ_T as appropriate.

The sign convention used in the tables is tension positive. Vertical stress is given for the inside surface. Reverse the sign for the outside surface.

Table D1 – Thermal stress coefficients – Pinned-base condition

(a) Vertical thermal stress – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	-0.022	-0.079	-0.175	-0.274	-0.379	-0.478	-0.526	-0.492	-0.348	0.000
3.000	0.000	-0.014	-0.065	-0.144	-0.227	-0.331	-0.457	-0.547	-0.551	-0.400	0.000
4.000	0.000	-0.005	-0.034	-0.077	-0.158	-0.274	-0.398	-0.523	-0.566	-0.442	0.000
5.000	0.000	0.000	-0.006	-0.036	-0.096	-0.204	-0.342	-0.480	-0.564	-0.468	0.000
6.000	0.000	0.000	0.000	-0.014	-0.058	-0.137	-0.281	-0.446	-0.562	-0.490	0.000
8.000	0.000	0.000	0.000	0.019	0.000	-0.067	-0.192	-0.365	-0.547	-0.518	0.000
10.000	0.000	0.000	0.000	0.024	0.012	-0.024	-0.132	-0.300	-0.516	-0.540	0.000
12.000	0.000	0.000	0.000	0.014	0.029	0.000	-0.072	-0.245	-0.461	-0.562	0.000
14.000	0.000	0.000	0.000	0.017	0.017	0.017	0.000	-0.202	-0.437	-0.554	0.000
16.000	0.000	0.000	0.000	0.000	0.019	0.038	0.077	-0.154	-0.422	-0.557	0.000

(b) Vertical thermal stress – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.072	0.234	0.417	0.592	0.697	0.728	0.665	0.531	0.306	0.032
3.000	0.000	0.109	0.346	0.605	0.832	0.969	1.007	0.921	0.727	0.425	0.052
4.000	0.000	0.139	0.428	0.733	0.978	1.129	1.165	1.066	0.838	0.484	0.041
5.000	0.000	0.165	0.494	0.825	1.080	1.226	1.255	1.153	0.910	0.522	0.027
6.000	0.000	0.189	0.549	0.895	1.143	1.282	1.305	1.203	0.756	0.551	0.016
8.000	0.000	0.233	0.644	0.991	1.218	1.335	1.327	1.260	1.028	0.603	0.002
10.000	0.000	0.277	0.725	1.060	1.251	1.335	1.349	1.286	1.082	0.653	-0.001
12.000	0.000	0.320	0.797	1.116	1.266	1.321	1.335	1.297	1.125	0.702	-0.002
14.000	0.000	0.359	0.891	1.159	1.272	1.304	1.319	1.302	1.161	0.745	-0.001
16.000	0.000	0.397	0.916	1.191	1.274	1.287	1.301	1.304	1.187	0.786	-0.001

(c) Hoop thermal stress, inside – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.205	0.156	0.107	0.141	-0.038	-0.134	-0.267	-0.426	-0.609	-0.812	-1.000
3.000	0.074	0.076	0.069	0.049	0.008	-0.054	-0.163	-0.319	-0.524	-0.762	-1.000
4.000	0.017	0.036	0.047	0.053	0.040	-0.004	-0.093	-0.341	-0.455	-0.723	-1.000
5.000	-0.008	0.014	0.034	0.050	0.052	0.025	-0.045	-0.180	-0.399	-0.690	-1.000
6.000	-1.011	0.003	0.023	0.040	0.053	0.041	-0.012	-0.137	-0.354	-0.661	-1.000
8.000	-0.015	-0.004	0.008	0.027	0.043	0.052	0.026	-0.069	-0.277	-0.607	-1.000
10.000	-0.008	-0.005	0.000	0.015	0.030	0.048	0.042	-0.024	-0.215	-0.564	-1.000
12.000	-0.002	-0.003	-0.003	0.005	0.022	0.041	0.051	0.006	-0.163	-0.524	-1.000
14.000	0.000	-0.002	-0.003	0.002	0.011	0.034	0.059	0.025	-0.127	-0.487	-1.000
16.000	0.002	0.000	-0.002	-0.001	0.006	0.028	0.064	0.036	-0.100	-0.557	-1.000

Table D1 – Thermal stress coefficients – Pinned-base condition (continued)

(d) Hoop thermal stress, outside – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.205	0.164	0.135	0.205	0.060	0.002	-0.095	-0.236	-0.431	-0686	-1.000
3.000	0.074	0.082	0.093	0.101	0.090	0.066	0.001	-0.123	-0.326	-0.618	-1.000
4.000	0.017	0.038	0.059	0.081	0.098	0.094	0.051	-0.053	-0.251	-0.565	-1.000
5.000	-0.008	0.014	0.036	0.062	0.086	0.099	0.079	-0.008	-0.195	-0.522	-1.000
6.000	-1.011	0.003	0.023	0.046	0.073	0.091	0.090	0.023	0.152	-0.485	-1.000
8.000	-0.015	-0.004	0.008	0.021	0.043	0.076	0.096	0.063	-0.081	-0.421	-1.000
10.000	-0.008	-0.005	0.000	0.007	0.026	0.056	0.090	0.084	-0.029	-0.370	-1.000
12.000	-0.002	-0.003	-0.003	-0.001	0.012	0.041	0.077	0.094	0.003	-0.322	-1.000
14.000	0.000	-0.002	-0.003	-0.004	0.005	0.028	0.059	0.097	0.031	-0.287	-1.000
16.000	0.002	0.000	-0.002	-0.001	-0.000	0.014	0.036	0.092	0.052	-0.357	-1.000

(e) Hoop thermal stress, inside – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.273	0.647	0.947	1.177	1.343	1.446	1.494	1.487	1.420	1.289	1.076
3.000	0.247	0.676	1.002	1.229	1.379	1.464	1.504	1.493	1.427	1.285	1.037
4.000	0.260	0.777	1.053	1.061	1.382	1.448	1.477	1.472	1.418	1.281	1.013
5.000	0.276	0.815	1.098	1.282	1.376	1.420	1.442	1.446	1.410	1.283	1.002
6.000	0.290	0.816	1.136	1.297	1.364	1.389	1.407	1.420	1.402	1.291	0.997
8.000	0.304	0.884	1.195	1.314	1.336	1.336	1.321	1.378	1.392	1.307	0.996
10.000	0.310	0.938	1.238	1.322	1.313	1.294	1.304	1.347	1.387	1.325	0.997
12.000	0.310	0.982	1.269	1.324	1.293	1.263	1.272	1.323	1.382	1.342	0.999
14.000	0.309	1.019	1.292	1.322	1.277	1.243	1.251	1.305	1.376	1.353	1.000
16.000	0.309	1.050	1.307	1.317	1.264	1.230	1.236	1.291	1.367	1.361	1.000

(f) Hoop thermal stress, outside – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-1.728	-1.380	-1.138	-0.974	-0.871	-0.806	-0.769	-0.753	-0.772	-0.822	-0.936
3.000	-1.753	-1.364	-1.123	-0.990	-0.821	-0.885	-0.859	-0.840	-0.835	-0.869	-0.983
4.000	-1.741	-1.274	-1.102	-1.004	-0.971	-0.959	-0.943	-0.912	-0.885	-0.894	-1.002
5.000	-1.725	-1.245	-1.081	-1.015	-1.014	-1.022	-1.010	-0.970	-0.919	-0.906	-1.008
6.000	-1.711	-1.253	-1.062	-1.026	-1.049	-1.073	-1.064	-1.014	-0.943	-0.909	-1.009
8.000	-1.696	-1.200	-1.037	-1.043	-1.103	-1.145	-1.111	-1.077	-0.979	-0.911	-1.006
10.000	-1.691	-1.162	-1.024	-1.061	-1.138	-1.188	-1.182	-1.116	-1.003	-0.911	-1.003
12.000	-1.691	-1.134	-1.018	-1.079	-1.164	-1.213	-1.209	-1.144	-1.024	-0.912	-1.001
14.000	-1.691	-1.111	-1.019	-1.096	-1.182	-1.227	-1.224	-1.164	-1.043	-0.916	-1.000
16.000	-1.692	-1.093	-1.023	-1.113	-1.196	-1.235	-1.233	-1.179	-1.061	-0.922	-1.000

Table D1 – Thermal stress coefficients – Pinned-base condition (continued)

(g) Vertical thermal stress – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.025	0.078	0.121	0.159	0.159	0.125	0.070	0.019	-0.021	0.016
3.000	0.000	0.047	0.141	0.231	0.303	0.319	0.275	0.187	0.088	0.012	0.026
4.000	0.000	0.067	0.197	0.328	0.410	0.427	0.383	0.272	0.136	0.021	0.021
5.000	0.000	0.082	0.244	0.394	0.492	0.511	0.457	0.336	0.173	0.027	0.013
6.000	0.000	0.095	0.274	0.441	0.543	0.573	0.512	0.378	0.197	0.031	0.008
8.000	0.000	0.117	0.322	0.505	0.609	0.634	0.582	0.448	0.241	0.042	0.001
10.000	0.000	0.138	0.362	0.542	0.631	0.655	0.609	0.493	0.283	0.056	-0.001
12.000	0.000	0.160	0.398	0.565	0.648	0.661	0.631	0.526	0.332	0.070	-0.001
14.000	0.000	0.179	0.431	0.588	0.645	0.660	0.659	0.550	0.362	0.096	-0.001
16.000	0.000	0.198	0.458	0.595	0.646	0.663	0.689	0.575	0.382	0.114	-0.001

(h) Hoop thermal stress, inside – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.239	0.401	0.527	0.659	0.652	0.656	0.613	0.531	0.406	0.239	0.038
3.000	0.161	0.376	0.536	0.639	0.694	0.705	0.670	0.587	0.452	0.261	0.018
4.000	0.139	0.407	0.550	0.657	0.711	0.722	0.692	0.616	0.481	0.279	0.007
5.000	0.134	0.414	0.566	0.666	0.714	0.723	0.699	0.633	0.506	0.296	0.001
6.000	0.139	0.410	0.580	0.669	0.708	0.715	0.698	0.641	0.524	0.315	-0.001
8.000	0.156	0.440	0.602	0.671	0.390	0.694	0.674	0.655	0.557	0.350	-0.002
10.000	0.151	0.467	0.619	0.668	0.672	0.671	0.673	0.662	0.586	0.380	-0.001
12.000	0.154	0.490	0.633	0.664	0.658	0.652	0.662	0.665	0.610	0.409	-0.001
14.000	0.155	0.508	0.644	0.662	0.644	0.639	0.655	0.665	0.625	0.433	0.000
16.000	0.155	0.525	0.653	0.658	0.635	0.629	0.650	0.664	0.634	0.402	0.000

(i) Hoop thermal stress, outside – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-0.761	+0.608	-0.501	-0.385	-0.405	-0.402	-0432	-0.495	-0.602	-0.754	-0.968
3.000	-0.840	-0.641	-0.515	-0.445	-0.416	-0.410	0.429	-0.481	-0.580	-0.743	-0.992
4.000	-0.862	-0.618	-0.521	-0.461	-0.437	-0.432	-0.446	-0.483	-0.568	-0.729	-1.001
5.000	-0.866	-0.616	-0.522	-0.476	-0.464	-0.462	-0.466	-0.489	-0.557	-0.714	-1.004
6.000	-0861	-0625	-0.520	-0.490	-0.488	-0.491	-0.487	-0.495	-0.547	-0.697	-0.005
8.000	-0.856	-0.602	-0.515	-0.511	-0.530	-0.534	-0.508	-0.507	-0.530	-0.666	-1.003
10.000	-0.850	-0.584	-0.512	-0.527	-0.556	-0.566	-0.546	-0.516	-0.516	-0.640	-1.001
12.000	-0.846	-0.568	-0.511	-0.540	-0.576	-0.586	-0.566	-0.525	-0.510	-0.617	-1.000
14.000	-0.846	-0.557	-0.511	-0.550	-0.589	-0.600	-0.583	-0.533	-0.508	-0.601	-1.000
16.000	-0.845	-0.547	-0.513	-0.557	-0.598	-0.610	-0.598	-0.544	-0.504	-0.639	-1.000

Table D2 – Thermal stress coefficients – Fixed-base condition

(a) Vertical thermal stress – Average temperature change

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	-0.024	-0.086	-0.158	-0.211	-0.214	-0.142	-0.046	0.401	0.934	1.726
3.000	0.000	-0.025	-0.094	-0.184	-0.266	-0.328	-0.299	-0.151	0.191	0.803	1.739
4.000	0.000	-0.019	-0.072	-0.158	-0.250	-0.326	-0.360	-0.254	0.062	0.696	1.752
5.000	0.000	-0.012	-0.048	-0.114	-0.210	-0.306	-0.366	-0.312	-0.042	0.606	1.758
6.000	0.000	-0.007	-0.029	-0.079	-0.158	-0.259	-0.353	-0.346	-0.122	0.526	1.742
8.000	0.000	0.000	-0.010	-0.029	-0.077	-0.173	-0.298	-0.365	-0.230	0.380	1.766
10.000	0.000	0.000	0.012	0.000	-0.024	-0.108	-0.252	-0.360	-0.312	0.264	1.784
12.000	0.000	0.000	0.000	0.014	0.000	-0.058	-0.202	-0.346	-0.317	0.173	1.771
14.000	0.000	0.000	0.000	0.000	0.000	-0.034	-0.168	-0.302	-0.353	0.118	1.764
16.000	0.000	0.000	0.000	0.000	0.000	-0.019	-0.115	-0.230	-0.384	0.096	1.747

(b) Vertical thermal stress – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.070	0.232	0.432	0.633	0.814	0.964	1.080	1.164	1.219	1.252
3.000	0.000	0.100	0.320	0.569	0.797	0.981	1.113	1.198	1.247	1.269	1.272
4.000	0.000	0.129	0.397	0.680	0.916	1.087	1.193	1.249	1.270	1.271	1.261
5.000	0.000	0.156	0.465	0.770	1.005	1.157	1.237	1.269	1.271	1.261	1.247
6.000	0.000	0.183	0.527	0.847	1.072	1.203	1.260	1.272	1.264	1.249	1.236
8.000	0.000	0.232	0.633	0.964	1.164	1.252	1.272	1.263	1.246	1.234	1.222
10.000	0.000	0.277	0.722	1.049	1.216	1.270	1.268	1.248	1.232	1.222	1.219
12.000	0.000	0.320	0.797	1.113	1.247	1.272	1.257	1.237	1.224	1.219	1.218
14.000	0.000	0.359	0.861	1.159	1.263	1.269	1.247	1.229	1.220	1.218	1.219
16.000	0.000	0.397	0.916	1.193	1.270	1.261	1.238	1.224	1.218	1.218	1.219

(c) Hoop thermal stress, inside – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.253	0.140	0.025	-0.100	-0.232	-0.371	-0.511	-0.647	-0.742	-0.777	-0.689
3.000	0.160	0.107	0.044	-0.035	-0.136	-0.263	-0.408	-0.568	-0.708	-0.774	-0.687
4.000	0.085	0.070	0.044	0.000	-0.048	-0.172	-0.319	-0.493	-0.667	-0.770	-0.685
5.000	0.037	0.042	0.038	0.021	-0.023	-0.106	-0.241	-0.427	-0.629	-0.763	-0.684
6.000	0.010	0.023	0.033	0.031	0.005	-0.061	-0.185	-0.368	-0.592	-0.756	-0.686
8.000	-0.011	0.005	0.020	0.031	0.030	-0.005	-0.101	-0.278	-0.522	-0.742	-0.682
10.000	-0.011	-0.002	0.012	0.023	0.035	0.021	-0.049	-0.206	-0.465	-0.726	-0.682
12.000	-0.006	-0.003	0.003	0.017	0.031	0.033	-0.014	-0.151	-0.405	-0.707	-0.681
14.000	-0.003	-0.002	0.000	0.007	0.022	0.034	0.005	-0.105	-0.359	-0.685	-0.682
16.000	0.000	-0.001	-0.001	0.003	0.018	0.029	0.019	-0.066	-0.319	-0.662	-0.686

Table D2 – Thermal stress coefficients – Fixed-base condition (continued)

(d) Hoop thermal stress, outside – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.253	0.148	0.057	-0.042	-0.156	-0.295	-0.461	-0.663	-0.886	-1.113	-1.311
3.000	0.160	0.117	0.078	0.031	-0.040	-0.145	-0.300	-0.514	-0.776	-1.064	-1.313
4.000	0.085	0.076	0.070	0.058	0.042	-0.054	-0.139	-0.401	-0.689	-1.020	-1.315
5.000	0.037	0.046	0.056	0.063	0.053	0.004	-0.100	-0.315	-0.613	-0.981	-1.316
6.000	0.010	0.025	0.043	0.059	0.063	0.033	-0.057	-0.244	-0.548	-0.946	-1.314
8.000	-0.011	0.005	0.024	0.041	0.058	0.057	0.007	-0.146	-0.440	-0.880	-1.318
10.000	-0.011	-0.002	0.008	0.023	0.043	0.059	0.041	-0.076	-0.353	-0.822	-1.318
12.000	-0.006	-0.003	0.003	0.011	0.031	0.053	0.058	-0.027	-0.291	-0.769	-1.319
14.000	-0.003	-0.002	0.000	0.007	0.022	0.046	0.065	0.003	-0.231	-0.727	-1.318
16.000	0.000	-0.001	-0.001	0.003	0.012	0.035	0.061	0.016	-0.181	-0.696	-1.314

(e) Hoop thermal stress, inside – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.308	0.635	0.891	1.077	1.203	1.279	1.318	1.331	1.327	1.313	1.295
3.000	0.308	0.699	0.983	1.168	1.273	1.321	1.331	1.320	1.299	1.277	1.256
4.000	0.308	0.750	1.050	1.225	1.308	1.331	1.321	1.297	1.271	1.249	1.233
5.000	0.307	0.792	1.101	1.263	1.324	1.328	1.305	1.276	1.250	1.233	1.222
6.000	0.308	0.829	1.142	1.289	1.331	1.319	1.288	1.258	1.236	1.223	1.217
8.000	0.308	0.891	1.203	1.318	1.327	1.295	1.260	1.235	1.221	1.216	1.215
10.000	0.308	0.941	1.244	1.329	1.314	1.274	1.241	1.222	1.216	1.215	1.217
12.000	0.308	0.983	1.273	1.331	1.299	1.256	1.228	1.217	1.215	1.217	1.219
14.000	0.308	1.019	1.294	1.327	1.284	1.243	1.221	1.215	1.216	1.218	1.220
16.000	0.308	1.050	1.308	1.321	1.271	1.233	1.218	1.215	1.217	1.219	1.220

(f) Hoop thermal stress, outside – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-1.693	-1.391	-1.194	-1.079	-1.026	-1.015	-1.030	-1.059	-1.093	-1.126	-1.156
3.000	-1.693	-1.338	-1.133	-1.038	-1.015	-1.033	-1.070	-1.112	-1.151	-1.181	-1.203
4.000	-1.693	-1.297	-1.094	-1.020	-1.023	-1.061	-1.109	-1.153	-1.187	-1.209	-1.222
5.000	-1.693	-1.265	-1.067	-1.015	-1.038	-1.089	-1.141	-1.182	-1.208	-1.222	-1.228
6.000	-1.693	-1.237	-1.048	-1.016	-1.056	-1.115	-1.166	-1.201	-1.220	-1.228	-1.229
8.000	-1.693	-1.193	-1.026	-1.030	-1.093	-1.156	-1.199	-1.221	-1.228	-1.228	-1.226
10.000	-1.693	-1.160	-1.017	-1.049	-1.124	-1.184	-1.216	-1.228	-1.228	-1.225	-1.223
12.000	-1.693	-1.133	-1.015	-1.070	-1.150	-1.203	-1.225	-1.229	-1.226	-1.223	-1.221
14.000	-1.693	-1.111	-1.017	-1.091	-1.171	-1.215	-1.228	-1.228	-1.224	-1.221	-1.220
16.000	-1.693	-1.094	-1.023	-1.109	-1.187	-1.222	-1.229	-1.226	-1.222	-1.220	-1.220

Table D2 – Thermal stress coefficients – Fixed-base condition (continued)

(g) Vertical thermal stress – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.023	0.073	0.137	0.211	0.300	0.411	0.563	0.782	1.076	1.489
3.000	0.000	0.037	0.113	0.192	0.265	0.327	0.407	0.523	0.719	1.036	1.506
4.000	0.000	0.055	0.162	0.261	0.333	0.380	0.417	0.497	0.666	0.984	1.507
5.000	0.000	0.072	0.208	0.328	0.398	0.425	0.436	0.478	0.615	0.934	1.502
6.000	0.000	0.088	0.249	0.384	0.457	0.472	0.454	0.463	0.571	0.887	1.489
8.000	0.000	0.116	0.312	0.468	0.544	0.539	0.437	0.449	0.508	0.497	1.494
10.000	0.000	0.138	0.367	0.525	0.596	0.581	0.508	0.444	0.460	0.743	1.491
12.000	0.000	0.160	0.398	0.564	0.623	0.607	0.528	0.446	0.453	0.696	1.494
14.000	0.000	0.179	0.431	0.580	0.631	0.618	0.539	0.463	0.434	0.668	1.491
16.000	0.000	0.198	0.458	0.597	0.645	0.621	0.562	0.497	0.417	0.657	1.483

(h) Hoop thermal stress, inside – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.280	0.387	0.458	0.489	0.485	0.454	0.403	0.342	0.292	0.268	0.303
3.000	0.234	0.403	0.513	0.566	0.568	0.529	0.462	0.376	0.296	0.251	0.285
4.000	0.196	0.410	0.547	0.613	0.630	0.580	0.501	0.402	0.302	0.240	0.274
5.000	0.172	0.417	0.570	0.642	0.651	0.611	0.532	0.424	0.311	0.235	0.269
6.000	0.159	0.426	0.583	0.660	0.668	0.629	0.552	0.445	0.322	0.233	0.265
8.000	0.148	0.448	0.611	0.674	0.679	0.645	0.580	0.478	0.349	0.237	0.267
10.000	0.148	0.469	0.628	0.676	0.674	0.647	0.596	0.508	0.375	0.244	0.267
12.000	0.151	0.490	0.638	0.674	0.665	0.644	0.607	0.533	0.405	0.255	0.269
14.000	0.152	0.508	0.647	0.667	0.653	0.638	0.613	0.555	0.429	0.267	0.269
16.000	0.154	0.524	0.653	0.662	0.645	0.631	0.618	0.574	0.449	0.279	0.267

(i) Hoop thermal stress, outside – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-0.720	-0.621	-0.569	-0.561	-0.591	-0.655	-0.745	-0.861	0.990	-1.120	-1.233
3.000	-0.767	-0.611	-0.528	-0.503	-0.527	-0.589	-0.695	-0.813	-0.963	-1.122	-1.258
4.000	-0.804	-0.610	-0.512	-0.481	-0.490	-0.558	-0.649	-0.777	-0.938	-1.115	-1.269
5.000	-0.828	-0.609	-0.506	-0.476	-0.493	-0.543	-0.625	-0.748	-0.911	-1.102	-1.272
6.000	-0.842	-0.606	-0.502	-0.479	-0.497	-0.541	-0.612	-0.722	-0.884	-1.087	-1.271
8.000	-0.852	-0.594	-0.501	-0.494	-0.518	-0.549	-0.596	-0.684	-0.834	-1.054	-1.272
10.000	-0.852	-0.581	-0.504	-0.513	-0.541	-0.562	-0.588	-0.652	-0.791	-0.023	-1.270
12.000	-0.850	-0.568	-0.506	-0.529	-0.560	-0.575	-0.583	-0.628	-0.759	-0.996	-1.270
14.000	-0.848	-0.557	-0.509	-0.542	-0.574	-0.584	-0.581	-0.612	-0.728	-0.974	-1.269
16.000	-0.847	-0.547	-0.512	-0.553	-0.588	-0.593	-0.584	-0.605	-0.701	-0.958	-1.267

Table D3 – Thermal stress coefficients – Sliding-base condition

(a) Vertical thermal stress – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

(b) Vertical thermal stress – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.032	0.068	0.176	0.292	0.377	0.407	0.377	0.292	0.176	0.068	0.032
3.000	0.052	0.149	0.346	0.547	0.689	0.742	0.689	0.547	0.346	0.149	0.052
4.000	0.041	0.181	0.447	0.709	0.889	0.954	0.889	0.709	0.447	0.181	0.041
5.000	0.027	0.198	0.516	0.819	1.022	1.093	1.022	0.819	0.516	0.198	0.027
6.000	0.016	0.212	0.571	0.899	1.113	1.186	1.113	0.899	0.571	0.212	0.016
8.000	0.002	0.243	0.659	1.007	1.216	1.283	1.216	1.007	0.659	0.243	0.002
10.000	-0.001	0.279	0.734	1.077	1.264	1.320	1.264	1.077	0.734	0.279	-0.001
12.000	-0.002	0.318	0.800	1.130	1.283	1.325	1.283	1.130	0.800	0.318	-0.002
14.000	-0.001	0.356	0.861	1.168	1.290	1.318	1.290	1.168	0.861	0.356	-0.001
16.000	-0.001	0.394	0.914	1.197	1.288	1.303	1.288	1.197	0.914	0.394	-0.001

(c) Hoop thermal stress, inside – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Table D3 – Thermal stress coefficients – Sliding-base condition (continued)

(d) Hoop thermal stress, outside – Average temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

(e) Hoop thermal stress, inside – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.383	0.729	0.997	1.188	1.301	1.338	1.301	1.188	0.997	0.729	0.383
3.000	0.344	0.755	1.062	1.268	1.384	1.422	1.384	1.268	1.062	0.755	0.344
4.000	0.321	0.779	1.101	1.302	1.409	1.442	1.409	1.302	1.101	0.779	0.321
5.000	0.309	0.805	1.132	1.319	1.409	1.435	1.409	1.319	1.132	0.805	0.309
6.000	0.304	0.832	1.158	1.327	1.399	1.417	1.399	1.327	1.158	0.832	0.304
8.000	0.303	0.887	1.204	1.332	1.367	1.371	1.367	1.332	1.204	0.887	0.303
10.000	0.305	0.936	1.240	1.332	1.335	1.327	1.335	1.332	1.240	0.936	0.305
12.000	0.306	0.980	1.268	1.328	1.308	1.292	1.308	1.328	1.268	0.980	0.306
14.000	0.307	1.017	1.290	1.323	1.286	1.266	1.286	1.323	1.290	1.017	0.307
16.000	0.308	1.049	1.305	1.316	1.269	1.246	1.269	1.316	1.305	1.049	0.308

(f) Hoop thermal stress, outside – Differential temperature change

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-1.629	-1.297	-1.067	-0.918	-0.836	-0.809	-0.836	-0.918	-1.067	-1.297	-1.629
3.000	-1.676	-1.299	-1.064	-0.930	-0.865	-0.846	-0.865	-0.930	-1.064	-1.299	-1.676
4.000	-1.695	-1.287	-1.061	-0.954	-0.912	-0.902	-0.912	-0.954	-1.061	-1.287	-1.695
5.000	-1.701	-1.267	-1.055	-0.977	-0.960	-0.959	-0.960	-0.977	-1.055	-1.267	-1.701
6.000	-1.702	-1.245	-1.048	-0.977	-1.003	-1.011	-1.003	-0.977	-1.048	-1.245	-1.702
8.000	-1.699	-1.202	-1.034	-1.031	-1.072	-1.092	-1.072	-1.031	-1.034	-1.202	-1.699
10.000	-1.696	-1.165	-1.025	-1.057	-1.121	-1.149	-1.121	-1.057	-1.025	-1.165	-1.696
12.000	-1.694	-1.136	-1.021	-1.079	-1.155	-1.186	-1.155	-1.079	-1.021	-1.136	-1.694
14.000	-1.693	-1.112	-1.021	-1.098	-1.179	-1.209	-1.179	-1.098	-1.021	-1.112	-1.693
16.000	-1.693	-1.094	-1.025	-1.115	-1.196	-1.223	-1.196	-1.115	-1.025	-1.094	-1.693

Table D3 – Thermal stress coefficients – Sliding-base condition (continued)

(g) Vertical thermal stress – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.016	0.034	0.088	0.146	0.188	0.204	0.188	0.146	0.088	0.034	0.016
3.000	0.026	0.074	0.173	0.273	0.345	0.371	0.345	0.273	0.173	0.074	0.026
4.000	0.021	0.090	0.223	0.354	0.445	0.477	0.445	0.354	0.223	0.090	0.021
5.000	0.013	0.099	0.258	0.409	0.511	0.547	0.511	0.409	0.258	0.099	0.013
6.000	0.008	0.106	0.285	0.450	0.556	0.593	0.556	0.450	0.285	0.106	0.008
8.000	0.001	0.121	0.329	0.503	0.608	0.642	0.608	0.503	0.329	0.121	0.001
10.000	-0.001	0.140	0.367	0.539	0.632	0.660	0.632	0.539	0.367	0.140	-0.001
12.000	-0.001	0.159	0.400	0.565	0.642	0.662	0.642	0.565	0.400	0.159	-0.001
14.000	-0.001	0.178	0.431	0.584	0.645	0.659	0.645	0.584	0.431	0.178	-0.001
16.000	-0.001	0.197	0.457	0.598	0.644	0.651	0.644	0.598	0.457	0.197	-0.001

(h) Hoop thermal stress, inside – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.191	0.364	0.499	0.594	0.650	0.669	0.650	0.594	0.499	0.364	0.191
3.000	0.172	0.378	0.531	0.634	0.692	0.711	0.692	0.634	0.531	0.378	0.172
4.000	0.160	0.390	0.550	0.651	0.705	0.721	0.705	0.651	0.550	0.390	0.160
5.000	0.155	0.402	0.566	0.659	0.705	0.718	0.705	0.659	0.566	0.402	0.155
6.000	0.152	0.416	0.579	0.664	0.699	0.709	0.699	0.664	0.579	0.416	0.152
8.000	0.151	0.443	0.602	0.666	0.683	0.685	0.683	0.666	0.602	0.443	0.151
10.000	0.152	0.468	0.620	0.666	0.667	0.664	0.667	0.666	0.620	0.468	0.152
12.000	0.153	0.490	0.634	0.664	0.654	0.646	0.654	0.664	0.634	0.490	0.153
14.000	0.154	0.508	0.645	0.661	0.643	0.633	0.643	0.661	0.645	0.508	0.154
16.000	0.154	0.524	0.653	0.658	0.634	0.623	0.634	0.658	0.653	0.524	0.154

(i) Hoop thermal stress, outside – Total effects

S_{FACT}	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-0.815	-0.648	-0.533	-0.459	-0.418	-0.405	-0.418	-0.459	-0.533	-0.648	-0.815
3.000	-0.838	-0.649	-0.532	-0.465	-0.432	-0.423	-0.432	-0.465	-0.532	-0.649	-0.838
4.000	-0.848	-0.643	-0.530	-0.477	-0.456	-0.451	-0.456	-0.477	-0.530	-0.643	-0.848
5.000	-0.851	-0.633	-0.527	-0.488	-0.480	-0.479	-0.480	-0.488	-0.527	-0.633	-0.851
6.000	-0.851	-0.622	-0.524	-0.499	-0.501	-0.505	-0.501	-0.499	-0.524	-0.622	-0.851
8.000	-0.849	-0.601	-0.517	-0.515	-0.536	-0.546	-0.536	-0.515	-0.517	-0.601	-0.849
10.000	-0.848	-0.583	-0.513	-0.529	-0.560	-0.574	-0.560	-0.529	-0.513	-0.583	-0.848
12.000	-0.847	-0.568	-0.510	-0.540	-0.578	-0.593	-0.578	-0.540	-0.510	-0.568	-0.847
14.000	-0.847	-0.556	-0.511	-0.549	-0.590	-0.605	-0.590	-0.549	-0.511	-0.556	-0.847
16.000	-0.846	-0.547	-0.512	-0.558	-0.598	-0.612	-0.598	-0.558	-0.512	-0.547	-0.846

NOTES

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