NZS 3106 : 1986

**Amendment No 1 &2
Erratum Appended**

Code of practice for
**CONCRETE STRUCTURES FOR THE
STORAGE OF LIQUIDS**

Superseding NZS 3106P:1978

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COMMITTEE REPRESENTATION

This Standard was prepared under the direction of the Building and Civil Engineering Divisional Committee (30/-) for the Standards Council, established under the Standards Act 1965.

The Concrete Structures for the Storage of Liquids Committee (31/10) was responsible for the preparation of the Standard and consisted of the following persons:

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AMENDMENTS

No.	Date of issue	Description	Entered by, and date

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CONTENTS

(contents continued overleaf)

NOTES

RELATED DOCUMENTS

In this document reference is made to the following:

NEW ZEALAND STANDARDS

NZS 1900: ____	Model building bylaw
NZS 3101 : 1982	The design of concrete structures
NZS 3103 : 1976	Sands for mortars and internal and external plasters
NZS 3104 : 1983	Concrete production – High grade and special grade
NZS 3109 : 1980	Concrete construction
NZS 3112 : ____	Methods of test for concrete
	Part 2:1986 Tests relating to the determination of strength of concrete
NZS 3113 : 1979	Chemical admixtures for concrete
NZS 3121 : 1986	Water and aggregate for concrete
NZS 3402P : 1973	Hot rolled steel bars for concrete reinforcement
NZS 3422 : 1975	Welded fabric of drawn steel wire for concrete reinforcement
NZS 4203 : 1984	General structural design and design loadings for buildings
NZS 4441 : 1985	Swimming pools

BRITISH STANDARDS

BS 1485 : 1983	Zinc coated hexagonal steel wire netting
BS 4102 : 1971	Steel wire for fences
BS 5337 : 1976	The structural use of concrete for retaining aqueous liquids

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FOREWORD

The objectives in drafting this Code have been to provide an up-to-date document covering materials, design, and construction of concrete structures for the storage of liquids. In writing the sections, particular attention has been given to producing provisions that would whenever possible be compatible with, and appropriate for use with, other modern New Zealand design loading codes – particularly NZS 4203:1984 *Code of practice for general structural design and design loadings for buildings* and NZS 3101:1982 *Code of practice for the design of concrete structures.*

However, some differences of approach have been necessary, and it is appropriate that these be identified in this foreword.

Thermal loading: NZS 4203 includes only generalized clauses relating to the design of concrete structures for thermal effects. As thermal stresses induced by temperature gradients through the wall thickness can be of major significance to concrete storage tanks, this Code includes specific data on levels of temperature change to be used in design.

Creep and shrinkage effects: Again, detailed specific requirements are included to cover these effects which are covered in a general way only in NZS 4203.

Earthquake loads: While this Code was being prepared, a major exercise re-evaluating the seismicity of New Zealand, and the correspondingly appropriate seismic design loads, was under way. The 'loadings code' NZS 4203 has been felt to be in need of updating in this area for some time. Of particular concern to the writers of this Code was that seismic coefficients for very short-period and very long-period modes of response were not well covered by NZS 4203. It is these modes that are of major concern to the designer of ground supported concrete tanks.

It became evident that a revised loadings code would not be available until at least 1988, while there was an urgent need to finalize this Code in 1986. Consequently seismic provisions included in this Code are based on the best information available at the time: a draft report on 'Seismic design of storage tanks' by a Study Group of the New Zealand National Society for Earthquake Engineering. The provisions incorporated in this Code have been formulated in such a way as to be compatible with NZS 4203 where possible.

Design philosophy: Design is solely based on working stress methods. Ultimate strength design using factored loads is seen as inappropriate when the basic design load (that of the contained liquid) is known to a comparatively high precision. Thus a load factor reflecting the uncertainty of magnitude of fluid loading would be close to unity. Conversely, performance under service loads is of prime importance, and a detailed control of service conditions is essential. Two categories of load combination are listed: permanent or long duration loads, and combinations including infrequent combinations of transient loads. For the two categories different limiting stresses are allowed.

An approach adopted in an early draft of this Code, based on limitations to calculated crack widths, has not been favoured, because of uncertainty of equations predicting crack widths, particularly when applied to, for example, circular shell structures, subjected to fluid and thermal loading.

The format of this Code reflects that adopted in recent Standards Association of New Zealand structural design codes.

NOTES

NEW ZEALAND STANDARD

Code of practice for
CONCRETE STRUCTURES FOR
THE STORAGE OF LIQUIDS

1
COMMENTARY

1
GENERAL

1.1
Scope

1.1.1
This New Zealand Standard Code of Practice sets out requirements for concrete, including cement mortar, structures for the storage of liquids, and is approved as a means of compliance with the relevant requirements of NZS 1900.

1.1.2
Sections 1,2,3, and 9 of this Standard apply to all concrete structures for the storage of liquids. The remaining sections apply specifically to structures constructed in either reinforced concrete, pre-stressed concrete or cement mortar.

C1.1.3
For liquids having a detrimental effect on con-crete, 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 and even collapse can occur before a leak is detected. In tanks where such impervious linings are used, the allowable stresses of the alternative design method of NZS 3101 may be used in lieu of those of this Code.

1.1.3
Appropriate special precautions additional to the requirements of this Standard shall be taken where the structure is to contain highly penetrating liquids, or liquids having detrimental effect on concrete.

1.1.4
Swimming pools shall in addition comply with NZS 4441.

1.2
Interpretation

1.2.1
In this Standard the word "shall" indicates a requirement that is to be adopted in order to comply with the Standard, while the word "should" indicates a recommended practice.

1.2.2
Cross-references to other clauses or subdivisions within this Standard quote the number only, for example: "... as required by 6.4.1".

1.2.3
The full titles of reference documents cited in this Standard are given in the "List of related documents" immediately preceding the Foreword.

1.3
Support structures

1.3.1
Support structures for elevated tanks shall be designed to the requirements of NZS 4203 together with an appropriate design code for the material to be used.

1.4
Roofs

1.4.1
Every structure containing water intended for potable water supply without further treatment shall have a roof designed to prevent contamination of its contents.

1.5
Notation

1.5.1
In this Standard, symbols shall have the following meanings, provided that other meanings for symbols that are defined immediately adjacent to formulae or diagrams shall apply in relation to those formulae or diagrams only:

a Radius of circular tank, m.

A The design loads, or their related internal moments and forces, resulting from a combination of group A loads.

A_o Peak horizontal ground acceleration coefficient (see 2.2.9.3).

b One half of the width of a rectangular tank, perpendicular to the direction being considered, m.

B The design loads, or their related internal moments and forces, resulting from a combination of group B loads.

C Temporary loads, or their related internal moments and forces, occurring during construction.

C_C The horizontal convective seismic coefficient.

C_I The horizontal impulsive seismic coefficient.

C_T The axial force induced in a circular wall due to a temperature gradient through the wall, N.

D Dead loads, or their related internal moments and forces.

E Earthquake loads, or their related internal moments and forces.

E_c Modulus of elasticity of concrete, MPa.

E_H The horizontal component of E.

E_S	Modulus of elasticity of prestressed reinforcement, MPa.
E_V	The vertical component of E.
EP	The loads, or their related moments and forces, resulting from backfilled earth pressure.
f'_c	Specified compressive strength of concrete or cement mortar, MPa.
$\sqrt{f'_c}$	The square root of the specified compressive strength of concrete or cement mortar, MPa.
f_i	The fibre stress on the inside face of a tank wall or roof resulting from a temperature gradient through the wall or roof, MPa.
f_o	The fibre stress on the outside face of a tank wall or roof resulting from a temperature gradient through the wall or roof, MPa.
f_y	The specified yield strength of non-prestressed reinforcement, MPa.
F	Liquid load, the internal static force exerted by the stored contents when filled to overflow level or its related moments and forces.
F_T	A reduction factor modifying temperature stresses for section rigidity (see C2.2.8.1).
h	The height above the base of the wall of the centre of gravity of either convective or impulsive horizontal seismic force exerted by the contained liquid, m.
h_C	The height of the centre of gravity of the convective horizontal force exerted by the contained liquid, m.
h_I	The height of the centre of gravity of the impulsive horizontal force exerted by the contained liquid, m.
h_S	The height to the centre of gravity of the tank shell, m.
H	The height of the tank wall to the surface level of the liquid, m.
ℓ	One half of the length of a rectangular tank in the direction being considered, m.
L	Live loads, or their related internal moments and forces.
M	The total overturning moment acting on the foundation or support structure, N m.
M_B	Overturning moment on the floor of a tank resulting from hydrodynamic seismic pressures, N m.

M_{BC}	The convective component of M_B, N m.
M_{BI}	The impulsive component of M_B, N m.
M_W	Overturning moment acting at the foot of the tank wall, N m.
M_{WC}	The convective component of M_W, N m.
M_{WI}	The impulsive component of M_W, N m.
M_T	The moment induced in an element due to a temperature gradient across the element, N m.
P_b	The hydrodynamic seismic pressure of the contained liquid at the bottom of the tank wall, MPa.
P_t	The hydrodynamic seismic pressure of the contained liquid at the top of the tank wall, MPa.
P	The prestressing load, or its related moments and forces, provided to counteract forces and deformations induced by other loads.
r	The radius of a dome roof, m.
R	Risk factor as defined in NZS 4203.
S_h	Loads, or their related moments and forces, resulting from shrinkage.
S_w	Loads, or their related moments and forces, resulting from swelling.
t	The thickness of the wall or roof, m.
T	Loads or their related moments and forces resulting from variations in temperature.
T_C	The natural period of the first mode of sloshing, s.
T_I	Fundamental period of oscillation of a tank, s.
V_C	The convective component of V_H, N.
V_H	The total horizontal seismic force per unit length exerted on the wall of a tank, N.
V_I	The impulsive component of V_H, N.
w	Maximum load (dead plus live) for unit area on dome roof, N/m^2.
W	Wind loads, or their related internal moments and forces.
W_C	The equivalent weight of the convective contents, N.
W_I	The equivalent weight of the impulsive contents, N.
W_S	The weight of the tank shell, including the roof if connected to the walls and including the floor, only if the tank is elevated, N.

α	Linear coefficient of thermal expansion.
β	A constant depending on the shape of a tank used in determining seismic pressure (see 2.2.9.5).
λ_L	The weight density of liquid, kN/m^3.
ε_{sh}	Shrinkage strain.
ε_{sw}	Swelling strain.
θ	Change in temperature at a point in a tank wall, °C.

1.6
Definitions

1.6.1
The following terms are defined for general use in this Code. Specialized definitions appear in individual sections.

ADMIXTURE. A material other than portland cement, aggregate, or water, added to concrete to modify its properties.

AGGREGATE. Inert material which is mixed with portland cement and water to produce concrete.

ANCHORAGE. See section 5 of NZS 3101. Also, the means by which the prestress force is permanently transferred to the concrete.

BONDED TENDON. Prestressing tendon that is bonded to concrete either directly or through grouting.

CEMENT MORTAR. A mixture of portland cement or any other hydraulic cement, sand and water.

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 adequate strength and serviceability.

DUCTILITY. The ability of a structure or member to undergo repeated and reversing inelastic deflections beyond the point of first yield while maintaining a substantial proportion of its initial maximum load carrying capacity.

ELASTIC ANALYSIS. Analysis based on the assumption of linear relationships between stress and strain for reinforcement and concrete.

ELEVATED TANK. A liquid retaining structure which is elevated above grade by a support structure.

ENGINEER. The Local Authority's principal Engineer who shall be registered under the Engineers Registration Act 1924 and who is the holder of a current annual practicing certificate; his deputy or assistant appointed by the Local Authority to control the erection of buildings.

FREEBOARD. Additional wall height above tank overflow level.

MOVEMENT JOINT. A specially formed joint intended to accommodate relative movement between adjoining parts of a structure.

PARTIALLY PRESTRESSED CONCRETE. Concrete in which prestressing is used to provide part of the reinforcement requirement, and where cracking of the concrete is permitted under specified design load combinations.

PRECAST CONCRETE. A concrete element cast in other than its final position in the structure.

PRESTRESSED CONCRETE. Concrete in which there has been introduced internal stresses of such magnitude and distribution that the stresses resulting from loads are counteracted to a desired degree.

REINFORCED CONCRETE. Concrete containing steel reinforcement, and designed and detailed so that the two materials act together in resisting forces.

REINFORCEMENT, DEFORMED. Round deformed reinforcing steel conforming to NZS 3402P.

REINFORCEMENT, MESH. Welded mesh formed of hard-drawn round steel bars, welded at intersections, conforming to NZS 3422.

REINFORCEMENT, PLAIN. Round, smooth reinforcing steel conforming to NZS 3402P.

SHRINKAGE. Time dependent compressive strain of concrete, resulting from loss of moisture to the surrounding environment.

SPECIFIED COMPRESSIVE STRENGTH 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.

SUPPORT STRUCTURE. A structure supporting a liquid retaining structure at a required height above grade.

SWELLING. Tensile strain of concrete resulting from absorption of moisture.

TANK. Structure designed for the containment and storage of liquids.

TENDON. Steel elements such as wire, cable, bar, rod, or strand used to impart prestress to concrete when the element is tensioned.

UNBONDED TENDONS. Tendons which are not bonded to the concrete either directly or through grouting.

WATERSTOP. Continuous impervious membrane placed across a construction joint to inhibit moisture flow through the joint.

COMMENTARY

C2.1
Design method

Ultimate strength design methods are considered to be of doubtful relevance to design of concrete storage tanks, as standard load factors specified in NZS 4203 were developed without considering the special types and intensities of loading to which concrete storage tanks are subjected. Serviceability of the structure under the design loads is of paramount importance. A working stress approach is therefore required. Provided allowable stress levels are not exceeded under the service load combinations of 2.3.2 of this Code, ultimate behaviour should be satisfactory.

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 fixity, the implications of which may need to be assessed. Because the effects of edge restraint are of fundamental importance to tank design, detailed information can usually be found in almost any publication on tank design. BS 5337 is a suitable reference on these matters.

C2.2
Design loads

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 and shrinkage.

C2.2.1
Dead load

Dead load definition differs from that given in NZS 4203 which includes such things as earth pressure, temperature effects and construction loads. This Code deals with these aspects separately.

C2.2.3
Backfill loading

Earth pressures should take into account the distribution and characteristics of the backfill (symmetrical or asymmetrical) and should be determined by rational methods of soil mechanics based on foundations and soil investigations.

2
DESIGN CONSIDERATIONS

2.1
Design method

Tank design shall be based on elastic analysis methods and shall take into account effects of all loads and conditions of edge restraint at wall junctions with floor and roof. Maximum stresses shall not exceed allowable working stresses given in sections 5, 6 or 7 as appropriate.

2.2
Design loads

A number of loadings and load combinations need to be considered in the design of a tank. In general, the nature of the loads and their intensity should be as prescribed by NZS 4203. This section deals with special considerations peculiar to tanks.

2.2.1
Dead load (D)

Dead load is the self weight of the structure including that of equipment and appurtenances.

2.2.2
Live load (L)

Live loads are the superimposed loads such as snow, people, vehicles, plant etc. and shall conform to the requirements of NZS 4203.

2.2.3
Backfill loading (EP)

Backfill loading is the external pressure exerted by backfill, including the effects of surcharge and water table.

C2.2.4
Construction loads
Examples of construction loads are the stacking, lifting and propping of precast panels.

C2.2.5
Liquid load
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 class B design load.

C2.2.6
Prestress
In circular tanks circumferential prestressing forces are sometimes used to offset content pressures. In domed tanks, circumferential prestress may be applied at the dome ring to counterbalance the horizontal component of the thrust due to dome dead and live load.

Vertical prestressing may be provided to counteract bending stresses in the vertical direction.

C2.2.7
Wind
In general the effects of wind load on ground supported tanks are not severe and will not govern design. The possibility of suction over the entire shell surface should be recognized in the design of dome roofs.

C2.2.8
Temperature

C2.2.8.1
Wall
The temperature distributions of fig. 2.1 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.

Design tables have been developed[2.1] (and included as Appendix A) suitable for the common range of circular tanks. Thermal stresses are presented in the form:

$$f = CE_c \alpha \theta \quad \text{................................... (Eq. C2.1)}$$

where C is a coefficient dependent on the shape factor $H^2/2at$, position up 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 F_T to account for the reduced section rigidity that accompanies cracking. The amount of reduction

2.2.4
Construction loads (C)
Construction loads are temporary loads resulting from equipment, materials and methods to be used during construction.

2.2.5
Liquid load (F)
Liquid load is the internal static pressure exerted by the stored contents when filled to the overflow level.

2.2.6
Prestress (P)
This is the prestressing force provided to counteract forces and deformation induced by other loads.

2.2.7
Wind (W)
Maximum design loading for wind shall conform to the requirements of NZS 4203.

2.2.8
Temperature (T)

2.2.8.1
Wall
Except as required by 2.2.8.3 and in the absence of thermal analysis based on known local meteorological conditions, the wall shall be designed for the temperature distributions defined by fig. 2.1.

depends on the extent of tension stiffening in the concrete, the reinforcing content and the degree of in-plane force.

A method of evaluating F_T for reinforced concrete design is described in section 5 and the implications for partially prestressed design discussed in section 6. There is no reduction ($F_T = 1.0$) for fully prestressed design as the concrete is assumed to remain uncracked.

Temperature moments and axial forces are given by:

$$M_T = F_T \ \frac{(f_i - f_o)}{2} \ \frac{.t^2}{6} \ \dots\dots\dots \ (Eq. \ C2\text{-}2)$$

$$C_T = F_T \ \frac{(f_i + f_o).t}{2} \ \dots\dots\dots \ (Eq. \ C2\text{-}3)$$

where f_i and f_o are the total temperature stresses on the inside and outside surfaces respectively for an uncracked wall as shown in fig. C2.1. A tank designed on the basis of a cracked section should be checked under the stresses resulting from the application of the moment and force given by equations C2.2 and C2.3. Note that direct factoring of the stresses f_i and f_o will not produce the same results.

Appendix A gives tables for thermal stress for various base and temperature conditions.
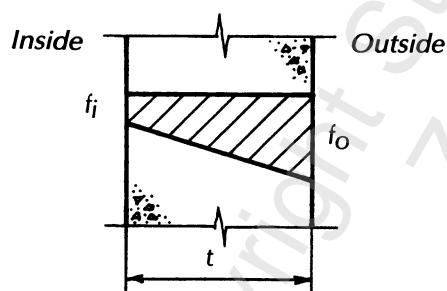
Fig. C2.1
THERMAL STRESSES

Tank full

Tank empty
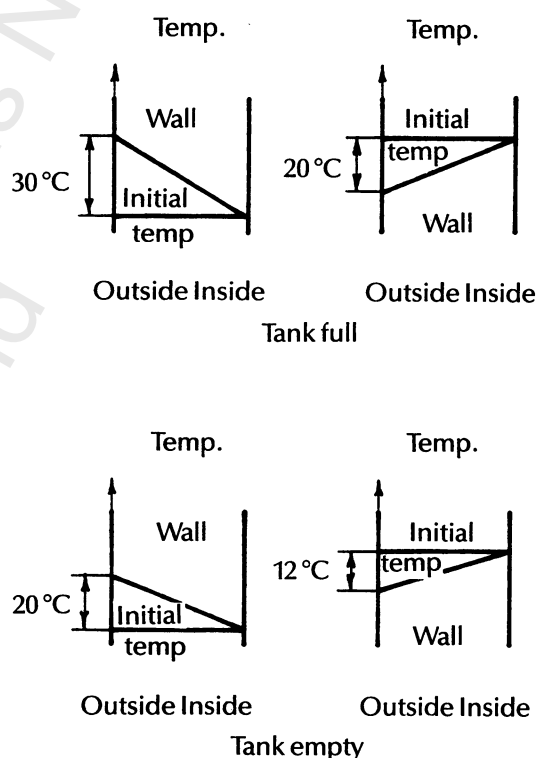
Fig. 2.1
TEMPERATURE DISTRIBUTIONS IN TANK WALLS

C2.2.8.2
Roof

Temperature effects on the roof are in general small and of little significance unless the roof is cast monolithically with the wall.

In snow regions, appropriate consideration should be given to the effects of a reverse temperature gradient (outside colder than inside). In the absence of specific information, a reverse gradient of 10 °C should be adopted.

C2.2.8.3
Special structures

The evaluation of the thermal response of concrete tanks subjected to unusual temperature conditions may require sophisticated heat flow analyses. A description of the procedures involved in such analyses is given in reference (2.1). 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 thermal stresses [2.2].

C2.2.9
Earthquake

The earthquake analysis should include the inertia forces generated by the horizontal acceleration of the structure itself and the hydrodynamic forces generated by the horizontal acceleration of the contained liquid. The hydrodynamic pressure of the contained liquid can be considered to consist of two components: the "impulsive" (inertia) pressure caused by the portion of the liquid accelerating with the tank and the "convective" pressure caused by the portion of liquid oscillating in the tank.

Consideration should also 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 1971 San Fernando earthquake, the damage apparently caused by movement of the surrounding ground. Earthquake earth pressures for which tanks are to be designed are given in reference (2.3).

C2.2.9.2
Horizontal earthquake force

For horizontal convective and inertia earthquake coefficients C_C and C_I respectively, see 2.2.9.3.

2.2.8.2
Roof

Allowance shall be made for stresses and movements resulting from:

(a) A ±20°C variation in the mean temperature
(b) A linear temperature gradient of 50°C per 100 mm of roof thickness (outside hotter than inside).

2.2.8.3
Special structures

A structure containing heated or cooled fluids shall be subject to a special study to establish the range of temperature conditions appropriate for design.

2.2.9
Earthquake (E)

2.2.9.1
General

The structure shall be designed for the forces, shears and moments resulting from earthquake accelerations of liquid mass, dead mass and external mass responding with the structure.

2.2.9.2
Horizontal earthquake force

Unless calculated on the basis of a more rigorous analysis, the total horizontal earthquake force,

W_C and W_I are the weights of the convective and impulsive contents respectively and are given in fig. C2.2 for both circular and rectangular tanks.

W_S is the weight of the tank shell (including roof if connected to walls).

V_H, exerted on the tank, shall be calculated in accordance with 2.2.9.4 by a combination of two components; a convective component V_C and an inertia component V_I where:

$$V_C = C_C W_C \quad\quad\quad\quad\quad\quad\quad\quad\quad\quad (Eq.\ 2\text{-}1)$$

and

$$V_I = C_I(W_I + W_S) \quad\quad\quad\quad\quad (Eq.\ 2\text{-}2)$$
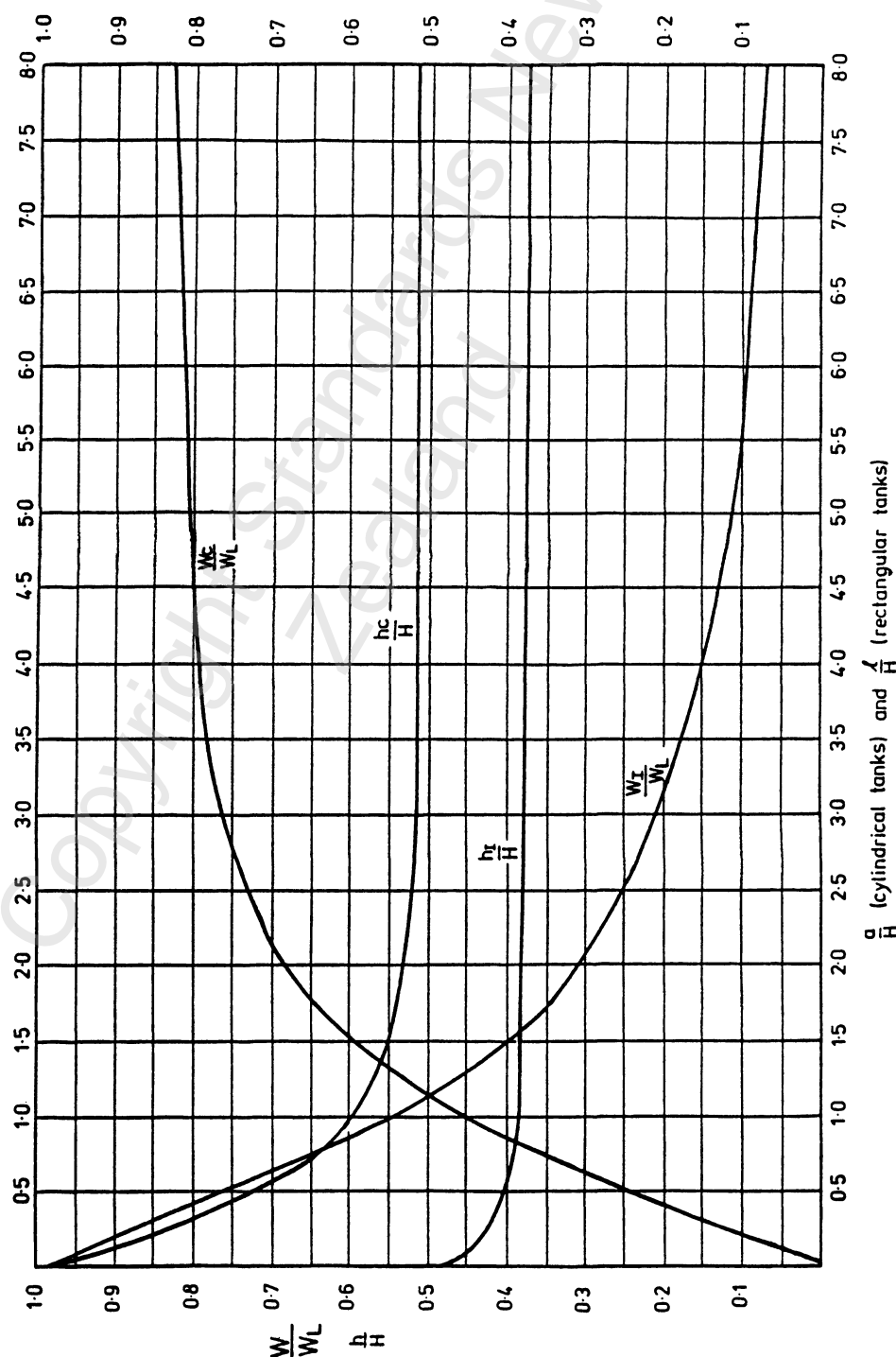
Fig. C2.2
EQUIVALENT WEIGHTS OF CONVECTIVE AND IMPULSIVE CONTENTS
(CIRCULAR AND RECTANGULAR TANKS)

C2.2.9.3
Horizontal seismic coefficient
(a) General
Convective and inertia earthquake coefficients should be determined from the revised basic acceleration response spectra given in NZS 4203, taking into account peak ground acceleration, period of vibration, geographical location and the appropriate ductility and damping factors. (In the absence of the loadings code revisions, data recommended by the NZNSEE Study Group on Seismic Design of Storage Tanks [2.4] has been substituted). Alternatively, the earthquake coefficients may be determined from response spectra established for the specific tank site taking into account the dynamic characteristics of the tank.

NOTE – It was originally envisaged that the latest revision of NZS 4203 would be completed before this Code was published. Unfortunately this has not happened and it has been necessary to include interim data to facilitate the calculation of earthquake coefficients.

(b) Convective coefficient C_C
Equation 2-3 has been derived from the average of spectra for soft rock and deep alluvium recommended by the NZNSEE Study Group,[2.4] using the following assumptions:

(i) $T_C > 1.0$ second.
(ii) ½ % viscous damping.
(iii) Elastic response.

The period of vibration of the convective mode, T_C, can be determined from fig. C2.3 for both rectangular and circular tanks.

(c) Peak ground acceleration
The peak ground acceleration is a function of the intensity of the design earthquake (earthquake for which the tank is to remain functional). This intensity varies with geographical location and is generally identified in terms of an annual probability of exceedance. To be consistent with NZS 4203, the intensity of the design earthquake is scaled in accordance with the appropriate seismic risk factor R. This risk factor is as defined in NZS 4203 with values and categories appropriate for tank design given in table C2.1 along with the implied probability of exceedance for each category.

2.2.9.3
Horizontal seismic coefficient
(a) General. Unless a more rigorous analysis is undertaken, the basic horizontal earthquake coefficients for the tank and its contents shall be determined in accordance with 2.2.9.3(b) and (d)
(b) Convective coefficient C_C. The convective earthquake coefficient shall be determined as a function of peak ground acceleration A_O and natural period of the first mode of sloshing, T_C, in accordance with:

$$C_C = \frac{2.7\,A_O}{T_C{}^{1.4}} \quad\text{.......................} \quad \text{(Eq. 2-3)}$$

(c) Peak ground acceleration.
Peak ground acceleration shall be as given in table 2.1.
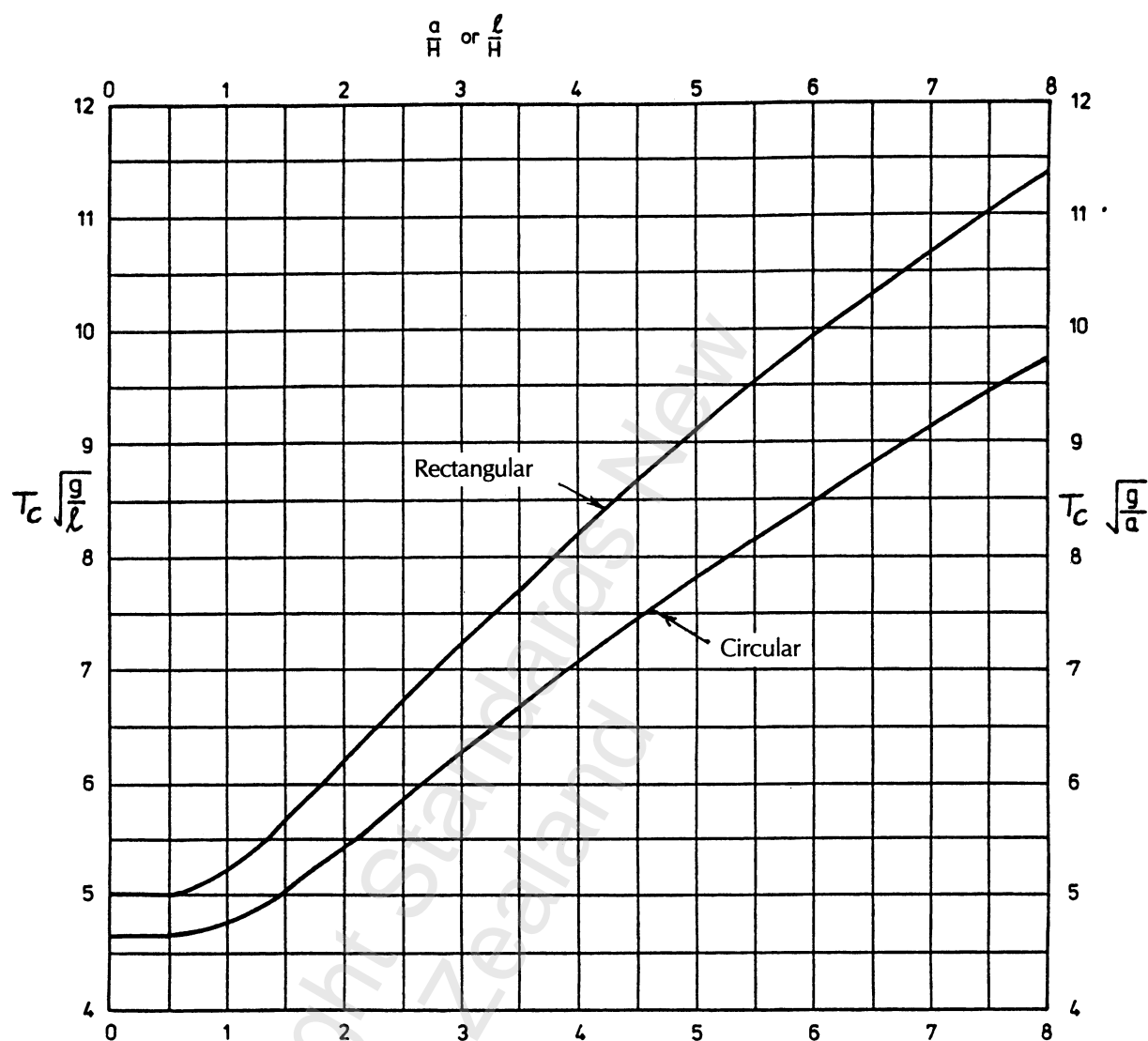
Fig. C2.3
DIMENSIONLESS PERIOD FOR FUNDAMENTAL SLOSHING MODE

Table C2.1
RISK FACTORS AND IMPLIED ANNUAL PROBABILITY OF EXCEEDANCE OF DESIGN EARTHQUAKE

Category	Description	Risk factor R	Annual probability of exceedance
1	Tanks containing highly valuable or hazardous contents, for example, emergency services or toxic chemicals	2.0	.0004
2	Tanks which are intended to remain functional in the emergency period for a major earthquake, for example, firefighting water	1.6	.001
3	Tanks which should be functional in the restoration period for a major earthquake, for example, potable water	1.3	.003
4	Tanks which should be functional in the restoration period for a moderate earthquake	1.0	.01
5*	Tanks which should be functional after a minor earthquake, for example, farm tanks	0.7	.05

sk category 5 is specific in this Code and not part of the NZS 4203 general classifications.

(d) Inertia coefficient C_I

Equation 2-4 states that the inertia earthquake coefficient is equal to the peak ground acceleration, which is also the zero period ordinate on the response spectrum. This assumption is realistic for a circular concrete tank, which because of its stiffness, has a very low period of oscillation. Rectangular tanks are generally more flexible than circular tanks which in some instances may lead to structural amplification effects. In lieu of the revised NZS 4203 response spectrum, the inertia coefficient for elastically responding flexible tanks may be calculated from the following expressions:

For $0.05 \leq T_I < 0.13$ seconds

$$C_I = A_O (1 + 13T_I) \quad \text{...................... (Eq. C2-4)}$$

For $0.13 < T_I < 0.60$ seconds

$$C_I = 2.7 A_O \quad \text{...................... (Eq. C2-5)}$$

For $0.6 \leq T_I < 10$ seconds

$$C_I = \frac{1.6 A_O}{T_I} \quad \text{...................... (Eq. C2-6)}$$

Equations C2-4 to C2-6 have been derived from the NZNSEE recommended response spectrum[2.4] using the following assumptions:

(i) Elastic response ($\mu = 1$).
(ii) 2% viscous damping.

Table 2.1
PEAK GROUND ACCELERATION

Seismic zone*	Peak ground acceleration
A	0.4R
B	0.3R
C	0.2R

* As defined in fig. 4 of NZS 4203.

(d) Inertia coefficient C_I. For rigid tanks with fundamental period of oscillation, T_I, less than 0.05 seconds, C_I is given by:

$$C_I = A_O \quad \text{...................... (Eq.2-4)}$$

For flexible tanks ($T_I > 0.05$ seconds) C_I values shall be in accordance with the basic coefficient given in NZS 4203 taking into account the period of vibration, geographical location and appropriate values of damping and ductility of the tank and its support structure.

Where the tank is supported by a ductile sup-porting system detailed for ductility in accor-dance with the appropriate material code, C_I should be taken as the basic coefficient, C_d, app-licable for the structural type and material of the support in accordance with NZS 4203.

The period of vibration of the inertia mode, T_I, should include the effects of flexibility from wall deformations. An approximate method of calcula-ting T_I for a rectangular concrete tank is given in Appendix B.

For an elevated tank, T_I is significantly influenced by the flexibility of the support structure. A method of calculating the period of vibration for the inertia mode for an elevated tank is given by Housner[2.5].

C2.2.9.4
Combinations of horizontal components
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.

For a flexible structure, for example, an elevated tank, it is possible for the response periods for both inertia and convective components to be of the same order of magnitude. Again the square root of the sum of the squares method of combi-nation better simulates the reduced probability of coincidence of peak inertia and convective responses.

C2.2.9.5
Earthquake pressures
Circumferential earthquake pressure distribution can be represented by a sinusoidal variation for a circular tank (see fig. C2.4(a)). Vertical earthquake pressure as presented by Jacobsen[2.6] and Housner[2.7] are of the form shown by the 'exact' curves in fig. C2.4(b). The equivalent linear distri-butions, although a simplification of the actual distribution, are sufficiently accurate for design purposes, and form the basis for equations 2-6 and 2-7. Stresses can be calculated using standard design charts.

2.2.9.4
Combinations of horizontal components
Inertia and convective responses shall be com-bined as follows:

$$V_H = \sqrt{V_C^2 + V_I^2} \qquad \text{(Eq. 2-5)}$$

2.2.9.5
Earthquake pressures
In the absence of a more rigorous analysis which takes into account the exact and complex vertical and horizontal variations in hydrodynamic pres-sures, the tank shall be designed for a horizon-tally uniform pressure distribution that varies lin-early from p_t at the surface of the liquid to p_b at the base,

where

$$p_t = \frac{V_H (6h-2H)}{\beta H^2} \qquad \text{(Eq. 2-6)}$$

and

$$p_b = \frac{V_H (4H-6h)}{\beta H^2} \qquad \text{(Eq. 2-7)}$$

where β is a constant whose value depends on the cross-sectional shape of the tank;

for a circular tank $\beta = \pi a$.
for a rectangular tank $\beta = 2b$.
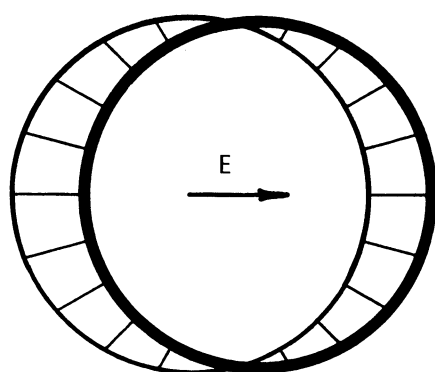
(a) Circumferential
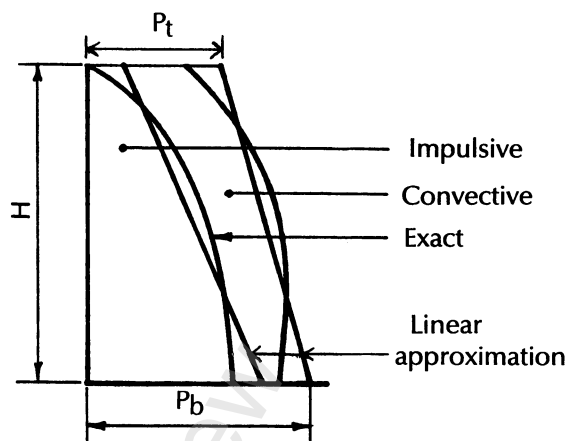
(b) Vertical

Fig. C2.4
EARTHQUAKE PRESSURE DISTRIBUTION IN CIRCULAR TANKS

The heights above the base of the centres of gravity of the convective and impulsive horizontal earthqake forces are obtained separately from fig. C2.2. Approximate pressure distributions may also be computed separately using the linear approximations given by equations 2-6 and 2-7 and shown in fig. C2.4. Tank hoop forces and vertical bending moments are computed separately and the combined (convective plus impulsive) forces are computed using the square root of the sum of the squares of the separate components. Normally tank inertial forces are combined with fluid inertial forces. This approach involves the approximation that the fluid height and the wall height are of the same order. For partially filled tanks, the forces due to impulsive response of the tank and liquid should be calculated separately.

C2.2.9.6
Overturning moment
(a) *Wall. The overturning moment acting at the base of the wall is conveniently expressed in terms of an equivalent weight (W_I, W_C, W_S) and an equivalent height (h_I, h_C, h_S) where subscripts I, C and S refer to inertia, convective and shell respectively. The variation of*

2.2.9.6
Overturning moment
(a) The overturning moment acting at the base of the wall shall be determined by:

$$M_W = \sqrt{(C_IW_Ih_I+C_IW_Sh_S)^2+(C_CW_Ch_C)^2} \quad\ldots\ldots \text{(Eq. 2-8)}$$

these parameters with the a/H ratio (circular) or ℓ/H ratio (rectangular) is shown in fig. C2.2.

(b) *Floor.* In order to design the tank foundation (or support structure) it is necessary to know the moment M_B arising from the hydro-dynamic pressures acting on the base. The variation of this moment with the ratio a/H (circular) or ℓ/H (rectangular) is shown in fig. C2.5. The moment on the base has been plotted as a dimensionless moment M_B defined by:

(i) *Circular tank*

$$M'_{BI} = \frac{M_{BI}}{C_{I\gamma L}\, a^2 H^2} \quad \ldots\ldots\ldots\ldots \text{(Eq. C2-7)}$$

$$M'_{BC} = \frac{M_{BC}}{C_{C\gamma L}\, a^2 H^2} \quad \ldots\ldots\ldots\ldots \text{(Eq. C2-8)}$$

(ii) *Rectangular tank*

$$M'_{BI} = \frac{M'_{BI}}{2C_{I\gamma L}\, \ell b H^2} \quad \ldots\ldots\ldots\ldots \text{(Eq. C2-9)}$$

$$M'_{BC} = \frac{M_{BC}}{2C_{C\gamma L}\, \ell b H^2} \quad \ldots\ldots\ldots\ldots \text{(Eq. C2-10)}$$
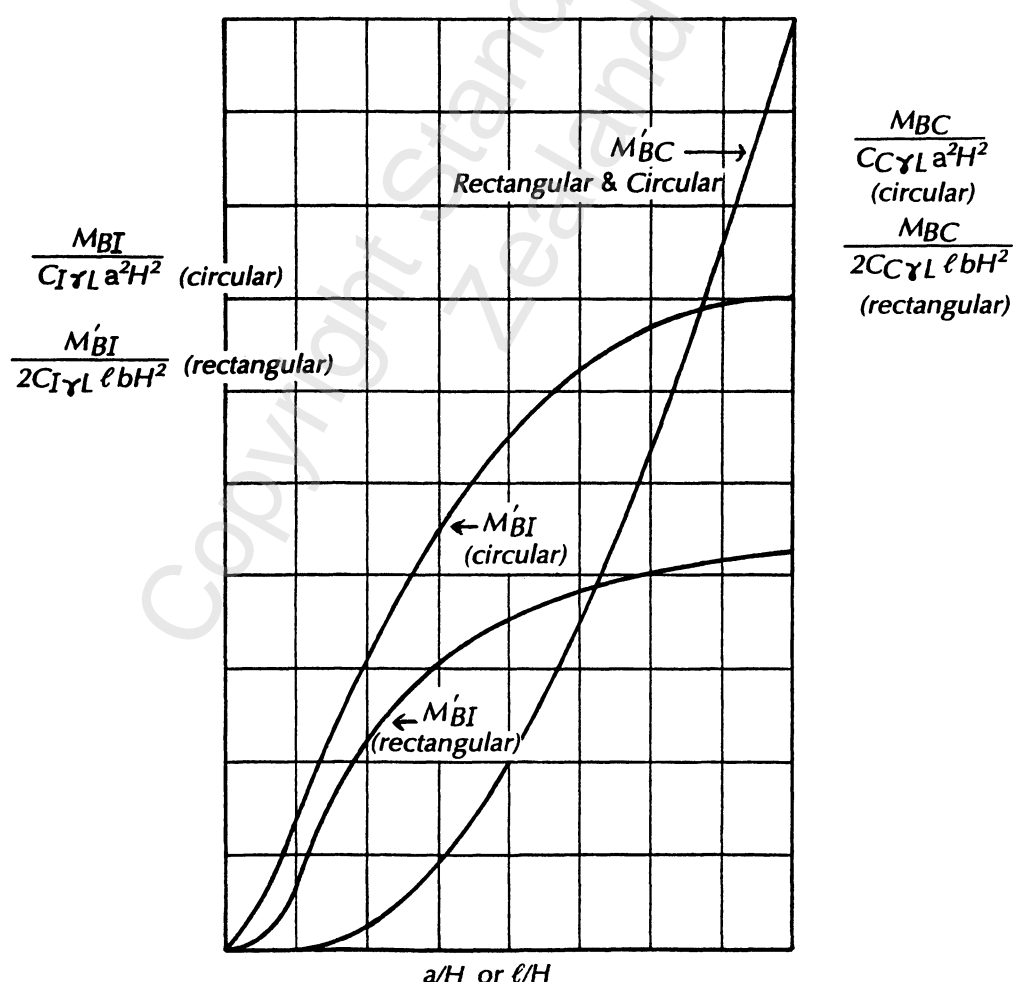
(b) The overturning moment acting on the floor of the tank shall be determined by:

$$M_B = \sqrt{M_{BI}^2 + M_{BC}^2} \quad \ldots\ldots\ldots\ldots \text{(Eq. 2-9)}$$

(c) The total overturning moment acting on the foundation or support structure is given by:

$$M = \sqrt{(M_{WI}+M_{BI})^2 + (M_{WC}+M_{BC})^2} \quad \ldots\ldots \text{(Eq. 2-10)}$$

Fig. C2.5
IMPULSIVE AND CONVECTIVE MOMENTS ON BASE (CIRCULAR AND RECTANGULAR TANKS)

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C2.2.9.7
Vertical earthquake acceleration

The effect on 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 earthquake coefficients given in table 2.2 correspond to the peak horizontal accelerations for the zone and magnitude of earthquake considered, but reduced by a factor of 0.67.

C2.2.9.8
Combination of horizontal and vertical responses

The stresses from peak horizontal and vertical ground accelerations are combined by their root mean square to account for the reduced probability of their concurrence. E_H is the stress caused by the horizontal component of earthquake acceleration. By way of explanation, considering the example of an upward ground acceleration of 0.25 g, the incremental stress E_V is equal to 0.25f where f is the stress caused by the static containment pressure. Then combined stress is:

$$E = \sqrt{E_H^2 + (0.25f)^2} \quad \text{.................. (Eq. C2-11)}$$

C2.2.9.9
Shear transfer

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:

$$Q = 0.8 V_H \quad \text{................................. (Eq. C2-12)}$$

To transmit this shear Q, a shear flow q is required at the wall/footing interface where:

$$q = \frac{Q \sin\delta}{\pi a} \quad \text{.............................. (Eq. C2-13)}$$

The distribution is illustrated in fig. C2.6.

2.2.9.7
Vertical earthquake acceleration

The vertical earthquake coefficient shall be as given in table 2.2.

Table 2.2
VERTICAL EARTHQUAKE COEFFICIENT

Seismic zone*	Vertical seismic coefficient C_V
A	0.27R
B	0.20R
C	0.13R

* As defined in fig. 4 of NZS 4203.

2.2.9.8
Combination of horizontal and vertical responses

Concurrence of horizontal and vertical motions shall be considered. Stresses shall be combined according to:

$$E = \sqrt{E_H^2 + E_V^2} \quad \text{....................... (Eq. 2-11)}$$

2.2.9.9
Shear transfer

Earthquake shear forces shall be considered in the design of wall to footing and wall to roof joints.
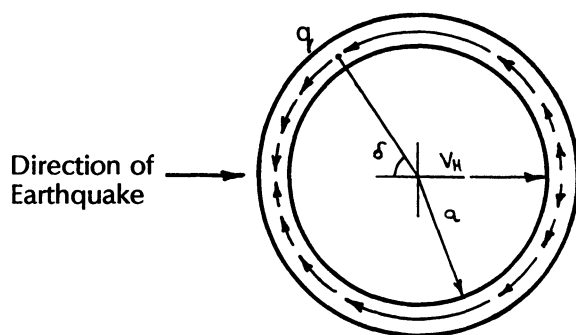
Fig. C2.6
MEMBRANE SHEAR DISTRIBUTION

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

$$q_{max.} = \frac{Q}{\pi a} = \frac{0.8 V_H}{\pi a} \quad \text{.................. (Eq. C2-14)}$$

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 galvanized 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 fig. C2.7.
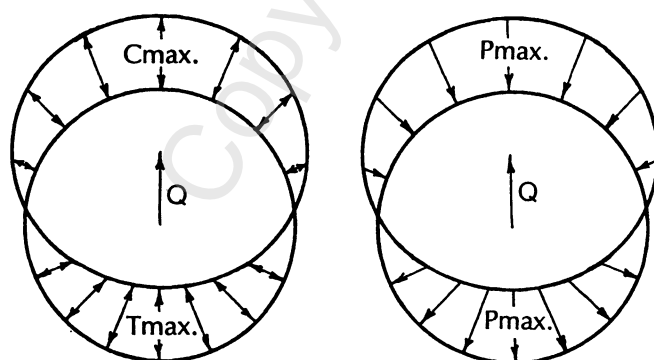
(a) Wall base hoop forces

$$C_{max.} \cdot T_{max.} \cdot \frac{Q}{\pi}$$

(b) Wall base reactions

$$P_{max.} = \frac{Q}{\pi a}$$

Fig. C2.7
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 fig. C2.6 with maximum shear given by:

$$q_{max.} = \frac{0.8\,F_R}{\pi a} \quad \dots\dots\dots\dots\dots\dots \text{(Eq. C2-15)}$$

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 fig. C2.7(b) but reacting on only half of the circumference. The maximum reaction force will be given by:

$$P_{max.} = \frac{2\,F_R}{\pi a} \quad \dots\dots\dots\dots\dots\dots \text{(Eq. C2-16)}$$

C2.2.9.10
Freeboard
The horizontal earthquake acceleration causes the contained fluid to slosh with vertical displacement of the fluid surface. The maximum vertical displacement, $d_{max.}$ may be calculated from the following expressions for circular and rectangular tanks.

(a) Circular tank

   $$d_{max.} = 0.84\,aC_C \dots\dots\dots\dots\dots \text{(Eq. C2-17)}$$

(b) Rectangular tank

   $$d_{max.} = \ell C_C \dots\dots\dots\dots\dots\dots \text{(Eq. C2-18)}$$

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), where overtopping may result in scouring of the foundation materials or cause damage to pipes and/or roof, then provisions should be made by:

   (i) Freeboard allowance, and/or
   (ii) Designing the roof structure to resist the resulting uplift pressures.

C2.2.10
Shrinkage and swelling

C2.2.10.1
Walls
The shrinkage and swelling strains given in table 2.3 were derived using the CEB-FIP (1978) Model Code, except that predicted shrinkages were doubled in accordance with the recommendations of reference (2.8).

2.2.9.10
Freeboard
Provision shall be made to accommodate the maximum wave oscillation generated by earthquake acceleration.

2.2.10
Shrinkage (S_h) and swelling (S_w)

2.2.10.1
Walls
In the absence of a rational analysis to determine shrinkage and swelling strains appropriate to the expected construction/loading history, the tank walls shall be designed for the shrinkage and swelling strains given in table 2.3.

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:

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

where ϕ 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: $e^{-\phi\infty}$ where $\phi\infty$ is the long term creep factor.

For tanks 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 the swelling rate is much faster than the shrinkage rate and hence in the short term less affected by creep relaxation. (The swelling strains given in table 2.3 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 *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 2.3.

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

Table 2.3
SHRINKAGE AND SWELLING STRAINS (CREEP ADJUSTED)

Wall thickness (mm)	Strain (x 10⁻⁶)			
	Shrinkage (ε_{sh})		Swelling (ε_{sw})	
	Precast	In situ	Precast	In situ
100	70	120	300	250
125	55	105	265	215
150	50	85	205	170
175	50	75	175	150
200	45	70	160	135
225	40	65	145	120
250	35	60	135	110

shrinkage gradients between the inside and outside faces of sections of the walls 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:

$$T = \frac{\varepsilon_s}{\alpha} \quad\text{..................................... (Eq. C2-19)}$$

In many practical cases it will be found that shrinkage induced stresses in the combinations specified in 2.3 will not control design.

C2.2.10.2
Roof
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.

C2.2.11
Non-symmetric loads for circular tanks
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.

C2.3
Load combinations

C2.3.2
Transient loads that should be omitted, if beneficial, are earth pressure (EP), shrinkage (S_h), swelling (S_w) and temperature (T).

The prestress force may vary between $P_{max.}$ and $P_{min.}$, the maximum and minimum due to in-time losses respectively. To ensure that the more adverse condition is incorporated in design, both $P_{max.}$ and $P_{min.}$ 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.

2.2.10.2
Roof
The roof shall be designed for the shrinkage and swelling strains given in table 2.3.

2.2.11
Non-symmetric loads for circular tanks
For thin walled circular tanks (t/a < 0.03) non-symmetric loads may be considered to be rotationally symmetric and equal to the value at the section under consideration.

2.3
Load combinations

2.3.1
Structures and members shall be designed in accordance with the allowable stresses to resist the loading combinations specified in 2.3.2 as applicable.

2.3.2
The loads described in 2.2 shall be combined in groups as defined below. In any group, if a worse effect is obtained by omitting one or more of the transient items, this case shall also be considered.

(a) Group A loads

wall $A = D + EP + P + (S_h \text{ or } 0.5\,S_w)$ .... (Eq. 2-12)
$A = D + F + EP + P + 0.5\,S_w$ .... (Eq. 2-13)
roof $A = D + P + S_h$ .... (Eq. 2-14)

Load case 2-12 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.

(b) *Group B loads. Group B load cases are permanent loads plus infrequent combinations of transient loads.*

Load case 2-15 specifies the basic load combination for seismic loads, and is compatible with the 'alternative method' loading specified by NZS 4203. However, in some cases, notably squat rectangular concrete tanks where the load is predominantly carried by one action, such as vertical bending, it may be that the ultimate strength can be predicted with accuracy. In such cases, the tank could be designed for an ultimate load combination of:

$$U = D + F + EP + P + E + 0.5\,S_w \quad (Eq.\ C2\text{-}20)$$

and section strength based on normal ultimate strength equations required by NZS 3101.

Where fluid pressure loading in concrete tanks is carried by a combination of membrane action and vertical bending, the proportion of load carried by each of the mechanisms depends on the tank geometry and their relative stiffness. Although these are well established for elastic response, the influence of cracking as the ultimate condition is approached will alter the proportion of load carried by the two mechanisms. Because of the difficulty in analysis, and the inappropriateness of elastic actions to the inelastic stage in this case, most storage tanks will need to be designed to the load combination of equation 2-15 using the stress levels permitted by the appropriate section of this Code.

Load case 2-17 applies equally to shrinkage and swelling; shrinkage – tank empty prior to filling, swelling – tank empty for maintenance.

The earthquake component in load case 2-21 refers to the pressure exerted on the roof by sloshing of the contained liquid.

(b) Group B loads

wall
$$B = D + F + EP + P + 0.8E + 0.5\,S_w \quad (Eq.\ 2\text{-}15)$$
$$B = D + F + EP + P + 0.7\,S_w + T \quad (Eq.\ 2\text{-}16)$$
$$B = D + EP + P + T + (0.7S_h\ or\ 0.35S_w) \quad (Eq.\ 2\text{-}17)$$

roof
$$B = D + L + P + T \quad (Eq.\ 2\text{-}18)$$
$$B = 0.8D + W + P \quad (Eq.\ 2\text{-}19)$$
$$B = D + P + T + (0.7S_h\ or\ 0.7S_w) \quad (Eq.\ 2\text{-}20)$$
$$B = 0.8D + 0.8E \quad (Eq.\ 2\text{-}21)$$

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NOTES

COMMENTARY

C3.1
General
Guidance is given in this section on the materials to be used in construction of concrete structures for the storage of liquids, and on their properties to be used for design purposes.

C3.2
Concrete

C3.2.2
The comparatively high strengths and low water/cement ratios specified for concrete are to ensure high quality dense concrete with low shrinkage, to minimize cracking and permeability.

C3.2.4
Minimum cement contents are specified to ensure dense durable concrete with low permeability. Maximum cement contents are specified in order to minimize shrinkage, and shrinkage cracking in thin sections, and heat-of-hydration cracking in thick sections. The limits correspond to those specified by BS 5337 for concrete of 20mm maximum aggregate size. Note that cement contents approaching the upper limit for prestressed concrete should only be adopted for very high strength concrete (50-60 MPa). Concrete roofs are specifically included, because condensation will mean the inside surface will be continuously wet. This conforms with the approach taken by BS 5337.

C3.2.5
Air entrainment reduces water demand and therefore reduces shrinkage. It also changes the nature of the voids between the particles in the concrete matrix from interconnected tubular voids to discrete spherical voids, thereby decreasing permeability. Pending further experience, waterproofing additives are not recommended for use in water retaining structures.

C3.2.6
Stresses induced in storage tanks by thermal loading and creep and shrinkage effects are directly proportional to the value of the modulus of elasticity, E, pertaining at the time the action is applied. Consequently it is important that realistic values

3
MATERIALS

3.1
General
This section applies to materials to be used in the construction of reinforced or prestressed concrete or cement mortar elements in liquid retaining structures.

3.2
Concrete

3.2.1
Mix constituents
All concrete shall comply with requirements of NZS 3109, and NZS 3104.

3.2.2
Compressive strength
The specified compressive strength of reinforced and prestressed concrete elements shall not be less than 35 MPa.

3.2.3
Water/cement ratio
The water/cement ratio of concrete used for liquid-retaining elements shall not exceed 0.50.

3.2.4
Cement content
For all concrete subject to continuous or frequent water contact or condensation, such as reservoir floors, walls and roof, the following limits to cement contents shall apply:

minimum cement content, reinforced or
    prestressed concrete: .... 320kg/m^3
maximum cement content, reinforced
    concrete: .... 400kg/m^3
maximum cement content, prestressed
    concrete: .... 550kg/m^3

3.2.5
Admixtures
Admixtures shall comply with NZS 3113. Air entrainment complying with the requirements of NZS 3104 or water reducing admixtures shall be used wherever practical for all concrete subject to continuous or frequent water contact, or condensation.

3.2.6
Modulus of elasticity
The design value of the modulus of elasticity of concrete shall be determined in accordance with provisions of NZS 3101, unless otherwise established by testing.

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 compression strength f'_C will normally exceed the specified strength by 20% or more. This will result in increased values of E_C when using the following equations, which are duplicated from NZS 3101, clause 3.3.4.

$$E_C = 0.043w_C{}^{1.5} \sqrt{f'_C} \text{ MPa} \quad \text{............ (Eq. C3-1)}$$

where $1400 \leq w_C \leq 2500$ is the concrete density in kg/m³

For normal weight concrete the value

$$E_C = 4700 f'_C \text{ MPa} \quad \text{...................... (Eq. C3-2)}$$

may be used.

C3.2.7
Thermal stresses in tanks are also proportional to the concrete coefficient of thermal expansion α_C which can vary between $5 \times 10^{-6}/°C$ and $15 \times 10^{-6}/°C$ depending primarily on aggregate type, with Andesites and Limestones giving lowest values and Quartzites typically giving highest values. Some typical values are listed in table C3.1. Limited data on New Zealand concretes is available in Reference (3.1).

Table C3.1
TYPICAL COEFFICIENTS OF THERMAL EXPANSION FOR WATER-CURED CONCRETE MADE FROM DIFFERENT AGGREGATE TYPES

Aggregate	Coefficient of thermal expansion $\times 10^{-6}/°C$
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.

C3.3
Cement mortar
Modulus of elasticity and coefficient of thermal expansion of cement mortars should be assessed on the behaviour of representatives testing wherever possible. Where characteristics of constituents are not known at the time of testing, E_C and

3.2.7
Coefficient of thermal expansion
The design value of the coefficient of thermal expansion shall be determined with due regard for the constituent materials to be used for the construction.

3.3
Cement mortar

3.3.1
Pneumatically placed mortar

should be assessed as for equivalent strength con-crete. It should, however, be recognised that because of high cement-aggregate ratios required for cement mortar structures, values for α_c will tend to be higher than for equivalent concretes. It is recommended that values listed in table C3.1 should all be increased by 1×10^{-6} for cement mortar as a consequence.

3.3.1.1

The aggregate for penumatically placed mortar shall, unless otherwise approved, comply with the requirements of NZS 3121 for sand, and in addition shall have a fineness modulus lying between the limits of 2.50 and 3.30.

3.3.1.2

Except for special works requiring other mixes, the proportions for pneumatic mortar shall be one part of cement to not more than four parts of moist sand (3 % to 5 % moisture content) volumetrically batched, or weigh-batched to equivalent proportions.

3.3.1.3

Compressive strength

The specified compressive strength of nominal 100 mm x 50 mm diameter cores taken from representative work panels shall not be less than:

(a) For reinforced concrete elements,
.... $f'_c = 25$ MPa

(b) For prestressed concrete elements,
.... $f'_c = 35$ MPa

The panels shall be fabricated, cured and cored as described in 3.3.1.4 and testing of cores shall be in accordance with NZS 3112: Part 2, section 9. Test results shall meet the requirements of NZS 3109.

3.3.1.4

Representative work panels of such number and at such times as the Engineer may specify shall be fabricated by gunning on to horizontal rigid plywood forms. These panels shall be approximately 300 mm square and 125 mm thick. Immediately after manufacture the panels shall be covered with impermeable plastics sheeting to prevent water loss, and shall be protected from direct sunlight. After not less than 16 hours they shall then be forwarded to the laboratory in a container which prevents damage or loss of moisture. The panels shall be stored in the laboratory at a temperature of 21 ±2 °C under moist conditions such that free water is maintained on their surfaces at all times. Between 3 and 7 days after manufacture the panels shall be cored. Three cores shall be taken from near the centre of each panel, and each set of three cores shall constitute a test sample. The cores shall remain in moist storage at 21 ±2 °C until tested at 28 days after forming the panel.

3.3.1.5

Modulus of elasticity

The design value of the modulus of elasticity of pneumatically placed mortar shall be determined in accordance with provisions of NZS 3101 for concrete of equivalent strength and density, unless otherwise established by testing.

3.3.1.6
Coefficient of thermal expansion
The design value of coefficient of thermal expansion shall be determined with due regard for the constituent materials to be used for the construction.

3.3.2
Hand-placed and mechanically-placed mortars

3.3.2.1
The aggregates for hand-placed and mechanically placed mortars shall comply with the requirements for sands for mortars and external rendering as specified in NZS 3103.

3.3.2.2
The proportions of hand-placed and mechanically-placed mortar shall be two parts of cement to not more than five parts of moist sand (3 % to 5 % moisture content) volumetrically batched, or weight-batched to equivalent proportions.

3.3.2.3
Compressive strength
The specified compressive strength of nominal 100 mm x 50 mm diameter cylinders moulded, cured and tested in accordance with NZS 3112 shall be not less than $f_c' = 25$ MPa. Test results shall meet the requirements of NZS 3109.

3.3.2.4
Modulus of elasticity
The design value of the modulus of elasticity of hand-placed or mechanically-placed mortar shall be determined in accordance with provisions of NZS 3101 for concrete of equivalent strength and density, unless otherwise established by testing.

3.3.2.5
Coefficient of thermal expansion
The design value of the coefficient of thermal expansion shall be determined with due regard for the constituent materials to be used for the construction.

3.4
Non-prestressed reinforcement

3.4.1
All hot rolled reinforcing steel and welded wire fabric shall comply with requirements of NZS 3402P, or NZS 3422 as appropriate.

3.4.2
Galvanized or zinc coated netting and twisted or loosely linked steel wire shall comply with BS 1485 or BS 4102, as appropriate.

3.4.3
Modulus of elasticity of hot-rolled reinforcement shall be taken as 200 GPa, unless otherwise established by testing.

C3.5
Prestressed reinforcement
Modulus of elasticity of prestressed reinforcement E_S is generally lower than the value of 200 GPa commonly used for normal strength reinforcement, and may be as low as 150 GPa. Values of E_S should be based on manufacturers' test results, or on results from independent testing.

3.5
Prestressed reinforcement

3.5.1
Prestressed reinforcement shall comply with the requirements for prestressing steel of NZS 3109.

3.5.2
Modulus of elasticity of prestressed reinforcement shall be established by tensile testing.

REFERENCES

(3.1) BOULT, B.F. Thermal Properties of Concrete, Report GLR15, New Zealand Concrete Research Association, April 1979.

NOTES

COMMENTARY

C4.1

The requirements set out in this section are those which are additional to the requirements of NZS 3109:1980 Specification for concrete construction. The additional requirements are related to the unusual service conditons of reservoirs and other water retaining structures and the need to provide watertight construction and durability.

C4.2.2
Construction joints

A construction joint is a joint in the concrete introduced for convenience in construction at which measures are taken to achieve subsequent continuity with no provision for further relative movement. Joints prepared in accordance with all of the requirements of NZS 3109 and with sufficient reinforcement through them to prevent all relative movement should perform satisfactorily. If complete continuity cannot be obtained the joint should be treated as a movement joint and taken into account in the design (see 2.1).

Construction joints should be located in positions selected by the design engineer and shown on the drawings. Typical applications for construction joints are in floor joints, and between successive lifts in a reservoir wall. Generally all joints should be either vertical or horizontal.

Waterstops are not usually required at construction joints but may be included at the discretion of the design engineer. Before a waterstop can be effective some relative movement (debonding) must take place first and strictly speaking such joints are movement joints.

A special case of construction joints is the infill between precast wall elements of circular prestressed reservoirs. It is not good practice to use dry pack mortar in the joint as uneven compaction can cause uneven bearing stresses on the joint faces. It is now common practice to make such joints with an in situ concrete infill placed in one lift using a super-plasticiser additive to aid placing. The dimensions of the infill should be sufficient for adequate placing of the concrete. The joint between the precast and in situ concrete

4
CONSTRUCTION REQUIREMENTS

4.1
General

4.1.1
This section applies to the construction of all reinforced and prestressed concrete liquid retaining structures covered by this Standard. The requirements for the construction of cement mortar structures are covered in section 7.

4.1.2
Except where otherwise specified the requirements of NZS 3109 also apply.

4.2
Concrete placing

4.2.1
The placing of concrete for water retaining structures shall be carried out with particular care but generally in accordance with the requirements of NZS 3109.

4.2.2
Construction joints
Construction joints shall be prepared as specified in NZS 3109.

should be planar and square to the panel face. Shear keys, grooves or other stress raisers should be avoided as they can lead to uneven stresses on the joint faces. Such details have been known to cause spalling to the precast element when pre-stress was applied.

Sometimes a small rectangular groove is formed on the waterface of the joint line and a sealant applied. .

C4.2.3
Movement joints
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 water-tightness 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 subfloor or blinding layer.*

The effectiveness of movement joints in controlling cracking depends on their correct location, which may be characterized 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. .

4.2.3
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 the loss of water.

Joints in floors. *The floor of a reservoir may be designed to permit shrinkage and thermal contraction by minimizing 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. This gap should not exceed 1m. Joints should be complete contraction joints except where allowance is made for expansion and an expansion joint provided.*

Strip casting and cutting to form contraction joints is also an acceptable method of shrinking control provided the concrete is cut before initial shrinkage cracking occurs. It should be noted that depending on mix design and weather conditions, this may be within a short time of placing.

Alternatively, the floor may be designed as fully restrained against shrinkage and thermal contraction. It should be cast direct on to the blinding concrete. 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 5337.*

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.*

Appendix D of BS 5337 discusses the problem and solution.

Recent experience has shown that polysulphide rubber compounds are attacked by chlorine and are not suitable for use in reservoirs containing treated water. An initial filling with a higher than usual chlorine content for sterilisation hastens the attack.

C4.3.1
Unbonded tendons
Unbonded tendons are not allowed because sufficiently reliable protective systems for the environment have not been developed for reservoirs in New Zealand.

4.3
Prestressing, stressing and grouting

4.3.1
Unbonded tendons
Unbonded tendons shall not be used in water retaining structures.

C4.3.2
Wound tanks

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 [4.1].

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 [4.1].

4.3.2
Wound tanks

Cylindrical tanks constructed using wound prestressing 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 not less than 25 mm.

REFERENCES

(4.1) ACI Committee report 344R-70, Design and Construction of Circular Prestressed Concrete Structures. ACI Committee 344, 1970, 16 pp. Also ACI Manual of Concrete Practice, Part 4, American Concrete Institute, Detroit.

COMMENTARY

C5.1.3
BS 5337 has severe requirements for water proofing of roofs but in New Zealand the atmosphere is cleaner and the worst potential contaminants are probably bird droppings. Reasonable protection should be provided by any reinforced concrete roof designed in accordance with this Code.

C5.2
Stiffness of cracked section
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 effect 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 F_t 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 F_t are plotted in figure C5.1 for a range of wall thickness and steel ratios.

5
DESIGN OF REINFORCED CONCRETE ELEMENTS

5.1
General

5.1.1
This section applies to the design of all unlined reinforced concrete elements of liquid retaining structures covered by this Standard that are in close proximity to the contained liquid, such as reservoir walls, floors and roofs.

5.1.2
The design of reinforced concrete elements of liquid retaining structures shall comply with the requirements of NZS 3101 Appendix B except as modified by the requirements of this Section.

5.1.3
For structures containing drinking water, roofs shall be waterproof and shall be graded so that they do not pond water.

5.2
Stiffness of cracked section

5.2.1
Except as provided by 5.2.2 strain induced forces (such as thermal and shrinkage) in the walls of circular reinforced concrete reservoirs may be calculated on the basis of an uncracked section and reduced by a factor F_t representing the local reduction in stiffness resulting from cracking in each direction.

5.2.2
A more rigorous approach to that given in 5.2.1 including an analysis of the cracked section stiffness everywhere in the wall may be used.

The I_{cr} values used in tabulating F_t 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 thermal/shrinkage stresses. However, unless allowable compressive stresses are exceeded in the concrete, this load case is unlikely to result in an adverse service condition.

The I_{cr}/I_g ratio 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 F_t 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 p, and with increasing moment level (after cracking). Approximate maximum figures are 100 % increase at p = 0.005 and 30 % increase at p = 0.02. Because of its significance, tension stiffening has been included in the derivation of the F_t values.

5.3
Maximum permitted concrete stresses

5.3.1
The allowable stresses in concrete for calculating the bending strength of the elements of the structure shall be as given in table 5.1

Table 5.1
MAXIMUM PERMITTED STRESSES IN REINFORCED CONCRETE

Load combinations	Compression stress
Group A combinations	$0.45\,f'_c$
Group B combinations	$0.60\,f'_c$
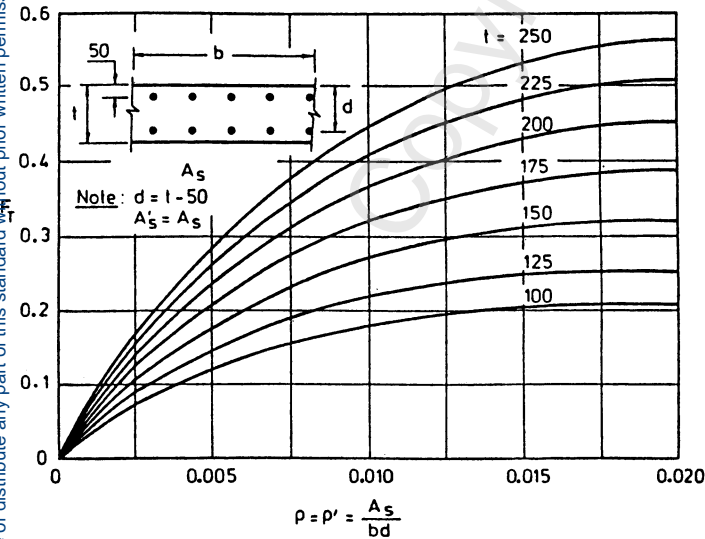
Fig. C5.1
REDUCTION OF STIFFNESS OF DOUBLY REINFORCED WALL ON CRACKING (INCLUDING TENSION STIFFENING EFFECT)

C5.3.2

NOTE – In the previous Code (NZS 3106P:Part 1:1978) the design philosophy adopted was that cracking was to be prevented by limiting steel and concrete tensile stresses. Thus it was necessary to calculate concrete tensile stresses based on a transformed section.

In this Code the design philosophy adopted is to recognize that cracks will inevitably occur and to limit their width by limiting steel stresses and more rigorously calculating stresses due to transient loads and secondary effects. Thus it is not necessary to calculate concrete tensile stresses and limits are not given in table 5.1 as cracking is assumed to occur.

C5.4.1

The permissible steel stresses given in table 5.2 are varied to suit the degree of exposure. For walls less than 225 mm thick the exposure on one face is deemed to affect the reinforcement on both faces. This is consistent with the provisions of BS 5337. However it is not as conservative as the provisions of ACI 350R – 77 "Concrete Sanitary Engineering Structures", which is intended for use with more aggressive liquids.

5.3.2

In stress calculations the tensile strength of concrete shall be ignored, except that, in respect of provisions of shear in flexural members, the concrete may be assumed to take diagonal tension for the purposes of Appendix B of NZS 3101.

5.4
Reinforcing steel stresses

5.4.1

Maximum permissible stresses for plain bars, deformed bars, and hard drawn steel mesh conforming to NZS 3422 shall be as given in table 5.2.

Table 5.2
MAXIMUM PERMITTED STRESSES IN REINFORCING STEEL FOR STRENGTH CALCULATIONS

		Steel stresses (MPa)			
Type of reinforcement		Deformed bars		Hard drawn steel mesh	
Load combinations		A	B	A	B
Members in direct tension		110	150	120	165
Members in bending	On liquid retaining face	110	150	120	165
	On face remote from liquid — Members less than 225 mm thick	110	150	120	165
	Members 225 mm or more thick	138	190	185	250
In shear reinforcement		110	150	120	165

5.5
Minimum reinforcement

5.5.1

The minimum reinforcement in each of two directions at right angles, in those parts of the structure not restrained from movement, shall have an area of not less than 0.3 % of the gross concrete section normal to that direction for plain round bars, 0.25 % for deformed bars, and 0.2 % for welded hard drawn steel mesh.

C5.5.2

The most common situation where partial restraint of shrinkage will occur is for a floor slab with an integral slab/foundation beam construction. Consider the case of a circular reservoir, where, as is generally the case, the foundation ring beam supporting the wall is cast first, with the slab cast later. Radial shrinkage of the slab is

5.5.2

For those parts of the structure partially restrained against shrinkage, the minimum ratio of reinforcement area to gross concrete area shall be based on rational analysis of the degree of restraint to shrinkage, with reinforcement stresses not exceeding those given in table 5.2, but shall not be less than that required by 5.5.1.

restrained by hoop compression in the foundation beam. The degree of restraint to slab shrinkage will depend on the relative stiffness of the slab in radial tension, and the ring beam in hoop compression.

A simple compatibility analysis will show that the shrinkage stress induced will be given by

$$f_s = \frac{1}{1 + K_R} \cdot E_{cs} \cdot \varepsilon_{sh} \quad \text{............... (Eq. C5-1)}$$

where ε_{sh} is the creep-compensated shrinkage strain, E_{cs} is the slab modulus of elasticity and

$$K_R = \frac{E_{cs} t_s R}{E_{cb} A_b} \quad \text{.................... (Eq. C5-2)}$$

is the ratio of slab to beam radial stiffness (t_s = slab thickness, R = radius to the slab/beam interface, E_{cb} = beam modulus of elasticity and A_b = cross-sectional area of beam).

The shrinkage strain ε_{sh} in equation C5-1 may be taken from 2.3 for a slab of effective thickness $2t_s$, since moisture loss will occur only from the top surface.

Stresses resulting from frictional restraint of the slab sliding on its sub-base must be added to the stress computed in equation C5-2. Note that since shrinkage occurs with the tank empty, the weight causing frictional restraint is the weight of the slab alone.

For a design separating the floor slab from the ring beam by a movement joint, the only restraint to shrinkage will be from frictional restraint to sliding.

Similar restraint of slab shrinkage will occur in rectangular tanks where floor slab and walls are monolithic. To avoid problems, it is adviseable for the floor slab to be cast at the same time, or before, the walls are cast. Occasionally corner pours in floors and restrained roof slabs are delayed to minimise shrinkage.

C5.5.3
All structures are subject to the effects of temperature change, creep and shrinkage. The contraction which results from shrinkage, from the dissipation of the heat of hydration of the cement, and from seasonal variations in temperature, can produce excessive tension in lightly reinforced restrained members. Thin sections such as walls and slabs, for example, are particularly susceptible to drying shrinkage, and must therefore contain certain minimum quantity of steel in any direction in which they are restrained if they are not to be rendered unserviceable by the formation of wide cracks.

All concrete expands after placing, due to the heat of hydration of the cement. This expansion is followed by a contraction as the concrete cools to the temperature of its surroundings. This contrac-

5.5.3
When hard drawn steel mesh is used, welded intersections shall not be further apart than 200 mm.

tion can produce tension in a restrained plain concrete member which exceeds the tensile strength of the concrete and thus produces a single wide crack. Such cracking can be controlled by the provision of a relatively small amount of reinforcement in the form of small-diameter bars or fabric. Mesh reinforcement is recommended for slabs less than 150 mm in thickness as bars 10 mm diameter or greater tend to act as crack inducers parallel to the bar.

C5.6.1
Reinforcement, especially at anchorage lengths and lapped splices, should be detailed in a manner that will minimise congestion which might inhibit the achievement of dense, void-free concrete. For this reason hooks and contact splices should not usually be used. Spliced bars which are separated by less than the greater of 1½ times the bar diameter or 40 mm should be considered to be contact splices requiring compliance with section 5 of NZS 3101. No more than one third of the bars at any cross section should be spliced within a length of 40 bar diameters unless special precautions are taken in accordance with section 5 of NZS 3101.

C5.7.1
The minimum covers specified are minimum requirements and should be increased for aggressive liquids or abrasive conditions.

5.5.4
In concrete sections of thickness 200 mm or greater, the steel shall be distributed between two layers, one near each face of the section.

5.5.5
Where two layers of reinforcement are required by 5.5.4, no one layer shall contain less than one third of the total required area in that direction.

5.6
Bond and anchorage

5.6.1
Bond and anchorage requirements shall be as set out in section 5 of NZS 3101.

5.7
Minimum cover to reinforcement

5.7.1
The minimum cover to any reinforcement shall be as specified in section 5 of NZS 3101 with additional requirements for faces in contact with liquid as follows:

(a) For surfaces in contact with liquid the minimum cover shall be 40 mm
(b) The underside of a roof shall also be designed as a liquid retaining face except that for a shell roof the cover may be reduced to a minimum of 25 mm in both faces
(c) For structures containing or surrounded by sewage, sewage sludge or seawater the minimum cover to any reinforcement shall be 50 mm
(d) For other liquids having a detrimental effect on the structure, approved precautions or protective coating shall be applied
(e) Where epoxy coated reinforcement is used covers may be reduced but shall be the subject of a special study.

C5.8
Floor slabs

Leaks in floor slabs can be undetected for a considerable period and in certain ground conditions even small flows can lead to severe damage. When designing floor systems the special conditions of the site should be considered. If risks and consequences warrant, then consideration should be given to providing a secondary collection system under floor slabs to give early warning of leakage. Drainage layers on a flexible membrane might provide such a collection system at a low cost.

C5.8.1

Site concrete shall not be considered as part of the structural thickness.

C5.8.2

Slabs should not be considered as unrestrained unless they are on a low friction surface and are not connected into perimeter beams or other footings. Restrained slabs should be provided with additional reinforcement to carry the retsraining forces as set out in 5.5.

C5.9
Roofs

C5.9.1

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.

C5.9.2

Some of the transient loading cases for roofs are different from those in walls. For example differential temperature effects are thought to be less severe than for the case of liquid in contact with the inside face and that is recognised in section 2.

Roofs constructed monolithically with the walls should be designed for the additional stresses due to differential shrinkage between the wall and the roof.

On large rectangular tanks with monolithic roofs the walls at the corners provide rigid constraint to shrinkage thereby inducing a severe loading condition.

It is sometimes considered worthwhile delaying the placing of the corner sections of rectangular roofs until some of the initial shrinkage has taken place.

C5.9.3

Equation 5-1 is based on the ACI 344R recommendations [5.1] to avoid the possibility of buckling of the shell. It is valid for domes with rise-to-span ratios between 1:6 and 1:10 and r/t less than 800. A factor of safety against buckling of 5 has been adopted in formulating equation 5-1. This is

5.8
Floor slabs

5.8.1

The minimum thickness of floor slabs shall be 125 mm which shall be placed in one layer.

5.8.2

Minimum reinforcement for unrestrained slabs shall be as set out in 5.5 and such reinforcement shall pass through all joints not specifically designed for movement.

5.9
Roofs

5.9.1

Reinforcement of concrete roofs shall be designed in accordance with the provisions of this section as though they were in contact with the retained liquid notwithstanding any air space below the roof soffit.

5.9.2

Design loads shall be as set out in section 2 of this Code.

5.9.3

The thickness of a dome roof shall be not less than

$$t = r\sqrt{\frac{20w}{E_c}} \quad \dots\dots\dots\dots\dots\dots\dots\dots \text{(Eq. 5-1)}$$

a relatively low factor of safety against buckling failure but is justifiable because the function of a dome is merely to cover a storage area [5.2].

5.9.4
The ring beam for a dome roof shall be prestressed or reinforced with sufficient tension capacity to carry the full horizontal component of the dome reaction without a contribution from hoop steel in the walls or roof.

REFERENCES

(5.1) ACI Committee report 344R-70, Design and Construction of Circular Prestressed Concrete Structures. ACI Committee 344, 1970, 16 pp. Also ACI Manual of Concrete Practice, Part 4, American Concrete Institute, Detroit.

(5.2) SCORDELIS, Alex C. Stability of Reinforced Concrete Domes and Hyperbolic Paraboloid Shells, ACI publication SP-67, 1981. Concrete shell buckling, Paper No. 3.

NOTES

COMMENTARY

C6.1
General
Design of prestressed concrete elements of liquid retaining structures will generally be governed by those clauses of NZS 3101 section 13 that refer to behaviour under service load conditions. In particular, reference should be made to NZS 3101 for requirements for estimating prestress losses, for anchorage design and for general serviceability requirements.

C6.2
Materials
Materials and construction methods used for prestressed elements of liquid retaining structures must be of the highest quality to avoid the possibility of unexpected cracking and possible corrosion of prestressing steel. For this reason, hand-placed or mechanically-placed mortar is not permitted, and until more conclusive test data are available establishing the long-term corrosion resistance of unbonded tendons, all prestressing tendons must be fully bonded.

C6.3
Allowable stresses
Table 6.1 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.

Fig. C6.1 illustrates the procedure for a typical tank wall subjected to axial compression force P, moment M_v, and shear V, per unit length of wall. The shear force V may be found from the slope of the vertical bending moment diagram, and will be a maximum at the wall base. The axial load and bending moment combine to give a linear distribution of direct stress, (fig. C6.1(b)) given by:

6
DESIGN OF PRESTRESSED CONCRETE ELEMENTS

6.1
General
Design of prestressed concrete elements of liquid-retaining structures shall comply with the requirements of NZS 3101, section 13, except as modified by the requirements of this section.

6.2
Materials

6.2.1
Materials for prestressed concrete elements of liquid-retaining structures shall comply with the requirements of section 3.

6.2.2
Hand-placed or mechanically-placed mortar shall not be used for prestressed elements of liquid-retaining structures.

6.2.3
Unbonded prestressing tendons shall not be used in the construction of liquid-retaining structures.

6.3
Allowable stresses

6.3.1
Prestressing reinforcement
Temporary and working stresses in prestressing steel shall comply with the requirements of NZS 3101, section 13.

$$f = \frac{P}{t} + \frac{12 M_v y}{t^3} \quad \text{................... (Eq. C6-1)}$$

The distribution of shear stress will be parabolic, (fig. C6.1(c)) and may be expressed as:

$$V = \frac{1.5V}{t}\left[1 - \left(\frac{2y}{t}\right)^2\right] \quad \text{................. (Eq. C6-2)}$$

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

$$f_t = \frac{f}{2} \sqrt{\frac{f^2}{4} + v^2} \quad \text{................. (Eq. C6-3)}$$

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 fig. C6.1(d). For most cases it will be sufficient to check f_t at the wall centre line (y =0). Thus

$$f_{to} = \frac{P}{t} \sqrt{\left(\frac{P}{t}\right)^2 + \left(\frac{1.5V}{t}\right)^2} \quad \text{.......... (Eq. C6-4)}$$

In equations C6-3 and C6-4 the sign convention used is compression positive.

6.3.2
Concrete and pneumatically placed mortar
Maximum stresses in prestressed concrete and prestressed pneumatically placed concrete shall not exceed the limits specified in table 6.1, except as allowed by 6.6.

Table 6.1
MAXIMUM PERMITTED STRESSES IN PRESTRESSED CONCRETE, MPa

Type of stress	Load combinations	
	Group A loads	Group B loads
Compression	$0.4f'_c$	$0.55f'_c$
Tension in monolithic concrete	-0.7MPa*	$0.5 \sqrt{f'_c}$
Tension across construction joints	-0.7MPa	0^{**}
Principal tension stress resulting from shear	$0.3 \sqrt{f'_c}$	$0.5 \sqrt{f'_c}$

* i.e. residual compression of 0.7 MPa required
** Refer to 6.3.3.

(a) Wall element

(b) Direct stress

(c) Shear stress

(d) Principal tension stress
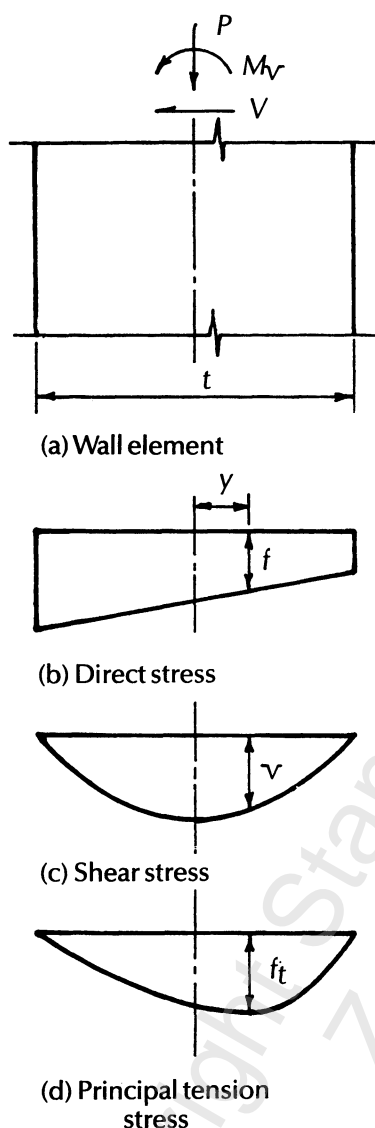
Fig. C6.1
PRINCIPAL TENSION STRESS DUE TO SHEAR

C6.3.3
Cracking at vertical construction joints is permitted provided that the crack is controlled by mild steel reinforcement, or by a flexible sealant. In the latter case, a redistribution of stress occurs between the uncracked panels and the cracked construction joint, as shown in fig. C6.2. In this figure, the wall is subjected to circumferential hoop compression of P (including prestress force) and moment M per unit wall height. At section A-A, the surface stresses are given by:

$$f_{CA} = \frac{P}{t} + \frac{6M}{t^2}$$

$$f_t = \frac{6M}{t^2} - \frac{P}{t}$$

At section B-B, the neutral axis position must

6.3.3
Group B load configurations
Cracking at construction joints under Group B load combinations is permitted provided:

(a) Non-tensioned reinforcement is provided across the construction joint in the tension zone to carry the entire calculated tension force, based on uncracked-section analysis, with tension stress in the reinforcement satisfying the limits of table 5.2 or

(b) The construction joint is protected against moisture ingress by an approved joint sealant, and compression stresses at the joint are calculated on the basis of a cracked-section analysis, assuming zero concrete tension capacity.

propagate away from the tension face for the forces to balance the applied moments. If x_B is the distance of the neutral axis from the compression face, equilibrium requires that:

$$P = \frac{1}{2} f_{CB} x_B$$

and

$$M = Pe = P\left(\frac{t}{2} - \frac{x_B}{3}\right)$$

hence

$$x_B = 3\left(\frac{t}{2} - \frac{M}{P}\right)$$

and

$$f_{CB} = \frac{2P}{3\left(\frac{t}{2} - \frac{M}{P}\right)} \leq 0.6 f'_c \quad \text{.......} \quad (Eq \ C6\text{-}5)$$

Note equation C6-5 applies only for

$$\frac{t}{6} \leq \frac{M}{P} \leq \frac{t}{2}$$

For $\frac{M}{P} < \frac{t}{6}$, the entire section is in compression and stresses at A-A and B-B are identical.

As $\frac{M}{P} \to \frac{t}{2}$, $f_{CB} \to \infty$

Fig. C6.2
STRESSES THROUGH WALL THICKNESS

C6.4
Secondary prestress stresses
This clause refers in particular to the distribution of stress induced in walls of circular prestressed storage tanks by circumferential prestress. Vertical bending moments will be induced by circumferential prestress with the magnitude and

6.4
Secondary prestress stresses

6.4.1
Liquid retaining structures shall be designed for the full effects of prestressing, including secondary stresses, and time-dependent creep redistribution of prestress.

distribution depending on the degree of base restraint, as well as the level of distribution of applied circumferential prestress force. The instantaneous vertical bending moments (secondary moments) induced by circumferential prestress can be modelled by considering the prestress to be an equivalent radially inward pressure, as shown in fig. C6.3. For tendon forces of F_p, spaced s apart, the equivalent pressure per unit height of wall will be:

$$f_p = F_p/s \quad\text{.......................................} \quad \text{(Eq. C6-6)}$$

The equivalent radially inwards pressure is thus

$$P_p = f_p/a = F_p/(a.s) \quad\text{.........................} \quad \text{(Eq. C6-7)}$$

The application of circular prestress forces to walls with pinned or moment resisting connections to the base will result in no circumferential stresses being induced at the level of the wall base because the rigidity of the base prevents development of radial displacement, and hence circumferential strain. Consequently it is common to apply some or all of the prestress with the wall initially free to slide radially. This enables compression stresses to be developed at the base of the wall.

If the base is pinned or fixed after the application of prestress, radially inwards creep displacements are restrained at the base, but may still develop at levels higher up the wall, resulting in an in-time radial displacement of the form shown in fig. C6.4. From the curvature of the wall it is apparent that vertical bending moments have been developed, whereas the initial linear deflection indicates no vertical bending.

(a) Plan $p = \dfrac{F}{a}$

(b) Elevation
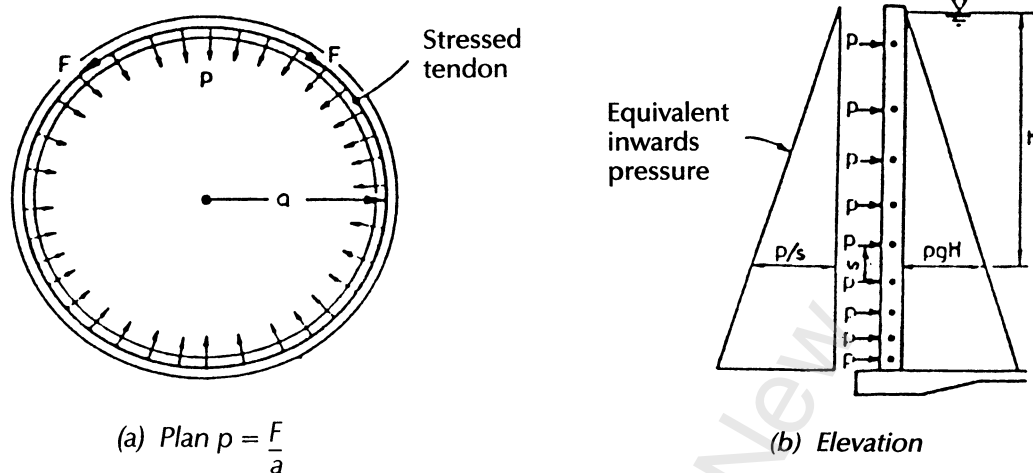
Fig. C6.3
SIMULATION OF PRESTRESS AS RADIALLY INWARD PRESSURE
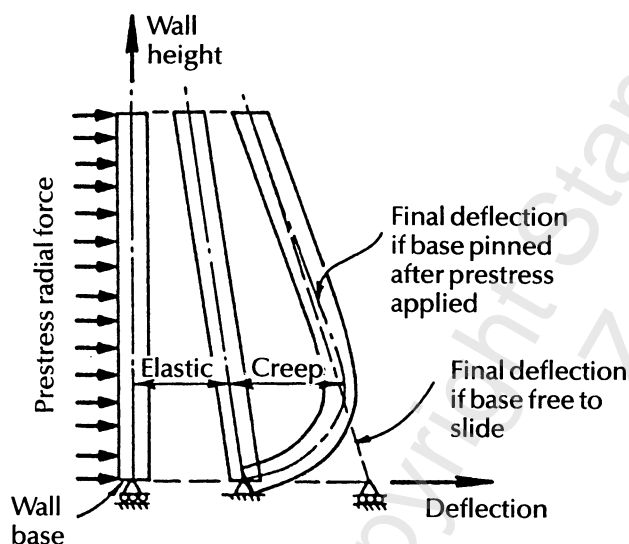
Fig. C6.4
RADIAL DEFLECTION OF WALL UNDER PRESTRESS

The effect of the structural modification provided by pinning or fixing the base after prestress application is to produce circumferential and vertical stresses that are the same as those that would result from a fraction of the prestress, p_i, being applied with the base free to slide, and the remainder, $p_f(=1-p_i)$ of the prestress being applied with the base pinned (or fixed). A rate-of-creep method of analysis results in the following expression for p_i.

$$p_i = e^{-C_t} \qquad\qquad (Eq.\,C6\text{-}8)$$

where C_t is the part of the creep function relative to time of applying circumferential prestress, still remaining at time of pinning the base. A typical

value of C_t for a tank stressed at age = 3 months and pinned one week after stressing is $C_t = 0.9$. This results in an estimate that 40% of the prestress may be considered to act in the initial (sliding) condition, and 60% of the prestress may be considered to act in the final (pinned, or fixed) condition.

6.5
Non-tensioned reinforcement
Non-tensioned reinforcement shall be provided in prestressed elements in:

(a) End anchorage zones, as shear and bursting reinforcement
(b) Between end anchorages, where prestress is calculated to be inadequate to sustain app-lied forces
(c) In other regions, such as construction joints, shown by analysis to require crack control.

6.6
Partial prestressing
A partially prestressed design approach, permit-ting cracking of concrete may be used, provided non-tensioned reinforcement is provided in the tension zone, and stresses in this reinforcment, taking full account of creep and shrinkage effects, satisfy the limits of table 5.2.

C6.6
Partial prestressing
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 6.1.

In adopting a partially prestressed approach, however, considerable care is needed in calcula-ting crack widths because of the effects of creep and shrinkage. Prior to cracking, the non-ten-sioned reinforcement in a partially prestressed section is subject to an initial compression stress which gradually increases due to creep of the con-crete 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 rein-forcement 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 rein-forcement 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 con-crete tension, to a tension stress after crack initia-tion) is larger than if the effects of creep on the initial stress distribution has been ignored. Con-sequently the crack width will be proportionally larger. It is this change in reinforcement stress which must not exceed the allowable stress levels given in table 5.2. The residual compression stress in the reinforcement at zero concrete stress may be calculated from the expression

$$f_{sr} = E_s C_t \cdot \frac{f_c}{E_c} \qquad (Eq. C6\text{-}9)$$

where f_c 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 C6-9, since the normal operating condition for the tank will be with the 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-induced forces, such as those resulting from thermal load. However, the reduction factor given in fig. C5.1 applies only to non-prestressed elements, and a rational analysis, taking tension stiffening effects into account, must be adopted for partially prestressed elements. At the time of drafting this Code, 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 F_t should be taken as 1.0.

COMMENTARY

C7.1
General

Tanks constructed of reinforced cement mortar have been in service in New Zealand and other countries for many years.

They differ from conventional reinforced concrete tanks in detailing and method of construction.

Typically they are reinforced with layers of very small diameter reinforcing separated by separately applied layers of high strength mortar. The high specific surface of the reinforcement produces a reduced characteristic bond length resulting in diminished crack widths. Added to this is the corrosion inhibiting properties of the dense fine grained mortar with its cement rich mix providing an alkaline environment.

It therefore follows that because of the inherent properties of cement mortar some of the code provisions for reinforced concrete elements can be relaxed for cement mortar elements.

Ferrocement structures, while having similar constituent materials, but generally higher percentages of reinforcement, are not covered by the requirements of this Standard.

C7.2
Construction

The Code allows high stresses and smaller covers for cement mortar and consequently tolerances in construction should be closer. The manufacture and application of mortar is a highly skilled trade and it is important that high standards of construction are maintained. It is for these reasons that the Code requires that construction is only carried out by those able to demonstrate the necessary skills.

C7.3
Pneumatically-placed mortar

It is difficult to codify good practice in this area. An account of American practice is described in reference (7.1).

7
CEMENT MORTAR ELEMENTS

7.1
General

7.1.1
This section applies to the elements of circular non-prestressed tanks which are constructed in multiple layers of hand-placed, mechanically-placed or pneumatically placed mortar.

7.1.2
Except where otherwise specified in this section the relevant requirements of sections 1, 2 and 3 should apply to cement mortar elements.

7.2
Construction

7.2.1
Cement mortar elements of tanks shall be constructed by specialist firms able to demonstrate satisfactory experience.

7.2.2
Continuous supervision shall be provided to the approval of the Engineer.

7.3
Pneumatically-placed mortar

7.3.1
The aggregate and cement shall be mixed and applied using special purpose equipment operated by skilled operators.

7.3.2
The velocity of the material leaving the nozzle shall be maintained uniform and such as to produce minimum rebound of sand.

7.3.3
Care shall be taken to attain uniform application behind reinforcing and other obstructions.

7.3.4
Immediately after the pneumatic mortar has been placed it shall be protected against premature

drying by shading from strong sunshine and shielding from the wind.

7.3.5
As soon as pneumatic mortar has hardened just sufficiently to avoid damage, it should be thoroughly wetted and thereafter kept continuously wet for at least seven days, or alternatively protected by an approved curing compound.

7.3.6
Adequate protection by shading and shielding shall be given against fluctuations in temperature.

7.4
Hand-placed mortar

7.4.1
Every layer of mortar shall be brushed or otherwise treated after initial set to provide adequate bond for the succeeding layer.

7.4.2
The thickness of any layer of mortar shall be such that no slumping occurs.

7.4.3
Hand-placed mortar shall be cured as described in 7.3.4, 7.3.5 and 7.3.6.

7.5
Mechanically-placed mortar

7.5.1
The provisions of 7.4 shall apply also to mechanically-placed mortar.

7.6
Minimum wall thickness for watertightness

7.6.1
Where any wall is constructed entirely with pneumatically-placed mortar, the minimum thickness shall be:

(a) For factory made portable tanks up to $25\,m^3$: Three mortar coats aggregating not less than 33 mm

(b) For other tanks not exceeding $40\ m^3$: Four mortar coats aggregating not less than 44 mm

(c) For tanks exceeding $40\,m^3$ or with walls exceeding 3 m in height: 75 mm.

7.7
Reinforcement

C7.7.1
An essential characteristic of cement mortar elements is a high density of closely spaced small diameter reinforcement.

7.7.1
Plain or galvanized netting, loosely linked steel wire mesh (chain netting), hard drawn steel wire and mesh, and plain and deformed bar may be used as reinforcement of mortar tanks.

C7.7.2
Conventional wire netting is not regarded as a good engineering material and therefore its working strength has been downgraded in the Code to 0.1 of f_y.

C7.7.3
Other small diameter fabric such as woven or welded mesh and chain netting are considered to be satisfactory and the allowable working stress for them is given as 0.55 of f_y.

Small diameter wires and bars (less than 6 mm) are also useful for increasing the percentage of reinforcement in one direction and the allowable working stress for them is the same as for fabric.

C7.7.4
Bars of 6 mm or greater are considered to behave as reinforcement in conventional reinforced concrete and corresponding limitations on working levels are appropriate.

Plain bars of 6 mm and greater should not be used in water retaining structures where the improved bond characteristics of deformed bars will provide better control of cracking.

C7.8
Minimum cover to reinforcement
The code requirements for cover to reinforcement in cement mortar elements are reduced from those required for conventional reinforced concrete because of the better corrosion resistance of cement mortar as described in C7.1.

7.7.2
Where netting is used the working strength shall be limited to 0.1 of the breaking strength of the effective area of the netting resolved in the direction of stress.

7.7.3
For loosely linked steel wire mesh and hard drawn steel wire and mesh the working strength shall be:

200 MPa for Group A loading combinations, and
300 MPa for Group B loading combinations

but in no case more than 0.55 f_y.

7.7.4
For plain and deformed reinforcing bar 6 mm or more in diameter the permissible stresses given in table 5.2 shall apply.

7.8
Minimum cover to reinforcement

7.8.1
The minimum cover to reinforcement shall be as specified in 5.7, except that where mortar is hand-placed by trowel, or pneumatically-placed mortar is used, cover may be reduced to a minimum of 15 mm or 25 mm respectively.

7.8.2
Where the reinforcing layer nearest the face is galvanized the cover may be further reduced to 10 mm where the member is hand-placed by trowel and to 15 mm when the member is pneumatically-placed.

7.8.3
For factory made portable tanks up to $25\,m^3$ the minimum cover on the inside face shall be not less than:

(a) 10 mm where the internal coat is applied last, or
(b) 12 mm where the internal coat is applied first.

7.8.4
Where steel-trowelled hand-placed mortar is used the outer surface shall be applied with a minimum of two layers.

7.8.5
For structures containing or surrounded by sewage, sewage sludge, or sea water the minimum cover to any reinforcement shall be 25mm using at least two layers of hand-placed

mortar, or 40 mm using pneumatically-placed mortar.

7.8.6
For other liquids having a detrimental effect on the structure, approved precautions shall be taken or a protective coating shall be applied.

7.9
Cement mortar roofs

7.9.1
Tanks may be covered with conical or domed shell roofs of cement mortar designed in accordance with established principles of structural design and adequately tied into the top of the wall.

7.9.2
The minimum thickness of a cement mortar shell roof shall be the greater of 40 mm and the figure given by equation 5-1 subject to the provisions of minimum steel coverages, as specified in 7.8, except that the thickness at the springing of the shell shall be not less than the thickness of the wall at that point.

For factory made portable tanks up to 40 m³ with galvanized reinforcement the minimum thickness of a shell roof shall be 33 mm, subject to the provisions of minimum steel cover.

7.9.3
A cement mortar roof may be constructed in one layer provided that:

(a) The minimum thickness and the cover to reinforcment shall be as required for a roof construction in several layers

(b) Special care shall be taken to ensure the accurate location and secure fixing of the reinforcment and

(c) Special care shall be taken to prevent premature drying and to ensure adequate curing conditions.

REFERENCES

(7.1) ACI Committee report 344R-70, "Design and Construction of Circular Prestressed Concrete Structures". ACI Committee 344, 1970, 16 pp. Also ACI Manual of Concrete Practice, Part 4, American Concrete Institute, Detroit.

APPENDIX A
CYLINDRICAL TANK THERMAL TABLES

A1
General

A1.1
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:

$$S_{FACT} = H^2/2a.t \qquad \text{(see C2.2.8.1)}$$

A1.2
Tables are given for three thermal conditions as set out in A2, A3 and A4 and for three base conditions:

Table A1 : Pinned-base condition
Table A2 : Fixed-base condition
Table A3 : Sliding-base condition

A1.3
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 C2.2.8.1.

A2
Average temperature change

$\equiv$ Uniform temperature change of θ_A.

Tables assume θ_A is a temperature increase.

Reverse sign for temperature decrease.

A3
Differential temperature change

$\equiv$ Differential temperature change

of $\pm \theta_D$ (see figure)

Note total gradient through wall

$= 2\theta_D$

Tables assume outside hotter than inside

Reverse sign for inside hotter than outside.
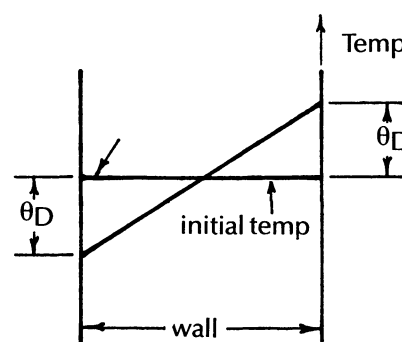
A4
Total temperature change

= Temperature variation on *outside*

surface only = θ_T (see figure)

Tables assume outside hotter than inside.

Reverse sign for outside colder than inside.
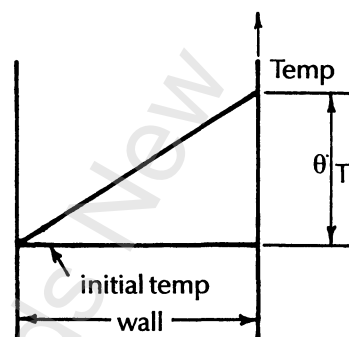
A5
Stresses
Thermal stress is given by:

$$f = C.E_C \, \alpha \, \theta$$

where C =   Coefficient from appropriate table
      E_C =   Modulus of elasticity of the concrete
      α =   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 A1 THERMAL STRESS CO-EFFICIENTS – 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	0.956	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	0.861	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	-0.241	-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	-0.011	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

(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	-0.686	-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	-0.011	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	1.261	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	-0.921	-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

(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	0.145	0.440	0.602	0.671	0.690	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

(j) 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	-0.432	-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	-0.861	-0.625	-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

NOTES

Table A2 THERMAL STRESS CO-EFFICIENTS – FIXED-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	-0.024	-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	0.384	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	0.019	-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

(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

(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

(j) 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	-1.023	-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

NOTES

Table A3 THERMAL STRESS CO-EFFICIENTS – 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	1.120	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

(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	0.303

(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	-0.997	-1.003	-1.011	-1.003	-0.997	-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

(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	0.138	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

(j) 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

APPENDIX B
FUNDAMENTAL PERIOD OF VIBRATION OF THE INERTIA COMPONENT OF A RECTANGULAR TANK (T_I)

B1
General

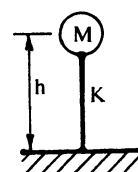#### B1.1
This appendix gives an appropriate method of calculating the fundamental period of vibration (T_I) of the inertia component of a rectangular tank (see C2.2.9.3(d)).

B1.2
T_I for an at-grade rectangular tank can be roughly estimated using a lumped mass/hypothetical column type model. (See fig. B1).
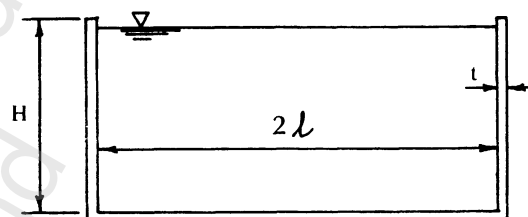
(a) Rectangular tank

(b) Hypothetical column model.

Fig. B1:
TANK/MODEL SYSTEM

The fundamental period T_I is given by:

$$T_I = 2\pi \sqrt{M/k}$$

where

$M = M'_W + M'_I$

M'_W = mass of wall per unit width (kg/m)
= $Ht P_C$ (where P_C = density of wall)

M'_I = half of the impulsive mass of contained liquid per unit width (kg/m)

$= \left(\dfrac{W_I}{W}\right) \ell H \rho_L$ (where ρ_L = density of liquid)

$\dfrac{W_I}{W}$ can be obtained from fig. C2.2

k = flexural stiffness of a unit width cantilever (N/m)

= the horizontal force which when applied at the centre of mass translates the centre of mass by a unit horizontal distance

= $P/\triangle$ (where P = the applied force, and the $\triangle$ = horizontal deflection of the centre of mass)

(see fig. B2)

h = effective height of M

h = $\dfrac{h_W M_W + h_I M_I}{M}$

h_W = centre of gravity of wall = H/2

h_I = height of impulsive mass

$$\left(\left(\dfrac{h_I}{H}\right) \text{ can be obtained from fig. C2.2} \right)$$
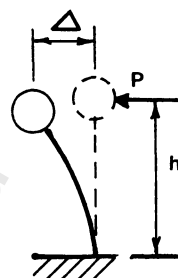
Fig. B2
DEFLECTION MODEL

B2
WORKED EXAMPLE

liquid = water $\quad \rho_L$ = 1000 kg/m³

concrete tank $\quad \rho_C$ = 2400 kg/m³

2ℓ = 20 m $\quad$ H = 6.0 m $\quad$ t = 300 m $\quad$ E_C = 25 GPa

M_W = $Ht\rho_C$ = 6 x 0.3 x 2400 = 4300 kg/m

M_I = $\left(\dfrac{W_I}{W}\right) H\ell\rho_L$ = 0.36 x 6 x 10 x 1000 = 21 600 kg/m

$$\left(\left(\dfrac{W_I}{W}\right) = 0.36 \text{ for } \dfrac{\ell}{H} = 1.67 \text{ in fig. C2.2} \right)$$

M = $M'_W + M'_I$ = 25 900 kg/m

h_I = 0.385H (from fig.C2.2 for $\dfrac{\ell}{H}$ = 1.67)

= 0.385 x 6 = 2.31

h_W = 0.5H (for a wall of constant thickness) = 3.0 m

h = $\dfrac{h_W M'_W + h_I M'_I}{M}$

= $\dfrac{3 \times 4300 + 2.31 \times 21\,600}{25\,900}$ = 2.42 m

For this particular case, the wall is assumed to act as a cantilever and modelled by a lumped mass on a stem of stiffness k

The defelection $\triangle$ for a simple cantilever $= \dfrac{Ph^3}{3F}$

For a wall of constant thickness and unit width $I = t^3/12$

and $\triangle = \dfrac{4P}{E}\left(\dfrac{h}{t}\right)^3$

$k = \dfrac{P}{\triangle} = \dfrac{E}{4}\left(\dfrac{t}{h}\right)^3 = \dfrac{25 \times 10^9 \times 0.3^3}{4 \quad 2.42} = 11.9 \times 10^6\,N/m$

$T_I = 2\pi\sqrt{\dfrac{M}{k}} = 2\pi\sqrt{\dfrac{25\,900}{11.9 \times 10^6}} = 0.29\,seconds$

NOTES

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NEW ZEALAND STANDARD

NZS 3106:1986

| Pr AA |

Code of practice for
CONCRETE STRUCTURES FOR THE
STORAGE OF LIQUIDS

AMENDMENT No. 1

April 1990

EXPLANATORY NOTE - Amendment No. 1 corrects minor errors and updates related documents.

To ensure receiving advice of the next amendment to NZS 3106:1986 please complete and return the amendment request form.

DECLARATION

Amendment No. 1 was declared on 27 April 1990 by the Standards Council to be an amendment to NZS 3106:1986 pursuant to the provisions of section 10 of the Standards Act 1988.

(Amendment No. 1, April 1990)

RELATED DOCUMENTS

After "NZS 1900:- - - - Model building bylaw" **add** "Division 11.1:1986 Structures for the storage of liquids".

Delete "NZS 3109:1980" and **substitute** "NZS 3109:1987".

Delete "NZS 3402P:1973 Hot rolled steel bars for concrete reinforcement" and **substitute** "NZS 3402:1989 Steel bars for the reinforcement of concrete".

Delete "BS 4102:1971 Steel wire for fences" and **substitute** "BS 4102:1986 Steel wire and wire products for fences".

Delete "BS 5337:1976 The structural use of concrete for retaining aqueous liquids" and **substitute** "BS 8007:1987 Design of concrete structures for retaining aqueous liquids".

(Amendment No. 1, April 1990)

1.1
Scope
In clause 1.1.1, **delete** the last words "the relevant requirements of NZS 1900", and **substitute** "the requirements of NZS 1900, Division 11.1".

(Amendment No. 1, April 1990)

1.5
Notation
In clause 1.5.1, after "W_c" **add** "W_L The weight of the tank contents, N."

The fourth symbol from the end: **Delete** "λ_L" and **substitute** "γ_L".

(Amendment No. 1, April 1990)

2.2
Design loads
In clause 2.2.8.2(b), **delete** "50 °C and **substitute** "5 °C".

In clause 2.2.9.5, **delete** the last line "for a rectangular tank $\beta = 2b$" and **substitute** "for a rectangular tank $\beta = 4b$".

In commentary clause C2.2.9.6, equation C2-9, in the numerator of the fraction, **change** "M'_{BI}" to M_{BI}".

In fig. C2.5 (also subject to erratum of March 1987) **make the same change** at the left hand ordinate.

(Amendment No. 1, April 1990)

3.2
Concrete
In commentary clause C3.2.4, in line 7 and in the last line **delete** "BS 5337" and **substitute** "BS 8007".

In commentary clause C3.2.6, equation C3-2, **delete** "f_c" and **substitute** "$\sqrt{f_c}$".

(Amendment No. 1, April 1990)

3.4
Non-prestressed reinforcement
In clause 3.4.1, **delete** "NZS 3402P" and **substitute** "NZS 3402".

(Amendment No. 1, April 1990)

4.2
Concrete placing
In commentary clause C4.2.3, paragraph 6, **delete** "BS 5337" and **substitute** "BS 8007".
In paragraph 9, **delete** "Appendix D of BS 5337" and **substitute** "Appendix C of BS 8007".

(Amendment No. 1, April 1990)

5.1
General
In commentary clause C5.1.3, **delete** "BS 5337" and **substitute** "BS 8007".

(Amendment No. 1, April 1990)

5.2
Stiffness of cracked section
In commentary clause C5.2, paragraph 1, line 6, **delete** "effect" and **substitute** "affect".

(Amendment No. 1, April 1990)

5.4
Reinforcing steel stresses
In commentary clause C5.4.1, line 6, **delete** "BS 5337" and **substitute** "BS 8007".
In line 7, **delete** ACI 350R-77 and **substitute** ACI 350R-83.

(Amendment No. 1, April 1990)

6.3
Allowable stresses
In figure C6.1, to the title of diagram (a), **add** "(plan view)".

In commentary clause C6.3.3, in the expression immediately above fig. C6.2, **delete** "00" and **substitute** "∞".

(Amendment No. 1, April 1990)

APPENDIX B
In B2 Worked example, in the fourth line on page 77, **delete** the denominator of the fraction, "3F" and **substitute** "3EI".

(Amendment No. 1, April 1990)

NEW ZEALAND STANDARD

NZS 3106:1986

Code of practice for
CONCRETE STRUCTURES FOR THE
STORAGE OF LIQUIDS

Pr Gratis

AMENDMENT No. 2

July 1992

EXPLANATORY NOTE – This Amendment applies when this Standard is used as a Verification Method that is referenced in Approved Document B1 Structure – General, to the New Zealand Building Code. The Amendment need not apply when this Standard is used under the Model Building Bylaw system which remains in operation until 31 December 1992.

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APPROVAL

Amendment No. 2 was approved in July 1992 by the Standards Council to be an amendment to NZS 3106:1986 pursuant to the provisions of section 10 of the Standards Act 1988.

(Amendment No. 2, July 1992)

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1.6.1
Delete the definition of "Engineer".

Add a new definition:

"DESIGN ENGINEER. Any person who, on the basis of experience or qualifications, is competent to design structural elements of the structure under consideration to safely resist the loads likely to be imposed on the structure."

(Amendment No. 2, July 1992)

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3.3.1.4
In line 2, **delete** the word "Engineer" and **substitute** "design engineer".

(Amendment No. 2, July 1992)

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C4.2.3
In the penultimate line of the second paragraph, **delete** the word "engineer" and **substitute** "design engineer".

(Amendment No. 2, July 1992)

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NZS 3106:1986

Code of practice for
CONCRETE STRUCTURES FOR THE
STORAGE OF LIQUIDS

ERRATUM

March 1987

Fig. C2.5, page 23.

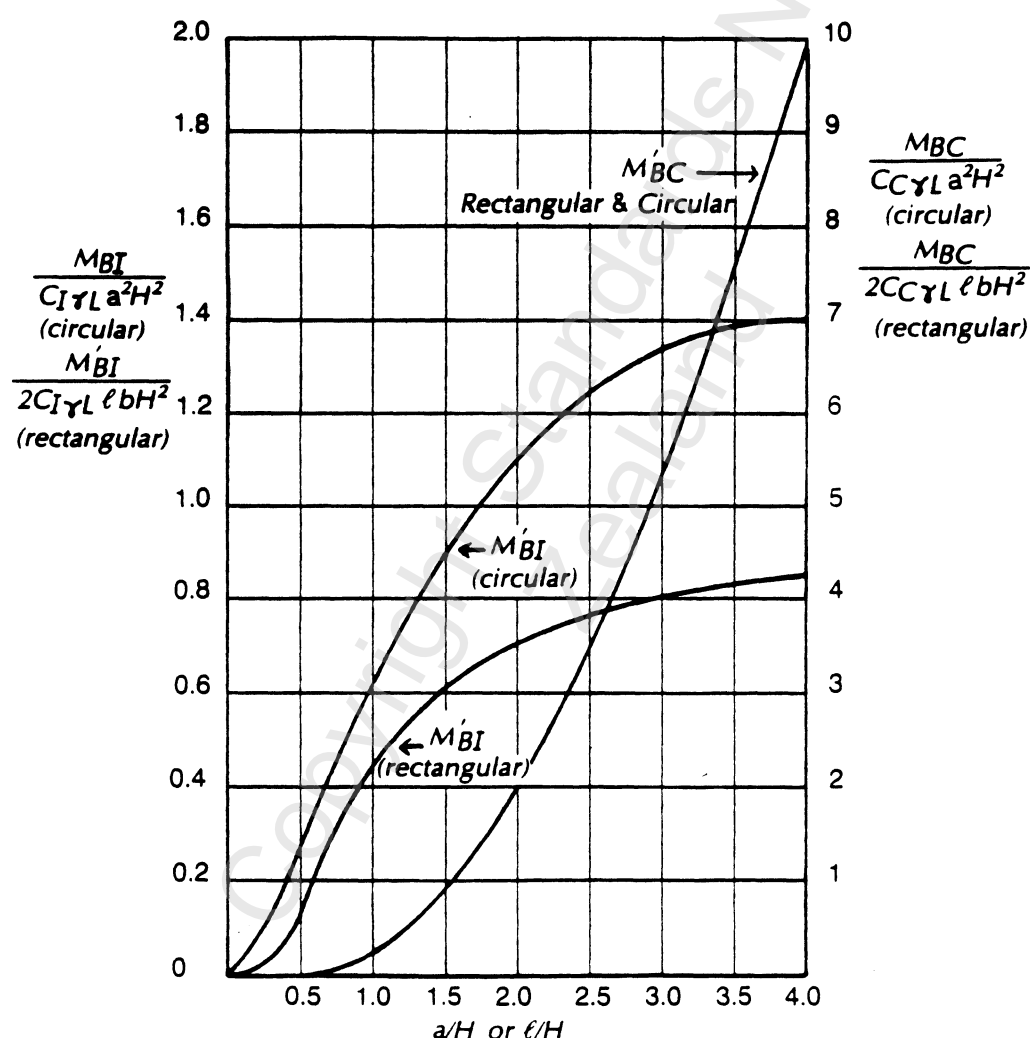**Delete** the figure and **substitute** the following:

Fig. C2.5
IMPULSIVE AND CONVECTIVE MOMENTS ON BASE
(CIRCULAR AND RECTANGULAR TANKS)

Fig. C2.7(a), page 25.

Delete the equation below the left hand diagram and **substitute** the following:

Cmax. = Tmax. = $\dfrac{Q}{\pi}$