AS 3735 Supp1

Australian Standard™

Concrete structures for retaining liquids—Commentary

(Supplement to AS 3735—2001)

Standards Australia

This Australian Standard was prepared by Committee CE-022, Concrete Structures for Retaining Liquids. It was approved on behalf of the Council of Standards Australia on 17 November 2000 and published on 13 March 2001.

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- Australian Pre-mixed Concrete Association
- Institution of Engineers Australia
- University of Queensland
- Water Services Association of Australia

Additional interests participating in the preparation of this Standard:

- Association of Consulting Engineers Australia
- Australian Chamber of Commerce and Industry
- Australian Post Tensioning Association
- Australian Water and Wastewater Association
- Brisbane City Council
- Department of Public Works and Services, N.S.W.
- Melbourne Water
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Concrete structures for retaining liquids—Commentary

(Supplement to AS 3735—2001)

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PREFACE

This Supplement was prepared by the Standards Australia Committee CE-022, Concrete Structures for Retaining Liquids, to supersede AS 3735 Suppl—1991.

This Commentary provides background information to AS 3735—2001, and is intended to be used in conjunction with that document. It is not intended to be used as a Standard, or other reference document.

The paragraph numbers of this Commentary are prefixed with the letter C and refer directly to the respective Clause numbers of AS 3735, e.g. Paragraph C5.3.1 refers to Clause 5.3.1. Where there is no commentary to a clause of the Standard, the paragraph number does not appear. Figures and tables are designated C2.2, C3.1, etc., and do not correspond to those in the Standard.

References are listed as the last paragraph of the Section in which they occur.

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STANDARDS AUSTRALIA

Australian Standard

Concrete structures for retaining liquids—Commentary (Supplement to AS 3735—2001)

SECTION C1 SCOPE AND GENERAL

C1.1 SCOPE

A lower concrete strength limit of 20 MPa has been imposed, as strength grades less than this are not considered suitable for structures.

An upper concrete strength limit of 50 MPa has been adopted, because much of the research on which the Standard is based involved concrete strengths at or below this value. Nevertheless, higher strength concretes are being used in Australia and overseas (Refs 1 and 2). The Standard may be applied to such concretes, provided that the physical properties appropriate to them are used in design.

The Standard also limits the use of lightweight aggregate in structural concrete. Most Australian structural concretes employ only lightweight coarse aggregate, resulting in concrete with a surface-dry density that is seldom less than 1800 kg/m^3.

In the preparation of a Standard such as this, a certain level of knowledge and competence of the majority of users has been assumed. As indicated by the Note to Clause 1.1, it was assumed that the predominant users of this Standard would be professionally qualified civil engineers experienced in the design of concrete structures for retaining liquids, or equally qualified but less experienced persons working under their guidance. It is therefore intended that the Standard be applied and interpreted primarily by such persons.

C1.5 USE OF ALTERNATIVE MATERIALS OR METHODS

The designer is usually required to seek approval from the appropriate Authority for the use of alternative materials or methods and such approval would not mean a relaxation of the requirements of the Standard, e.g. the use of fibre impregnated concrete would not mean an automatic relaxation of the requirements for conventional reinforcement or tendons.

C1.5.3 Ferrocement

Ferrocement should comply with the following:

(a) Sand/cement ratio, by volume, of not more than 3:1.

(b) For galvanized and epoxy coated steel reinforcement, the minimum cover given in Table 4.3 may be reduced by 15 mm, provided no negative tolerance is permitted and the surface is hand trowelled.

C1.6 DRAWINGS AND SPECIFICATIONS

The information applicable to most concrete members may be shown on only one of the drawings, usually the first sheet.

REFERENCES

1 CHOY, R.S. *High-strength concrete* Cement and Concrete Association of Australia. Sydney, 1988. Technical Report TR/F122.

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SECTION C2 LOADS, LOAD COMBINATIONS FOR STABILITY, STRENGTH AND SERVICEABILITY

C2.1 GENERAL

The combinations in Ref. 1 are expressed in more general terms than those specified in Clauses 2.3 and 2.4.

The serviceability of a concrete structure designed for the retention (or exclusion) of a liquid is of paramount importance. It is this (together with the possible need to resist flotation) that necessitates special design considerations. The purpose of this Standard is therefore primarily to provide information to ensure the structure is designed to be serviceable under the working design loads and conditions determined for the design life of the structure. Clause 2.3 provides design load combinations for stability design and Clause 2.4 provides load combinations for serviceability under working conditions. While in many situations the load combinations for serviceability will be more critical than the ultimate load combinations for strength, the engineer must ensure that the strength and other applicable serviceability requirements, e.g. ductility and deflection, as specified in Ref. 2, are met.

There are two broad types of loading: that resulting from the application of forces and that resulting from deformation (strain). In a tank, the force loading is exemplified by contained fluid pressure; the strain-induced loads are temperature and shrinkage. Strain-induced loads can also be produced in concrete members that are connected to members being post tensioned.

C2.2 LOADS AND OTHER ACTIONS

C2.2.1 Temperature

The temperature distributions as given in Table 2.1 and shown in Figure 2.1 are appropriate for the walls of tanks subject to direct solar radiation, and in the absence of a thermal analysis based on local meteorological records. 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.

A method, using thermal stress coefficients (*C*) developed in Ref. 3, for the calculation of thermal wall stresses for a common range of cylindrical tanks is given in Appendix A.

Thermal wall stresses calculated in accordance with Appendix A relate to surface stresses assuming uncracked wall stiffness. If the tank is designed on the basis of a cracked wall, then these stresses should be modified by a reduction factor (*RF*) to account for the reduced section rigidity that accompanies cracking. The amount of reduction depends on the extent of tension stiffening in the concrete, the reinforcing content and the degree of inplane force.

Ref. 3 describes a method for evaluating *RF* for reinforced concrete design and discusses the implications for partially prestressed design. There is no reduction for fully prestressed design as concrete is assumed to remain uncracked, i.e. *RF* = 1.0.

Temperature moments and axial forces for an uncracked wall as shown in Figure C2.1 are calculated from the following equations:

$$M_{\Delta\theta} = RF \frac{(\sigma_{\mathrm{i}} - \sigma_{\mathrm{o}})}{2} \frac{t_{\mathrm{w}}^2}{6} \qquad \text{. . . C2.2.1(1)}$$

$$F_{\Delta\theta} = RF \frac{(\sigma_{\mathrm{i}} + \sigma_{\mathrm{o}})}{2} t_{\mathrm{w}} \qquad \text{. . . C2.2.1(2)}$$

where

$M_{\Delta\theta}$ = moment induced in element due to temperature gradient across the element, in newton millimetres per millimetre length of wall

RF = reduction factor modifying temperature stresses for section rigidity

σ_{i} = fibre stress on inside face of a tank wall resulting from a temperature gradient through the roof or wall, in megapascals

σ_{o} = fibre stress on outside face of a tank roof or wall resulting from a temperature gradient through the wall, in megapascals

t_{w} = thickness of wall, in millimetres

$F_{\Delta\theta}$ = axial force induced in a cylindrical wall due to a temperature gradient through the wall, in newtons per millimetre length of wall

A tank designed on the basis of a cracked wall should be checked under the stresses resulting from the application of the moment and force given by Equations C2.2.1(1) and C2.2.1(2). Note that direct factoring of the stresses σ_{i} and σ_{o} will not produce the same results.

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

Increasing temperature or shrinkage subjects 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 factor 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 *RF* are shown in Figure C2.2 for a range of wall thickness and steel ratios.

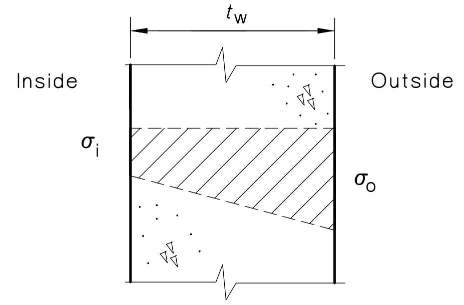
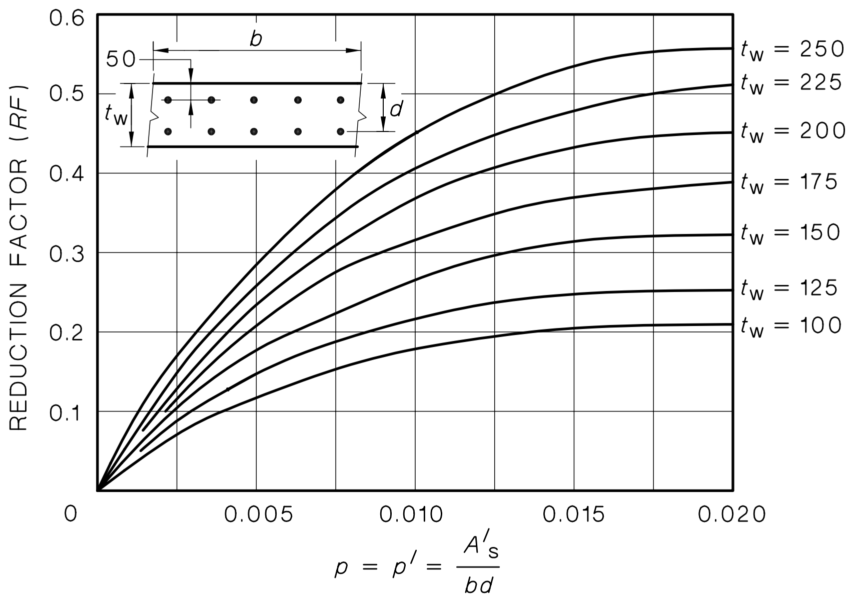


FIGURE C2.1 THERMAL STRESSES



LEGEND:

RF = reduction factor, i.e. reduction in stiffness with cracking
b = the width of cross-section, in millimetres
t_w = the thickness of a wall, in millimetres
d = the effective depth of a cross-section, in millimetres
A_s = the cross-sectional area of tension reinforcement, in square millimetres
A'_s = the cross-sectional area of compression reinforcement, in square millimetres
p and p' = reinforcement ratios

NOTES:

1 $d = t_w - 50$

2 $A'_s = A_s$

FIGURE C2.2 REDUCTION OF STIFFNESS OF DOUBLY REINFORCED WALL ON CRACKING (INCLUDING TENSION STIFFENING EFFECT)

The I_{cr} values used in tabulating *RF* 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 or shrinkage stress. Axial compression on the other hand increases section stiffness with a corresponding increase in thermal or shrinkage stress. 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 *RF* 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 tension stiffening effect decreases with increasing reinforcement ratio *(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 *RF* values.

C2.2.2 Moisture variation

Effects due to moisture variation should be considered as follows:

(a) *Roofs* 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.

(b) *Walls* The shrinkage and swelling strains given in Table 2.2 were derived using Ref. 4, except that predicted shrinkages were doubled in accordance with the recommendations of Ref. 5.

The following assumptions are made:

(i) Shrinkage commences immediately after casting.

(ii) Shrinkage regain is 100% and occurs immediately the tank is filled.

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

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

(v) Shrinkage strains are reduced by a creep reduction factor calculated from the following equation:

$$\phi_{rf} = \frac{1 - e^{-\phi}}{\phi} \qquad \text{. . . C2.2.2}$$

where

ϕ_{rf} = creep reduction factor

e = base of Napierian logarithms

ϕ = creep factor for the concrete between the time shrinkage stresses commence, i.e. when shrinkage movement is restrained, to the time the tank is filled

(vi) The creep reduction factor used to assess long-term (500 d after filling) swelling (implied by the load combinations) is given by $e^{-\phi(x)}$ (where $\phi(x)$ is the long-term creep factor).

For tanks cast in situ, shrinkage stresses develop between the time the walls are cast and when 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 2.2 are net strains, that is, counteracting shrinkage strains have been deducted.) Although swelling strains are generally similar for both types of construction, because the counteracting shrinkage strains are lower for precast panels than those for cast in situ construction, the net swelling strains are correspondingly higher.

The initial swelling strains caused by the shrinkage regain that occurs when the tank is filled are in time reduced by creep relaxation. This reduction is taken into account in the value given to the load factor used in the load combinations specified in Clause 2.4.

Exposure of a tank to wind and sun causes the outside surface to dry out resulting in a shrinkage gradient through the wall. Few data are available relating to the extent of the differential and to the distribution through the wall thickness. It appears, however, that the gradient is low for much of the wall thickness with most of the differential occurring in the outer 15%. This results in crazing of the outer surface, which, while relieving the shrinkage stresses, has negligible effect on the serviceability of the tank. Consequently, differential shrinkage gradients 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.

In many practical cases it will be found that shrinkage-induced stresses in the combinations specified in Clause 2.4 will not affect design.

C2.2.3 Earthquake

Ref. 6 or other appropriate reference should be used for the analysis of earthquake loads. Movement generated in the contained liquid due to movement of the structure is assessed to determine the earthquake action (F_{eq}).

Particular care should also be taken with the stability and fixing details of individual members and units. It has been found that it is often simply the arrangement or brittleness of these fixing details which cannot cope with any shaking action rather than a flaw in the design for resisting particular load.

C2.2.4 Other actions

Other actions that should be considered are as follows:

(a) *Backfill* 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.

(b) *Construction loads* Examples of construction loads are the stacking, lifting and propping of precast panels.

(c) *Liquid load* Normally, concrete structures for retaining liquids are designed on the basis of the liquid level being at the top of the wall, even though the normal operating overflow level is below this. Mechanical overflow devices can fail and hence reliance on their successful operation is normally not assumed. Additionally, such mechanical devices require a small surcharge to initiate operation. Nevertheless, the engineer may waive this if he/she is confident of the successful operation of such devices.

Liquid loads will develop—

(i) vertical moments and horizontal hoop forces in the walls of cylindrical tanks (see Ref. 7); and

(ii) vertical and horizontal moments in the wall of rectangular tanks (see Ref. 8).

The design tables of these References, based on homogeneous and elastic behaviour, are to assist in the determination of the resulting moments and forces. While the assumption of homogeneous and elastic behaviour is generally satisfactory for design purposes, the engineer should recognize that cracking and any other non-linearity will affect the relativity between the moments and the forces.

(d) *Wind* Generally the effects of wind load on ground-supported tanks are not severe and will not affect design. The possibility of suction over the entire roof surface should be recognized in the design.

C2.3 STABILITY DESIGN

This section has been amended to comply with the requirements of AS 1170.1.

C2.4 LOAD COMBINATIONS FOR SERVICEABILITY

Transient loads should be disregarded if their effect is beneficial. These loads include earth pressure (F_{ep}), shrinkage (F_{sh}), swelling (F_{sw}) and temperature *(T)*.

Forces due to prestress vary between maximum and minimum limits as a result of instantaneous losses (friction, elastic deformation) and deferred or time dependent losses (creep, shrinkage, steel relaxation). The more adverse prestressing force should be considered for each load combination.

Group A loads encompass permanent loads, variable loads of long duration and frequently repeating loads. Shrinkage is a long duration load. Swelling can be either a long or a short duration load: the difference is accounted for in the respective load factor applied. In the case of the first Group A load combination, shrinkage applies when the tank is empty prior to filling, and swelling applies when the tank is emptied for maintenance.

Group B loads encompass permanent loads plus infrequent combinations of transient loads. When 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 relative stiffness and on whether the gross section properties or the cracked section properties are used in the analysis. The final Group B load combination applies equally to shrinkage and swelling. Shrinkage is appropriate with the tank empty prior to filling and swelling is appropriate with the tank empty for maintenance. For guidance on earthquake requirements, see Ref. 6.

The load combinations for serviceability include a long-term load factor (ψ_l) of 0.0 and a short-term load factor (ψ_s) of 1.0. Where more accurate data are available, the applicable values of ψ_l and ψ_s should be introduced (see Ref. 2).

REFERENCES

1 STANDARDS AUSTRALIA. AS 1170.1. Minimum design loads on structures Part 1: Dead and live loads and load combinations Sydney: Standards Australia, 1989.

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3 PRIESTLEY, M.J.N. Ambient thermal stresses in circular prestressed concrete tanks. *ACI Journal*, Oct. 1976, vol. 73, no. 10, pp. 553560.

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13 HEMSLEY, J.A. Plane strain flexure of a strip foundation with edge walls. Proc. I.C.E. Part 2, Sept. 1987, no. 83, pp. 541-560.

SECTION C3 DESIGN FOR SERVICEABILITY AND STRENGTH

C3.1 GENERAL

The control of cracking in concrete structures is a complex and controversial subject. Numerous equations for crack widths have been reported in the literature, but many of them conflict. The results of these equations are, therefore, viewed with some scepticism. Even if the results could be relied upon, the significance of a specific crack width is also a subject of controversy. There is still considerable disagreement on the manner in which crack widths relate to performance and durability, yet there is no doubt that there is some relationship.

Excessive cracking generally leads to poor performance in terms of any or all of the following:

(a) Aesthetics of the structure.

(b) Leakage.

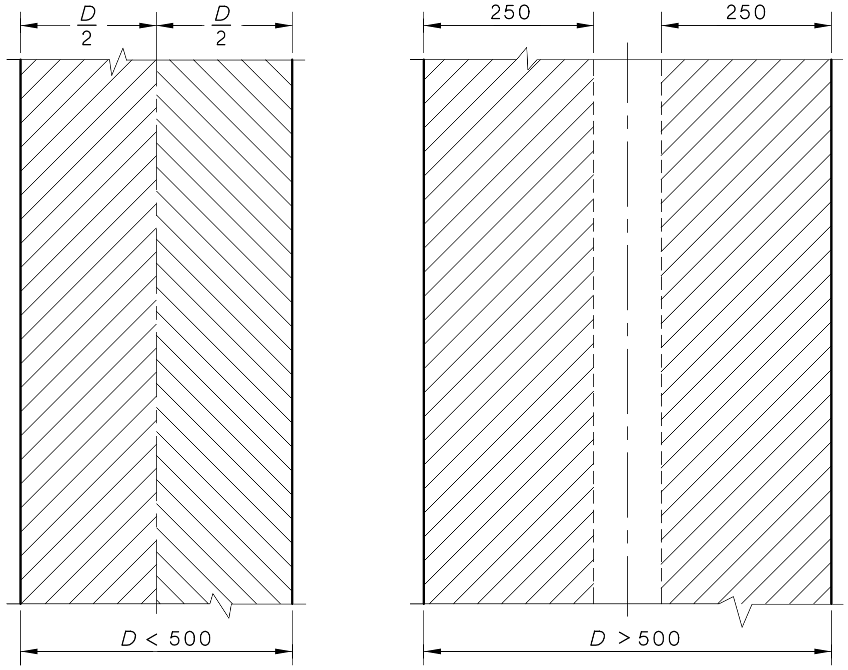
(c) Reinforcement corrosion.

Cracking, therefore, needs to be controlled, and this is achieved in the Standard by limiting crack widths implicitly in terms of steel stresses. The resulting provisions are far more convenient for the designer to use, and are less controversial.

C3.2 REINFORCED CONCRETE

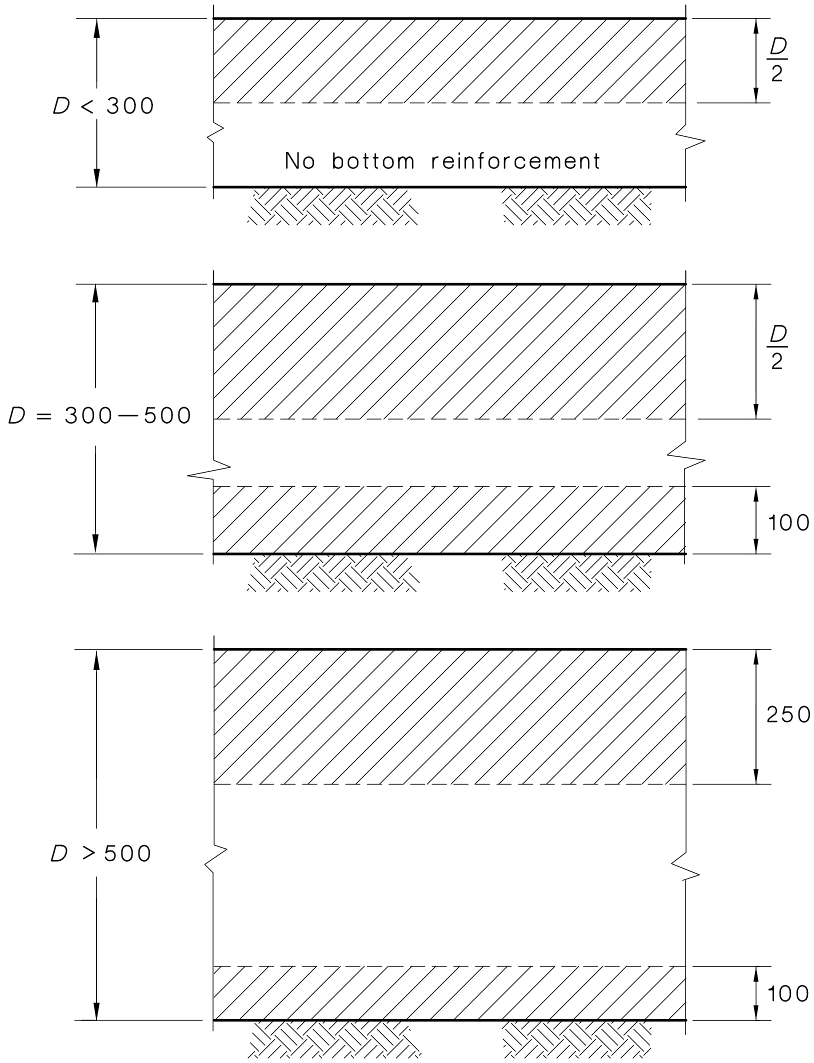
C3.2.1 General

The reinforcement ratio (*p*) is based on the appropriate hatched area of surface zone shown in Figures C3.1 and C3.2.



DIMENSIONS IN MILLIMETRES

FIGURE C3.1 SURFACE ZONES—WALLS AND SUSPENDED SLABS



DIMENSIONS IN MILLIMETRES

FIGURE C3.2 SURFACE ZONES—GROUND SLABS

C3.2.2 Minimum reinforcement ratio

When restrained concrete shrinks, cracks will invariably occur. However, by provision of sufficient reinforcement, the cracks can be both well distributed and of an acceptable width. It is the intention of Clause 3.2.2 to ensure that these two properties are addressed.

The cracking behaviour is dependent on a number of factors including:

(a) Degree of restraint.

(b) Type of restraint.

(c) Bar diameter and spacing.

(d) Amount of reinforcement.

(e) Concrete and steel strength.

When concrete shrinks, it is essential that, once the first crack occurs, subsequent shrinkage will result in the formation of further cracks and not in an uncontrolled increase in the width of the first crack. Thus reinforcement in excess of the minimum reinforcement required for normal slab on ground design (based on a sliding frictional force under the base) is required so that cracking is controlled under all circumstances.

Reinforcement specified in Clause 3.2.2(a) will ensure an even distribution of cracks. Suggested values for f'_t are given in Refs 1 and 2. For spacing of movement joints at a maximum of about 5 m, Ref. 1 specifies a reduction to 67% of the value specified in Clause 3.2.2(a).

When sufficient reinforcement is provided to ensure that the reinforcement does not yield and that the cracks are distributed, the width of the cracks so formed must be controlled. The amount of reinforcement required to control the crack widths is dependent on, amongst other items, the bond characteristics of the concrete to steel interface, the fall in temperature between the hydration peak in the concrete and the ambient, the creep properties and the coefficient of thermal expansion of immature concrete.

Thus owing to the large number of factors and their highly variable nature, it is difficult to evaluate precisely the amount of reinforcement required. The values specified in Clause 3.2.2(b) have been evaluated using the provisions of Appendix A, Ref. 1 with modifications to suit Australian conditions (Ref. 2) and an assumed mean crack width of 0.15 mm.

For thick sections or extremely hot climates, with a high cement content, higher values may be required than those given in Table 3.1 and the joint spacings reduced below those specified in Clause 3.2.2(b). For the extension of Table 3.1 for the above conditions and for plain bars and welded wire fabric reinforcement see Ref. 1 and Appendix A, Ref. 2.

C3.2.3 Limiting steel stresses for serviceability

The provisions of Clause 3.2.3 are intended to ensure crack widths at locations of reinforcements, are within acceptable limits. For Group A load combinations (i.e. long-term loads), for example, the provisions are intended to ensure that mean crack widths (b_m) do not exceed the values as given in Table C3.1.

TABLE C3.1

LIMITING MEAN CRACK WIDTHS (b_m)

Predominant stress state	Type of exposure	
	Continuously submerged	Intermittent wetting and drying
Tension	0.10 mm	0.10 mm
Flexure	0.15 mm	

The basis for deriving the provisions of Clause 3.2.3, Ref. 3 includes an equation that defines crack width in terms of parameters such as steel stress, bar diameter, stress distribution, bond strength, bar spacing and angle of cracking. All these parameters have been included in Clause 3.2.3 except for the following:

(a) *Bar spacing* The effect of changing bar spacing varies, depending on the value of the steel stress and the influence of tension stiffening in the concrete. For large steel stresses, the effect of tension stiffening is small, and a reduction in bar spacing for a fixed bar diameter and steel stress leads to a reduction in crack width. For small steel stresses, the converse applies. The effect of tension stiffening is large, and a reduction in bar spacing for a fixed bar diameter and steel stress leads to an increase in crack width. For moderate steel stresses, such as the design stresses as given in Table 3.2, the effect of changing bar spacing tends to be negligible. Bar spacing was therefore not included in the provisions of the Standard.

(b) *Angle of cracking* In contrast to bar spacing, the angle of cracking has a significant effect on crack width. Referring again to Ref. 3, a change in the angle of cracking can lead to an increase in crack width of the order of a factor of two. Inclined cracking is characterized by a biaxial stress condition in the reinforcement, and usually occurs in the presence of inplane shear stresses. Such stresses occur, for example, in buttressed walls, two-way slabs, deep beams, cylindrical roofs or skew plates. In some instances orthogonal cracking instead of diagonal cracking can occur. This has been observed in circular tanks at about one-third height of the wall, possibly due to the effect of combining a flexural stress with a hoop stress. However, the possibility of a diagonal crack occurring should be assessed.

If an inclined crack is anticipated, the limiting stresses in Table 3.2 should be multiplied by an additional factor which, in the absence of more precise information, can be obtained from Table C3.2.

TABLE C3.2

FACTOR FOR ANGLE OF CRACKING

Angle of cracking	**75° – 90°**	**60°**	**45°**
Steel stress ratio, σ_{SY}/σ_{SX}	0	0.30	1.00
Factor	1.00	0.85	0.75

NOTES:

1 The table refers to walls and slabs with orthogonal reinforcement as shown in Figure C3.3. The X direction is arbitrarily assumed to be the direction of the more highly stressed reinforcement.

2 The angle of cracking (α) is also illustrated in Figure C3.3. Its value can be determined using elastic theory, and is a function of the ratio of stresses in the two directions (Ref. 4). This ratio in turn is a function of the applied actions at the section and the reinforcement section areas (A_{SX}) and (A_{SY}) in the two directions.

3 Intermediate values of the factor in the above table may be determined by interpolation.

C3.2.4 Limiting concrete thickness

Where hoop force is the principal force and the concrete is assumed to be fully cracked, i.e. the reinforcement carries the full hoop force, the concrete thickness must be limited by the required cover to the reinforcement (see Clause 4.4), the size of the reinforcement, adequate space for placing and compacting the concrete and, where appropriate, the—

(a) concrete stresses due to —

  (i) moment, shear and torsion; and

  (ii) transfer of applied load to tension reinforcement (fully cracked section);

(b) stability, i.e. buckling due to backfill or wind load; and

(c) serviceability, i.e. deflection.

C3.3 PRESTRESSED CONCRETE

C3.3.1 General

Usually materials and construction methods used for prestressed elements of liquid retaining structures are 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.

C3.3.2 Analysis

The following considers 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 distribution depending on the degree of base restraint, as well as the level and distribution of the applied circumferential prestress force. The instantaneous vertical bending moments induced by circumferential prestress can be modelled by considering the prestress to be an equivalent radially inward pressure, as shown in Figure C3.4, and calculated from the following equation:

$$P_p = P/s_t a \qquad \text{. . . C3.3.2}$$

where

P_p = equivalent radially inward pressure, in megapascals

P = tendon forces, in meganewtons

s_t = centre-line spacing of tendons, in metres

a = radius of tank, in metres

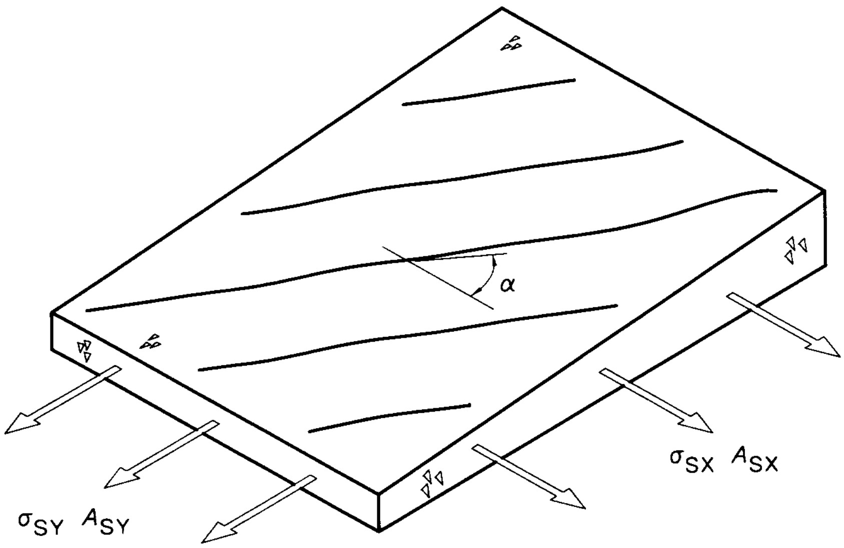


FIGURE C3.3 INCLINED CRACKING

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 inward 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 Figure C3.5. 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.

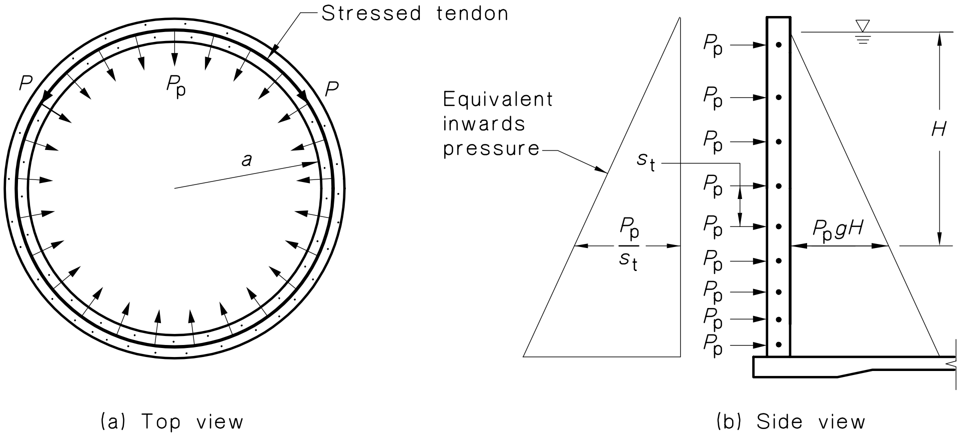


FIGURE C3.4 SIMULATION OF PRESTRESS AS RADIALLY INWARD PRESSURE

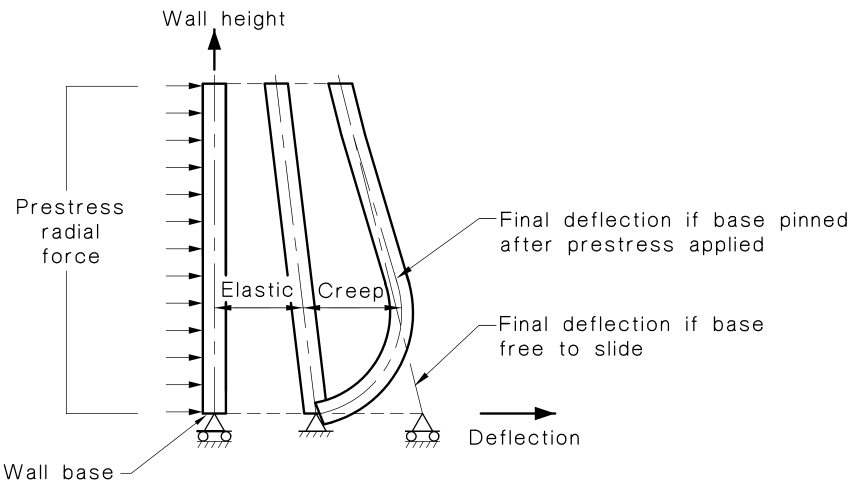


FIGURE C3.5 RADIAL DEFLECTION OF WALL UNDER PRESTRESS

C3.3.3 Limiting concrete stresses for serviceability

The limiting concrete stresses given in Table 3.5 ensure there is residual compression under long duration loads, but allow significant tension stresses under strain-induced load combinations or seismic loading. Shear stresses are unlikely to affect 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 operating through the wall thickness.

Figure C3.6 illustrates the procedure for a typical tank wall subjected to axial compression force, moment and shear per unit length of wall. The shear force may be found from the slope of the vertical bending moment diagram, and will be a maximum at the base of the wall. The linear distribution of direct stress, due to axial load and bending moment (see Figure C3.6(b)) is calculated from the following equation:

$$f_d = \frac{F_a}{t_w} + \frac{12My}{t_w^3} \qquad \text{. . . C3.3.3(1)}$$

where

f_d = linear distribution of direct stress, in megapascals

F_a = axial compression force per metre length of wall, in meganewtons

M = bending moment per metre length of wall, in meganewton metres

y = distance from centre-line of wall to the plane considered, in metres

t_w = thickness of wall, in metres

The parabolic distribution of shear stress may be expressed by the following equation:

$$\tau = \frac{1.5V}{t_w}\left[1 - \left(\frac{2y}{t_w}\right)^2\right] \qquad \text{. . . C3.3.3(2)}$$

where

τ = shear stress, in megapascals

V = shear force per metre length of wall, in meganewtons

From Mohr's circle for stress, the principal tensile stress is calculated from the following equation:

$$f_t = \frac{f_d}{2} - \sqrt{\left(\frac{f_d^2}{4} + \tau^2\right)} \qquad \text{. . . C3.3.3(3)}$$

where

f_t = principal tensile stress, in megapascals

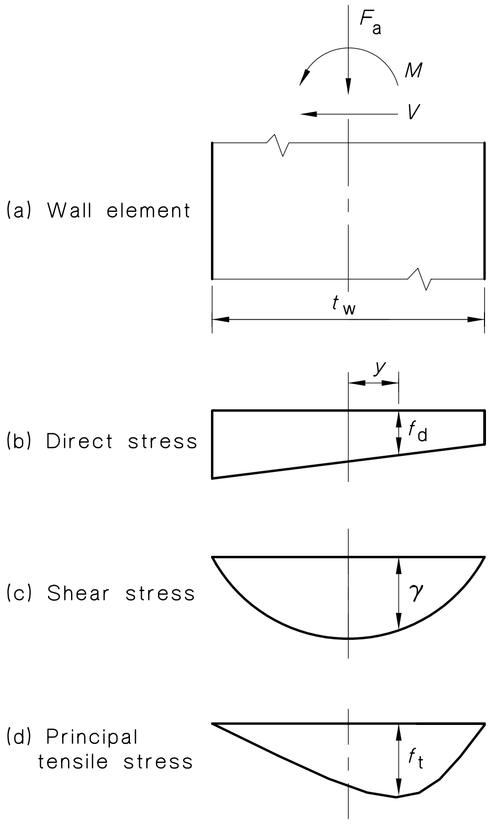
The distribution of f_t is asymmetrical, with the maximum occurring close to the centre of the wall, but offset to the side of reduced flexural compression stress, as shown in Figure C3.6(d). For most cases it is only necessary to check f_t at the centre of the wall, i.e. $y = 0$, and this may be calculated from the following equation:

$$f_{to} = \frac{F_a}{t_w} - \sqrt{\left[\left(\frac{F_a}{t_w}\right)^2 + \left(\frac{1.5V}{t_w}\right)^2\right]} \qquad \text{. . . C3.3.3(4)}$$

where

f_{to} = principal tensile stress at the centre of the wall, in megapascals

In Equations C3.3.3(3) and C3.3.3(4) the sign convention used is compression positive.



LEGEND:

F_a = axial compression force, in meganewtons per metre length of wall
M = bending moment per unit length of wall, in meganewton metres
V = shear force, in meganewtons per metre length of wall
t_w = thickness of wall, in metres
y = distances from centreline of wall to the plane considered, in metres
f_d = linear distribution of direct stress, in megapascals
γ = shear stress, in megapascals
f_t = principle tensile stress, in megapascals

FIGURE C3.6 PRINCIPAL TENSILE STRESS DUE TO SHEAR

REFERENCES

1 BRITISH STANDARDS INSTITUTE. BS 8007 *Design of concrete structures for retaining aqueous liquids*, Milton Keynes: 1987.

2 CAMPBELL-ALLEN D.C. and HUGHES G.W. Reinforcement to control thermal and shrinkage cracking. Transactions of the Institution of Engineers Australia, *Civil Engineering*, August 1981, vol. CE 23, no. 3.

3 CEBFIP *Model code for concrete structures*, Paris: Comite Euro International du Beton, 1978. MC 78

4 LEONHARDT F. and MONNIG E. *Vorlesungen uber massivbau Zweiter Berlin*: Teil, sonderfalle der Bemessung in Stahlbetonbau, Springer-Verlag, 1975.

SECTION C4 DESIGN FOR DURABILITY

C4.1 GENERAL

The design for durability gives requirements for structures built from Portland cement concrete having a design life of 40 to 60 years. It should not be construed that structures built to these requirements will last 40 to 60 years. The true useful life will depend on maintenance, the suitability of the structure for its purpose as time goes by and a variety of other factors applicable to each individual situation.

The designer's attention is drawn to the principal factors that require attention to ensure the concrete structure will have the appropriate standard of durability.

Concrete, especially reinforced concrete, is a very complex product made up of the chemical characteristics of the individual components, such as aggregates, cement, additives, embedded steel reinforcement, and is not the inert, stable, 'artificial stone' which it has in the past been thought to be. It is a very good building product, provided, like any other material, precautions are taken to ensure it remains durable. Again, like any other building material, the Standard provides for durability by specifying—

(a) the material to be of a suitable quality;

(b) an additional amount or thickness of material (cover) that, although it would deteriorate, would provide protection for the required life; and

(c) a coating or liner that although it may have to be renewed during the required life would protect the material from deterioration.

The designer should choose the most appropriate combination of requirements.

The Standard provides minimum requirements for durability rather than forcing designers to provide an inflexible and conservative solution. Thus, because of the highly complex nature of an individual situation it is the responsibility of the designer to assess the situation and the related economics, and provide the most appropriate solution.

The cement matrix of hydrated Portland cement is basic to the durability of concrete as it determines the strength and provides a protective film on the steel reinforcement. Although involved, this matrix is basically composed of calcium silicates and aluminates plus some smaller amounts of iron and magnesium. During hardening, the cement hydrates and precipitates in the water-filled capillaries as an extensive coiled flake-like molecule of mainly hydrated calcium silicates and some aluminates. The water in the capillaries becomes saturated with calcium hydroxide and solid calcium hydroxide precipitates. This process continues until the silica/water/lime system is in equilibrium.

All the water cannot be used up in the hydration process, and the cement matrix is a finely balanced system of solids and lime-saturated water. Any addition of water will dilute the water in the pores and thus, while unhydrated cement remains, will induce further hydration until the system regains its balance; however, migration of water through the gel will also gradually dissolve and leach out the lime. As the system attempts to regain its equilibrium, calcium is lost from each component and the concrete is weakened. Consequently the cement gel is susceptible to permeating liquids and deterioration can result from the following:

(i) Direct chemical reaction.

(ii) The development of pressures in the gel pores resulting from the swelling of gel products into those pores and osmotic pressures. Sulfate can produce this effect by its combination with calcium hydroxide to form calcium sulfate (gypsum) and then of gypsum and hydrated tricalcium aluminate (C_3A) to form calcium sulfoaluminate (ettringite). Although ettringite has a greater volume than its original components it appears to be the osmotic forces that produce the bursting pressures in the pores.

(iii) Loss of strength resulting from leaching of gel components.

The rate at which this deterioration occurs represents the rate of corrosion of the concrete. Factors that affect this include the following:

(A) *The aggressiveness of the liquid* Water saturated with calcium carbonate (i.e. with a positive Langelier Index) will not dissolve and leach the lime whereas liquids with a low pH, i.e. low alkalinity, free carbon dioxide content will be particularly active.

(B) *Carbonation of the lime* This produces a much more insoluble and slightly more impervious gel. However, the pH of the water in the gel pores will be reduced so that the protective film on steel reinforcement will be destroyed.

(C) *Access of liquid to the gel pores* Reduced permeability and area presented to the liquid will reduce the rate of attack or leaching.

(D) *Rate of replenishment of the liquid* With adequately low rates of replenishment, the liquid in the surface pores will be neutralized. Thus the rate of corrosion will be controlled by the rate of diffusion of the liquid and its capacity to dissolve lime or react with the gel components. This has little bearing, however, on moving or agitated liquids, as the liquid would be replaced before it is neutralized. Also, agitated or moving liquids are more likely to remove weakened surface layers that would have provided a barrier to the liquid.

The silica/water/lime balance in the cement gel provides a high pH environment, which is ideal for depositing a protective film on black or zinc coated steel reinforcement. These films can be destroyed by either of the following:

(1) Carbonation of the lime in the cement gel and lowering of the pH of the pore water. The protective film will then become depassivated and corrosion can occur. Black steel reinforcement will depassivate below about pH 11 and zinc coated reinforcement about pH 8.

(2) Introduction of chloride ions to the region of the reinforcement, which will cause the protective film to go into solution and leave the reinforcement susceptible to corrosion. Chloride ion contents in the range 0.2–0.4% (by mass) of cement have been found to result in corrosion of black steel. Zinc coated reinforcement has been shown to be able to sustain 4 to 5 times the chloride ion content (compared with black steel) before depassivation occurs.

The corrosion of reinforcement will be enhanced by circumstances that promote the permeation of carbon dioxide, chlorides, oxygen and aggressive liquids into the reinforcement zone of the concrete. Alternate wetting and drying, hydrostatic head differences, impacting flow in combination with permeable concrete and small covers are important typical factors.

C4.2 EXPOSURE CLASSIFICATION

The exposure classification refers to the exposed surfaces of a concrete member and indicates the aggressiveness of the environment to these surfaces.

Four basic exposure classifications for exposure to chemical or penetrating agents are defined in terms of their resultant effect on a concrete member as follows:

A—Where the concrete is in a non-aggressive environment or is protected from aggressive agents. This classification may be appropriate for surfaces protected or isolated from the attacking environment or where a lower level of durability is applicable (see Ref. 1).

B—Where the concrete is in an aggressive environment but only subjected to agents to which normal concrete of adequate quality is resistant. This is the lowest category applicable to concrete members in contact with water or condensation.

C—Where aggressive agents will attack the concrete but provision of a superior quality will enable the member to remain serviceable for the required design life.

D—where the concrete is subject to an environment that will attack the concrete to such an extent that the required design life cannot be met, i.e. cannot retain or exclude the liquid in an acceptable manner.

These exposure classifications are further subdivided by a number such that the sequence A1, A2, B1, B2, C, D indicates an increasing severity of attack. An exposure classification U indicates no guidance is given.

Although a concrete surface can be protected from the environment by a variety of means, such protection must remain intact and effective for the life of the structure if the benefit is to be realized.

A guide to the exposure classification to be assigned for a particular liquid environment is given in Table 4.1. Additionally, reference will need to be made to Ref. 1 to determine if any other exposure classification is more critical.

Table 4.1 is a general table only indicating exposure classifications for liquids that are more common and the range of exposure classifications for liquids that are less common. Additional details to assist in selecting an appropriate exposure classification from within these ranges is given in the following commentary.

Water supplies throughout Australia vary greatly in their characteristics. They also vary greatly as they flow along cementitious pipelines or pass through treatment plants. Characterizing these waters in terms of their aggressive reaction with concrete is very complex. However, provided significant quantities of highly reactive aggressive dissolved materials are not present, the sign of the Langelier Saturation Index and the pH provide a simple indicator of the applicable exposure classification (see Table 4.1, Item 1). With current technology this is considered to be as reliable as factors obtained by more complex analysis, e.g. aggressive CO_2.

Some examples of Langelier Saturation Indexes (*LI*) and pH determined for a range of water supplies across Australia are given in Table C4.1.

If a chemical analysis of the water is not available, an indication of the sign of the Langelier Saturation Index can be determined by immersing an electrically connected copper bar and a mild steel bar in the water. If calcium carbonate deposits at the copper bar, then the Langelier Index is positive and the water will be passive or protective of any grade of concrete.

In some instances the water will deposit a protective film or slime on the concrete. In other instances algal or other growths can occur on the surface. When these die they produce acid conditions, which would attack the concrete. Both of these conditions would be difficult to predict at the design stage. However, the exposure classifications indicated in Table 4.1 would remain satisfactory in most instances.

Domestic sewage, both fresh and stale, is basically non-aggressive to concrete. The main risk is the anaerobic generation of sulfides within the sewage, which occurs particularly in slowly moving or stagnant systems where slimes occur, and the consequent formation and release of gaseous hydrogen sulfide (H_2S) into the space above the liquid. Turbulence increases the release of H_2S and care should be taken in assessing the exposure classification for structures downstream of discharge from rising mains and drops. Air with H_2S concentrations greater than 0.5 mg/L should be rated as a high risk for H_2S corrosion (see Table 4.1, Item 2(b)). Aggressive attack on the material in the gaseous space will occur due to the anaerobic bacterial conversion of gaseous H_2S and oxygen to sulfuric acid in the condensed moisture film on these exposed surfaces. This problem would not occur with fresh sewage or with stale sewage where adequate precautions have been taken, e.g. adequate ventilation. If not avoided, a very severe attack will occur, which is indicated by an exposure classification of D (see Table 4.1, Item 2(b)).

Sewage from common effluent schemes should be considered in the same manner as any other sewage. Generally however, where these schemes are small, predominantly domestic and do not involve long rising mains then the exposure classification for fresh sewage (see Table 4.1, Item 2(a)) would be applicable.

Industrial sewage may contain various aggressive substances additional to those in domestic sewage. Similarly, waste water may contain a variety of substances of varying severity, the effects of which determine the most applicable exposure classification (see Table 4.1, Items 1, 2, 4 and 5).

Care should be taken with structures founded in soils containing decomposing marine vegetable matter such as old tidal seaweed deposit sites or mangrove swamplands. Ground waters in these areas will be high in dissolved hydrogen sulphide. Thus any situation that permits the escape of the hydrogen sulphide into a concrete enclosed air space will result in an exposure classification similar to that of stale sewage.

Concrete structures permanently submerged in seawater suffer minimal attack. The attack that does occur is generally limited to what appears to be a mild sulfate attack at the surface of the concrete. However, in other situations such as in the splash or tidal zones and where a hydrostatic head difference exists, the consequences of salt and chloride penetration are the principle forms of degradation. Degradation resulting from the sulfates in the sea water appears to be retarded by the chloride (see Table 4.1, Item 3).

Typically, the pH of seawater is 8.0 ±0.5. For unusual conditions such as protected areas where large amounts or organic matter collect and decay, the pH may drop below 7.5 indicating a higher level of aggressive carbon dioxide.

Research (Ref. 2) has shown that cements containing higher contents of tricalcium aluminate are more effective in protecting reinforced concrete exposed to seawater from corrosion. While this is of secondary importance to ensuring low permeability, the use of sulfate-resisting cement containing low levels of tricalcium aluminate is discouraged. Cements containing more tricalcium aluminate than Type A should not be used as early investigations indicate significantly increased chemical attack and spalling (Refs 13 and 14).

The use of pozzolanic material (e.g. fly ash) with normal Portland cement has been shown to be beneficial in increasing the resistance to attack by seawater provided the concrete is adequately cured (Ref. 14). It should also be noted that Ref. 1 requires the concrete to be air entrained should freezing conditions be encountered.

Because depassivation of the reinforcement is a serious concern for concrete immersed in or retaining seawater, galvanizing or epoxy coating (without defect) all steel reinforcement, tendons, ducts and embedded steel should be considered. However, for this to be successful it must be done correctly and to an appropriate standard. Details are provided in Section C5 of this commentary.

In special circumstances the use of austenitic stainless steel would be appropriate. Waterproofing agents have also been successful in providing resistance to permeation of salts and chloride ions. To be effective they should either have been demonstrated to reduce the permeability by at least 100 times or have been shown to be successful by testing in the field.

Adoption of galvanized, epoxy coated or austenitic stainless steel or a suitable waterproofing agent could significantly reduce the exposure classification given in Table 4.1; however, no recommendation for a reduction in the exposure classification can presently be given.

TABLE C4.1

LANGELIER SATURATION INDEXES (*LI*) VALUES FOR TYPICAL AUSTRALIA WATER SUPPLIES

	LI	pH	Basis	Applicable Item in Table 4.1
Kulin bore water supply (W.A)	+1.3	9.7	Single sample	1(a)
Roe Creek Mereenie borefield Alice Springs (N.T.)	0.0	7.5	single sample	1(a)
North Pine Water Treatment Plant (Qld) — raw water — treated water	–0.4 –2.1 –1.1 –1.9	8.2 6.9 7.3 7.0	year average min. value year average min. value	1(b)
Coal River at Crigburn Rd. (Tas.)	+0.1 –3.3	7.9 6.6	mean value min. value	1(b)
River Murray at Mannum (S.A.) — before chlorination — after chlorination	 –1.3 –1.8	 7.3 6.8	 single sample single sample	1(b)
Anstey Hill Water Filtration Plant (S.A.) — pre alum — post alum — end of filtration	 –0.9 –1.3 –0.8	 7.5 7.2 7.5	 single sample single sample single sample	1(b)
Middle River Reservoir (S.A.) — at dam — at Kingscote (A/C pipeline)	 –1.6 +0.8	 7.2 9.1	 single sample single sample	1(b)
Darwin River Dam (N.T.)	–2.7	7.8	Average	1(b)
Darling Range rivers and storage (W.A.)	–1.8 –3.6	7.5 6.5	max. value min. value	1(b)
Cardinia Reservoir (Vic.)	–2.9	7.0	year average	1(b)
Campbelltown Reservoir (N.S.W.)	–4.0 –4.8	6.8 6.3	year average year min.	1(c)
Henty River downstream of Zeehan Rd. Bridge (Tas.)	–3.4 –6.8	6.8 5.2	mean value min. value	1(c)

Table 4.1, Item 4 provides a guide to the exposure classification when in contact with liquids or associated vapours or gases that will chemically attack or corrode the concrete. Since this attack is a direct chemical reaction, no increased exposure classification for intermittent submergence is generally relevant. However, care must be taken with any aggressive gases released at the surface of a liquid. Situations where displacement of corrosion products would occur (e.g. agitation, flow or mechanical means) must be taken into account in assessing the severity. Examples indicating the severity used in Table 4.1 are given in Table C4.2.

Table 4.1, Item 5 provides an exposure classification for other liquids that can be stored in or associated with concrete structures. Guidance for the selection of an appropriate exposure classification from within the range indicated is given in Table C4.3. Although some of these liquids do not corrode the concrete or the reinforcement, they are very penetrating. Care must be taken with light oils as they can reduce the strength of the concrete as well as the bond strength between the reinforcement and concrete. Unless very low permeability concrete without cracks can be provided, sealing of the concrete surface or lining of the storage is recommended.

Care must be taken—

(a) to ensure the coating or lining is unaffected by the liquid or its vapours; and

(b) to prevent the accumulation of gas or vapours that may permeate the concrete members of a structure, e.g. petrol vapour.

Table 4.1, Item 6 gives the range of exposure classifications for concrete members in or on the ground and in contact with ground water. It should be noted that the lowest exposure classification for a concrete surface retaining and in contact with water is B1. Exposure classifications A1 and A2 would only be applicable when an impermeable membrane is used or for some other completely non-aggressive (passive) liquid.

Table C4.4 is provided to assist in selecting a specific exposure classification from within the range given in Table 4.1, Item 6.

Impermeable soils such as plastic clays will greatly reduce or even prevent the replenishment of the ground water at the face of the concrete structure. As the ground water would be quickly neutralized, the attack would be minimized and a lower exposure classification would be applicable.

This has been allowed for in the 'low ground water replenishment rate' column. With greater ease of replenishment such as with rubble or sand backfill or less well drained areas the exposure classification would have to be increased toward that shown in the 'high ground water replenishment rate' column.

The use of calcareous aggregate has been found to be beneficial in corrosive and particularly acid conditions.

Provided—

(i) the coarse aggregate is calcareous and has a minimum acid solubility of 90% when tested in accordance with Ref. 3; and

(ii) cover is increased by an amount equal to the loss that would occur over the design life and this additional cover is not considered to make any contribution to the strength of the member,

then a lower exposure classification than shown in Table 4.1 would provide the equivalent durability.

The amount by which the exposure classification is reduced depends on the amount of additional cover provided. However, it is recommended that the exposure classification should not be reduced more than one level and cover should be increased by at least 25%. Any reduction in exposure classification D should be carefully assessed.

TABLE C4.2

SEVERITY OF CORROSIVE LIQUIDS, VAPOURS OR GASES TO CONCRETE

Typical example of environment	Aggressiveness
Dilute cyanide solution Tanning liquor Tannic acid vapours and precipitates Oxalic acid Fresh animal fats (regularly cleaned away) Fruit juices containing citric or tartaric acid	Slight
Dry ammonia or chlorine gas Molasses Soap, glycerine solutions <2% concentration v/v Fresh organic oils (vegetable, seed, animal, fish) or high viscosity, and not exposed to air and moisture (e.g. stored in tank) Food processing wastes (dairy, fruit, vegetable, abattoir, distillery) that are cleaned away on a regular daily basis and where mechanical wear is limited by light use Apple juice (containing malic acid) Ferrous metallurgy wastes (e.g. blast furnace, rolling mill)	Mild
Wet ammonia gas (sprays, condensation) Sugar solutions Glycerine solutions 2–4% concentration v/v Acetic acid < 0.5% concentration v/v Fresh organic oils of low viscosity and not exposed to air and moisture Urine Industrial and winery effluent with pH 5.5 to 6.5 Inorganic (mineral) acids and waters with pH 5.5 to 6.5	Moderate
Ammonia salts (e.g. in a fertilizer factory) Wet chlorine gas (sprays, condensation) Bleaching dyes (containing chloride) Food processing wastes—normal condition but where mechanical wear is limited by light use Tannic acid Citric and malic acid <1% solution v/v Acetic acid 0.5–5% solution v/v (e.g. vinegar liquid)	
Vinegar vapours in well-ventilated room Glycerine solutions 4–10% concentration v/v Soap, oilseed where mechanical wear is limited by light use High viscosity organic oils that are exposed to air and moisture Photographic solutions Fermenting food products (e.g. sugar, stale beer) Dilute lactic acid <1 % solution v/v (e.g. whey) Dilute butyric acid <1 % solution v/v (e.g. sour silage)(fermentation of material containing chlorophyll) Rancid animal fats Inorganic acids and waters with pH 4.5 to 5.5	Severe

(*continued*)

TABLE C4.2 (*continued*)

Typical example of environment	Aggressiveness
High molecular weight organic acids (insoluble in water)(e.g. stearic, oleic, palmitic) other than as a constituent of thin oil Concentrated moist sewer gas Liquid chlorine Glycerine solutions >10 % concentration v/v Acetic acid > % concentration v/v Concentrated vinegar vapours Food pickling wastes Dairy—milk and cheese processing wastes Fermenting or rotting food products where mechanical wear occurs Low viscosity organic oils that are exposed to air and moisture High molecular weight organic acids that are constituents of thin oils Industrial wastes containing >1 % concentration v/v of magnesium chloride Industrial, winery and distillery wastes with pH <4.5 Inorganic acids and water with pH <4.5	Extreme

TABLE C4.3

EXPOSURE CLASSIFICATION—OTHER LIQUIDS

Characteristic of liquid in contact with concrete surface	Predominantly submerged		Alternate wet and dry (condensation, splashing or washing)
	Generally quiescent	Agitated or flowing	
Water with chloride ion (Cl) content >2 500 mg/L			
2 500 mg/L to 5 000 mg/L	B1	B1	B2
5 000 mg/L to 50 000 mg/L	B2	B2	C
> 50 000 mg/L (brine)	C	C	D
Water with sulfate ion (SO_4^{2-}) content > 400 mg/L (1)			
400 mg/L to 1 500 mg/L	B1	B2	B2
1 500 mg/L to 3 000 mg/L	B2SR	CSR	CSR
3 000 mg/L to 6 000 mg/L	CSR	CSR	CSR
> 6 000 mg/L	DSR	DSR	DSR
NOTE: Equivalent sulfur trioxide SO_3 = 0.83 × sulfate ion (SO_4^{2-})			
Water with magnesium ion (Mg^+) content > 1000 mg/L (2)			
Total (Mg^+ + SO_4^{2-})			
1 000 mg/L to 5 000 mg/L	B1	B2	C
5 000 mg/L to 7 000 mg/L	B2	C	C
> 7 000 mg/L	D	D	D
Water with ammonium ion ($NH^+{}_4$) content > 100 mg/L			
100 mg/L to 250 mg/L	B2	B2	B2
250 mg/L to 500 mg/L	C	C	C
> 500 mg/L	D	D	D
Wine	B1	—	—
Non-corrosive vegetable oils—			
Rosin	A1	A1	A1
Methylated spirits	D	D	A1
Turpentine	D	D	B1
Mineral oils—			
Heavy and medium petroleum oil	A1	A2	A1
Light petroleum oil			
—Sp.gr. > 0.875 at 15°C	B2	B2	B1
—Sp.gr. > 0.85 to 0.875 at 15°C	D	D	C
Pure petrol, diesel oil, etc.	D	D	A1
Kerosene	D	D	B1
Containing vegetable oil, e.g. cutting oil for machining	C	C	C
Coal tar products—			
Solvents (acetone, etc.)	D	D	B1
Phenols, cresols (e.g. Creosote, cresol), etc.	B2	B2	B2

NOTES:

1 The exposure classifications that contain the letters SR require the concrete to be sulfate resistant.

2 The following methods of providing sulfate resistance would be acceptable:

(a) Use a type D (sulfate-resistant) cement.

(b) Use pozzolanic material (e.g. fly ash or silica fume) with normal Portland cement (Type A) where this has been shown by tests to provide an equivalent sulfate resistance to Type D cement. A suitable test would be to soak mortar bars in a 5% sodium sulfate solution after they have been cured for 14 d, then compare the expansion at 28 d and 90 d and the time to 5% expansion.

(c) Use a waterproofing agent with normal Portland cement (Type A) provided it can be shown by tests (as in Item (b)) to be as effective as sulfate-resistant cement.

3 If sulfate resistant cement cannot be provided, then the exposure classification two higher is applicable.

4 The use of silica fume is discouraged (Ref. 34).

TABLE C4.4

EXPOSURE CLASSIFICATION—GROUND WATER

Nature of ground water in contact (see Note 1) with concrete surface	Ground water replenishment rate	
	Low, e.g. clay (see Note 3)	High, e.g. sand (see Note 3)
Normal inert or alkaline soils—	A1	A1
Saline (Chloride containing soils) (See Note 2)		
Mild—resistivity when damp > 50 Ω.m	A1	A2
Moderate—resistivity when damp 30 Ω.m to 50 Ω.m	A2	B1
High—resistivity when damp 10 Ω.m to 30 Ω.m	A2	B2
Very high—resistibility when damp < 10 Ω.m	B1	C
Salt-rich desert areas	B1	C
Sulfate-containing soils—		
NOTE: Equivalent sulfur trioxide SO_3 = 0.83 × sulfate ion SO_4^{2-}		
SO_4^{2-} < 0.2% in soil or 400 mg/L in ground water	A2	B1
SO_4^{2-} 0.2% to 0.6% in soil or 400 mg/L to 1500 mg/L in ground water	B1	B2 or B1SR
SO_4^{2-} 0.6% to 1.2% in soil or 1500 mg/L to 3000 mg/L in ground water	B1	B2SR
SO_4^{2-} 1.2% to 2.4% in soil or 3000 mg/L to 6000 mg/L in ground water	B2 or B1SR	CSR
SO_4^{2-} > 2.4% in soil or 6000 mg/L in ground water	B2SR	D
Acidic soils—		
pH > 6.5	A1	B1
pH 5.5 to 6.5 (e.g. very acidic natural soils)	A2	B2
pH 4.5 to 5.5 (e.g. ash from power station)	A2	B2CA
pH 3.5 to 4.5 (e.g. pyrites mine area)	B1	CCA
pH < 3.5 (e.g. pyrites mine tailings)	B1CA	D

NOTES:

1 Where it is essential to avoid the transmittance of moisture or salt damp through the concrete the ground water should be isolated from the structure by a suitable waterproof membrane. Where minimal transmittance of moisture or salt damp is acceptable, the concrete should be specified with a characteristic strength (f'_c) of at least 40 MPa or it should contain an effective waterproofing agent. An appropriate thickness of concrete, e.g. 100 mm + 0.05 × head of ground water in mm, should be provided.

2 Particularly in high and very high salinity soils that are permeable, evaporation and capillary action can result in spalling of concrete protruding above the ground. Therefore, consideration should be given to installing a membrane or adding an effective waterproofing agent.

3 The letters following the exposure classification indicate these classifications are dependent on the following:

4 SR—The concrete is sulfate resistant as specified in the Note to Table C4.3.CA—The coarse aggregate is calcareous with cover increased, detailed as follows.

C4.3 REQUIREMENTS FOR CONCRETE

Brief explanations of the requirements for concrete are as follows:

(a) *Characteristic compressive strength (f'_c)* The use of the characteristic compressive cylinder strength of concrete at 28 d (f'_c) as an indicator of durability is justified with modern concretes by the fact that this strength is similar to the compressive strength that would exist over the life of the structure. This was not so with older concretes. For example, the compressive strength of a sample of concrete cast at the turn of the century could now be twice as much as it was when it was three months old. Thus when modern cements that developed concrete compressive strength more quickly were produced, a weaker concrete (in the long term) would result from making a concrete with a similar early-life compressive strength to the older concretes. This then resulted in the production of lower durability concretes and the use of less cement. Clause 4.3 specifies characteristic strengths appropriate to the required durability.

Although there are deficiencies, it is considered that the use of characteristic strength as an indicator of durability is more practical to specify and control and with present technology more reliable than other indicators (see Ref. 4).

The aim of the Standard is to provide a set of requirements that are simple, consistent and the most reliable in ensuring a durable concrete. The Standard and Ref. 1 have adopted a similar approach.

Characteristic strength is specified so that—

(i) the mix will have to be designed so that the resultant permeability of the cement matrix will be appropriate for the required durability if cured as specified;

(ii) there will have to be sufficient cementitious material to adequately coat and embed all aggregates (this is ensured by the specified minimum cement content);

(iii) aggregates and bond to aggregates will have to be of sufficient strength; and

(iv) the mix will have to be compacted (as may be checked by tests on cored samples).

The use of f'_c has the benefit of avoiding the conflict that often occurs between specifying cement and water content for durability and characteristic strength for structural strength and sometimes again for durability. When pozzolanic materials are added to the cement or weaker aggregates are used or a greater proportion of sand is used, such as in a pump mix, it compensates for the possible loss in durability, which can occur by relying only on cement content and water/cement ratio.

Tables 4.2 and 4.3 indicate the lowest f'_c that may be used with a specific exposure classification, as concrete of a lower strength would be subject to attack and not be sufficiently durable for the exposure indicated.

(b) *Degree of curing* The curing of concrete is to ensure that the hydration of the cement continues until the average compressive strength of the concrete is sufficient to limit subsequent self-desiccation and micro-cracking, and the permeability of the cement matrix to that applicable for the specified durability.

The Standard specifies the degree of curing as a percentage of f'_c because it—

(i) is consistent and independent of the curing technique, e.g. water at ambient temperature or steam curing (provided an adequate quantity of curing water is maintained);

(ii) provides the appropriate curing periods for cements that cure at different rates;

(iii) provides for differing curing rates resulting from differing concrete temperatures; and

(iv) provides flexibility in enabling a higher strength concrete to be used with a shorter curing period, which tests have shown to be a valid technique.

Compensation is provided to alleviate problems such as the lower durability that can result from inadequate curing of blended cement concrete specified only in terms of the number of days to be cured.

In normal situations, the duration and method of curing applicable to the concrete mix are specified so that the required degree of curing is achieved. Determination of the required curing period from trial mixes would not normally be a problem. However, where only normal Portland cement (Type A) is used, 7 d of moist curing at ambient temperatures would be acceptable.

(c) *Cement content* A minimum cement content avoids the possibility of insufficient cement paste for the full embedment of the coarse aggregate.

(d) *Drying shrinkage* A maximum drying shrinkage limits cracking resulting from shrinkage.

It also limits the total amount of water that can be added to the mix and thus limits the porosity for the characteristic strength (f'_c) specified.

(e) *Reactive alkali content* A maximum total reactive alkali content avoids the disintegration of concrete that can occur, due to the chemical action. This is not normally a problem with Australian aggregates but can occur, with siliceous aggregates, e.g. opaline.

No provision has been made to limit alkali-carbonate reaction, as this has not been recognized as a problem with Australian concretes.

(f) *Temperature differentials* It is necessary to limit temperature differentials and the resulting cracking during the period of setting and cooling. Techniques used for this control include—

(i) limiting high cement contents;

(ii) using slower hydrating cements (Type C);

(iii) incorporating fly ash;

(iv) cooling mix water;

(v) cooling mix ingredients;

(vi) using larger size aggregate; and

(vii) limiting thickness of section.

C4.4 REQUIREMENTS FOR COVER TO REINFORCING STEEL AND TENDONS

C4.4.1 General

The required dimensions for cover are given in the following tables:

(a) Where standard formwork and compaction are used .... Table 4.2.

(b) Where rigid formwork and intense compaction are used .... Table 4.3.

The dimensions are greater than that specified in Ref. 1 because a reduction in cover due to the permitted tolerance in placing of reinforcement is not acceptable.

The required cover to provide durability was determined by the expected depth of carbonation after a consistent duration of exposure. Thus the same durability can be attained using a lesser cover with a greater strength concrete.

C4.4.3 Cover for corrosion protection

As in Ref. 1, the larger dimensions of cover in Tables 4.2 and 4.3 include an allowance for mild surface deterioration.

C4.4.4 Cover modifications for special circumstances

C4.4.4.4 *Embedded items in cover zone*

The use of reinforcement chairs is often not permitted for liquid-retaining structures. This is because the reinforcement chairs have been found to create a path to the reinforcement for the retained liquid. However, if this problem is overcome then Clause 4.4.5.4 permits the use of reinforcement chairs (which themselves are unaffected by the retained liquid).

C4.4.4.5 *Allowance for abrasion*

The minimum characteristic compressive strength for concrete members of liquid-retaining or liquid-excluding structures that are subject to hydraulic abrasion from liquid velocities greater than 4 m/s, e.g. inlets, outlets and energy dissipators, should be as given in Table C4.5.

TABLE C4.5

HYDRAULIC ABRASION RECOMMENDATIONS

Hydraulic abrasion	Minimum characteristic compressive strength (f'_c), MPa — Maximum velocity of flow: 8 m/s	Minimum characteristic compressive strength (f'_c), MPa — Maximum velocity of flow: 16 m/s
Unimpeded non-abrasive flow;		
—smooth faces and joints	25	32
—formed slightly wavy faces	32	40
Unimpeded flow containing suspended abrasive material	40	40 (see Note 1)
Turbulent flow containing debris	50 (see Note 1)	50 (see Notes 2 and 3)
Impacting non-abrasive flow	50	25 (see Note 4)
Light or infrequent cavitation or liquid containing at least 8% air by volume	40	50 (see Note 2)
Higher levels of cavitation	60 (see Notes 2 and 3)	25 (see Note 4)

NOTES:

1 Trowel monolithic metallic aggregate into surface or use a very hard coarse aggregate, e.g. chert. (Note that aggregates such as chert may be subject to alkali aggregate attack.)

2 The concrete should contain or be impregnated with a suitable polymer.

3 Line with 50 mm minimum thickness of steel-fibre-reinforced concrete. Recommended steel-fibre reinforcement is as follows:

(a)	Maximum size of coarse aggregate, mm	10	20
(b)	Quantity of straight fibres, kg/m^3	85	50

(c) For deformed end-fibres reduce quantity to 60% of that shown.

(d) Steel fibres should be at least as long as maximum aggregate size.

(e) Aspect ratio of steel fibres should not exceed 100.

4 The concrete should be protected with a suitable facing. Grade 316, stainless steel sheet (Ref. 5) tightly anchored to the concrete has been satisfactory with velocities greater than 50 m/s. Other facings such as thick rubber sheet may be used if testing or experience has shown them to be satisfactory.

C4.6 DURABILITY OF WATERSTOPS AND SEALANTS

Although waterstops and sealants may be considered minor parts, they are critical to the success of a structure. Therefore, it is important that the durability or maintainability of these items be carefully assessed and that they should be appropriately installed (e.g. to enable them to be maintained if their durability is less than the life of the structure).

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SECTION C5 MATERIALS AND CONSTRUCTION REQUIREMENTS

C5.2 CONCRETE

Admixtures such as accelerators, retarders, plasticizers, form release and waterproofing agents, curing compounds and others should be used only when the proof that such admixtures agents or agencies will not have deleterious effect on the essential qualities of concrete is supported by substantial evidence.

Blended cement can also be used as a design aid to minimize thermal stresses during the early stages of hydration. Blended cement containing fly ash or slag, or both, will reduce the heat of hydration, which can be advantageous if the ambient temperatures are high and the structure contains thick elements. Additional curing may be needed to achieve either stripping or long-term strength and durability requirements.

The reaction of the alkalis in cement with reactive constituents in the concrete mix may cause deterioration due to the expansion of the byproducts. The use of blended cements will reduce the alkali-aggregate reaction (AAR) potential by reducing the alkali level in the concrete. The occurrence of AAR in Australia is largely confined to West Australia and Queensland.

C5.2.2 Curing

To ensure full curing and the elimination of micro-cracking resulting from evaporation induced shrinkage, it is essential that the surface layer of the concrete remain moist for the full duration of the curing period.

The curing period determined in accordance with Clause 4.3(b) is based on full moist curing at 100% r.h.

Curing that satisfies this Standard may also be undertaken by one of the following:

(a) Applying a curing compound with a curing efficiency in excess of 75% while the concrete surface remains moist and maintaining the membrane for a duration equal to twice the curing period.

(b) Providing conditions to ensure the relative humidity will remain above 80% and wind speeds will remain below 12 kn (6 m/s) for a duration equal to twice the curing period.

(c) Other methods whereby it can be demonstrated that the curing conditions will remain between those indicated in Item (b) and full moist curing at 100% r.h.

The efficiency of a liquid membrane forming compound or sheet material shall be determined from Ref. 1. To provide the degree of curing required by this Standard the curing efficiency should not be less than 75% (i.e. the 72 h moisture loss should not exceed 25% of that of an untreated sample).

C5.3 REINFORCEMENT

C5.3.1 Protective coating

C5.3.1.1 *General*

Protective coating is not considered to permanently protect reinforcement exposed to an aggressive environment beyond the protection provided by the alkalinity of the concrete.

C5.3.1.2 *Galvanized*

Use of hot-dip galvanized non-prestressed reinforcement is subject to the following:

(a) The bars are passivated in a 0.2% sodium dichromate solution applied by the galvanizer not more than 60 d before casting in the concrete. For safety reasons the application of chromium trioxide to the concrete mix is not recommended.

(b) Use of only galvanized tie wires and steel chairs.

(c) No use of re-bent galvanized reinforcement.

(d) No welding of galvanized reinforcement.

(e) Galvanized steel is not connected with any ungalvanized steel except where it is more than 10 times the required cover away from any surface in contact with liquid.

(f) No galvanized steel within 10 times the required cover of any copper or other electrochemically dissimilar metal.

(g) Use of only non-galvanized cold-worked or cold-formed deformed bars. Hard drawn steel wire fabric may be galvanized.

C5.3.1.3 *Epoxy-coated*

Epoxy coating may affect the bond strength of the coated reinforcement particularly in coated plain surface bars.

NOTE: Cathodic protection of steel in reinforced and prestressed concrete structures for retaining liquids is beyond the scope of the Standard but where such protection is considered, the engineer should obtain specialist advice.

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SECTION C6 JOINTS, WATERSTOPS, AND SEALANTS

C6.1 JOINTS

C6.1.1 Purpose

Joints in liquid retaining structures are critical in the performance of the structure. They may be only a relatively small proportion of the cost of the structure but can be responsible for a large proportion of the problems with leaking of the contained liquid. Any joint deterioration could result in very expensive remedial measures that could have been avoided.

The size of the usual type of liquid containment structures usually requires that construction will consist of several stages requiring joints between each section. The number and spacing of these joints will usually decide the design of the structure especially for early elastic and thermal movements, shrinkage of the concrete and movements due to wetting and drying cycles.

C6.1.2 Non-movement

Construction joints (see Figure C6.1) provide convenient breaks in the construction of structures and should be specified by the designer. Reinforcement will pass continuously through the joint and ideally the elements will behave as a monolithic unit with full structural continuity. When there is doubt as to whether a watertight joint and long-term prevention of reinforcement corrosion is required, a waterstop or surface joint sealant groove, or both, may be used (see Figures C6.2 and C6.3).

There should not be any movement across the joint and this is achieved by careful control of the preparation of the joining surfaces. Laitance is removed from the concrete and the aggregate exposed so that satisfactory aggregate interlock can take place. This is achieved by the use of a water spray and brushing with a stiff brush (2 h to 4 h after placing concrete), by the use of grit or water blasting or scabbling with a small air tool (several days after placing concrete) as long as the aggregate is not disturbed.

The wall/floor joint may be shown with a small 'kicker' to allow for assembly of subsequent wall formwork.

C6.1.3 Movement

C6.1.3.1 *General*

All movement joints are a potential source of leakage and it is important that the methods of sealing them are carefully designed. Because of this, as well as their cost, it is usually desired to minimize the number of movement joints but this is to be balanced against the need to provide the appropriate flexibility of the structure.

C6.1.3.2 *Isolation*

Isolation joints (see Figure C6.4) are used to provide an effective means of avoiding unacceptable cracking due to large movements or a large degree of restraint.

C6.1.3.3 *Expansion*

Expansion joints (see Figure C6.5) are designed to accommodate all anticipated longitudinal movements in a structure. The reinforcement stops clear of the joint and a gap is left which is filled with a compressible filler. Waterstops are usually required to maintain the watertightness of the joint.

If relative transverse movement between the sides of the joint is to be prevented, dowel bars can be used. These bars should be accurately located to allow the joint to move freely and this is usually accomplished by casting one end of the bar into the concrete and allowing the other end of the bar to move freely within a PVC pipe sleeve or covering the bar with a debonding compound. These bars will also transmit shear from one slab to the other.

C6.1.3.4 *Full contraction*

Full contraction joints (see Figures C6.6 and C6.7) consist of a discontinuity of both reinforcement and concrete across the joint. Waterstops and, where there is any possibility of debris entering the joint, joint sealants are essential. Dowel bars can be used to provide a capacity for shear transfer across a joint (see Figure C6.8).

Some means of bond reduction on the dowel bars will be required such as wrapping one end of the dowel bars with a bond breaking tape or placing the dowel bars within PVC sleeves (taped over on the end to prevent concrete entering the sleeve). Shear transfer by the use of keys within the concrete thickness are less effective than dowels due to the reduction of strength of the slab and they are also very expensive to form.

C6.1.3.5 *Partial contraction*

Partial contraction joints (Figures C6.9 and C6.10) may be constructed with the use of a stop end or by inducing a crack by a local reduction of the depth of concrete by 25% to 35% of the concrete thickness. Providing the percentage of reinforcement is reduced, this crack will release any stresses in the concrete allowing the joint to act as a contraction joint. A reduction in the percentage of continuous reinforcement through the joint allows the steel to yield locally with the resulting final effect that the joint acts as a normal contraction joint. There should be no need for dowel bars as well, because the continuous reinforcement across the joint should be sufficient to transfer any shear forces.

C6.1.3.6 *Hinged*

Hinged joints (see Figure C6.11) are usually located at the intersection of the wall and floor of some tanks to fulfil the design condition of allowing rotation with minimal restraint and resisting thrust and shearing forces. A common method is to place the wall into a groove in the footing.

C6.1.3.7 *Sliding*

Sliding joints (see Figure C6.12 and C6.13) have a complete discontinuity in both the concrete and the reinforcement, and allow movement with minimal restraint in the plane of the joint. The surface of the concrete should be very smooth and flat with a separating layer of a suitable material to allow movement to take place. Prestressed concrete tanks with a relatively large movement between the wall and the footing generally have this type of joint.

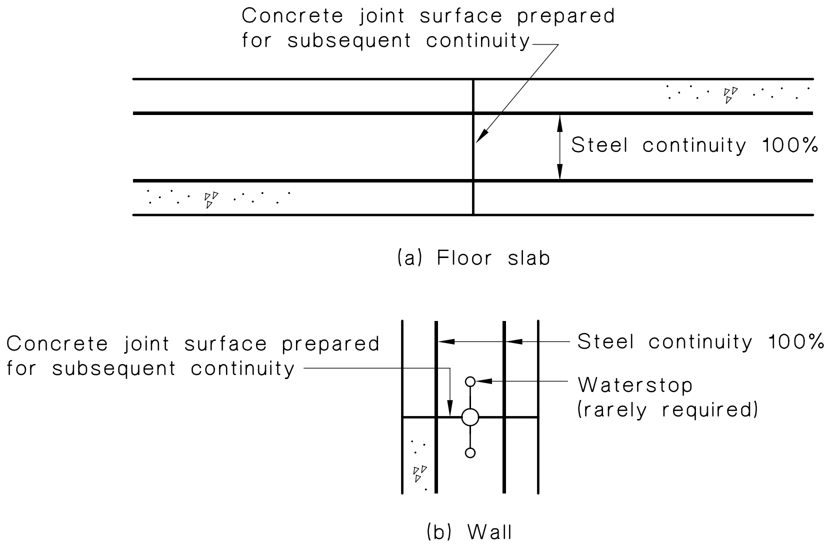

FIGURE C6.1 CONSTRUCTION JOINTS UNSEALED

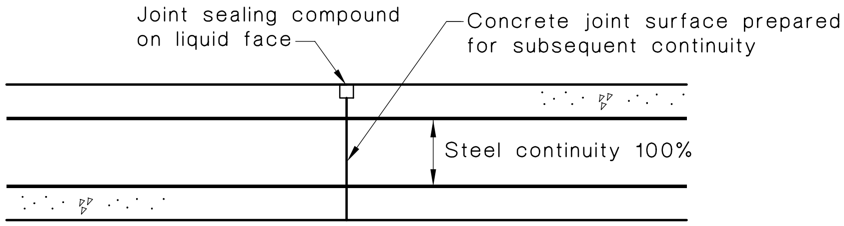

FIGURE C6.2 CONSTRUCTION JOINT—SEALED ON THE LIQUID FACE (NOT APPLICABLE FOR LOCATIONS WITH AN EXTERNAL WATER PRESSURE)

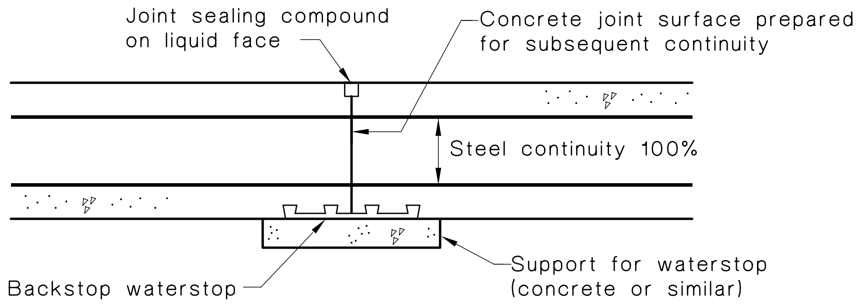

FIGURE C6.3 CONSTRUCTION JOINT—SEALED ON THE LIQUID FACE AND WITH A WATERSTOP (APPLICABLE FOR LOCATIONS WITH AN EXTERNAL WATER PRESSURE)

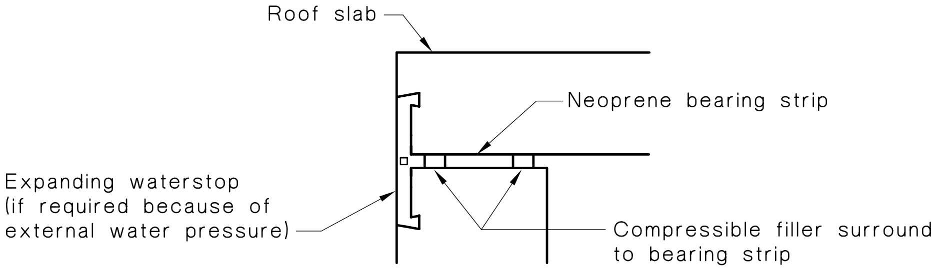

FIGURE C6.4 ISOLATION JOINT—BETWEEN WALL AND ROOF SLAB

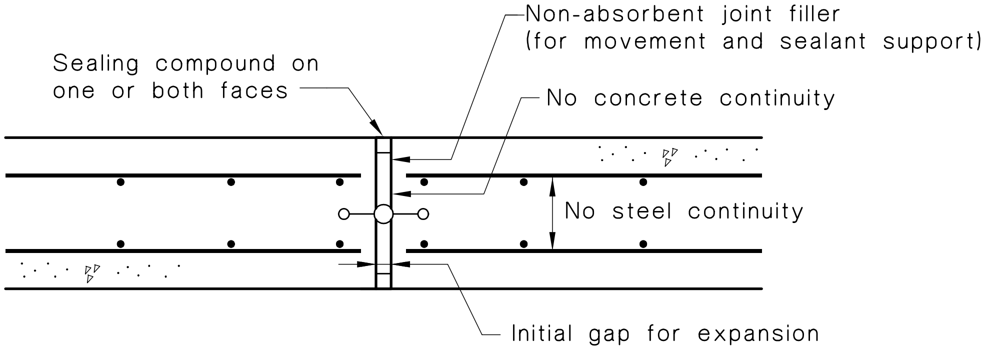

FIGURE C6.5 EXPANSION JOINT—WALL JOINT
(DOWEL BARS REQUIRED IF SHEAR TRANSFER REQUIRED)

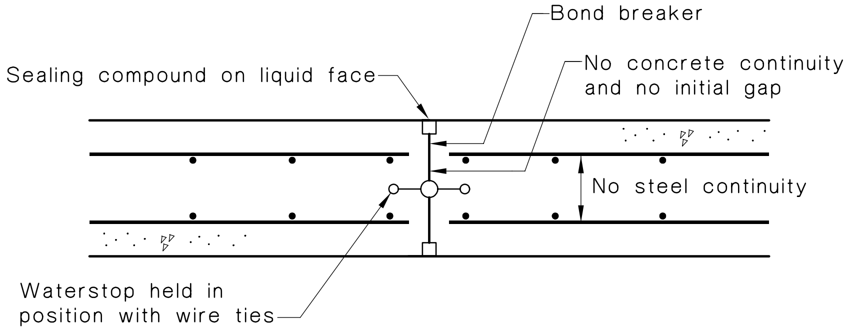

FIGURE C6.6 FULL CONTRACTION JOINT—WALL JOINT
(DOWEL BARS REQUIRED IF SHEAR TRANSFER REQUIRED)

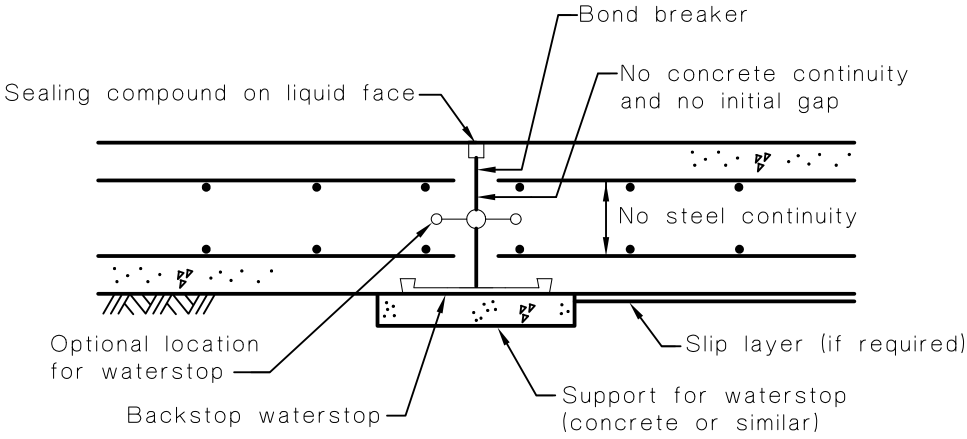

FIGURE C6.7 FULL CONTRACTION JOINT—FLOOR JOINT (DOWEL BARS REQUIRED IF SHEAR TRANSFER REQUIRED)

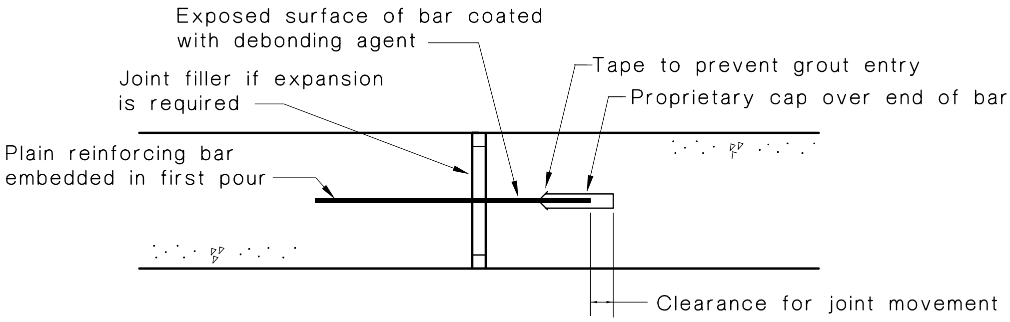

FIGURE C6.8 DOWEL BAR FOR SHEAR TRANSFER (NO STEEL CONTINUITY ACROSS JOINT)

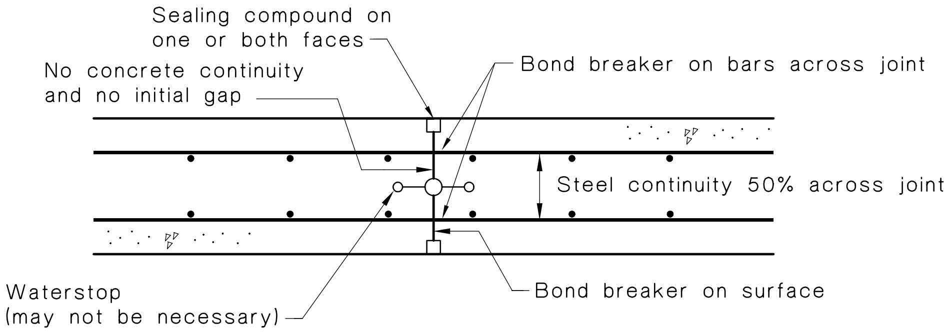

FIGURE C6.9 PARTIAL CONTRACTION JOINT—WALL JOINT

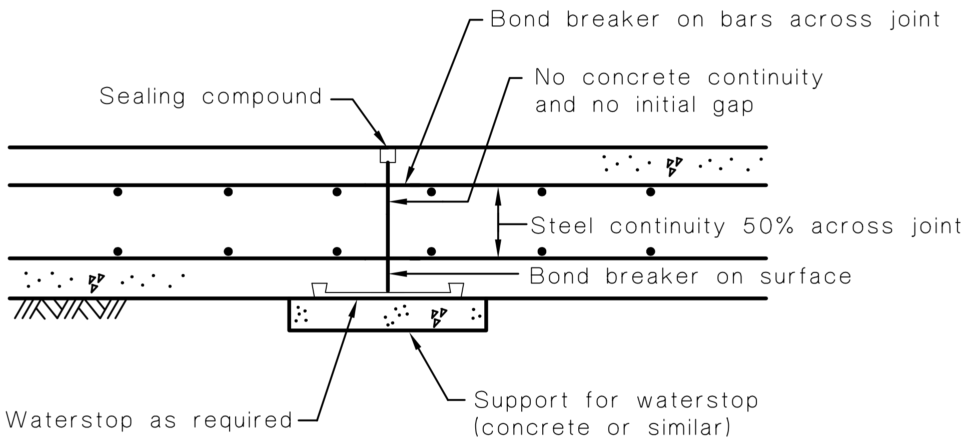

FIGURE C6.10 PARTIAL CONTRACTION JOINT—FLOOR JOINT
(APPLICABLE FOR LOCATIONS WITH AN EXTERNAL WATER PRESSURE)

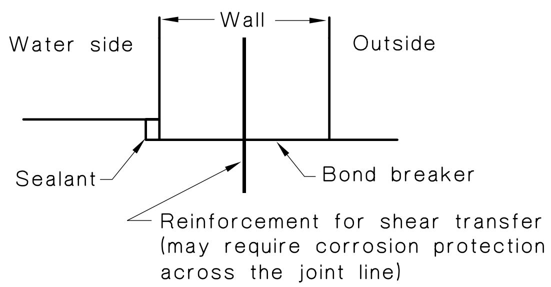

FIGURE C6.11 HINGED JOINT—BETWEEN BASE SLAB AND WALL

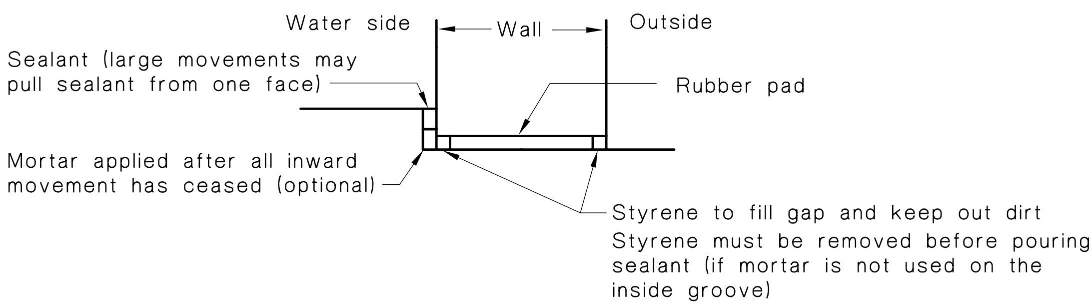

FIGURE C6.12 SLIDING JOINT—BETWEEN BASE SLAB AND WALL—
PRESTRESSED CONCRETE TANK WALL ON A RUBBER PAD

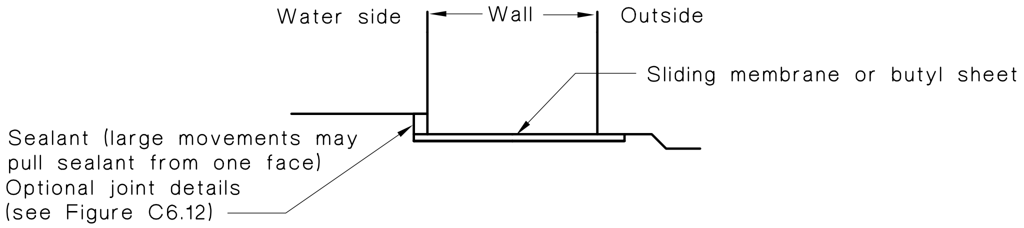

FIGURE C6.13 SLIDING JOINT—BETWEEN BASE SLAB AND WALL—PRESTRESSED CONCRETE TANK ON A MEMBRANE

C6.2 WATERSTOPS

Waterstops and other associated items should be shown to be durable in the environment or usage to which they will be exposed for at least the design life of the structure. Alternatively, they should be designed so as to be maintained or replaced at suitable intervals.

As the actual life of many concrete structures for retaining liquids will be well in excess of the design life of building structures (Ref. 1), consideration should be given to specifying a higher standard of durability for items that are inaccessible for replacement.

Materials used for waterstops should not be susceptible to biological attack.

Waterstops are usually proprietary items with determined performance characteristics. When specified, waterstops should be appropriate to the required design performance.

The different applications of waterstops are illustrated in Figures C6.1 and C6.3 to C6.10. It is essential that the concrete placed around the waterstop is well compacted and the waterstop is fixed and maintained firmly in position until the concrete placing is completed and the concrete has set. However, horizontal and near horizontal waterstops should not be wired firmly in position until the air beneath has been released and the waterstop is supported by compacted concrete.

Waterstops may be divided into four categories. The first category, known as the central bulb type, is used to seal across expansion, contraction and partial contraction joints. The central bulb is positioned across the joint, and the waterstop is set parallel to the liquid surface of the concrete wall. There is a solid bulb or wing at each end of this type of waterstop, which is made of rubber or flexible plastic such as PVC. The distance of the waterstop from the nearest exposed concrete face should not be less than either half the width of the waterstop or 1.5 times the maximum size of aggregate.

The second category is similar to the first category but has no central bulb. It is set in a similar manner to the central bulb type, but should be used only in partial contraction and construction joints.

The third category, consisting of surface types of waterstops, should only be used on the underside of concrete slabs where there is sufficient support from the foundation soil or blinding concrete to prevent the waterstop from being forced away from the underside of the slab by hydraulic pressure in the joint. These waterstops are set into the surface of the concrete each side of a contraction or partial contraction joint that is formed. They can also be used with a central crack inducing tongue for induced contraction joints. To achieve good compaction of the concrete against the waterstop, it should be fixed to the blinding concrete. The use of a surface waterstop is sometimes specified at construction joints. This type of waterstop is usually formed from rubber or flexible plastics such as PVC.

The fourth category of waterstop is a rigid type and is specified where, as in construction joints, no movement is expected at the joint but a positive waterstop is required because of the pressure of the contained liquid. Such waterstops are usually formed from copper, zinc or galvanized steel strip but they are not used as often as they were in the past years because of the possibility of embrittlement due to strain hardening. Waterstops made from Grade 316 stainless steel as specified in Ref. 2 with a large crimp or fold along the middle to permit some movement have recently been used. In some critical applications these waterstops have been used in parallel with a PVC waterstop.

The design of the structure should generally provide for the continuity of the waterstop system across all joints and particularly junctions between floor and wall systems. The correct procedure for making the running joints on site using heat-fused butt welds for PVC, vulcanized or pocketed sleeve joints for rubber and brazed or welded lap joints for copper or steel, needs to be adopted. All waterstop corners should be prefabricated in the factory.

With respect to non-metallic waterstops, both PVC and rubber exhibit satisfactory performance and durability; however, care should be taken when specifying PVC, to guard against the use of recycled or inferior grades of material during the manufacture of the waterstop.

Rubber-based materials that provide a sealing mechanism by their ability to absorb water and expand should be used with care. Some products can generate large expansion forces that can cause local shearing failure adjacent to rebates if the sealant is placed too close to the concrete surface. Although recognizing the potential benefits of these materials, insufficient data are available to offer any guidance for their use.

C6.3 JOINT FILLERS

Joint fillers are usually boards of compressed cork or impregnated fibre installed across nearly the full depth of the joint, but leaving space near the surface for the sealant and a foam backing rod if required.

Joint fillers give support to the sealant to enable it to resist pressure from the surface. They will, however, transmit some stress across the joint as it closes. Where stress transmission has to be avoided, an expanding foam joint filler can be used, but these do not usually provide the same level of resistance to external pressure from service loads as does cork or fibreboard.

C6.4 SEALANTS

Sealants and associated items should be shown to be durable in the environment or usage to which they will be exposed and to have no detrimental effect on the retained liquid for at least the design life of the structure. Alternatively, they should be designed to be maintained or replaced at suitable intervals. The sealing performance is obtained by permanent adhesion of the sealing compound to the concrete each side of the joint only, and most sealants should be applied in conditions of complete dryness and cleanness. There are joint sealing compounds that are produced for application to surfaces that are not dry. The recommendations of the manufacturer should be followed to ensure that the sealing compounds are applied correctly to adequately prepared surfaces. It is necessary that the corners of the concrete each side of the joint are accurately cast as detailed with impermeable concrete to avoid water bypassing the sealant through the concrete.

Recent experience has shown that polysulfide 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 sterilization hastens the attack. Polysulfide sealants have also shown degradation under anaerobic conditions in sewage treatment works.

Joint sealants are generally either bitumen based or elastomeric, and are selected to suit design requirements that may include—

(a) contact with chlorine;

(b) contact with sewage or waste water; and

(c) the ability to perform under water at prescribed depths.

Joint sealants in most liquid retaining structures are subject to much higher hydrostatic loads than are generally acceptable in building applications as shown in the literature of manufacturers.

The selection of sealant material will be based on the movement and hydraulic performance required by the designer. Guidance is given by reference to sealant manufacturers and their literature. Designers should be aware of the design differences for a sealant in Europe and, in some cases, more severe exposure situations in Australia.

The designer should determine—

(i) the size of the groove;

(ii) the shape of the groove;

(iii) the primer type and number of layers;

(iv) the sealant material;

(v) any action on the primer and sealant by the contained liquid; and

(vi) the life requirement and expectancy of the materials.

In floor joints, the sealing compounds are usually applied to a groove along the line of the joint. The shape factor of a groove in building applications is usually 2 (width is double the depth) to keep stresses in the deformed shape to a minimum. In deep liquid-retaining structures, there is high friction from the pressure head, which does not allow the sealant to develop its most efficient deformed shape.

In floor joints of the expansion type, the sealant must be supported and this is usually done with a joint filler. In many cases sealing can be delayed until just before the structure is put into service, so that the joint movement is stabilized and future movements will only be small. The groove should not be too narrow or too deep, which could hinder complete filling, and primer should be used before the sealant is applied. Many sealants perform better if two layers of primer are used.

A bitumen-based sealant is usually applied to a groove with a practical minimum depth of 15 mm. Sloping groove sides are more effective but are not practical in most cases. The minimum width is 20 mm if the joint is not active (no movement); however if there is a movement, the joint will open and the crack will fill with sealant and, hence, a reasonable reservoir of sealant is required, resulting in a wider groove. Narrow and deep grooves can cause overstressing of the sealant and are not recommended for bitumen sealants. Shape factors of 1.0 to 2.0 are usual for bitumen sealants with perhaps higher values if large joint movements are expected.

The action of an elastomeric sealant is elastic, rather than plastic for bitumen; however, the groove should be a minimum of 15 mm depth with a width of usually 15 mm to 20 mm. The shape factor should be in the range of 1.0 to 2.0. In building applications, a bond breaker is required on the bottom of the groove to allow a concave upper and lower surface to develop when the joint is subject to a tension force, but in deep liquid-retaining structures the hydrostatic pressure forces the sealant into the bottom of the groove and it is doubtful whether a bond breaking tape is effective. However, the tape may be required for the situation when the structure is empty and subject to movements without the hydrostatic pressure.

Vertical joints in walls should be primed where necessary and then sealed on the liquid face with a sealant that is usually pressured by gun or knife into the preformed chase. The sealants should have non-slumping properties and great extensibility.

The long-term performance of a joint sealing compound depends on its formulation, the workmanship with which it is prepared and applied, the quality and finish of the concrete to which it is applied and the circumstances of the structure. It would be unwise to depend on joint sealants for liquid tightness in the long term, as that security should be provided by the waterstop. The sealing compound should remain stable at the face of the joint and preclude the ingress of any hard objects that could impair joint movements.

C6.5 CONTAMINATION OF WATER

The effect of waterstops, joint fillers and sealants on potable water may be determined in accordance with Ref. 3. An assessment of the effect of the above on potable water, for a particular structure, should take account of the ratio of their exposed surface areas to the effected volume of water, i.e. product of storage volume and rate of turnover.

REFERENCES

1 STANDARDS AUSTRALIA. AS 3600, *Concrete structures*. Sydney:1994.

2 STANDARDS AUSTRALIA. AS 2837, *Wrought alloy steels—Stainless steel bars and semi-finished products*. Sydney:1986.

3 BRITISH STANDARDS INSTITUTE. BS 6920, *Suitability of non-metallic products for use in contact with water intended for human consumption with regard to their effect on the quality of the water*. Milton Keynes.

SECTION C7 TESTING

C7.1 GENERAL

Programs of regular post-commission inspection should be provided by the owner of any liquid retaining structure that has potential on failure to cause damage to property or endanger lives. The inspection should be carried out by an inspecting authority or under the direction of engineers suitably qualified and experienced in this field, who should investigate and record the safety and serviceability status of the structure as well as to determine repair works or issue an order to restrict or terminate the use of the structure.

APPENDIX A

CYLINDRICAL TANK THERMAL STRESSES

A1 GENERAL

This Appendix sets out a method for a common range of cylindrical tanks for the calculation of vertical and hoop thermal stresses at different heights in an uncracked wall.

A2 SHAPE FACTOR

The shape factor *(SF)* for a cylindrical tank must be calculated from the following equation:

$$SF = H^2 / at_w \qquad \text{. . . A1(1)}$$

where

SF = shape factor of the cylindrical tank

H = height of the tank wall to the surface level of the liquid, in metres

a = radius of tank, in metres

t_w = thickness of the tank wall, in metres

A3 THERMAL STRESS COEFFICIENT

The thermal stress coefficient (C) for each of the three thermal conditions specified in Paragraph A4 are given—

(a) in Table A1 for pinned base;

(b) in Table A2 for fixed base; and

(c) in Table A3 for sliding base.

The sign convention used in the above Tables is as follows:

(i) For tensile stress, positive.

(ii) For vertical stress—

(A) as shown for the inside face; and

(B) the reverse of that shown for the outside face.

Where the wall is designed on the basis of a cracked analysis, the forces and moments implied by the stresses may be reduced to reflect the reduced stiffness resulting from cracks (see Clause C2.2.1).

A4 TEMPERATURE CHANGE

A4.1 Average

The average temperature change is equivalent to a uniform temperature change (θ_A). Tables A1, A2 and A3 assume the temperature change is an increase so that for a decrease the sign of the thermal stress coefficient is reversed.

A4.2 Differential

For a differential temperature change of ($\pm\Delta\theta_D$) the total temperature gradient through the wall is $2\Delta\theta_D$ (see Figure A1). Tables A1, A2, and A3 assume the outside temperature is hotter than the inside temperature so that for the outside temperature colder than the inside temperature the sign of the thermal stress coefficients is reversed.

A4.3 Total

The total temperature change ($\Delta\theta_T$) is equivalent to the temperature variation on only the outside face of the wall (see Figure A2). Tables A1, A2 and A3 assume the outside temperature is hotter than the inside temperature, so that for an outside temperature colder than the inside temperature the sign of the thermal stress coefficient is reversed.

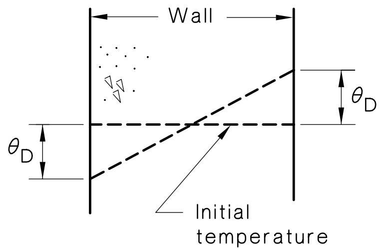

FIGURE A1 DIFFERENTIAL TEMPERATURE CHANGE

A5 THERMAL WALL STRESS

Thermal wall stress is calculated from the following equation:

$$\sigma = CE_c\alpha\Delta\theta \qquad \text{. . . A1(2)}$$

where

- σ = thermal wall stress, in megapascals
- C = thermal stress coefficient dependent on shape factor (*SF*) and base condition (see Table A1, A2 or A3 as appropriate)
- E_c = mean value of the modulus of elasticity of concrete at 28 days, in megapascals
- α = coefficient of thermal expansion of concrete, per degree Celsius (see Table A4)
- $\Delta\theta$ = change in temperature at point in tank wall $\Delta\theta_A$, $\Delta\theta_D$ or $\Delta\theta_T$ as appropriate, in degrees Celsius (see Paragraph A4)

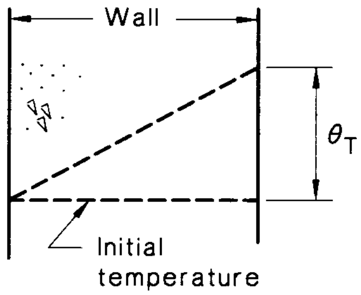

FIGURE A2 TOTAL TEMPERATURE CHANGE

TABLE A1

THERMAL STRESS COEFFICIENT (*C*)—PINNED-BASE CONDITION

(a) Vertical thermal stress—Average temperature change

SF	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

SF	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	0.655	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

SF	Top	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	Btm
2.000	0.025	0.156	0.107	0.141	–0.038	–0.267	–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

SF	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

SF	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	0.949	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	1.227	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

SF	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

SF	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	1.179	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

SF	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	0.688	0.672	0.671	0.673	0.662	0.586	0.380	–0.001
12.000	0.154	0.490	0.633	0.664	0.658	0.652	0.662	0.665	0.610	0.409	–0.001
14.000	0.155	0.508	0.644	0.662	0.644	0.639	0.655	0.665	0.625	0.433	0.000
16.000	0.155	0.525	0.653	0.658	0.635	0.629	0.650	0.664	0.634	0.402	0.000

(i) Hoop thermal stress, outside—Total effects

SF	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	–0.752	–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	–1.005
8.000	–0.856	–0.602	–0.515	–0.511	–0.530	–0.534	–0.508	–0.507	–0.530	–0.666	–1.003
10.000	–0.850	–0.584	–0.512	–0.527	–0.556	–0.566	–0.546	–0.516	–0.516	–0.640	–1.001
12.000	–0.846	–0.568	–0.511	–0.540	–0.576	–0.586	–0.566	–0.525	–0.510	–0.617	–1.000
14.000	–0.846	–0.557	–0.511	–0.550	–0.589	–0.600	–0.583	–0.533	–0.508	–0.601	–1.000
16.000	–0.845	–0.547	–0.513	–0.557	–0.598	–0.610	–0.598	–0.544	–0.504	–0.639	–1.000

TABLE A2

THERMAL STRESS COEFFICIENT (C)—FIXED-BASE CONDITION

(a) Vertical thermal stress—Average temperature change

SF	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

SF	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	0.918	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

SF	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	−0.048	−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

@Seismicisolation

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

SF	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	−0.041	−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	−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

SF	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

SF	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	−1.105	−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	−1.288	−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	−1.299	−1.226	−1.222	−1.220	−1.220

(g) Vertical thermal stress—Total effects

SF	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	1.719	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	0.797	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	0.613	0.539	0.463	0.434	0.666	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

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

(i) Hoop thermal stress, outside—Total effects

SF	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

TABLE A3

THERMAL COEFFICIENT (C) — SLIDING-BASE CONDITION

(a) Vertical thermal stress—Average temperature change

SF	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

SF	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	0.432
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	1.200	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

SF	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

SF	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

SF	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	1.422	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

SF	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

SF	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	0.320	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

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

(i) Hoop thermal stress, outside—Total effects

SF	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	−0.061	−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

TABLE A4

TYPICAL COEFFICIENTS OF THERMAL EXPANSION FOR WATER-CURED CONCRETE FOR DIFFERENT AGGREGATE TYPES

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

NOTE: In the absence of information on the aggregate type assume $\alpha = 11 \times 10^{6}$.

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