

Australian Standard™

**Concrete structures for retaining
liquids—Commentary**

(Supplement to AS 3735—2001)



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- Institution of Engineers Australia
- University of Queensland
- Water Services Association of Australia

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- Australian Chamber of Commerce and Industry
- Australian Post Tensioning Association
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- Brisbane City Council
- Department of Public Works and Services, N.S.W.
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Australian Standard™

**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

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**Concrete structures for retaining liquids—Commentary
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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/m³.

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.
- 2 RUSSELL, H.G. *High-strength concrete in North America* International symposium on utilization of high-strength concrete. Norway. Stavanger, 1987.

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:

$$M_{\Delta\theta} = RF \frac{(\sigma_i - \sigma_o)}{2} \frac{t_w^2}{6} \quad \dots \text{C2.2.1(1)}$$

$$F_{\Delta\theta} = RF \frac{(\sigma_i + \sigma_o)}{2} t_w \quad \dots \text{C2.2.1(2)}$$

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.

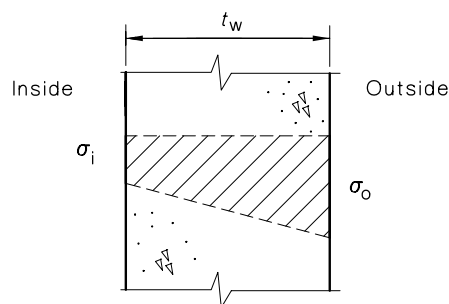
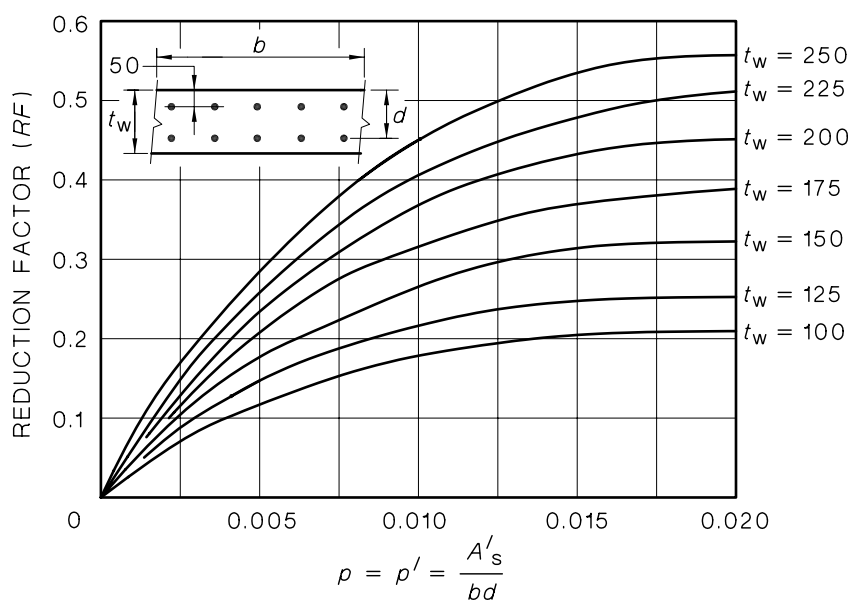


FIGURE C2.1 THERMAL STRESSES



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
 ρ and ρ' = 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:

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

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. 553-560.
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- 12 HEGER, F.J. Concrete domes for water and waste water tanks. *ACI Structural Journal*, July-August 1990.
- 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:

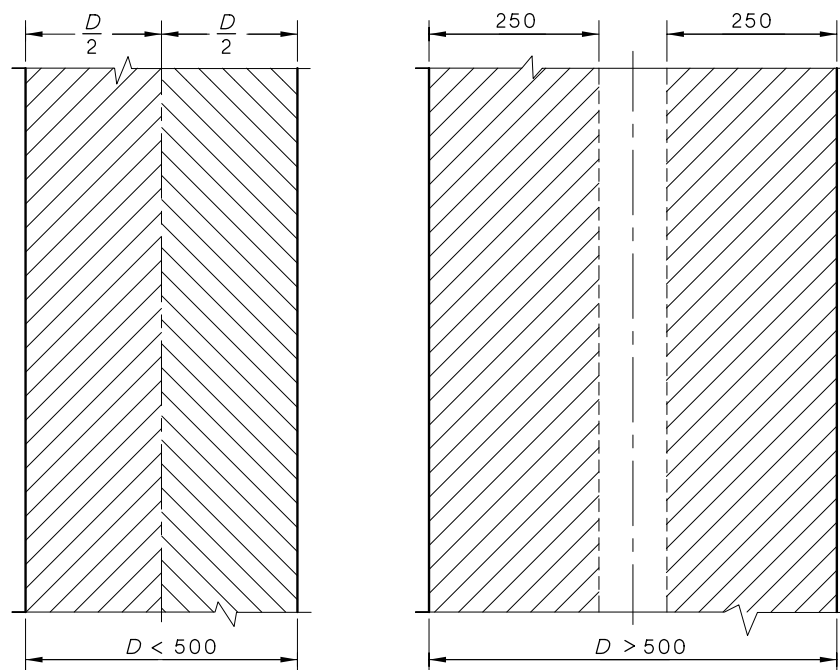
- (a) Aesthetics of the structure.
- (b) Leakage.
- (c) Reinforcement corrosion.

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

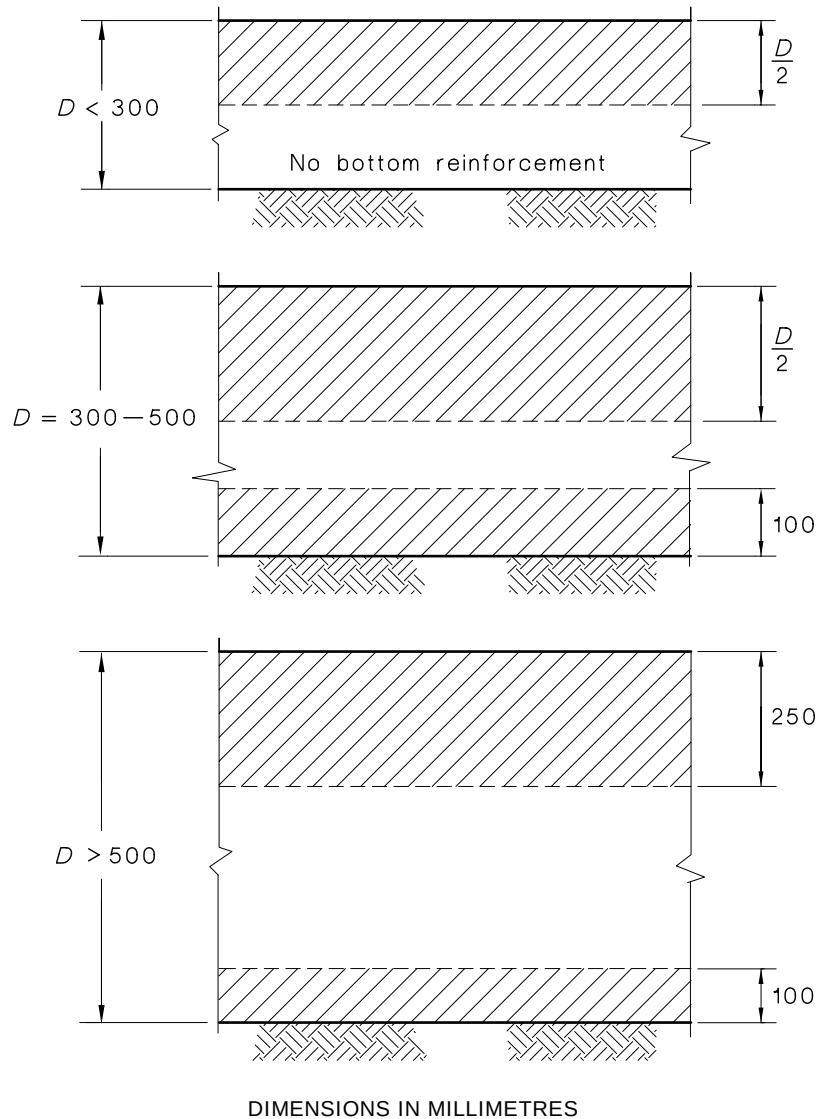


FIGURE C3.2 SURFACE ZONES—GROUND SLABS

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:

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

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

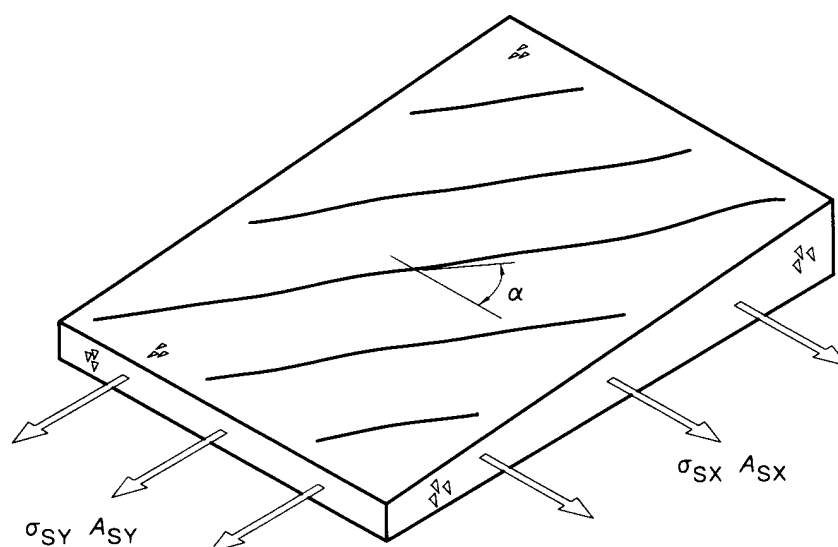


FIGURE C3.3 INCLINED CRACKING