IS 3370-2 (2009): Code of Practice Concrete structures for the storage of liquids, Part 2: Reinforced concrete structures [CED 2: Cement and Concrete]

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भारतीय मानक

द्रवों के संग्रहण के लिये कंक्रीट संरचनाएँ — रीति संहिता

भाग 2 प्रबलित कंक्रीट संरचनाएँ

(पहला पुनरीक्षण)

Indian Standard

CONCRETE STRUCTURES FOR STORAGE OF LIQUIDS — CODE OF PRACTICE

PART 2 REINFORCED CONCRETE STRUCTURES

(First Revision)

ICS 23.020.01; 91.080.40

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FOREWORD

This Indian Standard (Part 2) (First Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Cement and Concrete Sectional Committee had been approved by the Civil Engineering Division Council.

This standard was first published in 1965. The present revision has been taken up with a view to keeping abreast with the rapid development in the field of construction technology and concrete design and also to bring further modifications in the light of experience gained while applying the earlier version of this standard and the amendment issued.

The design and construction methods in reinforced concrete and prestressed concrete structures for the storage of liquids are influenced by the prevailing construction practices, the physical properties of the materials and the climatic condition. To lay down uniform requirements of structures for the storage of liquids giving due consideration to the above mentioned factors, this standard has been published in four parts, the other parts in the series are:

(Part 1) : 2009 General requirements

(Part 3) : 1967 Prestressed concrete structures

(Part 4) : 1967 Design tables

While the common methods of design and construction have been covered in this standard, for design of structures of special forms or in unusual circumstances, special literature may be referred to or in such cases special systems of design and construction may be permitted on production of satisfactory evidence regarding their adequacy and safety by analysis or test or by both.

In this standard it has been assumed that the design of liquid retaining structures, whether of plain, reinforced or prestressed concrete is entrusted to a qualified engineer and that the execution of the work is carried out under the direction of a qualified and experienced supervisor.

All requirements of IS 456 : 2000 'Code of practice for plain and reinforced concrete (*fourth revision*)' and IS 1343 : 1980 'Code of practice for prestressed concrete (*first revision*)', in so far as they apply, shall be deemed to form part of this standard except where otherwise laid down in this standard. For a good design and construction of structure, use of dense concrete, adequate concrete cover, good detailing practices, control of cracking, good quality assurance measures in line with IS 456 and good construction practices particularly in relation to construction joints should be ensured.

This revision incorporates a number of important modifications and changes, the most important of them being:

a) Scope has been clarified further by mentioning exclusion of dams, pipes, pipelines, lined structures and damp-proofing of basements;
b) A new sub-clause on loads has been added under the clause on design;
c) Regarding method of design, it has been specified that one of the two alternative methods of design, that is, limit state design and working stress design may be used; and
d) Provision for crack width calculations due to temperature and moisture and crack width in mature concrete have been incorporated as Annex A and Annex B, respectively.

The composition of the Committee responsible for formulation of this standard is given in Annex C.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the results of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

CONCRETE STRUCTURES FOR STORAGE OF LIQUIDS — CODE OF PRACTICE

PART 2 REINFORCED CONCRETE STRUCTURES

(First Revision)

1 SCOPE

1.1 This standard (Part 2) lays down the requirements applicable specifically to reinforced concrete structures for the storage of liquids, mainly water. These requirements are in addition to the general requirements laid down in IS 3370 (Part 1).

1.2 This standard does not cover the requirements for reinforced and prestressed concrete structures for storage of hot liquids and liquids of low viscosity and high penetrating power like petrol, diesel oil, etc. This standard also does not cover dams, pipes, pipelines, lined structures and damp-proofing of basements. Special problems of shrinkage arising in the storage of non-aqueous liquid and the measures necessary where chemical attack is possible are also not dealt with. The recommendations, however, may generally be applicable to the storage at normal temperatures of aqueous liquids and solutions which have no detrimental action on concrete and steel or where sufficient precautions are taken to ensure protection of concrete and steel from damage due to action of such liquids as in the case of sewage.

2 REFERENCES

The following standards contain provisions, which through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
1786 : 2008	Specification for high strength bars and wires for concrete reinforcement (*fourth revision*)
3370	Concrete structures for the storage of liquids — Code of practice:
(Part 1) : 2009	General requirements (*first revision*)
(Part 4) : 1967	Design tables

3 GENERAL REQUIREMENTS

Design and construction of reinforced concrete liquid retaining structures shall comply with the requirements of IS 3370 (Part 1) and IS 456 unless otherwise laid down in this standard.

4 DESIGN

4.1 General

Provisions shall be made for conditions of stresses that may occur in accordance with principles of mechanics, recognized methods of design and sound engineering practice. In particular, adequate consideration shall be given to the effects of monolithic construction in the assessment of axial force, bending moment and shear.

4.2 Loads

All structures required to retain liquids should be designed for both the full and empty conditions, and the assumptions regarding the arrangements of loading should be such as to cause the most critical effects. For load combinations, water load shall be treated as 'dead load'.

Liquid loads should allow for the actual density of the contained liquid and possible transient conditions, for example, suspended or deposited silt or grit where appropriate. For ultimate limit state conditions and working stress design, liquid levels should be taken to the maximum level the liquid can rise assuming that the liquid outlets are blocked. For serviceability, limit state conditions, the liquid level should be taken to the working top liquid level or the overflow level as appropriate to working conditions. Allowance should be made for the effects of any adverse soil pressures on the walls, according to the compaction and/or surcharge of the soil and the condition of the structure during construction and in service. No relief should be given for beneficial soil pressure effects on the walls of containment structures in the full condition. Loading effects due to temperature occurs when thermal expansion of a roof forces the walls of an empty structure into the surrounding backfill causing passive soil pressure. This effect can be reduced by providing a sliding joint between the top of the wall and under side of the roof which may be either a temporary free sliding joint that is not cast into a fixed or pinned connection, or a permanently sliding joint of assessed limiting friction. Movement of a roof may occur also where there is substantial variation in the temperature of the contained liquid. Where a roof is rigidly connected to a wall this may lead to additional loading in the wall that should be considered in the design. Earth covering on reservoir roof may be taken as dead load, but due account should be taken of construction loads from plant and heaped earth which may exceed the intended design load.

The junctions between various members (between wall and floor) intended to be constructed as rigid should be designed accordingly and effect of continuity should be accounted in design and detailing of each member.

4.3 Methods of Design

One of the two alternative methods of design given in **4.4** and **4.5** for design of water retaining structures shall be followed:

Additional provisions for design of floors, walls and roofs are given in **5**, **6** and **7** respectively. Structural elements that are not exposed to the liquids or to moist conditions shall be designed in accordance with IS 456.

4.4 Limit State Design

4.4.1 *Limit State Requirements*

All relevant limit states shall be considered in the design to ensure an adequate degree of safety and serviceability.

4.4.1.1 *Limit state of collapse*

The recommendations given in IS 456 shall be followed.

4.4.1.2 *Limit states of serviceability*

a) *Deflection* — The limits of deflection shall be as per IS 456.
b) *Cracking* — The maximum calculated surface width of cracks for direct tension and flexure or restrained temperature and moisture effects shall not exceed 0.2 mm with specified cover.

4.4.1.3 *Partial safety factors*

The recommendations given in IS 456 for partial safety factors for serviceability shall be followed.

4.4.2 *Basis of Design*

Design and detailing of reinforced concrete shall be as specified in Section 5 of IS 456 except that **37.1.1** of IS 456 shall not apply.

4.4.3 *Crack Widths*

Crack widths due to the temperature and moisture effects shall be calculated as given in Annex A and that in mature concrete shall be calculated as given in Annex B.

4.4.3.1 Crack widths for reinforced concrete members in direct tension and flexural tension may be deemed to be satisfactory if steel stress under service conditions does not exceed 115 N/mm^2 for plain bars and 130 N/mm^2 for high strength deformed bars.

4.5 Working Stress Design

4.5.1 *Basis of Design*

The design of members shall be based on adequate resistance to cracking and adequate strength. Calculation of stresses shall be based on the following assumptions:

a) At any cross-section plane section remains plane after bending.
b) Both steel and concrete are perfectly elastic and the modular ratio has the value given in IS 456.
c) In calculation of stresses, for both flexural and direct tension (or combination of both) relating to resistance to cracking, the whole section of concrete including the cover together with the reinforcement can be taken into account provided the tensile stress in concrete is limited to Table 1.
d) In strength calculations the concrete has no tensile strength.

4.5.2 *Permissible Stresses on Concrete*

4.5.2.1 *Resistance to cracking*

For calculations relating to the resistance to cracking, the permissible concrete stresses shall conform to the values specified in Table 1. Although cracks may develop in practice, compliance with assumption given in **4.5.1**(c) ensures that these cracks are not excessive.

Table 1 Permissible Concrete Stresses in Calculations Relating to Resistance to Cracking

[*Clauses* 4.5.1(c), 4.5.2.1 *and* 6.3(b)]

Sl No.	Grade of Concrete	Permissible Concrete Stresses, N/mm^2	
		Direct Tension	Tension Due to Bending
(1)	(2)	(3)	(4)
i)	M25	1.3	1.8
ii)	M30	1.5	2.0
iii)	M35	1.6	2.2
iv)	M40	1.8	2.4
v)	M45	2.0	2.6
vi)	M50	2.1	2.8

NOTE — The maximum values of shear stress in concrete shall be as given in IS 456.

4.5.2.2 *Strength calculation*

In strength calculations, the permissible concrete stresses shall be in accordance with Table 2 and Table 3.

Table 2 Permissible Stresses in Concrete

All values are in N/mm^2.

Sl No.	Grade of Concrete	Permissible Stress in Compression		Permissible Stress in Bond (Average) for Plain Bars in Tension
		Bending	Direct	
		σ_{cbc}	σ_{cc}	τ_{bd}
(1)	(2)	(3)	(4)	(5)
i)	M25	8.5	6.0	0.9
ii)	M30	10.0	8.0	1.0
iii)	M35	11.5	9.0	1.1
iv)	M40	13.0	10.0	1.2
v)	M45	14.5	11.0	1.3
vi)	M50	16.0	12.0	1.4

NOTES

1 The values of permissible shear stress in concrete are given in Table 3.

2 The bond stress given in col 5 shall be increased by 25 percent for bars in compression.

3 In case of deformed bars conforming to IS 1786, the bond stresses given above may be increased by 60 percent.

Table 3 Permissible Shear Stress in Concrete

(*Clause* 4.5.2.2, and *Table* 2)

Sl No.	$100 \frac{A_s}{bd}$	Permissible Shear Stress in Concrete τ_c, N/mm^2 Grade of Concrete			
		M25	M30	M35	M40 and Above
(1)	(2)	(3)	(4)	(5)	(6)
i)	$\leq$ 0.15	0.19	0.20	0.20	0.20
ii)	0.25	0.23	0.23	0.23	0.23
iii)	0.50	0.31	0.31	0.31	0.32
iv)	0.75	0.36	0.37	0.37	0.38
v)	1.00	0.40	0.41	0.42	0.42
vi)	1.25	0.44	0.45	0.45	0.46
vii)	1.50	0.46	0.48	0.49	0.49
viii)	1.75	0.49	0.50	0.52	0.52
ix)	2.00	0.51	0.53	0.54	0.55
x)	2.25	0.53	0.55	0.56	0.57
xi)	2.50	0.55	0.57	0.58	0.60
xii)	2.75	0.56	0.58	0.60	0.62
xiii)	3.00 and above	0.57	0.60	0.62	0.63

NOTE — A_s is that area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at supports where the full area of tension reinforcement may be used provided the detailing conforms to 26.2.2 and 26.2.3 of IS 456.

4.5.3 *Permissible Stresses in Steel*

4.5.3.1 *Resistance to cracking*

The tensile stress in the steel will necessarily be limited by the requirement that the permissible tensile stress in the concrete is not exceeded; so the tensile stress in steel shall be equal to the product of modular ratio of steel and concrete, and the corresponding permissible tensile stress in concrete.

4.5.3.2 *Strength calculations*

For strength calculations, the permissible stresses in steel shall conform to the values specified in Table 4.

Table 4 Permissible Stresses in Steel Reinforcement for Strength

Sl No.	Type of Stress in Steel Reinforcement	Permissible Stresses, N/mm^2	
		Plain Round Mild Steel Bars	High Strength Deformed Bars
(1)	(2)	(3)	(4)
i)	Tensile stress in members under direct tension, bending and shear	115	130
ii)	Compressive stress in columns subjected to direct load	125	140

4.5.4 *Stresses Due to Moisture or Temperature Changes*

No separate calculation is required for stresses due to moisture or temperature change in the concrete provided that:

a) The reinforcement provided is not less than that specified in 8,

b) The recommendations of the standard with regard to the provision of movement joints and for a suitable sliding layer beneath the tank given in IS 3370 (Part 1) are complied with,

c) The tank is to be used only for the storage of water or aqueous liquids at or near ambient temperature and the concrete never dries out, and

d) Adequate precautions are taken to avoid cracking of the concrete during the construction period and until the tank is put into use.

4.5.4.1 Shrinkage stresses may, however, be required to be calculated in special cases, when a shrinkage coefficient of 300×10^{-6} may be assumed.

4.5.4.2 Where reservoirs are protected with an internal impermeable lining, consideration should be given to the possibility of concrete eventually, drying out. Unless it is established on the basis of tests or experience that the lining has adequate crack bridging properties, allowance for the increased effect of drying shrinkage should be made in the design.

5 FLOORS

5.1 Provisions of Movement Joints

Movement joints shall be provided in accordance with IS 3370 (Part 1).

5.2 Floors of Tanks Resting on Ground

The floors of tanks resting on ground shall be in accordance with IS 3370 (Part 1).

5.3 Floors of Tanks Resting on Supports

If the tank is supported on walls or other similar supports, the floor slab shall be designed for bending moments due to water load and self weight. The worst conditions of loading may not be those given in **22.4.1** of IS 456, since water level extends over all spans in normal construction except in the case of multi-cell tanks, these will have to be determined by the designer in each particular case.

5.3.1 When the floor is rigidly connected to the walls (as is generally the case) the bending moments at the junction between the walls and floor shall be taken into account in the design of floor together with any direct forces transferred to the floor from the walls or from the floor to the wall due to the suspension of the floor from the wall.

6 WALLS

6.1 Provision of Joints

6.1.1 *Sliding Joints at the Base of the Wall*

Where it is desired to allow the walls to expand or contract separately from the floor, or to prevent moments at the base of the wall owing to fixity to the floor, sliding joints may be employed.

6.1.1.1 Constructions affecting the spacing of vertical movement joints are discussed in IS 3370 (Part 1). While the majority of these joints may be of the partial or complete contraction type, sufficient joints of the expansion type should be provided to satisfy the requirements of IS 3370 (Part 1).

6.2 Pressure on Walls

6.2.1 In liquid retaining structures with fixed or floating covers, the gas pressure developed above liquid surface shall be added to the liquid pressure.

6.2.2 When the wall of liquid retaining structure is built in ground or has earth embanked against it, the effect of earth pressure shall be taken into account as discussed in IS 3370 (Part 1).

6.3 Walls of Tanks Rectangular or Polygonal in Plan

While designing the walls of rectangular or polygonal concrete tanks, the following points should be taken care of:

a) In plane walls, the liquid pressure is resisted by both vertical and horizontal bending moments. An estimate of the bending moments in the vertical and horizontal planes should be made. The horizontal tension caused by the direct pull due to water pressure on end walls should be added to that resulting from horizontal bending moment.

b) On liquid retaining faces, the tensile stresses due to the combination of direct horizontal tension and bending action shall satisfy the following condition:

$$\frac{\sigma_{ct'}}{\sigma_{ct}} + \frac{\sigma_{cbt'}}{\sigma_{cbt}} \leq 1$$

where

$\sigma_{ct'}$ = calculated direct tensile stress in concrete,

σ_{ct} = permissible direct tensile stress in concrete (*see* Table 1),

$\sigma_{cbt'}$ = calculated tensile stress due to bending in concrete, and

σ_{cbt} = permissible tensile stress due to bending in concrete (*see* Table 1).

c) At the vertical edges where the walls of a reservoir are rigidly joined, horizontal reinforcement and haunch bars should be provided to resist the horizontal bending moments, even if the walls are designed to withstand the whole load as vertical beams or cantilever without lateral supports.

In the case of rectangular or polygonal tanks, the side walls act as two way slabs, whereby the wall is continued or restrained in the horizontal direction, fixed or hinged at the bottom and hinged or free at the top. The walls thus act as thin plates subject to triangular loading and with boundary conditions varying between full restraint and free edge. The analysis of moment and forces may be made on the basis of any recognized method. However, moment coefficients, for boundary conditions of wall panels for some common cases are given in IS 3370 (Part 4) for general guidance.

6.4 Walls of Cylindrical Tanks

While designing walls of cylindrical tanks, the following points should be borne in mind:

a) Walls of cylindrical tanks are either cast monolithically with the base or are set in grooves and keyways (movement joints). In either case deformation of the wall under the influence of liquid pressure is restricted at the base.

b) Unless the extent of fixity at the base is established by analysis with due consideration to the dimensions of the base slab, the type of joint between the wall and slab and the type of soil supporting the base slab, it is advisable to assume wall to be fully fixed at the base.

Coefficient for ring tension and vertical moments for different conditions of the walls for some common cases are given in IS 3370 (Part 4) for general guidance.

7 ROOFS

7.1 Provision of Movement Joints

To avoid the possibility of sympathetic cracking, it is important to ensure that movement joints in the roof correspond with those in walls if roof and walls are monolithic. If, however, provision is made by means of a sliding joint for movement between the roof and the wall, correspondence of joints is not important.

7.2 Water-Tightness

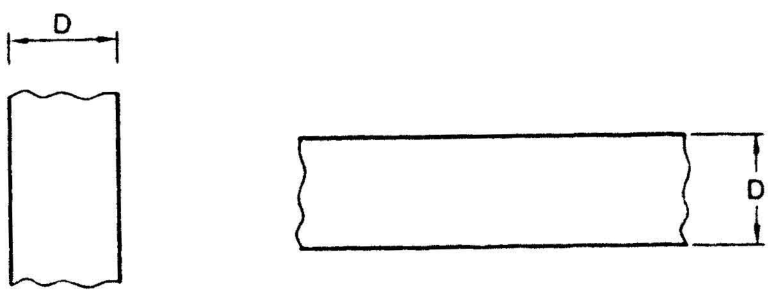
In case of tanks intended for the storage of water for drinking purposes, the roof must be made water-tight. This may be achieved by limiting the stresses as for the rest of the tank or by use of the covering of waterproof membrane or by providing slopes to ensure adequate drainage.

8 DETAILING

8.1 Minimum Reinforcement

8.1.1 The minimum reinforcement in walls, floors and roofs in each of two directions at right angles, within each surface zone shall not be less than 0.35 percent of the surface zone, cross section as shown in Fig. 1 and Fig. 2 for high strength deformed bars and not less than 0.64 percent for mild steel reinforcement bars. The minimum reinforcement can be further reduced to 0.24 percent for deformed bars and 0.40 percent for plain round bars for tanks having any dimension not more than 15 m. In wall slabs less than 200 mm in thickness, the calculated amount of reinforcement may all be placed in one face. For ground slabs less than 300 mm thick (*see* Fig. 2) the calculated reinforcement should be placed in one face as near as possible to the upper surface consistent with the nominal cover. Bar spacing should generally not exceed 300 mm or the thickness of the section, whichever is less.

8.2 Size of Bars, Distance Between Bars, Laps and Bends — Size of bars, distance between bars, laps and bends in bars, and fixing of bars shall be in accordance with IS 456.



NOTE — For $D < 500$ mm, assume each reinforcement face controls $D/2$ depth of concrete.

For $D > 500$ mm assume each reinforcement face controls 250 mm depth of concrete, ignoring any central core beyond this surface depth.

FIG. 1 SURFACE ZONES: WALLS AND SUSPENDED SLABS

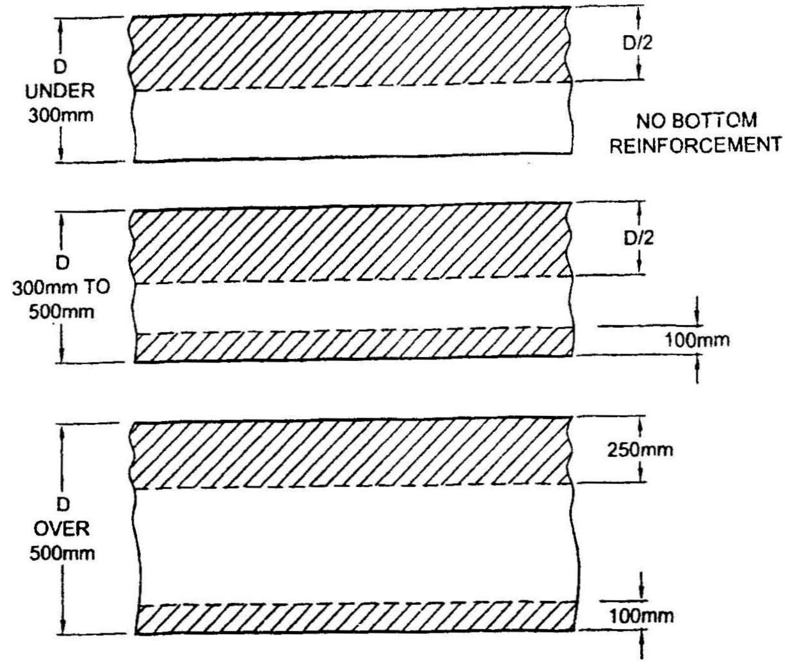


FIG. 2 SURFACE ZONES: GROUNDED SLABS

ANNEX A

(*Foreword*, and *Clause* 4.4.3)

CRACK WIDTH DUE TO TEMPERATURE AND MOISTURE

A-1 CALCULATION OF MINIMUM REINFORCEMENT CRACK SPACING AND CRACK WIDTHS IN RELATION TO TEMPERATURE AND MOISTURE EFFECTS IN THIN SECTION

A-1.1 The design procedures given in **A-1.2** to **A-1.3** are appropriate to long continuous wall or floor slabs of thin cross section. **A-2** considers thick sections.

A-1.2 Minimum Reinforcement

To be effective in distributing cracking, the amount of reinforcement provided needs to be at least as great as that given by the formula:

$$\rho_{crit} = \frac{f_{ct}}{f_y} \qquad \ldots(1)$$

where

ρ_{crit} = critical steel ratio, that is, the minimum ratio, of steel area to the gross area of the whole concrete section, required to distribute the cracking;

f_{ct} = direct tensile strength of the immature concrete, which is taken as given below:

Grade of concrete	M25	M30	M35	M40	M45	M50
f_{ct}, N/mm^2	1.15	1.3	1.45	1.6	1.7	1.8

f_y = characteristic strength of the reinforcement.

For ground slabs under 200 mm thick the minimum reinforcement may be assessed on the basis of thickness of 100 mm and placed wholly in the top surface with cover not exceeding 50 mm. The top surface zone for ground slab from 200 to 500 mm thick may be assessed on half the thickness of the slab. For ground slabs over 500 mm thick, consider them as 'thick' sections with the bottom surface zone only 100 mm thick.

A-1.3 Cracks can be controlled by choosing the spacing of movement joint and the amount of reinforcement. The three main options are summarized in Table 2 of IS 3370 (Part 1).

A-1.4 Crack Spacing

When sufficient reinforcement is provided to distribute cracking the likely maximum spacing of crack S_{Max} shall be given by the formula:

$$S_{Max} = \frac{f_{ct}}{f_b} \times \frac{\phi}{2\rho} \quad \dots (2)$$

where

$\frac{f_{ct}}{f_b}$ = ratio of the tensile strength of the concrete (f_{ct}) to the average bond strength between concrete and steel,

ϕ = size of each reinforcing bar, and

ρ = steel ratio based on the gross concrete section.

For immature concrete, the value of $\frac{f_{ct}}{f_b}$ may be taken as unity for plain round bars and 2/3 for deformed bars.

The above formula may be expressed for design purposes as:

$$n_b \phi \geq \frac{f_{ct}}{f_b} \times \frac{2bD}{\pi s_{Max}} \quad \dots (3)$$

where

n_b = number of bars in width of section,

b = width of section;

D = overall depth of member, and

S_{Max} = obtained from W_{Max}.

The width of a fully developed crack due to drying shrinkage and 'heat of hydration' contraction in lightly-reinforced restrained walls and slabs may be obtained from:

$$w_{Max} = s_{Max}\, \varepsilon \quad \dots (4)$$

where

ε = $[\varepsilon_{cs} + \varepsilon_{te} - (100 \times 10^{-6})]$

w_{Max} = estimated maximum crack width,

s_{Max} = estimated likely maximum crack spacing,

ε_{cs} = estimated shrinkage strain, and

ε_{te} = estimated total thermal contraction after peak temperature due to heat of hydration.

For immature concrete the effective coefficient of thermal contraction may be taken as one half of the value for mature concrete (due to the high creep strain in immature concrete).

For walls exposed to normal climatic conditions the shrinkage strain less the associated creep strain is generally less than the ultimate concrete tensile strain of about 100×10^{-6} unless high shrinkage aggregates are used. Hence the value of W_{Max} for cooling to ambient from the peak hydration temperature may be assumed to be:

$$w_{Max} = s_{Max} \times \frac{\alpha}{2} \times T_1 \quad \dots (5)$$

where

$\propto$ = coefficient of thermal expansion of mature concrete, and

T_1 = fall in temperature between the hydration peak and ambient.

The value of T_1 depends on the temperature of concreting, cement content, thickness of the member and material for shutters. As guideline, it is recommended to use T_1 = 30°C for concreting in summer and 20°C for concreting during winter, when steel shutters are used. For other conditions, the value of T_1 may be appropriately increased.

In addition to the temperature fall T_1, there can be a further fall in temperature, T_2 due to seasonal variations. The consequent thermal contractions occur in the mature concrete for which the factors controlling cracking behaviour are substantially modified. The ratio of the tensile strength of concrete to bond strength, $\frac{f_{ct}}{f_b}$, is appreciably lower for mature concrete. In addition, the restraint along the base of the member tends to be much more uniform and less susceptible to stress raisers, since a considerable shear resistance can be developed along the entire length of the construction joint.

Although precise data are not available for these effects a reasonable estimate may be assumed that the combined effect of these factors is to reduce the estimated contraction by half. Hence the value of w_{Max} when taking an additional seasonal temperature fall into account is given by:

$$W_{Max} = S_{Max} \times \frac{\alpha}{2} \times (T_1 + T_2) \quad \dots (6)$$

When movement joints are provided at not more than 15 m centres, the subsequent temperature fall, T_2 need not be considered.

A-2 THICK SECTIONS

For 'thick' sections, major causes of cracking are the differences which develop between the surface zones and the core of the section. The thickness of concrete which can be considered to be within the 'surface zone' is somewhat arbitrary. However, site observations have indicated that the zone thicknesses for $D > 500$ mm in Fig. 1 and Fig. 2 are appropriate for thick sections, and the procedure for calculating thermal crack control reinforcement in thick sections is same as that for thin sections.

The maximum temperature rise due to heat of hydration to be considered should be the average value for the entire width of section. The temperature rise to be considered for the core should be at least 10°C higher than the value which would be assumed for the entire section.

ANNEX B
(*Foreword*, and *Clause* 4.4.3)

CRACK WIDTHS IN MATURE CONCRETE

B-1 ASSESSMENT OF CRACK WIDTHS IN FLEXURE

Provided that the strain in the tension reinforcement is limited to 0.8 f_y/E_s and the stress in the concrete is limited to 0.45 f_{cu}, the design surface crack width should not exceed the appropriate value given in **4.4.1.2** and may be calculated from equation (7):

$$w = \frac{3a_{cr}\varepsilon_m}{1+\frac{2(a_{cr}-C_{Min})}{D-x}} \qquad \ldots (7)$$

where

w = design surface crack width,

a_{cr} = distance from the point considered to the surface of the nearest longitudinal bar,

ε_m = average strain at the level where the cracking is being considered. To be calculated in accordance with **B-2**,

C_{Min} = minimum cover to the tension steel,

D = overall depth of the members, and

x = depth of neutral axis.

B-2 AVERAGE STRAIN IN FLEXURE

The average strain at the level where cracking is being considered, is assessed by calculating the apparent strain using characteristic loads and normal elastic theory. Where flexure is predominant but some tension exists at the section, the depth of the neutral axis should be adjusted. The calculated apparent strain, ε_1 is then adjusted to take into account the stiffening effect of the concrete between cracks ε_2. The value of the stiffening effect may be assessed from **B-3**, and

$$\varepsilon_m = \varepsilon_1 - \varepsilon_2$$

where

ε_m = average strain at the level where cracking is being considered,

ε_1 = strain at the level considered, and

ε_2 = strain due to stiffening effect of concrete between cracks.

B-3 STIFFENING EFFECT OF CONCRETE IN FLEXURE

The stiffening effect of the concrete may be assessed by deducting from the apparent strain a value obtained from equations (8) or (9).

For a limiting design surface crack width of 0.2 mm:

$$\varepsilon_2 = \frac{b_t(D-x)(a'-x)}{3E_sA_s(d-x)} \qquad \ldots (8)$$

For a limiting design surface crack width of 0.1 mm:

$$\varepsilon_2 = \frac{1.5b_t(D-x)(a'-x)}{3E_sA_s(d-x)} \qquad \ldots (9)$$

where

ε_1 = strain at the level considered,

ε_2 = strain due to the stiffening effect of concrete between cracks,

b_t = width of section at the centroid of the tension steel,

D = overall depth of the member,

x = depth of the neutral axis,

E_s = modulus of elasticity of reinforcement,

A_s = area of tension reinforcement,

d = effective depth, and

a' = distance from the compression face to the point at which the crack width is being calculated.

B-4 ASSESSMENT OF CRACK WIDTHS IN DIRECT TENSION

Provided that the strain in the reinforcement is limited to 0.8 f_y/E_s, the design crack width should not exceed the appropriate value given in **8** of IS 3370 (Part 1) and may be calculated from equation (10):

$$w = 3\,a_{cr}\,\varepsilon_m \qquad \ldots (10)$$

where ε_m is assessed in accordance with **B-5**.

B-5 AVERAGE STRAIN IN DIRECT TENSION

The average strain is assessed by calculating the apparent strain using characteristic loads and normal elastic theory. The calculated apparent strain is then adjusted to take into account the stiffening effect of the concrete between cracks. The value of the stiffening effect may be assessed from **B-6**.

B-6 STIFFENING EFFECT OF CONCRETE IN DIRECT TENSION

The stiffening effect of the concrete may be assessed by deducting from the apparent strain a value obtained from equation (11) or (12).

For a limiting design surface crack width of 0.2 mm:

$$\varepsilon_2 = \frac{2b_t D}{3E_s A_s} \qquad \ldots (11)$$

For a limiting design surface crack width of 0.1 mm:

$$\varepsilon_2 = \frac{b_t D}{E_s A_s} \qquad \ldots (12)$$

ε_2 = strain due to stiffening effect,

b_t = width of the section at the centroid of the tension steel,

D = overall depth of the member,

E_s = modulus of elasticity of reinforcement, and

A_s = area of tension reinforcement.

The stiffening effect factors should not be interpolated or extrapolated and apply only for the crack widths stated.

ANNEX C

(*Foreword*)

COMMITTEE COMPOSITION

Cement and Concrete Sectional Committee, CED 2

Organization	*Representative(s)*
Delhi Tourism and Transportation Development Corporation Ltd, New Delhi	Shri Jose Kurian (*Chairman*)
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Central Public Works Department, New Delhi	Superintending Engineer (Design) Executive Engineer (Design) (*Alternate*)
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