

COMMITTEE BD-002

**DR AS 3600:2017**

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# Draft for Public Comment Australian Standard

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**FOR COMMENT:**

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**Concrete structures**

**(Revision of AS 3600—2009)**

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## Draft for Public Comment Australian Standard

The committee responsible for the issue of this draft comprised representatives of organizations interested in the subject matter of the proposed Standard. These organizations are listed on the inside back cover.

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**Draft for Public Comment**

STANDARDS AUSTRALIA

Committee BD-002—Concrete Structures

**DRAFT**

**Australian Standard**

Concrete structures

(Revision of AS 3600—2009)

(To be AS 3600:201X)

Comment on the draft is invited from people and organizations concerned with this subject. It would be appreciated if those submitting comment would follow the guidelines given on the inside front cover.

***Important: Please read the instructions on the inside cover of this document for the procedure for submitting public comments***

This document is a draft Australian Standard only and is liable to alteration in the light of comment received. It is not to be regarded as an Australian Standard until finally issued as such by Standards Australia.

## PREFACE

This Standard was prepared by Standards Australia Committee BD-002, Concrete Structures, to supersede AS 3600—2009.

### Objective of the Standard

The principal objective of this Standard is to provide users with nationally acceptable unified rules for the design and detailing of concrete structures and members, with or without steel reinforcement or prestressing tendons, based on the principles of structural engineering mechanics. The secondary objective is to provide performance criteria against which the finished structure can be assessed for conformance with the relevant design requirements.

Statements expressed in mandatory terms in notes to tables are deemed to be requirements of this Standard.

The terms ‘normative’ and ‘informative’ are used in a Standard to define the application of the appendices or annexes to which they apply. A ‘normative’ appendix or annex is an integral part of a Standard, whereas an ‘informative’ appendix or annex is only for information and guidance.

## CONTENTS

	<i>Page</i>
<b>SECTION 1 SCOPE AND GENERAL</b>	
1.1 SCOPE AND APPLICATION.....	8
1.2 NORMATIVE REFERENCES.....	9
1.3 EXISTING STRUCTURES.....	9
1.4 DOCUMENTATION.....	9
1.5 CONSTRUCTION.....	10
1.6 DEFINITIONS.....	10
1.7 NOTATION.....	17
<b>SECTION 2 DESIGN PROCEDURES, ACTIONS AND LOADS</b>	
2.1 DESIGN PROCEDURES.....	30
2.2 DESIGN FOR STRENGTH.....	31
2.3 DESIGN FOR SERVICEABILITY.....	35
2.4 DESIGN FOR FATIGUE.....	37
2.5 ACTIONS AND COMBINATIONS OF ACTIONS.....	38
<b>SECTION 3 DESIGN PROPERTIES OF MATERIALS</b>	
3.1 PROPERTIES OF CONCRETE.....	41
3.2 PROPERTIES OF REINFORCEMENT.....	47
3.3 PROPERTIES OF TENDONS.....	49
3.4 LOSS OF PRESTRESS IN TENDONS.....	51
3.5 MATERIAL PROPERTIES FOR NON-LINEAR STRUCTURAL ANALYSIS.....	54
<b>SECTION 4 DESIGN FOR DURABILITY</b>	
4.1 GENERAL.....	55
4.2 METHOD OF DESIGN FOR DURABILITY.....	55
4.3 EXPOSURE CLASSIFICATION.....	55
4.4 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATIONS A1, A2, B1, B2, C1 AND C2.....	58
4.5 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATION U.....	59
4.6 ABRASION.....	59
4.7 FREEZING AND THAWING.....	59
4.8 AGGRESSIVE SOILS.....	60
4.9 RESTRICTIONS ON CHEMICAL CONTENT IN CONCRETE.....	62
4.10 REQUIREMENTS FOR COVER TO REINFORCING STEEL AND TENDONS....	62
<b>SECTION 5 DESIGN FOR FIRE RESISTANCE</b>	
5.1 SCOPE.....	65
5.2 DEFINITIONS.....	65
5.3 DESIGN PERFORMANCE CRITERIA.....	67
5.4 FIRE RESISTANCE PERIODS (FRPs) FOR BEAMS.....	68
5.5 FIRE RESISTANCE PERIODS (FRPs) FOR SLABS.....	72
5.6 FIRE RESISTANCE PERIODS (FRPs) FOR COLUMNS.....	74
5.7 FIRE RESISTANCE PERIODS (FRPs) FOR WALLS.....	78
5.8 INCREASE OF FIRE RESISTANCE PERIODS (FRPs) BY USE OF INSULATING MATERIALS.....	81

*Page***SECTION 6 METHODS OF STRUCTURAL ANALYSIS**

6.1	GENERAL.....	83
6.2	LINEAR ELASTIC ANALYSIS .....	86
6.3	ELASTIC ANALYSIS OF FRAMES INCORPORATING SECONDARY BENDING MOMENTS .....	88
6.4	LINEAR ELASTIC STRESS ANALYSIS.....	88
6.5	NON-LINEAR FRAME ANALYSIS .....	89
6.6	NON-LINEAR STRESS ANALYSIS.....	89
6.7	PLASTIC METHODS OF ANALYSIS .....	90
6.8	ANALYSIS USING STRUT-AND-TIE MODELS .....	91
6.9	IDEALIZED FRAME METHOD OF ANALYSIS .....	91
6.10	SIMPLIFIED METHODS OF FLEXURAL ANALYSIS .....	93

**SECTION 7 STRUT-AND-TIE MODELLING**

7.1	GENERAL.....	101
7.2	CONCRETE STRUTS.....	101
7.3	TIES .....	106
7.4	NODES.....	106
7.5	ANALYSIS OF STRUT-AND-TIE MODELS .....	107
7.6	DESIGN BASED ON STRUT-AND-TIE MODELLING.....	107

**SECTION 8 DESIGN OF BEAMS FOR STRENGTH AND SERVICEABILITY**

8.1	STRENGTH OF BEAMS IN BENDING.....	108
8.2	STRENGTH OF BEAMS IN SHEAR .....	114
8.3	GENERAL DETAILS .....	125
8.4	LONGITUDINAL SHEAR IN COMPOSITE AND MONOLITHIC BEAMS .....	128
8.5	DEFLECTION OF BEAMS.....	130
8.6	CRACK CONTROL OF BEAMS.....	133
8.7	VIBRATION OF BEAMS .....	136
8.8	T-BEAMS AND L-BEAMS .....	136
8.9	SLENDERNESS LIMITS FOR BEAMS.....	137

**SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY**

9.1	STRENGTH OF SLABS IN BENDING .....	138
9.2	STRUCTURAL INTEGRITY REINFORCEMENT .....	141
9.3	STRENGTH OF SLABS IN SHEAR.....	142
9.4	DEFLECTION OF SLABS.....	146
9.5	CRACK CONTROL OF SLABS .....	148
9.6	VIBRATION OF SLABS .....	152
9.7	MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS.....	152
9.8	LONGITUDINAL SHEAR IN COMPOSITE SLABS.....	152

**SECTION 10 DESIGN OF COLUMNS FOR STRENGTH AND SERVICEABILITY**

10.1	GENERAL.....	153
10.2	DESIGN PROCEDURES .....	153
10.3	DESIGN OF SHORT COLUMNS .....	154
10.4	DESIGN OF SLENDER COLUMNS .....	155
10.5	SLENDERNESS.....	156
10.6	STRENGTH OF COLUMNS IN COMBINED BENDING AND COMPRESSION .....	160
10.7	REINFORCEMENT REQUIREMENTS FOR COLUMNS.....	163
10.8	TRANSMISSION OF AXIAL FORCE THROUGH FLOOR SYSTEMS.....	171

	<i>Page</i>
SECTION 11 DESIGN OF WALLS	
11.1 GENERAL.....	173
11.2 DESIGN PROCEDURES .....	173
11.3 BRACED WALLS.....	174
11.4 EFFECTIVE HEIGHT.....	174
11.5 SIMPLIFIED DESIGN METHOD FOR WALLS SUBJECT TO VERTICAL COMPRESSION FORCES.....	175
11.6 DESIGN OF WALLS FOR IN-PLANE SHEAR FORCES .....	176
11.7 REINFORCEMENT REQUIREMENTS FOR WALLS .....	177
SECTION 12 DESIGN OF NON-FLEXURAL MEMBERS, END ZONES AND BEARING SURFACES	
12.1 GENERAL.....	179
12.2 STRUT-AND-TIE MODELS FOR THE DESIGN OF NON-FLEXURAL MEMBERS .....	179
12.3 ADDITIONAL REQUIREMENTS FOR CONTINUOUS CONCRETE NIBS AND CORBELS .....	181
12.4 ADDITIONAL REQUIREMENTS FOR STEPPED JOINTS IN BEAMS AND SLABS.....	181
12.5 ANCHORAGE ZONES FOR PRESTRESSING ANCHORAGES .....	181
12.6 BEARING SURFACES.....	183
12.7 CRACK CONTROL .....	183
SECTION 13 STRESS DEVELOPMENT OF REINFORCEMENT AND TENDONS	
13.1 STRESS DEVELOPMENT IN REINFORCEMENT.....	184
13.2 SPLICING OF REINFORCEMENT.....	191
13.3 STRESS DEVELOPMENT IN TENDONS .....	193
13.4 COUPLING OF TENDONS .....	195
SECTION 14 REQUIREMENTS FOR STRUCTURES SUBJECT TO EARTHQUAKE ACTIONS	
14.1 GENERAL.....	196
14.2 DEFINITIONS.....	196
14.3 STRUCTURAL DUCTILITY FACTOR ( $\mu$ ) AND STRUCTURAL PERFORMANCE FACTOR ( $S_p$ ).....	197
14.4 GENERAL EARTHQUAKE DESIGN REQUIREMENTS .....	198
14.5 INTERMEDIATE MOMENT-RESISTING FRAMES (IMRFs) .....	200
14.6 LIMITED DUCTILE STRUCTURAL WALLS .....	203
14.7 MODERATELY DUCTILE STRUCTURAL WALLS.....	204
SECTION 15 DIAPHRAGMS	
15.1 GENERAL.....	206
15.2 DESIGN ACTIONS.....	206
15.3 CAST IN-PLACE TOPPINGS.....	207
15.4 DIAPHRAGM REINFORCEMENT.....	207

	<i>Page</i>
<b>SECTION 16 STEEL FIBRE REINFORCED CONCRETE</b>	
16.1 GENERAL.....	209
16.2 DEFINITIONS.....	209
16.3 PROPERTIES OF SFRC .....	210
16.4 DESIGN OF SFRC MEMBERS CONTAINING REINFORCEMENT OR TENDONS .....	214
16.5 DURABILITY .....	219
16.6 FIRE .....	220
16.7 PRODUCTION OF SFRC .....	220
<b>SECTION 17 MATERIAL AND CONSTRUCTION REQUIREMENTS</b>	
17.1 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR CONCRETE AND GROUT .....	224
17.2 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR REINFORCING STEEL .....	226
17.3 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR PRESTRESSING DUCTS, ANCHORAGES AND TENDONS .....	229
17.4 CONSTRUCTION REQUIREMENTS FOR JOINTS AND EMBEDDED ITEMS .....	231
17.5 TOLERANCES FOR STRUCTURES AND MEMBERS .....	231
17.6 FORMWORK.....	232
17.7 PREFABRICATED CONCRETE STRUCTURES .....	235
<b>SECTION 18 DESIGN FOR FATIGUE</b>	
18.1 GENERAL.....	237
18.2 MAXIMUM COMPRESSIVE STRESS IN CONCRETE.....	237
18.3 PLAIN CONCRETE WITH COMPRESSION-TENSION STRESS.....	239
18.4 PLAIN CONCRETE WITH PURE TENSION OR COMBINED TENSION-COMPRESSION STRESS.....	239
18.5 SHEAR LIMITED BY WEB COMPRESSIVE STRESSES .....	239
18.6 SHEAR IN SLABS.....	239
18.7 BOND STRENGTH IN REINFORCEMENT AND PRESTRESSED STEEL .....	240
18.8 TENSILE STRESS RANGE IN STEEL .....	240
18.9 CALCULATION OF STRESSES IN REINFORCEMENT AND TENDONS OF FLEXURAL MEMBERS.....	243
<b>SECTION 19 JOINTS, EMBEDDED ITEMS AND FIXINGS</b>	
19.1 JOINTS.....	244
19.2 EMBEDDED ITEMS.....	245
19.3 FIXINGS .....	245
<b>SECTION 20 PLAIN CONCRETE PEDESTALS AND FOOTINGS</b>	
20.1 GENERAL.....	247
20.2 DURABILITY .....	247
20.3 PEDESTALS .....	247
20.4 FOOTINGS.....	247
<b>SECTION 21 SLAB-ON-GROUND FLOORS, PAVEMENTS AND FOOTINGS</b>	
21.1 GENERAL.....	249
21.2 DESIGN CONSIDERATIONS.....	249
21.3 FOOTINGS.....	249



## APPENDICES

A	REFERENCED DOCUMENTS.....	250
B	TESTING OF MEMBERS AND STRUCTURES.....	252
C	RESIDUAL TENSILE STRENGTH TEST FOR SFRC .....	258
BIBLIOGRAPHY.....		260

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## STANDARDS AUSTRALIA

**Australian Standard**  
**Concrete structures**

## SECTION 1 SCOPE AND GENERAL

**1.1 SCOPE AND APPLICATION****1.1.1 Scope**

This Standard sets out minimum requirements for the design and construction of concrete building structures and members that contain reinforcing steel or tendons, or both. It also sets out minimum requirements for plain concrete pedestals and footings.

## NOTES:

- 1 The general principles of concrete design and construction and the criteria embodied in this Standard may be appropriate for concrete structures other than buildings, members not specifically mentioned herein and to materials outside the limits given in Clause 1.1.2.
- 2 It is intended that the design of a structure or member to which this Standard applies be carried out by, or under the supervision of, a suitably experienced and competent person.
- 3 For guidance on the design of maritime structures refer to AS 4997.

This Standard is not intended to apply to the design of mass concrete structures.

**1.1.2 Application**

This Standard applies to structures and members in which the materials conform to the following:

- (a) Concrete with—
  - (i) characteristic compressive strength at 28 days ( $f'_c$ ) in the range of 20 MPa to 100 MPa; and
  - (ii) with a saturated surface-dry density in the range 1800 kg/m<sup>3</sup> to 2800 kg/m<sup>3</sup>.
- (b) Reinforcing steel of Ductility Class N in accordance with AS/NZS 4671.  
NOTE: These reinforcing materials may be used, without restriction, in all applications referred to in this Standard.
- (c) Reinforcing steel of Ductility Class L in accordance with AS/NZS 4671—
  - (i) may be used as main or secondary reinforcement in the form of welded wire mesh, or as wire, bar and mesh in fitments; but
  - (ii) shall not be used in any situation where the reinforcement is required to undergo large plastic deformation under strength limit state conditions.

NOTE: The use of Ductility Class L reinforcement is further limited by other clauses within the Standard.

- (d) Higher reinforcing steel grades >500 MPa to 800 MPa meeting the requirements of Table 3.2.1. For ultimate limit states the strength of the reinforcement in design models shall not be taken as greater than 600 MPa unless noted otherwise.
- (e) Prestressing tendons conforming with AS/NZS 4672.1 and tested in accordance with AS/NZS 4672.2.

### 1.1.3 Exclusions

The requirements of this Standard shall not take precedence over design requirements and material specifications set out in other Australian Standards that deal with specific types of structures.

## 1.2 NORMATIVE REFERENCES

Normative documents referred to in this Standard are listed in Appendix A.

NOTE: Informative documents referred to in this Standard are listed in the Bibliography at the end of this document.

## 1.3 EXISTING STRUCTURES

The general principles of this Standard shall be applied when evaluating the strength or serviceability of an existing structure.

NOTE: Existing structures are likely to contain materials that do not conform with the material specifications herein and may have been designed to different requirements, but the general principles of this Standard would apply. (See also Appendix B.)

## 1.4 DOCUMENTATION

The drawings and/or specification for concrete structures and members shall include, as appropriate, the following:

- (a) Reference number and date of issue of applicable design Standards.
- (b) Imposed actions (live loads) used in design.
- (c) The appropriate earthquake design category determined from AS 1170.4.
- (d) Any constraint on construction assumed in the design.
- (e) Exposure classification for durability.
- (f) Fire resistance level (FRL), if applicable.
- (g) Class and, where appropriate, grade designation of concrete.
- (h) Any required properties of the concrete.
- (i) The curing procedure.
- (j) Grade, Ductility Class and type of reinforcement and grade and type of tendons.
- (k) The size, quantity and location of all reinforcement, tendons and structural fixings and the cover to each.
- (l) The location and details of any splices, mechanical connections and welding of any reinforcement or tendon.
- (m) The maximum jacking force to be applied in each tendon and the order in which tendons are to be stressed.
- (n) The shape and size of each member.
- (o) The finish and method of control for unformed surfaces.
- (p) Class of formwork in accordance with AS 3610 for the surface finish specified.
- (q) The minimum period of time after placing of concrete before stripping of forms and removal of shores.
- (r) The location and details of planned construction and movement joints, and the method to be used for their protection.

## 1.5 CONSTRUCTION

All concrete structures, designed in accordance with this Standard, shall be constructed so that all the requirements of the design, as contained in the drawings and specifications, are achieved.

## 1.6 DEFINITIONS

### 1.6.1 General

For the purposes of this Standard, the definitions below apply.

### 1.6.2 Administrative definitions

#### 1.6.2.1 *Building authority or other relevant regulatory authority*

The body having statutory powers to control the design and construction of the structure in the area in which the structure is to be constructed.

#### 1.6.2.2 *Drawings*

The diagrams forming part of the documents setting out the work to be executed.

#### 1.6.2.3 *Specification*

The requirements forming part of the documents setting out the work to be executed.

### 1.6.3 Technical definitions

#### 1.6.3.1 *Action*

Set of concentrated or distributed forces acting on a structure (direct action), or deformation imposed on a structure or constrained within it (indirect action).

NOTE: The term 'load' is also often used to describe direct actions.

#### 1.6.3.2 *Action effects*

Internal forces and bending moments due to actions (stress resultants).

#### 1.6.3.3 *Anchorage zone*

Region between the face of the member where the prestress is applied and the cross-section at which a linear distribution of stress due to prestress is achieved.

#### 1.6.3.4 *Average ambient temperature*

Average value of the daily maximum and minimum ambient temperatures over the relevant period at a site.

#### 1.6.3.5 *Average axis distance*

See Clause 5.2.1.

#### 1.6.3.6 *Axis distance*

Distance from the centre-line axis of a longitudinal bar or tendon to the nearest surface exposed to fire (see Figure 5.2.2).

#### 1.6.3.7 *B-region*

Portion of a member in which the assumption that plane sections remain plane can be applied.

#### 1.6.3.8 *Basic creep coefficient*

Mean value of the ratio of final creep strain to elastic strain for a specimen loaded at 28 days under a constant stress of  $0.4f'_c$  (see Clause 3.1.8.2).

**1.6.3.9 Bottle-shaped compression field**

Compression field that is wider at mid-length than at its ends [see Figure 7.2.1(c)].

**1.6.3.10 Braced column**

Column in a structure for which the lateral actions, applied at the ends in the direction under consideration, are resisted by components such as masonry infill panels, shear walls or lateral bracing.

**1.6.3.11 Characteristic strength**

Value of the material strength, as assessed by standard test, that is exceeded by 95% of the material (lower characteristic strength).

**1.6.3.12 Closed fitment**

A unit or multiple units of reinforcement used as an external or internal tie that form a continuous perimeter around an element with the ends anchored using a minimum of 135 degree hooks around a longitudinal bar.

**1.6.3.13 Closed tie**

A unit of reinforcement used as an external or internal fitment having a straight leg and minimum of 135 degree hooks at each end fixed around a longitudinal bar.

**1.6.3.14 Collector**

An element that takes distributed lateral action from the diaphragm and delivers it to a vertical element. A distributor takes the actions from a vertical element and distributes it into the diaphragm. Also known as a drag bar, strut or distributor.

**1.6.3.15 Column strip**

See Clause 6.1.4.1.

**1.6.3.16 Composite concrete member**

Member consisting of concrete members constructed separately but structurally connected so the member responds as a unit to applied actions.

**1.6.3.17 Concrete**

Mixture of cement, aggregates and water, with or without the addition of chemical admixtures.

**1.6.3.18 Construction joint**

Joint that is located in a structure or part of a structure for convenience of construction and made so that the load-carrying capacity and serviceability of the structure, or part of the structure, will be unimpaired by the inclusion of the joint.

**1.6.3.19 Cover**

Distance between the outside of the reinforcing steel or tendons and the nearest permanent surface of the member, excluding any applied surface finish.

**1.6.3.20 Creep coefficient**

Mean value of the ratio of creep strain to elastic strain under conditions of constant stress.

**1.6.3.21 Critical opening**

Opening through the thickness of a slab where an edge, or part of the edge, of the opening is located at a clear distance of less than  $2.5b_o$  from the critical shear perimeter [see Figure 9.3(A)(b)].

### **1.6.3.22 Critical shear perimeter**

Perimeter defined by a line geometrically similar to the boundary of the effective area of a support or concentrated load and located at a distance of  $d_{om}/2$  therefrom [see Figure 9.3(A)].

### **1.6.3.23 Cross tie**

An internal fitment with a single leg and minimum of 135 degree hook at one end and a minimum of 90 degrees cog at the other end anchored around a longitudinal bar.

### **1.6.3.24 Design life**

Period for which a structure or a structural member is intended to remain fit for use for its intended purpose with appropriate maintenance.

### **1.6.3.25 Design strip**

See Clause 6.1.4.2.

### **1.6.3.26 Diaphragm**

A horizontal or near horizontal concrete element such as a floor or roof used to transmit horizontal actions into the vertical elements of the force resisting system.

### **1.6.3.27 Direct loading**

Loading on a structure that includes the self-weight of its component members and externally applied loads.

### **1.6.3.28 Discontinuity**

Abrupt change in geometry or loading, including prestress.

### **1.6.3.29 D-region**

Portion of a member within a distance equal to the member depth ( $D$ ), from a discontinuity.

### **1.6.3.30 Duct**

Conduit (plain or corrugated) to accommodate prestressing tendon(s) for post-tensioned installation.

### **1.6.3.31 Ductility Class**

Designation relating to the ductility of reinforcement ('L' designates 'low', 'N' designates 'normal', 'E' designates 'earthquake').

NOTE: For further information refer to AS/NZS 4671.

### **1.6.3.32 Durability**

Ability of a structure and its component members to perform the functions for which they have been designed, over a specified period of time, when exposed to their environment.

### **1.6.3.33 Effective area of a support or concentrated load for slabs in shear**

Area totally enclosing the actual support or load and for which the perimeter is a minimum [see Figure 9.3(A)].

### **1.6.3.34 Effective depth**

Distance from the extreme compressive fibre of the concrete to the resultant tensile force in the reinforcing steel and tendons in that zone, which will be tensile at the ultimate strength condition of pure bending.

### **1.6.3.35** *Embedded items*

Items, other than reinforcement and tendons, that are embedded in a concrete member or structure.

NOTE: Embedded items include pipes and conduits with their associated fittings, sleeves, permanent inserts for fixings and other purposes, prestressed anchorages, holding-down bolts and other supports.

### **1.6.3.36** *Exposure classification*

Designation indicative of the most severe environment to which a concrete member is to be subjected during its design life (see Table 4.3).

### **1.6.3.37** *Fan-shaped compression field*

Compression field that has non-parallel straight sides [see Figure 7.2.1(b)].

### **1.6.3.38** *Fire resistance*

Ability of a structure or part of it to fulfil its required functions (loadbearing and/or separating function) for a specified fire exposure, for a specified time.

### **1.6.3.39** *Fire resistance level (FRL)*

Fire resistance periods for structural adequacy, integrity and insulation expressed in that order.

NOTE: Fire resistance levels for structures, parts and elements of construction are specified by the relevant authority [e.g. in the Building Code of Australia (BCA)].

### **1.6.3.40** *Fire resistance period (FRP)*

Time, in minutes, for a member to reach the appropriate failure criterion (i.e. structural adequacy, integrity and/or insulation) if tested for fire in accordance with the appropriate Standard.

NOTE: For structures that need to conform with the BCA requirements, the appropriate Standard is AS 1530.4.

### **1.6.3.41** *Fire-separating function*

Ability of a boundary element of a fire compartment (e.g. wall, floor or roof) to prevent fire spread by passage of flames or hot gases (integrity) or ignition beyond the exposed surface (thermal insulation) during a fire.

NOTE: When tested in accordance with AS 1530.4, prototypes of such members are exposed to fire from only one direction at a time and are assumed to be similarly exposed for the purpose of interpreting Section 5.

### **1.6.3.42** *Fitment*

Unit of reinforcement commonly used to restrain from buckling the longitudinal reinforcing bars in beams, columns and piles; carry shear, torsion and diagonal tension; act as hangers for longitudinal reinforcement; or provide confinement to the core concrete.

NOTE: Also referred to commonly as a stirrup, ligature or helical reinforcement.

### **1.6.3.43** *Fixing or fastener or anchor or lifter*

Material cast into concrete for the purpose of maintaining in position reinforcement, tendons, ducts, formwork, inserts or a post fixed element or devices for lifting of members.

### **1.6.3.44** *Flat plate*

Flat slab without drop panels.

**1.6.3.45 Flat slab**

Continuous two-way solid or ribbed slab, with or without drop-panels, having at least two spans in each direction, supported internally by columns without beams and supported externally by walls or columns with or without spandrel beams, or both.

**1.6.3.46 Footing**

Part of a structure in direct contact with and transmitting load to the supporting foundation.

**1.6.3.47 Foundation**

Soil, subsoil or rock, whether built-up or natural, by which a structure is supported.

**1.6.3.48 Grout**

Mixture of cement and water, with or without the addition of sand, or chemical admixtures, proportioned to produce a pourable liquid without segregation of the constituents.

**1.6.3.49 Headed reinforcement**

Steel bar that achieves anchorage by means of a suitably sized head or end plate.

**1.6.3.50 Helical reinforcement**

Unit of reinforcement that is wound in a helical fashion around the main longitudinal reinforcing bars in a column or pile restraining them from buckling and to carry shear, torsion and diagonal tension or around tendons at an anchorage to resist bursting action effects.

**1.6.3.51 Hollow-core slab or wall**

Slab or wall having mainly a uniform thickness and containing essentially continuous voids.

**1.6.3.52 Initial force**

Force immediately after transfer, at a stated position in a tendon.

**1.6.3.53 Insulation (fire)**

The ability of a fire-separating member, such as a wall or floor, to limit the surface temperature on one side of the member when exposed to fire on the other side.

**1.6.3.54 Integrity (fire)**

Ability of a fire-separating member to resist the passage of flames or hot gases through the member when exposed to fire on one side.

**1.6.3.55 Jacking force**

Force in a tendon measured at the jack.

**1.6.3.56 Ligature (reinforcement)**

See 'Fitment'.

**1.6.3.57 Lightweight concrete**

Concrete having a saturated surface-dry density in the range of 1800 kg/m<sup>3</sup> to 2100 kg/m<sup>3</sup>.

**1.6.3.58 Limit state**

Limiting condition at which the structure ceases to fulfil its intended function.

**1.6.3.59 Loadbearing function**

Ability of a structure or member to sustain specified actions during the fire.



**1.6.3.60 Loadbearing member**

Member intended to support or transmit vertical loads additional to its own weight where the design axial force at mid-height of the member is greater than  $0.03 f'_c A_g$ .

**1.6.3.61 Mean strength**

Statistical average of a number of test results representative of the strength of a member, prototype or material.

**1.6.3.62 Middle strip**

See Clause 6.1.4.3.

**1.6.3.63 Movement joint**

Joint that is made between parts of a structure for the specific purpose of permitting relative movement between the parts of the structure on either side of the joint.

**1.6.3.64 Node**

Point in a joint in a strut-and-tie model where the axes of the struts, ties and concentrated forces acting on the joint intersect.

**1.6.3.65 Nodal zone**

Volume of concrete around a node, which is assumed to transfer strut-and-tie forces through the node.

**1.6.3.66 One-way slab**

Slab characterized by flexural action mainly in one direction.

**1.6.3.67 Plain concrete member**

Member either unreinforced or containing reinforcement but assumed to be unreinforced.

**1.6.3.68 Post-tensioning**

Tensioning of tendons after the concrete has hardened.

**1.6.3.69 Prestressed concrete**

Concrete into which internal stresses are induced deliberately by tendons.

NOTE: It includes concrete commonly referred to as 'partially prestressed'.

**1.6.3.70 Prestressing steel**

See 'Tendon'.

**1.6.3.71 Pretensioning**

Tensioning of tendons before the concrete is placed.

**1.6.3.72 Prismatic compression field**

Compression field that is parallel sided [see Figure 7.2.1(a)].

**1.6.3.73 Reinforcement**

Steel bar, wire or mesh but not tendons.

NOTE: Commonly referred to as reinforcing steel.

**1.6.3.74 Ribbed slab**

Slab incorporating parallel ribs in one or two directions.

**1.6.3.75 Shear wall**

Wall that is intended to resist lateral forces acting in or parallel to the plane of the wall.

**1.6.3.76 Short column**

Column in which the additional bending moments due to slenderness can be taken as zero.

**1.6.3.77 Slender column**

Column that does not satisfy the requirements for a short column.

**1.6.3.78 Span support**

See Clause 6.1.4.4.

**1.6.3.79 Strength grade**

Numerical value of the characteristic compressive strength of concrete at 28 days ( $f'_c$ ), used in design.

**1.6.3.80 Structural adequacy (fire)**

Ability of a member to maintain its structural function when exposed to fire.

**1.6.3.81 Strut-and-tie model**

Truss model made up of struts and ties connected at nodes.

**1.6.3.82 Tendon**

Wire, strand or bar (or any discrete group of such wires, strands or bars) that is intended to be pretensioned or post-tensioned.

**1.6.3.83 Tie**

Tension member in a strut-and-tie model.

**1.6.3.84 Torsion strip**

Strip of slab of width  $a$ , whose longitudinal axis is perpendicular to the direction of  $M_v^*$  [see Figure 9.3(B)].

**1.6.3.85 Transfer**

Time of initial transfer of prestressing forces from the tendons to the concrete.

**1.6.3.86 Transmission length**

Length, at transfer, over which the stress in a pretensioned tendon builds up from zero at one end to its full value.

**1.6.3.87 Transverse width**

See Clause 6.1.4.5.

**1.6.3.88 Two-way slab**

Slab characterized by significant flexural action in two directions at right angles to one another.

**1.6.3.89 Uniform strain**

Strain in the reinforcement at maximum stress, corresponding to the onset of necking.

**1.6.3.90 Upper characteristic strength**

Value of the material strength, as assessed by standard test, which is exceeded by 5% of the material.

## 1.7 NOTATION

The symbols used in this Standard, including their definitions, are listed below.

Unless a contrary intention appears, the following applies:

- (a) The symbols used in this Standard shall have the meanings ascribed to them below, with respect to the structure, or member, or condition to which a clause is applied.
- (b) Where non-dimensional ratios are involved, both the numerator and denominator shall be expressed in identical units.
- (c) The dimensional units for length, force and stress, in all expressions or equations, shall be taken as millimetres (mm), newtons (N) and megapascals (MPa) respectively, unless noted otherwise.
- (d) An asterisk (\*) placed after a symbol as a superscript (e.g.,  $M_y^*$ ) denotes a design action effect due to the design load.

Symbol	Definition
$A_b$	= cross-sectional area of a reinforcing bar
$A_{b,fit}$	= cross-sectional area of the fitment
$A_c$	= smallest cross-sectional area of the concrete strut at any point along its length and measured normal to the line of action of the strut (see Clauses 5.6.3 and 7.2.3); <i>or</i> = cross-sectional area bounded by the centre-line of the outermost fitments (see Clause 10.7.3.3)
$A_g$	= gross cross-sectional area of a member
$A_m$	= an area enclosed by the median lines of the walls of a single cell (see Clause 8.3.3)
$A_p$	= cross-sectional area of prestressing steel
$A_{pt}$	= cross-sectional area of the tendons in the zone that will be tensile under ultimate load conditions
$A_s$	= cross-sectional area of reinforcement (see Clauses 3.4.3.2 and 13.2.2); <i>or</i> = cross-sectional area of a single anchored bar of diameter $d_b$ (see Clause 13.1.2.3)
$A_{sc}$	= cross-sectional area of compressive reinforcement
$A_{sf}$	= area of fully anchored reinforcement crossing the interface
$A_{si}$	= cross-sectional area of steel bar (tendon, wire) (see Clause 5.2.1) = area of reinforcement in the $i$ th direction crossing a strut (see Clause 7.2.4)
$A_{st}$	= cross-sectional area of longitudinal tensile reinforcement; <i>or</i> = cross-sectional area of reinforcement in the zone that would be in tension under the design loads if the effects of prestress and axial loads are ignored
$A_{sv}$	= cross-sectional area of shear reinforcement
$A_{sv,min}$	= cross-sectional area of minimum shear reinforcement
$A_{sw}$	= cross-sectional area of the bar forming a closed fitment
$A_t$	= area of a polygon with vertices at the centre of longitudinal bars at the corners of the cross-section

$A_{tr}$	= cross-sectional area of a transverse bar along a development or lap length (see Clause 13.1.2.3)
$A_{tr.min}$	= cross-sectional area of the minimum transverse reinforcement along the development length (see Clause 13.1.2.3)
$A_1$	= a bearing area (see Clause 12.6)
$A_2$	= largest area of the supporting surface that is geometrically similar to and concentric with $A_1$ (see Clause 12.6)
$a$	= a distance; <i>or</i> = shear span, equal to the distance between the centroids of an applied load and a support reaction in a structure (see Clause 7.2.4); <i>or</i> = perpendicular distance from the nearer support to the section under consideration (see Clause 9.6); <i>or</i> = dimension of the critical shear perimeter measured parallel to the direction of $M_v^*$ [see Figure 9.3(B)]
$a_m$	= average axis distance (see Clause 5.2.1)
$a_s$	= axis distance (see Clause 5.2.2)
$a_{sup}$	= length of a support in the direction of the span (see Clause 6.1.4.4)
$b$	= width of a rectangular cross-section or member; <i>or</i> = width of beam at the centroid of the bottom reinforcement (see Clause 5.4.1); <i>or</i> = width of ribs [see Table 5.5.2(C) and Table 5.5.2(D)]; <i>or</i> = smaller cross-sectional dimension of a rectangular column or the diameter of a circular column (see Table 5.6.3 and Table 5.6.4); <i>or</i> = wall thickness (see Table 5.7.2)
$b_c$	= core dimension measured between the centre-lines of the outermost fitments measured across the width of the section (see Clause 10.7.3.3)
$b_{ef}$	= effective width of a compression face or flange of a member
$b_f$	= width of the shear plane (see Clause 8.4.3)
$b_l$	= size of rectangular, or equivalent rectangular column, capital, or bracket, measured in the direction of the span for which moments are being determined (see Clause 14.5.3.2)
$b_o$	= dimension of an opening (see Clause 9.3.1.2 and 9.3.1.5)
$b_t$	= size of rectangular, or equivalent rectangular column, capital, or bracket, measured transverse to the direction of the span for which moments are being determined (see Clause 14.5.3.2)
$b_v$	= effective width of a web for shear (see Clause 8.2.6)
$b_w$	= a width of the web; <i>or</i> = minimum thickness of the wall of a hollow section (see Clause 8.3.3)
$c$ ( $c_1$ )	= cover to reinforcing steel or tendons
$c_d$	= a dimension (in millimetres) corresponding to the smaller of the concrete cover to a bar developing stress and half the clear distance to the next parallel bar developing stress, as shown in Figure 13.1.2.2

- $D$  = overall depth of a cross-section in the plane of bending; *or*  
= depth or breadth of the symmetrical prism as appropriate (see Clause 12.5.6)
- $D_b$  = overall depth of a spandrel beam
- $D_c$  = smaller column cross-sectional dimension if rectangular, or the column diameter if circular (see Clause 10.7.4.3)
- $D_s$  = overall depth of a slab or drop panel  
= the member depth at the theoretical cut-off point or debonding point (see Clause 8.1.11.1)
- $d$  = effective depth of a cross-section in the plane of bending; *or*  
= nominal internal diameter of reinforcement bend or hook (see Clause 17.2.3.2)
- $d_b$  = nominal diameter of a bar, wire or tendon
- $d_c$  = width of the idealized strut (see Clause 7.2.4); *or*  
= core dimension measured between the centre-lines of the outermost fitments measured through the depth of the section (see Clause 10.7.3.3)
- $d_d$  = diameter of a prestressing duct (see Clause 8.2.6)
- $d_t$  = diameter of the bar forming the tie (see Paragraph C4.2.2, Appendix C)
- $d_o$  = distance from the extreme compressive fibre of the concrete to the centroid of the outermost layer of tensile reinforcement or tendons (not less than  $0.8D$  for prestressed concrete members)
- $d_{om}$  = mean value of  $d_o$ , averaged around the critical shear perimeter
- $d_p$  = distance from the extreme compressive fibre of the concrete to the centroid of the tendons in that zone, which will be tensile under ultimate strength conditions
- $d_s$  = overall dimension measured between centre-lines of the outermost fitments (see Clause 10.7.3.3)
- $d_{sc}$  = distance from the extreme compressive fibre of the concrete to the centroid of compressive reinforcement (see Clause 8.1.7)
- $EC_e$  = electrical conductivity (see Clause 4.8.2)
- $E_c$  = mean value of the modulus of elasticity of concrete at 28 days
- $E_{cj}$  = mean value of the modulus of elasticity of concrete at the appropriate age, determined in accordance with Clause 3.1.2
- $E_d$  = design action effect (see Clauses 2.2.2 to 2.2.6)
- $E_p$  = modulus of elasticity of tendons, determined in accordance with Clause 3.3.2
- $E_s$  = modulus of elasticity of reinforcement, determined in accordance with Clause 3.2.2
- $e$  = eccentricity of prestressing force or load; *or*  
= the base of Napierian logarithms
- $e_a$  = an additional eccentricity (see Clause 11.5.1)
- $F$  = total vertical component of the external load carried through the shear span (see Clause 12.2.1)

$F_d$	= uniformly distributed design load, factored for strength or serviceability, as appropriate
$F_{d,ef}$	= effective design service load per unit length or area, used in serviceability design
$f_{cm}$	= mean value of cylinder strength (see Clause 3.1.1.2)
$f_{cmi}$	= mean value of the in situ compressive strength of concrete at the relevant age (see Clause 3.1.1.2 and Table 3.1.2)
$f_{cp}$	= mean compressive strength of concrete at transfer
$f_{ct}$	= uniaxial tensile strength of concrete (see Clause 3.1.1.3)
$f_{ct,f}$	= measured flexural tensile strength of concrete (see Clause 3.1.1.3)
$f_{ct,sp}$	= measured splitting tensile strength of concrete (see Clause 3.1.1.3)
$f_{cv}$	= concrete shear strength (see Clause 9.2.4)
$f_{pb}$	= characteristic minimum breaking strength (see Clause 3.3.1)
$f_{py}$	= yield strength of tendons determined in accordance with Clause 3.3.1
$f_r$	= average confining pressure on the core cross-section taken at the level of the fitments (see Clause 10.7.3.3)
$f_{r,eff}$	= effective confining pressure applied to the core of a column (see Clause 10.7.3.3)
$f_{si}$	= stress in reinforcement in the $i$ th direction crossing a strut
$f_{sy}$	= characteristic yield strength of reinforcement (referred to as $R_e$ in AS/NZS 4671), determined in accordance with Clause 3.2.1
$f_{sy,f}$	= yield strength of reinforcement used as fitments
$f'_c$	= characteristic compressive (cylinder) strength of concrete at 28 days
$f'_{cc}$	= compressive strength of the concrete in the column (see Clause 10.8)
$f'_{ce}$	= effective compressive strength of the concrete in the joint (see Clause 10.8)
$f'_{cs}$	= compressive strength of the concrete in the slab or beams (see Clause 10.8)
$f'_{ct}$	= characteristic uniaxial tensile strength of concrete (see Clause 3.1.1.3)
$f'_{ct,f}$	= characteristic flexural tensile strength of concrete at 28 days (see Clause 3.1.1.3)
$G$	= permanent action (dead load)
$g$	= dead load, per unit length or area
$g_p$	= permanent distributed load normal to the shear interface per unit length, newtons per millimetre (N/mm) (see Clause 8.4.3)
$H_w$	= floor-to-floor unsupported height of a wall
$H_{we}$	= effective height of a wall
$h$	= overall depth of a joint (see Clause 10.8)
$h_s$	= flange thickness of a ribbed slab
$I$	= second moment of area of the uncracked concrete cross-section about the centroidal axis

$I_c$	= second moment of area of a column
$I_{cr}$	= second moment of area of a cracked section with the reinforcement transformed to an equivalent area of concrete
$I_{ef}$	= an effective second moment of area (see Clause 8.5.3)
$I_{ef,max}$	= maximum effective second moment of area (see Clause 8.5.3)
$I_f$	= second moment of area of a flexural member
$J_t$	= a torsional modulus
$j$	= time after prestressing, in days (see Clause 3.3.4.3)
$K$	= a factor that accounts for the weighted average effectiveness of transverse reinforcement in controlling potential splitting cracks along a development or lap splice length (see Clause 13.1.2.3)
$k$	= a coefficient, ratio or factor used with and without numerical subscripts
$k_{co}$	= cohesion coefficient (see Clause 8.4.3)
$k_{cs}$	= factor used in serviceability design to take account of the long-term effects of creep and shrinkage
$k_e$	= effectiveness factor accounting for the arrangement of the fitments
$k_m$	= coefficient calculated in accordance with Clause 10.4.2
$k_r$	= ratio of the depth, or breadth, of an anchorage bearing plate to the corresponding depth, or breadth, of the symmetrical prism (see Clause 12.5.4)
$k_u$	= neutral axis parameter being the ratio, at ultimate strength under any combination of bending and compression, of the depth to the neutral axis from the extreme compressive fibre to $d$
$k_{uo}$	= ratio, at ultimate strength, without axial force of the depth to the neutral axis from the extreme compressive fibre to $d_o$
$L$	= centre-to-centre distance between the supports of a flexural member
$L_e$	= effective length of a column
$L_{ef}$	= effective span of a member, taken as the lesser of $(L_n + D)$ and $L$ for a beam or slab; <i>or</i> $= L_n + D/2$ for a cantilever
$L_l$	= distance between centres of lateral restraints or from a lateral restraint to the free edge
$L_n$	= length of clear span in the direction in which moments are being determined, measured face-to-face of supporting beams, columns or walls, or for a cantilever, the clear projection
$L_o$	= $L$ minus 0.7 times the sum of the values of $a_{sup}$ at each end of the span (see Clause 6.10.4.2)
$L'_o$	= smaller value of $L_o$ for adjoining spans (see Clause 6.10.4.5)
$L_p$	= development length of tendons
$L_{pa}$	= length of the tendon from the jacking end to a point at a distance 'a' from that end (see Clause 3.4.2.4)
$L_{pt}$	= transmission length for pretensioned tendons
$L_s$	= span between formwork supports (see Clause 17.6.2.4)

- $L_{sc}$  = development length of a bar for a compressive stress less than the yield stress  
 $L_{st}$  = development length of a bar for a tensile stress less than the yield stress  
 $L_{sy,c}$  = development length in compression, being the length of embedment required to develop the yield strength of a deformed bar in compression (see Clause 13.1.5.1)  
 $L_{sy,cb}$  = basic development length of a deformed bar in compression (see Clause 13.1.5.2)  
 $L_{sy,t}$  = development length in tension, to develop the characteristic yield strength of a deformed bar in tension [see Clause 13.1.2 and Figure 13.1.2.3]  
 $L_{sy,t.lap}$  = the tensile lap length for either contact or non-contact splices (see Clause 13.2.2)  
 $L_{sy,tb}$  = basic development length of a deformed bar in tension (see Clause 13.1.2.2)  
 $L_t$  = width of a design strip [see Figure 6.1.4(A)]  
 $L_u$  = unsupported length of a column, taken as the clear distance between the faces of members capable of providing lateral support to the column. Where column capitals or haunches are present,  $L_u$  is measured to the lowest extremity of the capital or haunch  
 $L_w$  = overall length of a wall  
 $L_x$  = shorter effective span of a slab supported on four sides  
 $L_y$  = longer effective span of a slab supported on four sides  
 $l_b$  = length of the bursting zone (see Clause 7.2.4)  
 $l_x$  = shorter span of a two-way slab [see Table 5.5.2(A)]  
 $l_y$  = longer span of a two-way slab [see Table 5.5.2(A)]  
 $l_{0,fi}$  = effective length of a column under fire conditions (see Clause 5.6.3)  
 $M^*$  = design bending moment at a cross-section  
 $M_f^*$  = design moment in the fire situation (see Table 5.6.4)  
 $M_s^*$  = maximum bending moment at the section based on the short-term serviceability load or construction load (see Clause 8.5.3.1)  
 $M_{s,1}^*$  = design bending moment at the serviceability limit state, calculated with  $\psi_s = 1.0$  (see Clauses 8.6.1 and 9.4.1)  
 $M_v^*$  = design bending moment to be transferred from a slab to a support  
 $M_x^*, M_y^*$  = design bending moment in a column about the major and minor axes respectively; *or*  
= positive design bending moment, at midspan in a slab, in the  $x$  and  $y$  direction respectively  
 $M_1^*, M_2^*$  = smaller and larger design bending moment respectively at the ends of a column  
 $M_c$  = moment used in the calculation of the buckling load ( $N_c$ ) (see Clause 10.4.4)  
 $M_{cr,t}$  = bending moment at time  $t$  causing cracking of the section with due consideration to prestress, restrained shrinkage and temperature stresses  
 $M_o$  = total static moment in a span (see Clause 6.10.4.2); *or*



$M_u$	= ultimate strength in bending at a cross-section of an eccentrically loaded compressive member
$M_{ub}$	= particular ultimate strength in bending when $k_{uo} = 0.003/(0.003 + f_{sy}/E_s)$
$M_{uo}$	= ultimate strength in bending, without axial force, at a cross-section
$(M_{uo})_{min}$	= minimum required strength in bending at a critical cross-section (see Clause 8.1.6.1)
$M_{ux}, M_{uy}$	= ultimate strength in bending about the major and minor axes respectively of a column under the design axial force $N^*$
$m$	= number of fitments legs crossing the confinement plane (see Clause 10.7.3.3)
$N^*$	= axial compressive or tensile force on a cross-section
$N_f^*$	= design axial load in the fire situation (see Clause 5.6.3)
$N_c$	= buckling load used in column design
$N_u$	= ultimate strength in compression, or tension, at a cross-section of an eccentrically loaded compression or tension member respectively
	= ultimate strength per unit length of wall (see Clause 11.5.1)
$N_{ub}$	= particular ultimate strength in compression of a cross-section when $k_{uo} = 0.003/(0.003 + f_{sy}/E_s)$
$N_{uo}$	= ultimate strength in compression, without bending, of an axially loaded cross-section
$N_{uot}$	= ultimate strength in tension, without bending, of an axially loaded cross-section
$n$	= number of bars uniformly spaced around helical reinforcement (see Clause 13.2.4); or
	= number of laterally restrained longitudinal bars (see Clauses 10.7.3.3 and 10.7.3.4)
$P$	= force in the tendons; or
	= maximum force occurring at the anchorage during jacking (see Clause 12.5.4); or
	= applied loads (see Clause 12.2)
$P_e$	= total effective prestress force allowing for all losses of prestress
$P_v$	= vertical component of the prestressing force
$p$	= a reinforcement ratio
$p_{cw}$	= web reinforcement ratio for compressive reinforcement (see Clause 8.5.3.1)
$p_w$	= a reinforcement ratio in a wall; or
	= web reinforcement ratio for tensile reinforcement (see Clause 8.5.3.1)
$Q$	= imposed action (live load) including impact, if any
$q$	= imposed action (live load) per unit length or area
$R$	= design relaxation of a tendon, determined in accordance with Clause 3.3.4.3
$R_b$	= basic relaxation of a tendon, determined in accordance with Clause 3.3.4.2
$R_d$	= design capacity of a member or structure (equal to $\phi R_u$ or $\phi_{sys} R_{u,sys}$ )

$R_u$	= ultimate strength of a member (see Clause 2.2)
$R_{u.sys}$	= mean capacity of the structure (see Clause 2.2.5)
$r$	= radius of gyration of a cross-section
$S_p$	= structural performance factor (see Clause 14.3)
$s$	= centre-to-centre spacing of fitments including shear, torsional or confining reinforcement, measured parallel to the longitudinal axis of a member; <i>or</i> = standard deviation; <i>or</i> = maximum spacing of transverse reinforcement within $L_{sy.c}$ , or spacing of fitments, or spacing of successive turns of helical reinforcement, all measured centre-to-centre, in millimetres (see Clause 13.2.4); <i>or</i> = spacing of anchored shear reinforcement crossing interface (see Clause 8.4.3)
$s_b$	= clear distance between bars of the non-contact lapped splice (see Figure 13.2.2)
$s_L$	= clear distance between bars of the non-contact lapped splice (see Figure 13.2.2)
$T$	= a temperature; <i>or</i> = force resultant of transverse tensile stresses (see Clause 12.5.4)
$T^*$	= torsional moment at a cross-section
$T_b^*$	= bursting force calculated at the ultimate limit state (see Clause 7.2.4)
$T_{b.s}^*$	= bursting force calculated at the serviceability state (see Clause 7.2.4)
$T_{b.cr}$	= bursting (or splitting) force across a strut caused at the time of cracking of the strut (see Clause 7.2.4)
$T_u$	= ultimate torsional strength
$T_{u.max}$	= ultimate torsional strength of a beam limited by web crushing failure (see Clause 8.3.3)
$T_w$	= vertical component of the force carried by the secondary struts (see Clause 12.2)
$t$	= time
$t_d$	= difference between the actual effective thickness of the slab and the effective thickness specified in Table 5.5.1, for the required FRP (see Clause 5.8.2)
$t_f$	= thickness of topping or flange anchored by shear reinforcement (see Clause 8.4.4)
$t_h$	= hypothetical thickness of a member used in determining creep and shrinkage, taken as $2A_g/u_e$
$t_{nom}$	= nominal thickness of topping applied (see Clause 5.8.2)
$t_w$	= thickness of a wall
$u$	= length of the critical shear perimeter (see Clause 9.3.1.5)
$u_e$	= exposed perimeter of a member cross-section plus half the perimeter of any closed voids contained therein, used to calculate $t_h$
$V^*$	= design shear force at a cross-section

$V_o$	= shear force which would occur at a section when the bending moment at that section was equal to the decompression moment $M_o$
$V_u$	= ultimate shear strength
$V_{u,max}$	= ultimate shear strength limited by web crushing failure
$V_{uc}$	= ultimate shear strength excluding shear reinforcement (see Clause 8.2.3)
$V_{uo}$	= ultimate shear strength of a slab with no moment transfer (see Clause 9.3.3)
$V_{us}$	= contribution by shear reinforcement to the ultimate shear strength of a beam or wall (see Clauses 8.2.3 and 11.6.4)
$w$	= average clear spacing between adjacent tied longitudinal bars (see Clause 10.7.3.3); <i>or</i> = width of loaded area (see Figure 12.2.1) or node [see Figure 7.2.4(A)]
$X$	= a dimension [see Figure 9.3(A)]
$x$	= shorter overall dimension of a rectangular part of a cross-section
$Y$	= a dimension [see Figure 9.3(A)]
$y$	= larger overall dimension of a rectangular part of a cross-section
$y_1$	= larger overall dimension of a closed fitment (see Clause 9.3.1.5)
$Z$	= section modulus of the uncracked cross-section, referred to the extreme fibre at which flexural cracking occurs (see Clause 8.1.6.1)
$z$	= projection of the inclined compressive strut normal to the shear span (see Clause 7.2.4); <i>or</i> = internal moment lever arm of the section (see Clause 8.4.2)
$\alpha$	= coefficient; <i>or</i> = angle of divergence between bottle shape compression fields and idealized parallel sided strut (see Clause 7.2.4)
$\alpha_b$	= coefficient for beams (see Clause 8.1.6.1)
$\alpha_c$	= coefficient (see Clause 10.3.1)
$\alpha_n$	= coefficient (see Clause 10.6.4)
$\alpha_s$	= correlation factor (see Clause 10.4.3)
$\alpha_{tot}$	= sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length ( $L_{pa}$ ) (see Clause 3.4.2.4)
$\alpha_v$	= angle between the inclined shear reinforcement and the longitudinal tensile reinforcement (see Clause 8.2.3.3)
$\alpha_x, \alpha_y$	= short and long span bending moment coefficients respectively, for slabs supported on four sides (see Clause 6.10.3.2)
$\beta$	= an effective compression strength factor (see Clause 2.2.3); <i>or</i> = fixity factor (see Clause 10.5.4); <i>or</i> = a ratio (see Clauses 8.4.2 and 8.5.3.1); <i>or</i> = a factor with or without alphanumeric subscripts (see Clause 8.2.7)
$\beta_d$	= a factor (see Clause 10.4.3)
$\beta_h$	= a ratio (see Clause 9.3.1.5)

- $\beta_n$  = factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone (see Clause 7.4.2)
- $\beta_p$  = an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects (see Clause 3.4.2.4)
- $\beta_s$  = strut efficiency factor (see Clause 7.2.2)
- $\beta_x, \beta_y$  = short and long span bending moment coefficients respectively, for slabs supported on four sides (see Clause 6.10.3.2)
- $\gamma$  = the ratio, under design bending or design combined bending and compression, of the depth of the assumed rectangular compressive stress block to  $k_u d$
- $\gamma_1, \gamma_2$  = column end restraint coefficients, determined in accordance with Clause 10.5.3
- $\gamma_i$  = angle between the axis of a strut and the bars in the  $i$ th direction of reinforcement crossing that strut (see Clause 7.2.4)
- $\Delta$  = a deflection
- $\Delta\sigma_p$  = change in the stress due to the change in length of the prestressed tie (see Clause 7.3.2)
- $\delta, \delta_b, \delta_s$  = moment magnifiers for slenderness effects (see Clause 10.4)
- $\varepsilon$  = a strain
- $\varepsilon_{cc}$  = strain due to concrete creep (see Clauses 3.1.8.1 and 3.4.3.3)
- $\varepsilon_{cs}$  = design shrinkage strain, determined in accordance with Clause 3.1.7.1
- $\varepsilon_{cs}^*$  = final design shrinkage strain of concrete
- $\varepsilon_{csd}$  = drying shrinkage strain, determined in accordance with Clause 3.1.7.2
- $\varepsilon_{cse}$  = autogenous shrinkage strain, determined in accordance with Clause 3.1.7.2
- $\varepsilon_{cse}^*$  = final autogenous shrinkage strain, determined in accordance with Clause 3.1.7.2
- $\varepsilon_{csd,b}$  = basic drying shrinkage strain, determined in accordance with Clause 3.1.7.2
- $\varepsilon_{csd,b}^*$  = final drying basic shrinkage strain, determined in accordance with Clause 3.1.7.2
- $\varepsilon_{pu}$  = strain at maximum stress of a prestressing tendon
- $\varepsilon_{su}$  = uniform strain at maximum stress, corresponding to the onset of necking
- $\theta$  = angle measured between the axis of the strut and the axis of a tie passing through a common node (see Clauses 7.2.2 and 12.2); *or*  
= angle between tie leg and confinement plane (see Clause 10.7.3.3)
- $\theta_v$  = angle between the axis of the concrete compression strut and the longitudinal axis of the member (see Clause 8.2.3.3)
- $\lambda_1$  = a factor being 1.0 for normal concrete and 0,75 for lightweight concrete
- $\lambda_{uc}$  = a ratio of loads (see Clause 10.4.3)
- $\mu$  = friction curvature coefficient (see Clause 3.4.2.4); *or*  
= coefficient of friction (see Clause 8.4.3); *or*  
= structural ductility factor (see Appendix C)
- $\nu$  = Poisson's ratio for concrete, determined in accordance with Clause 3.1.5

$\rho$	= density of concrete, in kilograms per cubic metre (kg/m <sup>3</sup> ), determined in accordance with Clause 3.1.3
$\rho_p$	= transverse compressive pressure, in megapascals, at the ultimate limit state along the development length perpendicular to the plane of splitting (see Clause 13.1.2.3)
$\rho_s$	= volumetric ratio of the fitments relative to the volume of the core (see Clause 10.7.3.3)
$\sigma_{ci}$	= sustained stress in the concrete at the level of the centroid of the tendons, calculated using the initial prestressing force prior to any time-dependent losses and the sustained portions of all the service loads (see Clause 3.4.3.3)
$\sigma_{cp}$	= average intensity of effective prestress in concrete
$\sigma_{cs}$	= maximum shrinkage-induced tensile stress on the uncracked section at the extreme fibre at which cracking occurs (see Clause 8.5.3.1)
$\sigma_o$	= a constant sustained stress (see Clause 3.1.8.1)
$\sigma_{pa}$	= stress in the tendon at a distance 'a', measured from the jacking end (see Clause 3.4.2.4)
$\sigma_{p.ef}$	= effective stress in the tendon after allowing for all losses (see Clause 8.1.8)
$\sigma_{pi}$	= stress in the tendon immediately after transfer
$\sigma_{pj}$	= stress in the tendon at the jacking end (see Clause 3.4.2.4)
$\sigma_{pu}$	= maximum stress that would be reached in a tendon at ultimate strength of a flexural member
$\sigma_{sc}$	= a compressive stress being developed in a bar in compression (see Clause 13.1.5.4)
$\sigma_{scr}$	= tensile steel stress at the serviceability limit state for a beam in flexure or in tension (see Clause 8.6.1) or for a slab in flexure (see Clause 9.4.1)
$\sigma_{scr.l}$	= tensile stress in reinforcement at a cracked section, due to the short-term load combination for the serviceability limit states, calculated with $\psi_s = 1.0$ , when direct loads are applied (see Clause 8.6.1)
$\sigma_{st}$	= tensile stress in reinforcement (see Clause 13.1.2.4)
$\tau$	= age of concrete at the time of loading, in days
$\tau^*$	= design shear stress acting on the interface (see Clause 8.4.2)
$\tau_u$	= unit shear strength (see Clause 8.4.3)
$\phi$	= capacity reduction factor for design using linear elastic analysis (see Clause 2.2.2)
$\phi_s$	= stress reduction factor for design using linear stress analysis (see Clause 2.2.3)
$\phi_{st}$	= strength reduction factor for design using strut-and-tie analysis (see Clause 2.2.4)
$\phi_{sys}$	= system strength reduction factor for design using non-linear methods of analysis (see Clauses 2.2.5 and 2.2.6)
$\varphi_{cc}$	= design creep coefficient at any time $t$ (see Clause 3.1.8.3)
$\varphi_{cc}^*$	= final design creep coefficient (see Clause 3.1.8.3)

$\varphi_{cc,b}$	= basic creep coefficient of concrete, determined in accordance with Clause 3.1.8.2
$\psi_a$	= factor for reduction of imposed (live) floor loads due to area (refer to AS/NZS 1170.1)
$\psi_c$	= combination imposed action (live load) factor used in assessing the design load for strength (refer to AS/NZS 1170.0)
$\psi_s$	= short-term imposed action (live load) factor used in assessing the design load for serviceability (refer to AS/NZS 1170.0)
$\psi_l$	= long-term imposed action (live load) factor used in assessing the design load for serviceability (refer to AS/NZS 1170.0)
$\Omega$	= dimension of node [see Figure 7.2.4(A)]
$n_{bs}$	= number of longitudinal bars being developed or spliced at which a potential splitting crack can develop (see Table 13.1.2.3)
$n_f$	= number of fitment bars within longitudinal spacing or pitch $s$ that a potential splitting crack has to cross (see Table 13.1.2.3)
$D_{Ed}$	= fatigue damage factor
$d_i$	= mandrel pin diameter for bending reinforcement
$f_{c,fat}$	= compressive strength for fatigue effects
$f'_c$	= characteristic compressive (cylinder) strength of concrete at 28 days;
$m$	= number of intervals with constant amplitude
$n_i$	= foreseen number of constant amplitude cycles in interval 'i'
$N_i$	= ultimate number of constant amplitude cycles in interval "i" that can be carried before failure
$k_d$	= $0.35 + 0.026 (d_i/d_b) \leq 1.0$
$n_{sc}$	= foreseen number of effective stress cycles during the required design service life
$N$	= number of resisting stress cycles, to be calculated from the fatigue strength
$N_{Rsk}$	= number of cycles for resistance of constant amplitude stress $\Delta\sigma_{Rsk}$
$Q_{fat}$	= foreseen load level expected for the design situation
$S_{cd,max}$	= maximum compressive stress level
$S_{cd,min}$	= minimum compressive stress level
$s$	= factor related to type of cement
$\beta_{sc}$	= factor for strength gain in concrete with time
$\gamma_p$	= factor for representative value of prestress
$\gamma_F$	= load factor for fatigue loading
$\sigma_{ct,max}$	= maximum tension stress
$\sigma_{c,max}$	= maximum compression stress
$\phi_{c,fat}$	= fatigue strength capacity reduction factor for concrete
$\phi_{s,fat}$	= fatigue strength capacity reduction factor for steel
$\eta_c$	= factor for stress gradient for concrete in the compression zone of a cracked section

- $\eta_s$  = factor which increases the stress in the reinforcing steel due to differences in bond behaviour between prestressing and reinforcing steel
- $\sigma_{c,1}$  = minimum compressive stress within a distance of 300 mm from the surface under the relevant load combination of actions
- $\sigma_{c,2}$  = maximum compressive stress within a distance of 300 mm from the surface under the same load combination as that for which  $\sigma_{c,1}$  was determined
- $\sigma_{c,max}$  = maximum compressive stress at the extreme fibre under consideration, compression measured positive,  $\gamma_F [G, P, Q_{fat}]$
- $\sigma_{c,min}$  = minimum compressive stress at the extreme fibre under consideration, taken as zero if tensile,  $\gamma_F [\{G, P, \psi_s Q\}, Q_{fat}]$
- $\Delta\sigma_s$  = maximum nominal steel stress range under the acting load combination for steel
- $\Delta\sigma_{Rsk}(n_{sc})$  = resisting stress range relevant to  $n_{sc}$  cycles obtained from a characteristic fatigue strength function

## SECTION 2 DESIGN PROCEDURES, ACTIONS AND LOADS

### 2.1 DESIGN PROCEDURES

#### 2.1.1 Design for strength and serviceability

Concrete structures shall be designed for ultimate strength and serviceability limit states in accordance with the general principles and procedures for design as set out in AS/NZS 1170.0 and the specific requirements of Clauses 2.2 and 2.3.

Notwithstanding the requirements of Clauses 2.2 and 2.3, it shall be permissible to carry out design checks for strength and serviceability by testing a structure or a component member in accordance with Appendix B.

#### 2.1.2 Design for earthquake actions

Where structures are required by AS 1170.4 to be designed for earthquake actions they shall conform with that Standard, this Standard and the provisions of Section 14 of this Standard. Reinforcement shall be detailed to provide the structure with the assumed ductility when determining the static earthquake load for the structure to be able to resist the remainder of the earthquake loading inelastically.

#### 2.1.3 Design for robustness and structural integrity

Concrete structures shall be designed to be robust in accordance with the procedures and criteria given in Section 6 of AS/NZS 1170.0.

In the detailing of reinforcement and connections for structural integrity, members of a structure shall be effectively tied together to improve integrity of the overall structure. For cast in place concrete, see Sections 8, 9, 10 and 11 for specific requirements. For specific requirements for prefabricated concrete structures, see Section 17.

#### 2.1.4 Design for durability and fire resistance

Concrete structures shall be designed to be—

- (a) durable in accordance with the procedures and criteria given in Section 4; and
- (b) fire resistant in accordance with the procedures and criteria given in Section 5.

#### 2.1.5 Design for fatigue

Fatigue shall be considered in the design of structures and structural elements subject to regular cyclic loads such as vibrating machines, crane-rails, heavy traffic areas, but need not be considered where the foreseen effective number of stress cycles  $n_{sc}$  is less than 10 000.

Design actions for use in fatigue analysis shall be determined in accordance with the methods specified in Clause 6.1.3, except that the moment redistribution for elastic method of Clause 6.2.7 and plastic method of Clause 6.7 are not permitted. Non-linear analysis is permitted; however, sections shall not undergo large plastic deformations under fatigue load combinations.

NOTE: Linear elastic models may generally be used, and reinforced concrete in tension is considered to be cracked. The ratio of moduli of elasticity for steel and concrete may be taken as  $E_s/E_c = 10$ .



## 2.1.6 Material properties

The properties of materials used in the design shall be in accordance with Section 3.

When evaluating the behaviour of a concrete structure, member or cross-section, the values of concrete properties used in the calculation shall be appropriate to the age of the concrete, rate of loading and expected variations of material properties.

## 2.2 DESIGN FOR STRENGTH

### 2.2.1 General

Strength checks for concrete structures and their component members shall be carried out using the procedures specified in Clauses 2.2.2 to 2.2.6, and methods of structural analysis specified in Section 6, as appropriate to the strength check procedures being used.

It shall be permissible to use different strength check procedures for different members in a structure, and for the structure as a whole, provided it can be shown that all external actions and forces and calculated internal stress resultants are consistent with the requirements of equilibrium and compatibility for the entire structure.

### 2.2.2 Strength check procedure for use with linear elastic methods of analysis, with simplified analysis methods and for statically determinate structures

The strength check procedure for use in conjunction with—

- (a) linear elastic methods of analysis of indeterminate structures and members;
- (b) simplified methods of analysis of indeterminate structures and members; and
- (c) static analysis of determinate structures,

shall be carried out as follows:

- (i) It shall be confirmed that the design capacity is equal to or greater than the design action effect, for all critical cross-sections and regions—

$$R_d \geq E_d \quad \dots 2.2.2$$

where

$R_d$  = design capacity (equal to  $\phi R_u$ )

$E_d$  = design action effect

- (ii) The design capacity,  $R_d = \phi R_u$ , shall be obtained using the appropriate capacity reduction factor ( $\phi$ ), given in Table 2.2.2, and the ultimate strength ( $R_u$ ), determined in accordance with the relevant sections of this Standard using characteristic values for the material strengths.
- (iii) The design action effect ( $E_d$ ), shall be determined for the critical combination of factored actions specified in AS/NZS 1170.0 and Clause 2.4 by one of the following methods of analysis:
  - (A) Linear elastic analysis in accordance with Clause 6.2.
  - (B) Linear elastic analysis incorporating secondary bending moments due to lateral joint displacement in accordance with Clause 6.3.
  - (C) One of the simplified methods of analysis in accordance with Clauses 6.9 and 6.10.
  - (D) Equilibrium analysis of a statically determinate structure.

**TABLE 2.2.2**  
**CAPACITY REDUCTION FACTORS ( $\phi$ )**

Type of action effect	Capacity reduction factor ( $\phi$ )
(a) Axial force without bending:	
(i) Tension	
(A) members with Class N reinforcement and/or tendons	0.85
(B) members with Class L reinforcement	0.65
(ii) Compression	0.65
(b) Bending without axial tension or compression—	
(i) for members with Class N reinforcement and/or tendons	$0.65 \leq (1.24 - 13k_{uo}/12) \leq 0.85$
(ii) for members with Class L reinforcement	0.65
(c) Bending with axial tension—	
(i) for members with Class N reinforcement and/or tendons	$\phi + [(0.85 - \phi) (N_u/N_{uot})]$ and $\phi$ is obtained from Item (b)(i)
(ii) for members with Class L reinforcement	0.65
(d) Bending with axial compression, where—	
(i) $N_u \geq N_{ub}$	$\phi_o$
(ii) $N_u < N_{ub}$	$\phi_o + [(\phi - \phi_o) (1 - N_u/N_{ub})]$ and $\phi$ is obtained from Item (b)
	Short columns with $Q/G \geq 0.25$ , $\phi_o = 0.65$ ; otherwise, $\phi_o = 0.60$
(e) Shear and Torsion	
(i) for members where Class N fitments are provided meeting the requirements of Clause 8.2.1.7	0.75
(ii) otherwise	0.7
(f) Torsion	0.7
(g) Bearing	0.6
(h) Bending, shear and compression in plain concrete	0.6
(i) Bending, shear and tension in fixings	0.6
(j) Bending, shear and axial force in singly reinforced walls forming part of a primary lateral load resisting system	0.65

NOTE: In members where Class L reinforcement together with Class N reinforcement and/or tendons are used as longitudinal tensile reinforcement in the design for strength in bending, with or without axial force, the maximum value of  $\phi$  for calculating the member design strength should be taken as 0.65.

### 2.2.3 Strength check procedure for use with linear elastic stress analysis

The strength check procedure for use with a linear elastic stress analysis of a structure or member shall be made as follows:

- (a) The structure or member shall be analysed for the critical combination of factored actions, as specified in AS/NZS 1170.0 and Clause 2.4, by linear stress analysis, in accordance with Clause 6.4, assuming the concrete to be uncracked, and using accepted principles of mechanics.

- (b) The calculated principal compressive stresses shall not exceed the following value:

$$\phi_s \beta 0.9 f'_c \quad \dots 2.2.3$$

where

$\phi_s$  = stress reduction factor with values taken from Table 2.2.3

$\beta$  = an effective compressive strength factor, to be evaluated as follows:

- (i) In regions not containing effective confining reinforcement—  
 $\beta = 1.0$  when the principal tensile stress does not exceed  $f'_{ct}$ , otherwise  
 $\beta = 0.6$
- (ii) In regions where effective confining reinforcement is provided,  $\beta$  shall be evaluated by rational calculation taking account of the amount of confining steel and the details used, but shall not exceed two.
- (c) Reinforcement and/or tendons shall be provided to carry all of the internal tensile forces, with stresses not exceeding  $\phi_s f_{sy}$  and  $\phi_s f_{py}$  respectively, where values for the stress reduction factor ( $\phi_s$ ) are in accordance with Table 2.2.3.
- (d) In determining the areas of steel reinforcement, it shall be permissible to reduce the peak stresses by averaging the stresses over an area appropriate to the size of the member.
- (e) The stress development of the reinforcement and tendons shall be determined in accordance with Clauses 13.1 and 13.3 respectively.

**TABLE 2.2.3**  
**STRESS REDUCTION FACTORS ( $\phi_s$ )**

Material	Stress reduction factor ( $\phi_s$ )
Concrete in compression	0.65
Steel in tension	
Class N	0.85
Class L	0.65
Tendons	0.85

#### 2.2.4 Strength check procedure for use with strut-and-tie analysis

The strength check procedure for use with strut-and-tie analysis shall be carried out as follows:

- (a) The strut-and-tie model shall satisfy the requirements of Section 7.
- (b) The forces acting on all struts and ties and nodes shall be determined for the critical combination of factored actions as specified in AS/NZS 1170.0 and Clause 2.4 by an analysis of the strut-and-tie model in accordance with Section 7.
- (c) The compressive force in any concrete strut shall not exceed the design strength of that strut determined in accordance with Clause 7.2.3. The strength reduction factor ( $\phi_{st}$ ) to be used in determining the design strength shall be in accordance with Table 2.2.4.
- (d) The tensile force in any tie shall not exceed the design strength of the tie determined in accordance with Clause 7.3.2 where the strength reduction factor ( $\phi_{st}$ ) is given in Table 2.2.4.
- (e) The reinforcement and/or tendons in the ties shall be anchored in accordance with Clause 7.3.3.

- (f) The design strength of nodes shall be calculated in accordance with Clause 7.4.2 and shall not be exceeded. The strength reduction factor ( $\phi_{st}$ ) shall be in accordance with Table 2.2.4.
- (g) Tie reinforcement shall be provided by Class N reinforcement or tendons.

**TABLE 2.2.4**  
**STRENGTH REDUCTION FACTORS ( $\phi_{st}$ ) FOR**  
**DESIGN USING STRUT-AND-TIE ANALYSIS**

Material	Strength reduction factor ( $\phi_{st}$ )
Concrete in compression	0.65
Steel in tension	0.85

### 2.2.5 Strength check procedure for use with non-linear analysis of framed structures

The strength check procedure for use with non-linear analysis of framed structures at collapse shall be carried out as follows:

- (a) It shall be confirmed that the design capacity of the structure is equal to or greater than the design action effect—

$$R_d \geq E_d$$

... 2.2.5

where

$R_d$  = design capacity of the structure

$E_d$  = design action effect

- (b) The design action effect ( $E_d$ ) is the critical combination of factored actions as specified in AS/NZS 1170.0 and Clause 2.4.
- (c) The design capacity of the structure ( $R_d = \phi_{sys} R_{u,sys}$ ) shall be obtained using the appropriate system strength reduction factor ( $\phi_{sys}$ ), given in Table 2.2.5, and the mean capacity of the structure ( $R_{u,sys}$ ) determined for the same combination of actions adopted in Item (b) to evaluate  $E_d$ , by using non-linear frame analysis as specified in Clause 6.5, with mean values of material properties.

**TABLE 2.2.5**  
**SYSTEM STRENGTH REDUCTION FACTORS ( $\phi_{sys}$ )**  
**(For application with Clauses 2.2.5 and 2.2.6)**

Type of failure	System strength reduction factor ( $\phi_{sys}$ )
For structural systems in which the deflections and local deformations at high overload are an order of magnitude greater than those for service conditions; and yielding of the reinforcement and/or the tendon occurs well before the peak load is reached	0.7
In all other cases	0.5 (see Note)

NOTE: Larger values than 0.5 may be used if it can be shown that, at high overload, adequate warning is given of impending collapse.

### 2.2.6 Strength check procedure for use with non-linear stress analysis

The strength check procedure for use with non-linear stress analysis at collapse shall be carried out as follows:

- (a) It shall be confirmed that the design capacity of the structure or the component member is equal to or greater than the design action effect—

$$R_d \geq E_d$$

... 2.2.6

where

$R_d$  = design capacity of the structure or component

$E_d$  = design action effect on the structure or the design action effects for a component

- (b) The design action effect ( $E_d$ ) shall be the critical combination of factored actions (or action effects) as specified in AS/NZS 1170.0 and Clause 2.4.
- (c) The design capacity of the structure (or component) ( $R_d = \phi_{\text{sys}} R_{\text{u,sys}}$ ) shall be obtained using the appropriate system strength reduction factor ( $\phi_{\text{sys}}$ ) given in Table 2.2.5, and the mean capacity of the structure (or component) ( $R_{\text{u,sys}}$ ) which shall be determined for the same combination of actions adopted for  $E_d$ , by non-linear stress analysis as specified in Clause 6.6, with mean values of material properties.

## 2.3 DESIGN FOR SERVICEABILITY

### 2.3.1 General

Design checks shall be carried out for all appropriate service conditions to ensure the structure will perform in a manner appropriate for its intended function and purpose.

NOTE: Design limits given or implied in Clauses 2.3.2 and 2.3.3 are based on previous design experience, and reflect requirements for normal structures. In special situations other limits may be appropriate. For further guidance refer to Appendix C of AS/NZS 1170.0.

### 2.3.2 Deflection

The deflection of beams and slabs under service conditions shall be controlled as follows:

- (a) A limit for the calculated deflection of the member shall be chosen and shall be appropriate to the structure and its intended use. The chosen value shall be not greater than the value calculated from the appropriate deflection-to-span ratio given in Table 2.3.2.
- (b) The member shall be designed so that, under the design load for serviceability, the deflections, determined either by calculation or controlled by limiting the span-to-depth ratios in accordance with Clause 8.5 for beams and Clause 9.3 for slabs, do not exceed the deflection limit.

For unbraced frames and multistorey buildings subject to lateral loading, an appropriate limit for the inter-storey lateral drift shall be chosen, which does not exceed 1/500 of the storey height. The structure shall be designed so that, under the design lateral load for serviceability, the calculated inter-storey lateral drift does not exceed the chosen value.

**TABLE 2.3.2**  
**LIMITS FOR CALCULATED VERTICAL DEFLECTIONS**  
**OF BEAMS AND SLABS**

Type of member	Deflection to be considered	Deflection limitation ( $\Delta/L_{ef}$ ) for spans (Notes 1 and 2)	Deflection limitation ( $\Delta/L_{ef}$ ) for cantilevers (Note 4)
All members	The total deflection	1/250	1/125
Members supporting masonry partitions	The deflection that occurs after the addition or attachment of the partitions	1/500 where provision is made to minimize the effect of movement, otherwise 1/1000	1/250 where provision is made to minimize the effect of movement, otherwise 1/500
Members supporting other brittle finishes	The deflection that occurs after the addition or attachment of the finish	Manufacturer's specification but not more than 1/500	Manufacturer's specification but not more than 1/250
Members subjected to vehicular or pedestrian traffic	The imposed action (live load and dynamic impact) deflection	1/800	1/400
Transfer members	Total deflection	1/500 where provision is made to minimize the effect of deflection of the transfer member on the supported structure, otherwise 1/1000	1/250

**NOTES:**

- 1 In general, deflection limits should be applied to all spanning directions. This includes, but is not limited to, each individual member and the diagonal spans across each design panel. For flat slabs with uniform loadings, only the column strip deflections in each direction need be checked.
- 2 If the location of masonry partitions or other brittle finishes is known and fixed, these deflection limits need only be applied to the length of the member supporting them. Otherwise, the more general requirements of Note 1 should be followed.
- 3 Deflection limits given may not safeguard against ponding.
- 4 For cantilevers, the values of  $\Delta/L_{ef}$  given in this table apply only if the rotation at the support is included in the calculation of  $\Delta$ .
- 5 Consideration should be given by the designer to the cumulative effect of deflections, and this should be taken into account when selecting a deflection limit.
- 6 When checking the deflections of transfer members and structures, allowance should be made in the design of the supported members and structure for the deflection of the supporting members. This will normally involve allowance for settling supports and may require continuous bottom reinforcement at settling columns.

## **2.3.3 Cracking**

### **2.3.3.1 General**

Cracking in concrete structures shall be controlled so that structural performance, durability and appearance of the structure are not compromised.

### **2.3.3.2 Control of cracking**

The requirements for cracking set out in Clause 2.3.3.1 shall be deemed to be satisfied by designing the structure and members to conform to the following requirements:

- (a) Flexural cracking in concrete beams and slabs under service conditions shall be controlled in accordance with Clauses 8.6, 9.5.1, 9.5.2, 9.5.4 or 9.5.5, as appropriate.
- (b) Cracking caused by shrinkage and temperature in concrete slabs shall be controlled in accordance with Clause 9.5.3.

- (c) Cracking in concrete walls under service conditions shall be controlled in accordance with Clause 11.7.2.
- (d) Cracking in D-regions under service conditions shall be controlled in accordance with Clause 12.7.
- (e) Pre-hardening cracking shall be controlled by appropriate specification and construction measures so that the durability, serviceability and/or the behaviour of the structure or member is not adversely affected.

### 2.3.4 Vibration

Vibration in concrete structures and members shall be controlled so that the serviceability and structural performance are not adversely affected.

## 2.4 DESIGN FOR FATIGUE

The fatigue strength check procedure shall be undertaken as outlined in Clause 2.1.5 and the following:

- (a) It shall be confirmed that the design fatigue capacity is equal to or greater than the foreseen number of cyclic design action effect, for all critical cross-sections and regions—

$$\text{For } E_d \leq R_d \quad \dots 2.4(1)$$

$$n_{sc} \leq N \quad \dots 2.4(2)$$

where

$E_d$  = design action effect

$R_d$  = design capacity

$n_{sc}$  = foreseen number of cyclic design actions

$N$  = number of resisting stress cycles

- (b) The design capacity,  $R_d = \phi_{fat} R_{fat}$ , shall be obtained using the appropriate capacity reduction factor ( $\phi_{fat}$ ), given in Table 2.4.

The fatigue strength ( $R_{fat}$ ) is determined in accordance with the relevant sections of this Standard using the characteristic values of material strength.

- (c) The design action effect ( $E_d$ ), shall be determined for the critical combination of factored actions specified in AS/NZS 1170.0 and Clause 2.4 by one of methods as specified in Clause 2.1.5.

The fatigue strength in any concrete shall not exceed the design fatigue strength for  $n_{sc}$  effective stress cycles determined in accordance with Section 18. The fatigue strength reduction factor for fatigue ( $\phi_{c,fat}$ ) to be used shall be in accordance with Table 2.4.

The tensile stress range in any steel shall not exceed the design fatigue strength for  $n_{sc}$  effective stress cycles determined in accordance with Section 18. The fatigue strength reduction factor ( $\phi_{s,fat}$ ) is given in Table 2.4.

**TABLE 2.4**  
**STRENGTH REDUCTION FACTORS**  
**FOR FATIGUE**

Material	Strength reduction factor
Concrete ( $\phi_{s,fat}$ )	0.65
Steel ( $\phi_{s,fat}$ )	0.85

## 2.5 ACTIONS AND COMBINATIONS OF ACTIONS

### 2.5.1 Actions and loads

The minimum actions and loads used in the design shall be those set out in AS/NZS 1170.0.

### 2.5.2 Combinations of actions and loads

#### 2.5.2.1 General

The combinations of actions, loads and forces used in the design shall be in accordance with AS/NZS 1170.0. Additional combinations for prestressed concrete and for fatigue are given in Clauses 2.5.2.2 and 2.5.2.3, respectively.

#### 2.5.2.2 Additional combinations for prestressed members

Where applicable, the prestressing effect shall be included with a load factor of unity in all load combinations for both ultimate and serviceability design except for the following:

- (a) For the case of permanent action plus prestressing force at transfer, when the more severe of—

$$1.15G + 1.15P; \text{ and}$$

$$0.9G + 1.15P$$

shall be used.

NOTE: See also Clause 6.2.6.

- (b) For the case where the vertical component of the prestressing force at a section,  $P_v$ , is greater than the minimum design shear action  $V_{\min}^*$ , the following design action shall be considered—

$$1.2P_v - V_{\min}^*$$

where  $V_{\min}^*$  = minimum design shear action calculated for all load combinations.

NOTE: See also Clause 8.2.1.3.

#### 2.5.2.3 Actions and loads combinations for fatigue

The basic combinations for the fatigue limit states used in checking the stresses or the stress range in the structural materials in this section are calculated in accordance with the load combination of actions shown in Table 2.5.2.3(A), where  $\psi_s$  is the short-term combination factor given in AS 1170.0.

**TABLE 2.5.2.3(A)**  
**FATIGUE LOAD COMBINATIONS**

The maximum design stress range in the steel $\Delta\sigma$	$E_d = \gamma_F[Q_{\text{fat}}]$
The maximum and minimum concrete design compressive stress $\sigma_{\text{max}}, \sigma_{\text{min}}$	$E_d = \gamma_F[\{G, \gamma_P P, \psi_s Q\}, Q_{\text{fat}}]$
The maximum design tensile stress in plain concrete $\sigma_{\text{max}}$	$E_d = \gamma_F[\{G, \gamma_P P, \psi_s Q\}, Q_{\text{fat}}]$

The combinations in the brackets  $\{G, \gamma_P P, \psi_s Q\}$  represent the most adverse combination of permanent and non-cyclic service level actions acting with the fatigue design action  $Q_{\text{fat}}$ . The fatigue design action  $Q_{\text{fat}}$  to be used shall be the load level determined for the design situation.

The representative values factor for prestress  $\gamma_P$  for the fatigue load combinations shall be determined in accordance with Table 2.5.2.3(B).



**TABLE 2.5.2.3(B)**  
**REPRESENTATIVE VALUES FOR PRESTRESS**

<b>Tendon description</b>	<b>Increases safety</b>	<b>Decreases safety</b>
Pre-tensioning or unbonded tendons	0.95	1.05
Post-tensioning with bonded tendons	0.90	1.10
When appropriate measures are taken (e.g. direct measurements of pre-tensioning)	1.0	1.0

The load factor  $\gamma_F$  for the fatigue load combinations shall be taken as 1.1. If the stress analysis is sufficiently accurate or conservative, and can be verified by in situ observations,  $\gamma_F$  may be reduced to  $\gamma_F = 1.0$ .

### 2.5.3 Construction effects

In determining the critical design conditions for strength and serviceability, account shall be taken of the conditions during construction, and in particular—

- (a) the construction sequence;
- (b) the influence of the schedule for stripping of formwork; and
- (c) the method of back-propping, and its effect on the loads applied during construction.

### 2.5.4 Arrangements of vertical loads on continuous beams, frames and floor systems

When design checks are carried out for continuous beams and continuous floor systems, for two-dimensional framed structures and for three-dimensional framed structures and floor systems, alternative arrangements of the vertical loads shall be considered in order to determine the critical load combinations. Variations in the load intensity on individual spans shall be considered, including partial loading as specified in AS/NZS 1170.1, together with variations in the loading patterns, whereby some spans are loaded and others unloaded.

The loading arrangements to be considered shall include at least the following:

- (a) The factored permanent action (dead load), without variations in pattern.
- (b) For factored imposed action (live load), where the pattern of loaded and unloaded spans is fixed, the full factored imposed action (live load) applied in the specified pattern.
- (c) For imposed action (live load), where the pattern of loaded and unloaded spans is variable, as follows:
  - (i) For continuous beams and two-dimensional frames and floor systems—
    - (A) the factored imposed action (live load) on alternate spans;
    - (B) the factored imposed action (live load) on any two adjacent spans; and
    - (C) the factored imposed action (live load) on all spans.
  - (ii) For three-dimensional framed structures and floor systems, patterned variations of the factored imposed action (live load) shall be applied in chequerboard arrangements, including the principles of Items (A), (B) and (C) of Item (i) above, to determine the peak design action effects at each critical section.
  - (iii) Notwithstanding Items (i) and (ii), for beams and slabs at the strength limit state, for which the imposed action (live load) ( $Q$ ) is less than three-quarters of the permanent action (dead load) ( $G$ ), the factored imposed action (live load) on all spans.

NOTE: The load arrangements listed are the minimum to be considered for design. In particular, for deflection or vibration-sensitive structures and slender floor systems, additional load arrangements need to be considered.

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## SECTION 3 DESIGN PROPERTIES OF MATERIALS

### 3.1 PROPERTIES OF CONCRETE

#### 3.1.1 Strength

##### 3.1.1.1 Characteristic compressive strength

The characteristic compressive strength of concrete at 28 days ( $f'_c$ ) shall be either—

- (a) taken as equal to the specified strength grade, provided the appropriate curing is ensured and that the concrete complies with AS 1379; or
- (b) determined statistically from compressive strength tests carried out in accordance with AS 1012.9.

The characteristic compressive strengths of the standard strength grades are 20 MPa, 25 MPa, 32 MPa, 40 MPa, 50 MPa, 65 MPa, 80 MPa and 100 MPa.

##### 3.1.1.2 Mean in situ compressive strength

In the absence of more accurate data, the mean value of the in situ compressive strength ( $f_{cmi}$ ) shall be taken as 90% of the mean value of the cylinder strength ( $f_{cm}$ ) or shall be taken as those given in Table 3.1.2.

##### 3.1.1.3 Tensile strength

The uniaxial tensile strength ( $f_{ct}$ ) is the maximum stress that concrete can withstand when subjected to uniaxial tension.

The uniaxial tensile strength shall be determined from either the measured flexural tensile strength ( $f_{ct,f}$ ) or from the measured splitting tensile strength ( $f_{ct,sp}$ ) using—

$$f_{ct} = 0.6f_{ct,f} \quad \text{or} \quad f_{ct} = 0.9f_{ct,sp}$$

where  $f_{ct,f}$  and  $f_{ct,sp}$  are determined statistically from—

- (a) flexural strength tests carried out in accordance with AS 1012.11; or
- (b) indirect tensile strength tests carried out in accordance with AS 1012.10, respectively.

In the absence of more accurate data, the characteristic flexural tensile strength of concrete ( $f'_{ct,f}$ ) and the characteristic uniaxial tensile strength of concrete ( $f'_{ct}$ ) shall be taken as—

$$f'_{ct,f} = 0.6\sqrt{f'_c} \quad \text{and} \quad f'_{ct} = 0.36\sqrt{f'_c} \quad \text{at 28 days and standard curing,}$$

and where the mean and upper characteristic values are obtained by multiplying these values by 1.4 and 1.8, respectively.

#### 3.1.2 Modulus of elasticity

The mean modulus of elasticity of concrete at the appropriate age ( $E_{cj}$ ) shall be either—

- (a) taken as equal to—
  - (i)  $(\rho^{1.5}) \times (0.043\sqrt{f_{cmi}})$  (in megapascals) when  $f_{cmi} \leq 40$  MPa; or
  - (ii)  $(\rho^{1.5}) \times (0.024\sqrt{f_{cmi}} + 0.12)$  (in megapascals) when  $f_{cmi} > 40$  MPa,
 consideration being given to the fact that this value has a range of  $\pm 20\%$ ;

- (b) determined by test in accordance with AS 1012.17; and
- (c) for Standard strength grades at 28 days determined from Table 3.1.2.

**TABLE 3.1.2**  
**CONCRETE PROPERTIES AT 28 DAYS**

$f'_c$ (MPa)	20	25	32	40	50	65	80	100
$f_{cmi}$ (MPa)	22	28	35	43	53	68	82	99
$E_c$ (MPa)	24 000	26 700	30 100	32 800	34 800	37 400	39 600	42 200

### 3.1.3 Density

The density of concrete ( $\rho$ ) shall be determined by test in accordance with either AS 1012.12.1 or AS 1012.12.2. For normal-weight concrete, the density may be taken as 2400 kg/m<sup>3</sup>.

### 3.1.4 Stress-strain curves

The stress-strain curve for concrete shall be either—

- (a) assumed to be of curvilinear form defined by recognized simplified equations; or
- (b) determined from suitable test data.

For design purposes, the shape of the in situ uniaxial compressive stress-strain curve shall be modified so that the maximum stress is  $0.9f'_c$ .

### 3.1.5 Poisson's ratio

Poisson's ratio for concrete ( $\nu$ ) shall be either—

- (a) taken as equal to 0.2; or
- (b) determined by test in accordance with AS 1012.17.

### 3.1.6 Coefficient of thermal expansion

The coefficient of thermal expansion of concrete shall be either—

- (a) taken as equal to  $10 \times 10^{-6}/^{\circ}\text{C}$ , consideration being given to the fact that this value has a range of  $\pm 20\%$ ; or
- (b) determined from suitable test data.

### 3.1.7 Shrinkage

#### 3.1.7.1 Calculation of design shrinkage strain

The design shrinkage strain of concrete ( $\epsilon_{cs}$ ) shall be determined—

- (a) from measurements on similar local concrete;
- (b) by tests after eight weeks of drying modified for long-term value, in accordance with AS 1012.13; or
- (c) by calculation in accordance with Clause 3.1.7.2.

#### 3.1.7.2 Design shrinkage strain

When the design shrinkage strain of concrete ( $\epsilon_{cs}$ ) is to be calculated, it shall be determined as the sum of the chemical (autogenous) shrinkage strain ( $\epsilon_{cse}$ ) and the drying shrinkage strain ( $\epsilon_{csd}$ )—

$$\epsilon_{cs} = \epsilon_{cse} + \epsilon_{csd} \quad \dots 3.1.7.2(1)$$

The autogenous shrinkage strain shall be taken as—

$$\varepsilon_{cse} = \varepsilon_{cse}^* \times (1.0 - e^{-0.07t}) \quad \dots 3.1.7.2(2)$$

where  $t$  is the time (in days) after setting and  $\varepsilon_{cse}^*$  is the final autogenous shrinkage strain given by—

$$\text{For } f'_c > 50 \text{ MPa } \varepsilon_{cse}^* = (0.07f'_c - 0.5) \times 50 \times 10^{-6} \quad \dots 3.1.7.2(3)$$

$$\text{For } f'_c > 50 \text{ MPa } \varepsilon_{cse}^* = (0.08f'_c - 1.0) \times 50 \times 10^{-6}$$

At any time  $t$  (in days) after the commencement of drying, the drying shrinkage strain shall be taken as—

$$\varepsilon_{csd} = k_1 k_4 \varepsilon_{csd,b} \quad \dots 3.1.7.2(4)$$

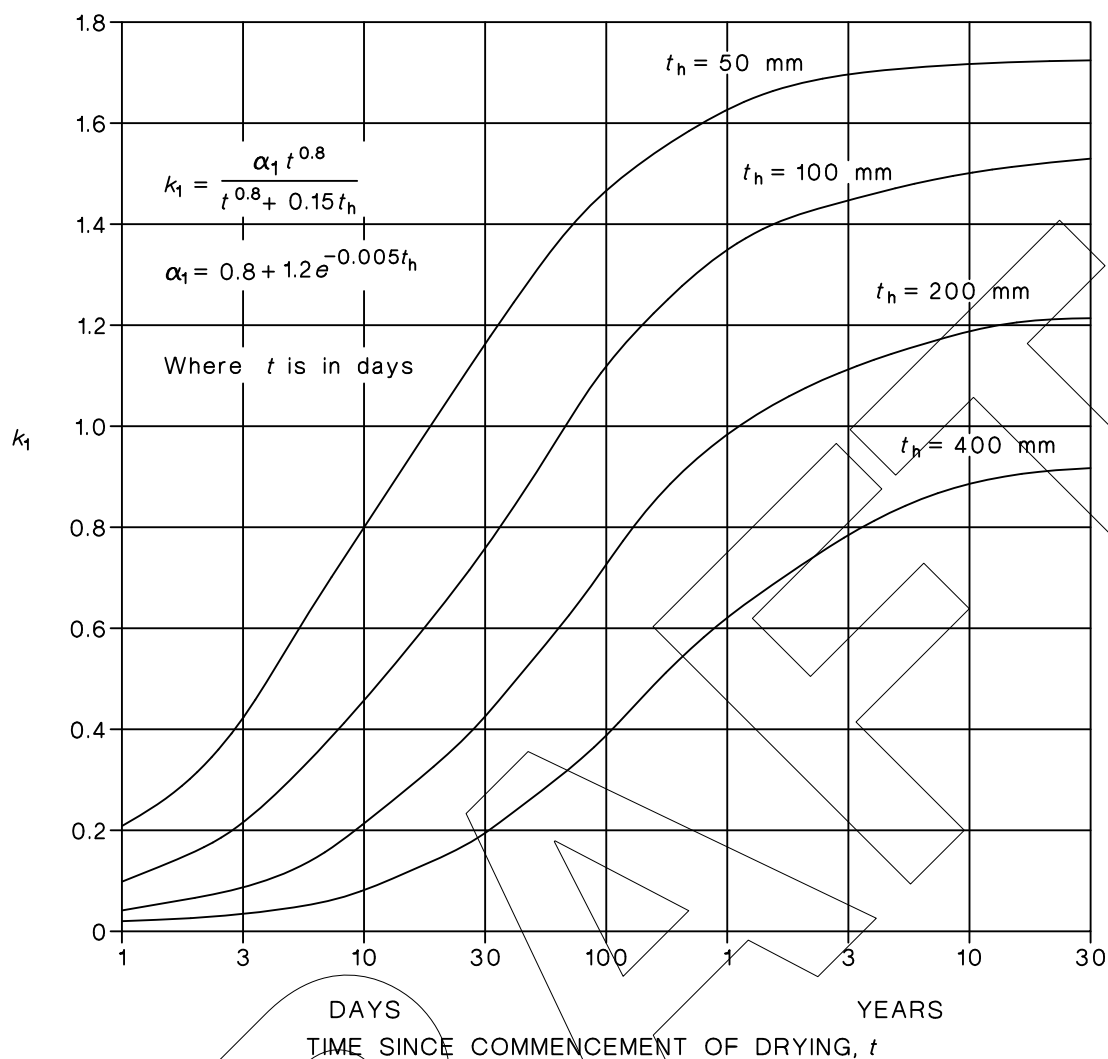
and  $k_1$  is obtained from Figure 3.1.7.2 and  $k_4$  is equal to 0.7 for an arid environment, 0.65 for an interior environment, 0.6 for a temperate inland environment and 0.5 for a tropical or near-coastal environment.

The basic drying shrinkage strain ( $\varepsilon_{csd,b}$ ) is given by—

$$\varepsilon_{csd,b} = (0.9 - 0.005f'_c) \times 800 \times 10^{-6} \quad \dots 3.1.7.2(5)$$

NOTE: Concrete exposed to early drying undergoes shrinkage due to capillary suction. This can result in cracking and poor service performance, particularly of exposed slabs. The amount of shrinkage from suction depends on the ambient conditions and the concrete mix, and can exceed the combined shrinkage from other causes. Therefore, it is important to prevent excessive drying of concrete between the commencement of casting and the application of curing at the completion of finishing.

Consideration shall be given to the fact that  $\varepsilon_{cs}$  has a range of  $\pm 30\%$ .

FIGURE 3.1.7.2 SHRINKAGE STRAIN COEFFICIENT ( $k_1$ ) FOR VARIOUS VALUES OF  $t_h$ 

**TABLE 3.1.7.2**  
**TYPICAL FINAL DESIGN SHRINKAGE STRAINS AFTER 30 YEARS**

$f'_c$ (MPa)	Final design shrinkage strain $\varepsilon_{cs}^*$ ( $\times 10^{-6}$ )															
	Arid environment				Interior environment				Temperate inland environment				Tropical, near-coastal and coastal environment			
	$t_h$ (mm)				$t_h$ (mm)				$t_h$ (mm)				$t_h$ (mm)			
	50	100	200	400	50	100	200	400	50	100	200	400	50	100	200	400
25	810	720	590	470	760	670	550	440	710	630	520	410	600	530	440	350
32	800	720	590	470	750	670	560	440	700	630	520	420	600	540	450	360
40	790	710	590	480	740	670	560	450	700	620	530	430	600	540	460	380
50	780	700	590	490	730	660	560	460	690	620	530	440	600	540	470	390
65	770	700	600	510	730	660	570	490	690	630	550	470	610	560	490	420
80	750	690	610	530	720	660	590	510	680	630	560	490	620	570	510	460
100	740	690	620	560	710	660	600	540	680	640	580	530	630	590	540	500

### 3.1.8 Creep

#### 3.1.8.1 General

The creep strain at any time ( $t$ ) caused by a constant sustained stress ( $\sigma_o$ ) shall be calculated from—

$$\varepsilon_{cc} = \varphi_{cc} \sigma_o / E_c \quad \dots 3.1.8.1$$

where

$E_c$  = mean modulus of elasticity of the concrete at 28 days

$\varphi_{cc}$  = design creep coefficient at time ( $t$ ) determined in accordance with Clause 3.1.8.3

#### 3.1.8.2 Basic creep coefficient

The basic creep coefficient of concrete ( $\varphi_{cc,b}$ ) is the mean value of the ratio of final creep strain to elastic strain for a specimen loaded at 28 days under a constant stress of  $0.4f'_c$  and shall be—

- determined from measurements on similar local concrete; or
- determined by tests in accordance with AS 1012.16; or
- taken as the value given in Table 3.1.8.2.

**TABLE 3.1.8.2**  
**BASIC CREEP COEFFICIENT**

Characteristic strength ( $f'_c$ ), MPa	20	25	32	40	50	65	80	100
Basic creep coefficient ( $\varphi_{cc,b}$ )	5.2	4.2	3.4	2.8	2.4	2.0	1.7	1.5

#### 3.1.8.3 Design creep coefficient

The design creep coefficient for concrete at any time,  $t$ , ( $\varphi_{cc}$ ) shall be determined from the basic creep coefficient ( $\varphi_{cc,b}$ ) by any accepted mathematical model for creep behaviour, calibrated such that  $\varphi_{cc,b}$  is also predicted by the chosen model.

In the absence of more accurate methods,  $\varphi_{cc}$  at any time shall be taken as—

$$\varphi_{cc} = k_2 k_3 k_4 k_5 k_6 \varphi_{cc,b} \quad \dots 3.1.8.3$$

where  $k_2$  is obtained from Figure 3.1.8.3 and  $k_3$  depends on the age of the concrete ( $\tau$ ) at the time of loading (in days) and is given by the following:

$$k_3 = 2.7/[1 + \log(\tau)] \text{ for } \tau \geq 1 \text{ day}$$

$$k_4 = 0.70 \text{ for an arid environment, } 0.65 \text{ for an interior environment, } 0.60 \text{ for a temperate inland environment and } 0.50 \text{ for a tropical or near-coastal environment}$$

$$k_5 = \text{a modification factor for high strength concrete, which shall be taken as—}$$

$$k_5 = 1.0$$

when  $f'_c \leq 50$  MPa; or

$$k_5 = (2.0 - \alpha_3) - 0.02(1.0 - \alpha_3) f'_c \quad \text{when } 50 \text{ MPa} < f'_c \leq 100 \text{ MPa}$$

the factor  $\alpha_3 = 0.7/(k_4 \alpha_2)$ ; and  $\alpha_2$  is defined in Figure 3.1.8.3

$k_6$  accounts for the non-linear creep that develops at sustained stress levels in excess of  $0.45f_{cmi}$ :

$$k_6 = 1.0$$

when  $\sigma_o \leq 0.45 f_{cmi}$

$$k_6 = e^{1.5 \left( \frac{\sigma_o}{f_{cmi}} - 0.45 \right)}$$

when  $\sigma_o > 0.45 f_{cmi}$

Consideration shall be given to the fact that  $\phi_{cc}$  has a range of approximately  $\pm 30\%$ . This range is likely to be exceeded if the concrete member is subjected to prolonged periods of temperature in excess of  $25^\circ\text{C}$ .

The final design creep coefficients ( $\phi_{cc}^*$ ) (after 30 years) predicted by this method for concrete first loaded at 28 days with a sustained stress level not exceeding  $0.45 f_{cmi}$  are given in Table 3.1.8.3.

NOTE: The compressive stress in concrete caused by the permanent effects (including prestress) should not exceed  $0.45 f_{cmi}$ .

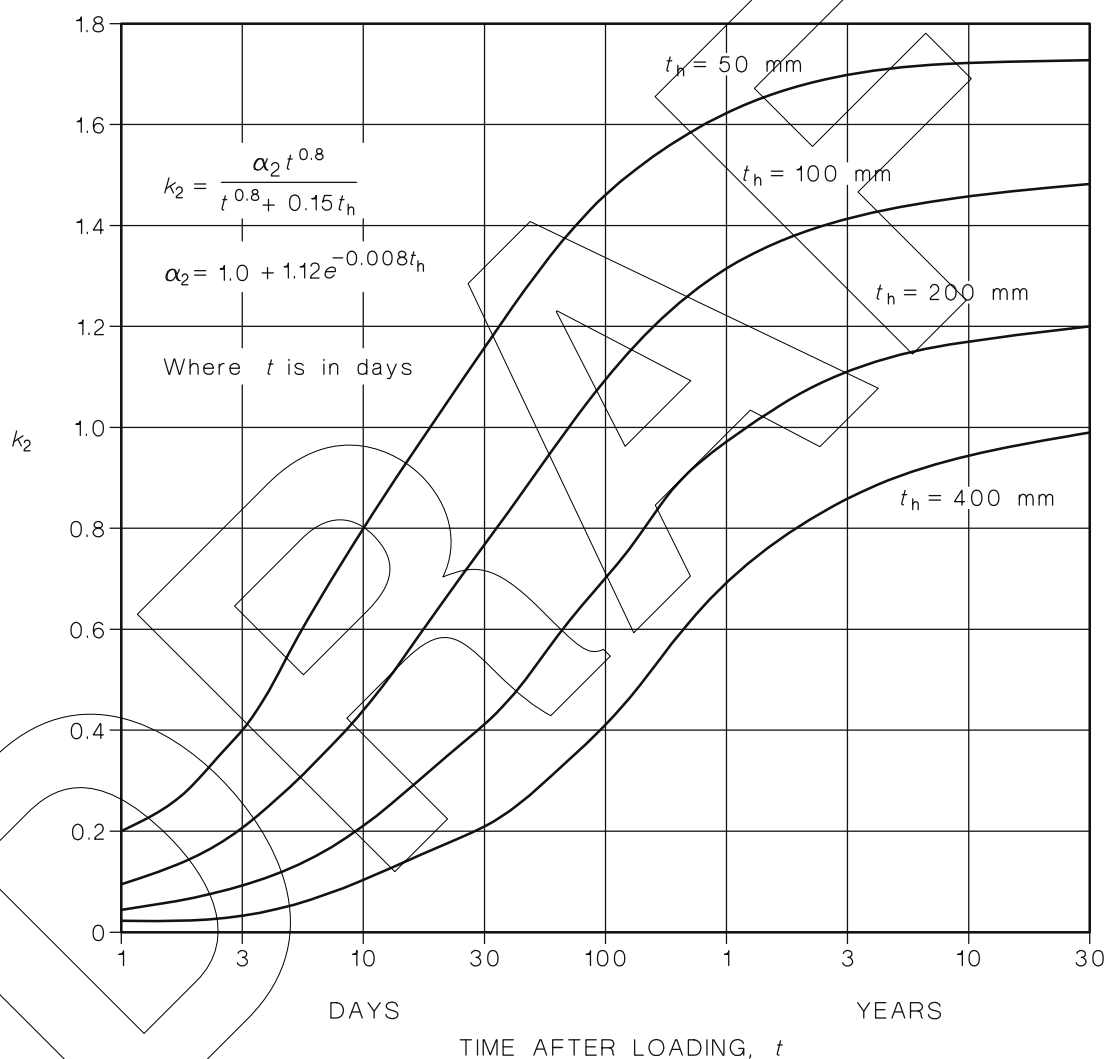


FIGURE 3.1.8.3 COEFFICIENT ( $k_2$ )



**TABLE 3.1.8.3**  
**FINAL CREEP COEFFICIENTS (AFTER 30 YEARS) FOR CONCRETE**  
**FIRST LOADED AT 28 DAYS**

$f'_c$ (MPa)	Final creep coefficient ( $\phi_{cc}^*$ )											
	Arid environment			Interior environment			Temperate inland environment			Tropical, near-coastal and coastal environment		
	$t_h$ (mm)			$t_h$ (mm)			$t_h$ (mm)			$t_h$ (mm)		
	100	200	400	100	200	400	100	200	400	100	200	400
25	4.82	3.90	3.27	4.48	3.62	3.03	4.13	3.34	2.80	3.44	2.78	2.33
32	3.90	3.15	2.64	3.62	2.93	2.46	3.34	2.70	2.27	2.79	2.25	1.90
40	3.21	2.60	2.18	2.98	2.41	2.02	2.75	2.23	1.87	2.30	1.86	1.56
50	2.75	2.23	1.89	2.56	2.07	1.73	2.36	1.91	1.60	1.97	1.59	1.33
65	2.07	1.75	1.53	1.95	1.66	1.46	1.84	1.59	1.38	1.61	1.38	1.23
80	1.56	1.40	1.29	1.50	1.36	1.25	1.45	1.32	1.22	1.33	1.23	1.14
100	1.15	1.14	1.11	1.15	1.14	1.11	1.15	1.14	1.11	1.15	1.14	1.11

## 3.2 PROPERTIES OF REINFORCEMENT

### 3.2.1 Strength and ductility

For the purposes of design, the characteristic yield strength of reinforcement ( $f_{sy}$ ) shall be taken as not greater than the value specified in Table 3.2.1 for the appropriate type of reinforcement (see also Clause 17.2.1.1).

The ductility of the reinforcement shall be characterized by its uniform strain ( $\epsilon_{su}$ ) and tensile-to-yield stress ratio and designated as low (L) or normal (N) Ductility Class as given in Table 3.2.1. For the purposes of design, values of these parameters for each Ductility Class shall conform with AS/NZS 4671.

NOTE: In AS/NZS 4671,  $\epsilon_{su}$  is referred to as  $A_{gt}$ , expressed as a percentage, and  $f_{sy}$  is referred to as  $R_e$ .

**TABLE 3.2.1**  
**YIELD STRENGTH AND DUCTILITY CLASS OF REINFORCEMENT**

Reinforcement		Characteristic yield strength ( $f_{sy}$ ) MPa	Characteristic uniform strain ( $\epsilon_{su}$ )	Ductility Class
Type	Designation grade			
Bar plain to AS/NZS 4671	R250N	250	0.05	N
Bar deformed to AS/NZS 4671	D500L (fitments only)	500	0.015	L
	D500N	500	0.05	N
Welded mesh, plain, deformed or indented to AS/NZS 4671	D500L	500	0.015	L
	D500N	500	0.05	N

**NOTES:**

- 1 Reference should be made to AS/NZS 4671 for explanation to designations applying to 500 MPa steels.
- 2 For higher reinforcing steel grades permitted in Clause 1.1.2(d) the following characteristic properties shall be met:
  - (a) The following limits for the chemical composition determined by cast analysis shall not be exceeded:
    - (i) Carbon – 0.33%, Phosphorus – 0.050%, Sulphur – 0.050%.
    - (ii) The carbon equivalent value shall not exceed 0.49.
  - (b) The maximum yield strength does not exceed the nominal yield strength by more than 150 MPa.
  - (c) For steels:
    - (i) >500 MPa – 700 MPa: uniform elongation  $\epsilon_{su} \geq 0.05$  and the tensile-to-yield stress ratio  $R_m/R_e \geq 1.08$ .
    - (ii) >700 MPa – 800 MPa: uniform elongation  $\epsilon_{su} \geq 0.04$  and the tensile-to-yield stress ratio  $R_m/R_e \geq 1.04$ .

### 3.2.2 Modulus of elasticity

The modulus of elasticity of reinforcement ( $E_s$ ) for all stress values not greater than the yield strength ( $f_{sy}$ ) shall be either—

- (a) taken as equal to  $200 \times 10^3$  MPa; or
- (b) determined by test.

### 3.2.3 Stress-strain curves

A stress-strain curve for reinforcement shall be either—

- (a) assumed to be of a form defined by recognized simplified equations; or
- (b) determined from suitable test data.

### 3.2.4 Coefficient of thermal expansion

The coefficient of thermal expansion of reinforcement shall be either—

- (a) taken as equal to  $12 \times 10^{-6}/^\circ\text{C}$ ; or
- (b) determined from suitable test data.

### 3.3 PROPERTIES OF TENDONS

#### 3.3.1 Strength

The following applies:

- (a) The characteristic minimum breaking strength ( $f_{pb}$ ) for commonly used tendons shall be as specified in Table 3.3.1. For tendons of dimensions not covered in Clause 3.3, refer to AS/NZS 4672.1.
- (b) The yield strength of tendons ( $f_{py}$ ) shall be taken either as the 0.1% proof stress as specified in AS/NZS 4672.1, or determined by test data. In the absence of test data it shall be taken as follows:
  - (i) For wire used in the as-drawn condition .....  $0.80f_{pb}$ .
  - (ii) For stress-relieved wire .....  $0.83f_{pb}$ .
  - (iii) For all grades of strand .....  $0.82f_{pb}$ .
  - (iv) For hot-rolled bars (super grade) .....  $0.81f_{pb}$ .
  - (v) For hot-rolled ribbed bars .....  $0.89f_{pb}$ .

**TABLE 3.3.1**  
**TENSILE STRENGTH OF COMMONLY USED WIRE STRAND AND BAR**

Material type and Standard	Nominal diameter	Area	Characteristic minimum breaking load	Characteristic minimum breaking strength ( $f_{pb}$ )
	mm	mm <sup>2</sup>	kN	MPa
As-drawn wire, AS/NZS 4672.1	5.0	19.6	34.7	1700
	7.0	38.5	64.3	1670
Stress-relieved wire, AS/NZS 4672.1	5.0	19.9	33.8	1700
	7.0	38.5	64.3	1670
7 wire ordinary strand, AS/NZS 4672.1	9.5	55.0	102	1850
	12.7	98.6	184	1870
	15.2	143	250	1790
	15.2	143	261	1830
	15.7	150	279	1860
7 wire compacted strand, AS/NZS 4672.1	15.2	165	300	1820
	18.0	223	380	1700
Hot-rolled bars, AS/NZS 4672.1 (Super grade only)	26	562	579	1030
	29	693	714	1030
	32	840	865	1030
	36	995	1025	1030
	40	1232	1269	1030
	56	2428	2501	1030
	75	4371	4502	1030

#### 3.3.2 Modulus of elasticity

The modulus of elasticity of commonly used tendons ( $E_p$ ) shall be either—

- (a) taken as equal to—
  - (i) for as-drawn wire, to AS/NZS 4672.1 .....  $205 \pm 10 \times 10^3$  MPa;
  - (ii) for stress-relieved steel wire, to AS/NZS 4672.1 .....  $205 \pm 10 \times 10^3$  MPa;
  - (iii) for stress-relieved steel strand, to AS/NZS 4672.1 .....  $200 \pm 5 \times 10^3$  MPa; or

(iv) for hot-rolled high tensile alloy steel bars, to AS/NZS 4672.1

.....205 ±10 × 10<sup>3</sup> MPa; or

(b) determined by test.

NOTE: Consideration should be given to the fact that the modulus of elasticity of tendons may vary by ±10% and will vary more when a multi-strand or multi-wire tendon is stressed as a single cable. This will influence the calculated extension.

### 3.3.3 Stress-strain curves

A stress-strain curve for tendons shall be determined from appropriate test data.

### 3.3.4 Relaxation of tendons

#### 3.3.4.1 General

This clause applies to the relaxation, at any age and stress level, of low-relaxation wire, low-relaxation strand, and alloy-steel bars.

#### 3.3.4.2 Basic relaxation

The basic relaxation of a tendon ( $R_b$ ) after one thousand hours at 20°C with an initial force of  $0.8f_{pb}$  for wire and strand and  $0.7f_{pb}$  for hot rolled steel bars shall be determined in accordance with AS/NZS 4672.1.

The basic relaxation of a tendon ( $R_b$ ) after one thousand hours at 20°C and  $0.8f_{pb}$  shall be determined in accordance with AS/NZS 4672.1.

#### 3.3.4.3 Design relaxation

The design relaxation of a tendon ( $R$ ) shall be determined from—

$$R = k_4 k_5 k_6 R_b \quad \dots 3.3.4.3$$

where

$k_4$  = a coefficient dependent on the duration of the prestressing force

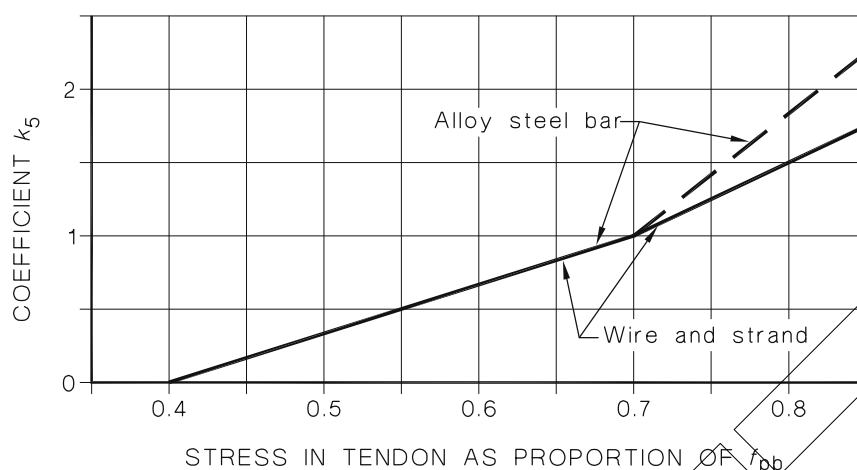
$$= \log [5.4(j)^{1/6}]$$

$j$  = time after prestressing, in days

$k_5$  = a coefficient, dependent on the stress in the tendon as a proportion of  $f_{pb}$ , determined from Figure 3.3.4.3

$k_6$  = a function, dependent on the average annual temperature ( $T$ ) in degrees Celsius, taken as  $T/20$  but not less than 1.0

When determining the design relaxation, consideration shall be given to the effects of curing at elevated temperatures, if applicable.

FIGURE 3.3.4.3 COEFFICIENT  $k_5$ 

### 3.4 LOSS OF PRESTRESS IN TENDONS

#### 3.4.1 General

The loss of prestress in tendons, at any given time, shall be taken to be the sum of the immediate loss of prestress and the time-dependent loss of prestress, calculated in accordance with Clauses 3.4.2 and 3.4.3 respectively.

For structures designed to operate above 40°C, special calculations, based on appropriate test data, shall be made.

NOTE: Tendons in structural members subject to elevated temperatures permanently or in a high temperature environment will have significantly higher losses of prestress. Reference to specialist literature is recommended to establish the effect of elevated temperatures on the behaviour of the materials.

#### 3.4.2 Immediate loss of prestress

##### 3.4.2.1 General

The immediate loss of prestress shall be estimated by adding the calculated losses of prestress due to elastic deformation of concrete, friction, anchoring and other immediate losses as are applicable.

##### 3.4.2.2 Loss of prestress due to curing conditions

Where curing of a prestressed member is carried out at ambient conditions, the design relaxation shall be as determined by Clause 3.3.4.3.

Where curing of a prestressed member is carried out at elevated temperature (such as steam curing), a part or all of the design relaxation, as determined from Clause 3.3.4.3, shall be deemed to be part of the immediate loss of prestress.

##### 3.4.2.3 Loss of prestress due to elastic deformation of concrete

Calculation of the immediate loss of prestress due to elastic deformation of the concrete at transfer shall be based on the value of modulus of elasticity of the concrete at that age.

##### 3.4.2.4 Loss of prestress due to friction

The stress variation along the design profile of a tendon due to friction in the jack, the anchorage and the duct shall be assessed in order to obtain an estimate of the prestressing forces at the critical sections considered in the design.

The extension of the tendon shall be calculated allowing for the variation in tension along its length, as follows:

- (a) *Friction in the jack and anchorage* The loss of prestress due to friction in the jack and anchorage shall be determined for the type of jack and anchorage system to be used.
- (b) *Friction along the tendon* Friction loss shall be calculated from an analysis of the forces exerted by the tendon on the duct. In the absence of more detailed calculations the stress in the tendon at a distance ( $a$ ) measured from the jacking end ( $\sigma_{pa}$ ) shall be taken as—

$$\sigma_{pa} = \sigma_{pj} e^{-\mu(\alpha_{tot} + \beta_p L_{pa})}$$

... 3.4.2.4

where

$\sigma_{pj}$  = stress in the tendon at the jacking end

$e$  = base of Napierian logarithms

$\mu$  = friction curvature coefficient for different conditions, which, in the absence of specific data and when all tendons in contact in the one duct are stressed simultaneously, shall be taken as—

- (i) for greased-and-wrapped coating, 0.15;
- (ii) for bright and zinc-coated metal sheathing, 0.15 to 0.20;
- (iii) for bright and zinc-coated flat metal ducts, 0.20.

$\alpha_{tot}$  = sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length of the tendon from the jacking end to a point at distance ( $a$ ) from that end ( $L_{pa}$ )

$\beta_p$  = an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects, which, as a first approximation, shall be taken as—

- (A) for sheathing containing tendons other than bars and having an internal diameter—
  - (1)  $\leq 50$  mm: 0.024 to 0.016 rad/m;
  - (2)  $> 50$  mm but  $\leq 90$  mm: 0.016 to 0.012 rad/m;
  - (3)  $> 90$  mm but  $\leq 140$  mm: 0.012 to 0.008 rad/m.
- (B) for flat metal ducts containing tendons other than bars: 0.024 rad/m to 0.016 rad/m;
- (C) for sheathing containing bars and having an internal diameter of 50 mm or less: 0.016 rad/m to 0.008 rad/m; and
- (D) for bars of any diameter in a greased-and-wrapped coating: 0.008 rad/m.

$L_{pa}$  = length of the tendon from the jacking end to a point at a distance ( $a$ ) from that end

The magnitude of the friction due to duct curvature and wobble used in the design shall be verified during the stressing operation.

### 3.4.2.5 Loss of prestress during anchoring

In a post-tensioned member, allowance shall be made for loss of prestress when the prestressing force is transferred from the tensioning equipment to the anchorage. This allowance shall be checked on the site and any correspondingly required adjustment shall be made.

### 3.4.2.6 Loss of prestress due to other considerations

Where applicable, loss of prestress due to the following shall be taken into account in design:

- (a) Deformation of the forms for precast members.
- (b) Differences in temperature between stressed tendons and the actual stressed structures during heat treatment of the concrete.
- (c) Changes in temperature between the time of stressing the tendons and the time of casting concrete.
- (d) Deformations in the construction joints of precast structures assembled in sections.
- (e) Permanently elevated temperatures in excess of 40°C.

## 3.4.3 Time-dependent losses of prestress

### 3.4.3.1 General

The total time-dependent loss of prestress shall be estimated by adding the calculated losses of prestress due to shrinkage of the concrete, creep of the concrete, tendon relaxation, and other considerations as are applicable.

### 3.4.3.2 Loss of prestress due to shrinkage of the concrete

The loss of stress in the tendon due to shrinkage of the concrete shall be taken as  $E_p \epsilon_{cs}$ , modified to allow for the effects of reinforcement, provided the shrinkage restraint effects of the reinforcement are included in the serviceability design of the member, where  $\epsilon_{cs}$  is determined in accordance with Clause 3.1.7.2.

Where reinforcement is distributed throughout the member so that its effect on shrinkage is mainly axial, the loss of prestress in the tendons may be taken as:

$$\frac{E_p \epsilon_{cs}}{1 + 15 A_s / A_g} \quad \dots 3.4.3.2$$

### 3.4.3.3 Loss of prestress due to creep of the concrete

The loss of prestress due to creep of the concrete shall be calculated from an analysis of the creep strains in the concrete. In the absence of more detailed calculations and provided the sustained stress in the concrete at the level of the tendons at no time exceeds  $0.5 f'_c$ , the loss of stress in the tendon due to creep of the concrete may be taken as  $E_p \epsilon_{cc}$ , in which  $\epsilon_{cc}$  is given by—

$$\epsilon_{cc} = 0.8 \phi_{cc} (\sigma_{ci} / E_c) \quad \dots 3.4.3.3$$

where

$\phi_{cc}$  = design creep coefficient calculated in accordance with Clause 3.1.8.3

$\sigma_{ci}$  = sustained stress in the concrete at the level of the centroid of the tendons, calculated using the initial prestressing force prior to any time-dependent losses and the sustained portions of all the service loads

### 3.4.3.4 Loss of prestress due to tendon relaxation

The loss of stress in a tendon due to relaxation of the tendon in the member shall be determined by modifying the percentage loss of stress due to the design relaxation of the tendon ( $R$ ) to take into account the effects of shrinkage and creep.

In the absence of more detailed calculations, the percentage loss of stress in the tendon in the member shall be taken as—

$$R \left( 1 - \frac{\text{the loss of stress due to creep and shrinkage}}{\sigma_{pi}} \right) \quad \dots 3.4.3.4$$

where

$\sigma_{pi}$  = stress in the tendon immediately after transfer

#### **3.4.3.5** *Loss of prestress due to other considerations*

Account shall be taken, if applicable, of—

- (a) losses due to deformations in the joints of precast structures assembled in sections; and
- (b) losses due to the effects of any increase in creep caused by frequently repeated loads.

### **3.5 MATERIAL PROPERTIES FOR NON-LINEAR STRUCTURAL ANALYSIS**

Where the structure is to be analysed in design in accordance with Clauses 6.5 and 6.6, mean values of all relevant material properties shall be used in the form of the stress-strain curve for the material.



## SECTION 4 DESIGN FOR DURABILITY

### 4.1 GENERAL

The requirements of this section apply to plain, reinforced and prestressed concrete structures and members with a design life of 50 years  $\pm 20\%$ .

#### NOTES:

- 1 More stringent requirements would be appropriate for structures with a design life in excess of 50 years (e.g. monumental structures), while some relaxation of the requirements may be acceptable for structures with a design life of less than 50 years (e.g. temporary structures).
- 2 Durability is a complex topic and conformance with these requirements may not be sufficient to ensure a durable structure.

### 4.2 METHOD OF DESIGN FOR DURABILITY

Durability shall be allowed for in design by determining the exposure classification in accordance with Clause 4.3 and, for that exposure classification, conforming with the appropriate requirements for concrete quality and curing, in accordance with Clauses 4.4 and 4.5. In addition—

- (a) members subject to abrasion from traffic (e.g. pavements and floors) shall satisfy the requirements of Clause 4.6;
- (b) members subject to cycles of freezing and thawing shall satisfy the requirements of Clause 4.7;
- (c) members subject to aggressive soils shall satisfy Clause 4.8;
- (d) members susceptible to damage due to alkali aggregate reaction (AAR) shall be assessed and appropriate management measures shall be taken; and

NOTE: Guidance on appropriate management measures may be found in HB 79.

- (e) members containing reinforcement and/or tendons, the chemical content restrictions of the concrete shall be in accordance with Clause 4.9 and the cover to reinforcement and tendons shall be in accordance with Clause 4.10.

### 4.3 EXPOSURE CLASSIFICATION

#### 4.3.1 General

The following are applicable:

- (a) The exposure classification for a surface of a member shall be determined from Table 4.3 and Figure 4.3.
- (b) For determining concrete quality requirements in accordance with Clauses 4.4 to 4.8 as appropriate, the exposure classification for the member shall be taken as the most severe exposure of any of its surfaces.
- (c) For determining cover requirements for corrosion protection in accordance with Clause 4.10.3, the exposure classification shall be taken as the classification for the surface from which the cover is measured.
- (d) Members that do not contain reinforcement shall have an exposure classification of A1, unless the environment is aggressive to the concrete [see also Clause 4.2, Items (a), (b), (c) and (d)].

### 4.3.2 Concession for exterior exposure of a single surface

Where the exterior exposure is essentially only one surface of a member, concrete of the next lower grade than would otherwise be required by Clause 4.4 may be used, provided the cover from that surface is increased by—

- (a) 20 mm from the value required by Clause 4.10.3.2; or
- (b) 15 mm from the value required by Clause 4.10.3.3.

**TABLE 4.3**  
**EXPOSURE CLASSIFICATIONS**

Surface and exposure environment	Exposure classification reinforced or prestressed concrete members (see Note 1)
1 Surface of members in contact with the ground:	
(a) Members protected by a damp-proof membrane.	A1
(b) Residential footings in non-aggressive soils.	A1
(c) Other members in non-aggressive soils.	A2
(d) Members in aggressive soils:	
(i) Sulfate bearing (magnesium content <1g/L).	See Table 4.8.1
(ii) Sulfate bearing (magnesium content ≥1g/L) (see Note 2).	U
(iii) Other.	U
(e) Salt rich soils and soils in areas affected by salinity.	See Table 4.8.2
2 Surfaces of members in interior environments:	
(a) Fully enclosed within a building except for a brief period of weather exposure during construction:	
(i) Residential.	A1
(ii) Non-residential.	A2
(b) In industrial buildings, the member being subject to repeated wetting and drying.	B1
3 Surfaces of members in above-ground exterior environments in areas that are:	
(a) Inland (>50 km from coastline) environment being:	
(i) Non-industrial and arid climatic zone (see Note 3).	A1
(ii) Non-industrial and temperate climatic zone.	A2
(iii) Non-industrial and tropical climatic zone.	B1
(iv) Industrial (see Note 4) and any climatic zone.	B1
(b) Near-coastal (1 km to 50 km from coastline), any climatic zone.	B1
(c) Coastal (see Note 5) and any climatic zone.	B2
4 Surfaces of members in water:	
(a) In freshwater (continuously).	B1
(b) In soft or running water (see Note 6).	U

(continued)

**TABLE 4.3** (continued)

Surface and exposure environment	Exposure classification reinforced or prestressed concrete members (see Note 1)
5 Surfaces of maritime structures in sea water:	
(a) Permanently submerged.	B2
(b) In spray zone (see Note 7).	C1
(c) In tidal/splash zone (see Note 8).	C2
6 Surfaces of members in other environments, that is, any exposure environment not specified in Items 1 to 5 above (see Note 9)	U

## NOTES:

- 1 In this context, reinforced concrete includes any concrete containing metals that rely on the concrete for protection against environmental degradation. Plain concrete members containing metallic embedments should be treated as reinforced members when considering durability.
- 2 Severity of sulfate attack depends on the type of sulfate. For example, magnesium sulfate is more aggressive than sodium sulfate. The use of sulfate-resisting cement and concrete would be adequate for sodium sulfate conditions. For the magnesium sulfate conditions, specific consideration should be given to the cement and concrete that are likely to resist this type of sulfate. For magnesium sulfate soil conditions in which the concentration of magnesium is more than 1000 ppm, special consideration shall be given to the cement and concrete and other protective measures that will be required to resist this type of sulfate attack.
- 3 The climatic zones referred to are those given in Figure 4.3, which is based on the Bureau of Meteorology map, *Major seasonal rainfall zones of Australia*, Commonwealth of Australia, 2005.
- 4 Industrial refers to areas that are within 3 km of industries that discharge atmospheric pollutants.
- 5 For the purpose of this table, the coastal zone includes locations within 1 km of the shoreline of large expanses of saltwater. Where there are strong prevailing winds or vigorous surf, the distance should be increased beyond 1 km and higher levels of protection should be considered.
- 6 Exposure classification guidance based on Langelier saturation index and pH can be obtained from AS 3735.
- 7 The spray zone is the zone from 1 m above wave crest level.
- 8 The tidal/splash zone is the zone 1 m below lowest astronomical tide (LAT) and up to 1 m above highest astronomical tide (HAT) on vertical structures, and all exposed soffits of horizontal structures over the sea.
- 9 Further guidance on measures appropriate in exposure classification U may be obtained from AS 3735.
- 10 In this table, classifications A1, A2, B1, B2, C1 and C2 represent increasing degrees of severity of exposure, while classification U represents an exposure environment not specified in this table but for which a degree of severity of exposure should be appropriately assessed. Protective surface coatings may be taken into account in the assessment of the exposure classification.

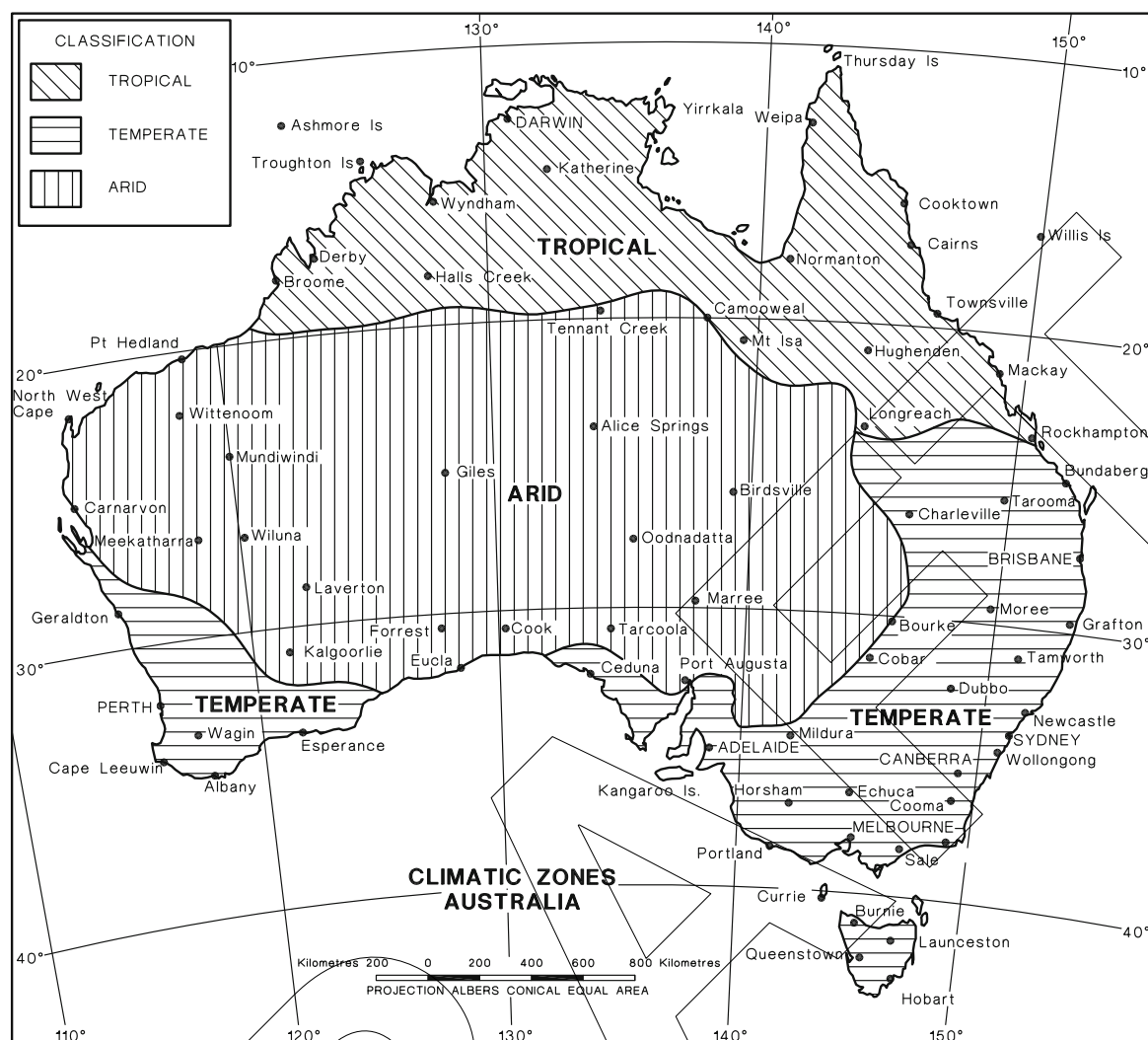


FIGURE 4.3 CLIMATIC ZONES REFERRED TO IN TABLE 4.3

#### 4.4 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATIONS A1, A2, B1, B2, C1 AND C2

Members subject to exposure classifications A1, A2, B1, B2, C1 and C2 shall have minimum  $f'_c$  as specified in Table 4.4, Column 2, and be cured as specified in Table 4.4, Column 3, or have a minimum average compressive strength of the concrete at the time of stripping of forms or removal from moulds as specified in Table 4.4, Column 4.

All concrete subject to exposure classifications B2, C1 or C2 shall be specified as special class as per AS 1379 and include in the specified parameters the exposure classification and any limitations on concrete quality.

**TABLE 4.4**  
**MINIMUM STRENGTH AND CURING REQUIREMENTS FOR CONCRETE**

Column 1	Column 2	Column 3	Column 4
Exposure classification	Minimum $f'_c$ (MPa)	Minimum initial curing requirement (see Clause 17.1.5.1)	Minimum average compressive strength at the time of stripping of forms or removal from moulds (MPa)
A1	20	Cure continuously for at least 3 days	15
A2	25		
B1	32	Cure continuously for at least 7 days	20
B2	40		25
C1	50		32
C2	50		

#### 4.5 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATION U

Members subject to exposure classification U shall have the concrete quality, cover to reinforcement/tendons, and other parameters specified, as appropriate, to ensure durability under the particular exposure environment.

#### 4.6 ABRASION

In addition to the other durability requirements of this section, concrete for members subject to abrasion from traffic shall have a characteristic compressive strength not less than the applicable value given in Table 4.6.

**TABLE 4.6**  
**STRENGTH REQUIREMENTS FOR ABRASION**

Member and/or traffic	Minimum characteristic compressive strength ( $f'_c$ ) MPa
Footpaths and residential driveways	20
Commercial and industrial floors not subject to vehicular traffic	25
Pavements or floors subject to:	
(a) Pneumatic-tyred traffic	32
(b) Non-pneumatic-tyred traffic	40
(c) Steel-wheeled traffic	To be assessed but not less than 40

NOTE:  $f'_c$  refers to the characteristic compressive strength of the wearing surface.

#### 4.7 FREEZING AND THAWING

In addition to the other durability requirements of this section, where the surface exposure includes exposure to cycles of freezing and thawing, concrete in the member shall—

- (a) have an  $f'_c$  not less than—
  - (i) 32 MPa for occasional exposure (<25 cycles p.a.); or
  - (ii) 40 MPa for frequent exposure (≥25 cycles p.a.); and

- (b) contain a percentage of entrained air not outside the following ranges—
- (i) for 10 mm to 20 mm nominal size aggregate..... 8% to 4%; or
  - (ii) for 40 mm nominal size aggregate .....6% to 3%,
- where the percentage of entrained air is determined in accordance with AS 1012.4.

## **4.8 AGGRESSIVE SOILS**

### **4.8.1 Sulfate and acid sulfate soils**

In addition to the other durability requirements of this section, where the surface exposure includes exposure to acid sulphate soils or to sulfate soils with a magnesium content of less than 1000 ppm, the exposure classification shall be as shown in Table 4.8.1.

**TABLE 4.8.1**  
**EXPOSURE CLASSIFICATION FOR CONCRETE IN SULFATE SOILS**

Exposure conditions			Exposure classification	
Sulfates (expressed as SO <sub>4</sub> )*		pH	Soil conditions A†	Soil conditions B‡
In soil ppm	In groundwater ppm			
<5000	<1000	>5.5	A2	A1
5000–10 000	1000–3000	4.5–5.5	B1	A2
10 000–20 000	3000–10 000	4–4.5	B2	B1
>20 000	>10 000	<4	C2	B2

\* Approximately 100 ppm SO<sub>4</sub> = 80 ppm SO<sub>3</sub>.

† Soil conditions A—high permeability soils (e.g. sands and gravels) which are in groundwater.

‡ Soil conditions B—low permeability soils (e.g. silts and clays) or all soils above groundwater.

**NOTES:**

- 1 This is a simplistic and sometimes conservative approach to the definition of aggressivity. It is common to find more than one chemical in the service environment and the effect of these chemicals may be modified in the presence of others. For example, sulfate ions become aggressive at levels of 600 ppm to 1000 ppm when combined with magnesium or ammonium ions. In the presence of chloride ions, however, attack by sulfate ions generally exhibits little disruptive expansion with the exception of conditions of wetting and extreme drying where crystallization can cause surface fretting of concrete.
- 2 Chemical concentrations relate only to the proportion of chemical present that is water-soluble.
- 3 Where exposure classifications 'B1', 'B2', 'C1' or 'C2' are indicated, it is recommended that the cement be Type SR.
- 4 Where exposure classifications 'B2', 'C1' or 'C2' are in acid sulfate soil conditions, it is recommended that a protective coating is used on the concrete surface.
- 5 Where a protective coating is used it may be possible to reduce the minimum required reinforcement cover to 50 mm.
- 6 Acidic ground conditions can be caused by dissolved 'aggressive' carbon dioxide, pure and very soft waters, organic and mineral acids and bacterial activity. Care is required in assessment of pH underground structure and lifetime conditions since pH can change over the lifetime of the member. Therefore, the pH should not be assessed only on the basis of a present-day test result, rather the ground chemistry should be considered over the design life of the ground structure. Testing for pH should be carried out either in situ or immediately after sampling as there is otherwise a risk of oxidation with time, leading to apparent acidity, which does not correctly represent in situ conditions.
- 7 pH alone may be a misleading measure of aggressivity without a full analysis of causes (e.g. still versus running water).
- 8 Contamination by the tipping of mineral and domestic wastes or by spillage from mining, processing or manufacturing industries presents special durability risks due to the presence of certain aggressive acids, salts and solvents, which can either chemically attack concrete or lead to a corrosion risk. Certain ground conditions cannot be properly addressed by reference only to Table 4.8.1. These conditions include, for example, areas where acid-sulfate soils exist, contamination by industrial and domestic waste, or spillage from mining, processing or manufacturing industries. In the absence of site-specific chemical information, the exposure condition should be assessed as 'exposure classification B2' for domestic refuse and 'exposure classification C2' for industrial/mining waste tips. Chemical analysis of the latter may, however, allow a lower risk classification.

The minimum reinforcement cover shall be the greater of the values given in Tables 4.8.2, 4.10.3.2 and 4.10.3.3, as varied by Clause 4.10.3.5, for the appropriate exposure classification, placement conditions and characteristic strength of concrete.

#### 4.8.2 Saline soils

Where the surface exposure includes exposure to saline soils, the minimum requirement shall be as shown in Table 4.8.2.

**TABLE 4.8.2**  
**STRENGTH AND COVER REQUIREMENTS FOR SALINE SOILS**

Soil electrical conductivity (EC <sub>e</sub> )	Exposure classification	Minimum $f'_c$	Minimum cover (mm)
4–8	A2	25	45
8–16	B1	32	50
>16	B2	40	55

**NOTES:**

- 1 EC<sub>e</sub> is saturated electrical conductivity in deciSiemens per metre.
- 2 Guidance on concrete in saline environments can be found in CCAA T56.

#### 4.9 RESTRICTIONS ON CHEMICAL CONTENT IN CONCRETE

Certain chemical constituents (e.g. chlorides) can have deleterious effects on the durability of concrete. For this reason, chemical admixtures added to concrete to be used in structures or members designed in accordance with this Standard shall conform with AS 1478.1 and chemical content in concrete shall conform with AS 1379.

#### 4.10 REQUIREMENTS FOR COVER TO REINFORCING STEEL AND TENDONS

##### 4.10.1 General

The cover to reinforcing steel and tendons shall be the greatest of the values determined from Clauses 4.8, 4.10.2 and 4.10.3, as appropriate, unless exceeded by the requirements for fire resistance given in Section 5.

##### 4.10.2 Cover for concrete placement

Designers shall specify appropriate covers to ensure the concrete can be satisfactorily placed and compacted around the reinforcement, tendons or ducts, or any combination of these, in accordance with the requirements of Clause 17.1.3.

In the determination of an appropriate cover, consideration shall be given to—

- (a) the size and shape of the member;
- (b) the size, type and configuration of the reinforcement and, if present, the tendons or ducts; and
- (c) the aggregate size, the workability of the concrete and the direction of concrete placement.

Where the presence of ducts is not a consideration, covers to reinforcement or tendons greater than their nominal size or the maximum nominal aggregate size, whichever is larger, shall be deemed to satisfy the requirements of the first two paragraphs of the clause.



### 4.10.3 Cover for corrosion protection

#### 4.10.3.1 General

For corrosion protection, the cover shall be not less than the appropriate value given in Clauses 4.10.3.2 to 4.10.3.7.

#### 4.10.3.2 Standard formwork and compaction

Where concrete is cast in formwork conforming with AS 3610 and compacted in accordance with Clause 17.1.3 of this Standard, the cover shall be not less than the value given in Tables 4.8.1, 4.8.2 and 4.10.3.2, as appropriate to the exposure classification and  $f'_c$ .

**TABLE 4.10.3.2**  
**REQUIRED COVER WHERE STANDARD FORMWORK**  
**AND COMPACTION ARE USED**

Exposure classification	Required cover, mm				
	Characteristic strength ( $f'_c$ )				
	20 MPa	25 MPa	32 MPa	40 MPa	≥ 50 MPa
A1	20	20	20	20	20
A2	(50)	30	25	20	20
B1	—	(60)	40	30	25
B2	—	—	(65)	45	35
C1	—	—	—	(70)	50
C2	—	—	—	—	65

NOTE: Bracketed figures are the appropriate covers when the concession given in Clause 4.3.2, relating to the strength grade permitted for a particular exposure classification, is applied.

#### 4.10.3.3 Required cover where repetitive procedures or intense compaction are used in rigid formwork

Where concrete members are cast in rigid formwork under repetitive procedures, with demonstrated process control systems including supervision, the cover shall be not less than the value given in Tables 4.8.1, 4.8.2 and 4.10.3.3, as appropriate to the exposure classification and  $f'_c$ . Cover to screeded surfaces of members shall be in accordance with Table 4.10.3.2.

**TABLE 4.10.3.3**  
**REQUIRED COVER WHERE REPETITIVE PROCEDURES**  
**AND INTENSE COMPACTION OR SELF-COMPACTING**  
**CONCRETE ARE USED IN RIGID FORMWORK**

Exposure classification	Required cover, mm				
	Characteristic strength ( $f'_c$ )				
	20 MPa	25 MPa	32 MPa	40 MPa	≥ 50 MPa
A1	20	20	20	20	20
A2	(45)	30	20	20	20
B1	—	(45)	30	25	20
B2	—	—	(50)	35	25
C1	—	—	—	(60)	45
C2	—	—	—	—	60

NOTE: Bracketed figures are the appropriate covers when the concession given in Clause 4.3.2, relating to the strength grade permitted for a particular exposure classification, is applied.

#### **4.10.3.4** *Required cover where self-compacting concrete is used*

Where concrete members are cast with self-compacting concrete, the cover shall be not less than the value given in Tables 4.8.1, 4.8.2 and 4.10.3.3. Screeded surfaces of members shall be in accordance with Table 4.10.3.2.

#### **4.10.3.5** *Cast against ground*

Where concrete is cast on or against the ground and compacted in accordance with Clause 17.1.3, the cover to a surface in contact with the ground shall be as given in Table 4.10.3.2 but increased by—

- (a) 10 mm if the concrete surface is protected by a damp-proof membrane; or
- (b) 20 mm otherwise.

#### **4.10.3.6** *Structural members manufactured by spinning or rolling*

Where structural members are manufactured by spinning and rolling concrete, the cover for corrosion protection shall be as specified in the appropriate Standard, where an equivalent exposure classification (as identified in Clause 4.3.1 of this Standard) for the required design life can be demonstrated.

#### **4.10.3.7** *Embedded items cover*

Embedded items, as defined in Clause 14.2, shall be protected from corrosion or deterioration. The cover to embedded items that are not corrosion resistant shall be as given in Table 4.10.3.2 and Table 4.10.3.3, as applicable.

Metals such as aluminium shall not be embedded in structural concrete unless effectively coated, covered, or treated to prevent chemical action between the metal and the concrete and electrolytic action between the metal and steel.

## SECTION 5 DESIGN FOR FIRE RESISTANCE

### 5.1 SCOPE

This section specifies the requirements for reinforced and prestressed concrete members used in buildings to meet the fire resistance levels (FRLs) required by the Building Code of Australia (BCA).

### 5.2 DEFINITIONS

For the purpose of this section, the definitions below apply.

#### 5.2.1 Average axis distance

When reinforcement is arranged in several layers as shown in Figure 5.2.1, and where it consists of either reinforcement or prestressing tendons with the same characteristic strength  $f_{sy}$  and  $f_{py}$  respectively, the average axis distance ( $a_m$ ) may be determined by—

$$a_m = \frac{A_{s1}a_1 + A_{s2}a_2 + \dots + A_{sn}a_n}{A_{s1} + A_{s2} + \dots + A_{sn}} = \frac{\sum A_{si}a_i}{\sum A_{si}} \quad \dots 5.2.1$$

where

$A_{si}$  = cross-sectional area of steel bar (tendon, wire) 'i'

$a_i$  = axis distance of steel bar (tendon, wire) 'i' from the nearest exposed surface

When reinforcement consists of steels with different characteristic strength,  $A_{si}$  should be replaced by  $A_{si}f_{syi}$  (or  $A_{si}f_{pyi}$ ) in the above equation.

Where reinforcement and prestressing tendons are used simultaneously (e.g. in a partially prestressed member), the axis distances of reinforcement and prestressing tendons shall be determined separately.

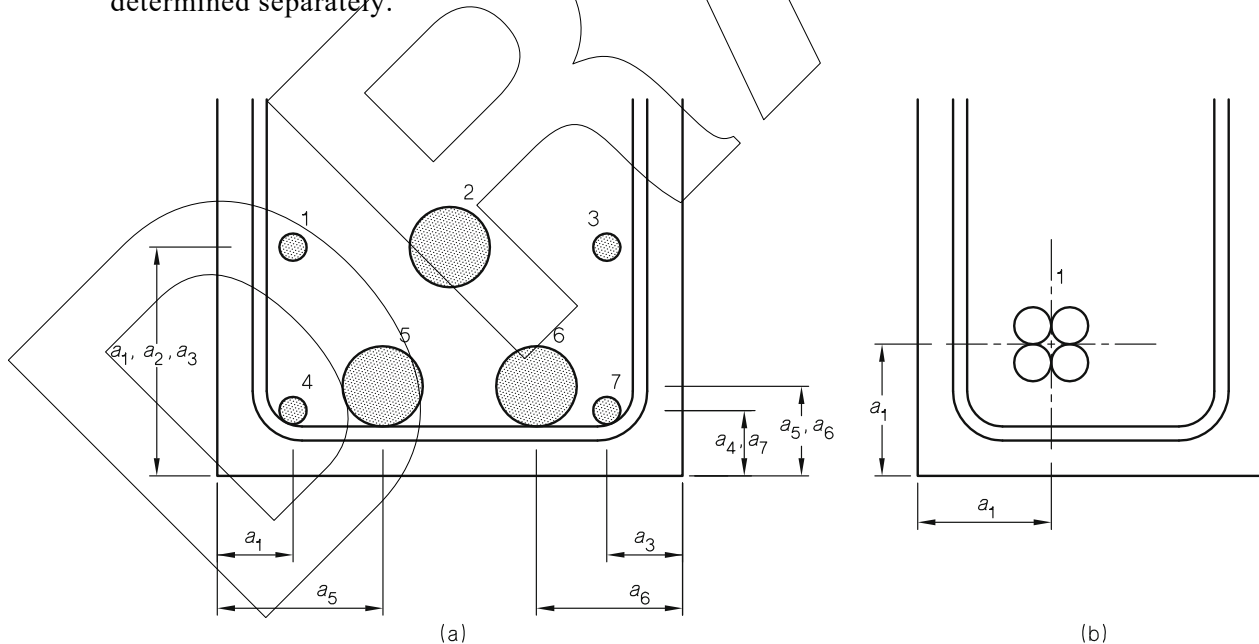
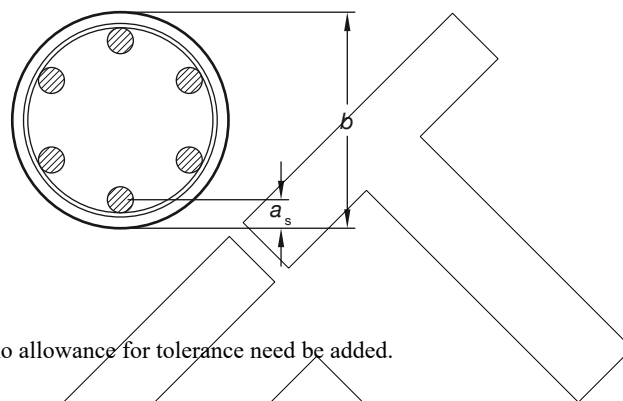
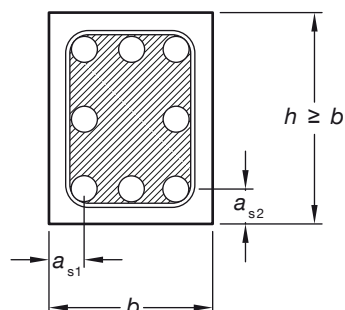


FIGURE 5.2.1 DIMENSIONS USED TO CALCULATE AVERAGE AXIS DISTANCE

### 5.2.2 Axis distance

Distance from the centre-line axis of a longitudinal bar or tendon to the nearest surface exposed to fire (see Figure 5.2.2).



NOTE: Axis distance ( $a_s$ ) is a nominal value and no allowance for tolerance need be added.

FIGURE 5.2.2 SECTIONS THROUGH STRUCTURAL MEMBERS SHOWING AXIS DISTANCE ( $a_s$ )

### 5.2.3 Fire resistance

Ability of a structure or part of it to fulfil its required functions (loadbearing and/or separating function) for a specified fire exposure, for a specified time.

### 5.2.4 Fire resistance level (FRL)

Fire resistance periods for structural adequacy, integrity and insulation, expressed in that order.

NOTE: Fire resistance levels for structures, parts and elements of construction are specified by the relevant authority, for example, in the Building Code of Australia (BCA).

### 5.2.5 Fire resistance period (FRP)

Time, in minutes, for a member to reach the appropriate failure criteria (i.e. structural adequacy, integrity and/or insulation) if tested for fire in accordance with the appropriate Standard.

NOTE: Where the Building Authority references the Building Code of Australia, the appropriate Standard is AS 1530.4.

### 5.2.6 Fire-separating function

Ability of a boundary element of a fire compartment (e.g. wall, floor or roof) to prevent fire spread by passage of flames or hot gases (integrity) or ignition beyond the exposed surface (thermal insulation) during a fire.

NOTE: When tested in accordance with AS 1530.4, prototypes of such members are exposed to fire from only one direction at a time and are assumed to be similarly exposed for the purpose of interpreting Section 5.

### 5.2.7 Insulation (fire)

The ability of a fire-separating member, such as a wall or floor, to limit the surface temperature on one side of the member when exposed to fire on the other side.

### 5.2.8 Integrity (fire)

Ability of a fire-separating member to resist the passage of flames or hot gases through the member when exposed to fire on one side.

### 5.2.9 Structural adequacy (fire)

Ability of a member to maintain its structural function when exposed to fire.

## 5.3 DESIGN PERFORMANCE CRITERIA

### 5.3.1 General performance criteria

A member shall be designed to have a fire resistance period (FRP) for structural adequacy, integrity and insulation of not less than the required fire resistance level (FRL).

If applicable, the criteria for integrity shall be considered to be satisfied if the member meets the criteria for both insulation and structural adequacy for that period.

The FRP for a member shall be established by either one of the following methods:

- (a) Determined from the tabulated data and figures given in this section. Unless stated otherwise within this section, when using the tabulated data or figures no further checks are required concerning shear and torsion capacity or anchorage details.
- (b) Predicted by methods of calculation. In these cases, checks shall be made for bending, and where appropriate, shear, torsion and anchorage capacities.

NOTE: Eurocode 2, Part 1.2 provides a method of calculation to predict the FRP of a member.

### 5.3.2 General rules for the interpretation of tabulated data and figures

Linear interpolation between values given in the tables and figures in this section is permitted. Values in the tables provide minimum dimensions for fire resistance. Some values of the axis distance of the reinforcement or tendons will result in covers less than those required for durability or compaction and are provided only to allow interpolation within the table or figure.

### 5.3.3 Increase in axis distance for prestressing tendons

The required axis distance for prestressing tendons shall be that determined for reinforcing bars as shown in the tables and figures of this section, increased by—

- (a) 15 mm for prestressing strand or wire, or
- (b) 10 mm for prestressing bars.

### 5.3.4 Dimensional limitations to achieve fire-rating

Where hollow-core slabs or walls are required to achieve a FRL, the thickness of concrete between adjacent voids and the thickness of concrete between any part of a void and the nearest surface shall be not less than the greater of one fifth the required effective thickness of the slab or wall and 25 mm.

Where ribbed slabs are required to achieve a FRL, the ribs shall be spaced at not greater than 1500 mm centre-to-centre.

### 5.3.5 Joints

Joints between members or between adjoining parts shall be constructed so that the FRL of the whole assembly is not less than that required for the member.

### 5.3.6 The effect of chases

In concrete members subject to fire, chases shall be kept to a minimum. The effect of chases on the FRPs of walls shall be taken into account in accordance with the requirements of Clause 5.7.4. The effect of chases in other members shall be taken into account using a rational method of analysis.

### 5.3.7 Increasing FRPs by the addition of insulating materials

The FRP for insulation and structural adequacy of a concrete member may be increased by the addition to the surface of an insulating material, to provide increased thickness to the member or greater insulation to the longitudinal reinforcement or tendons, or both, in accordance with the requirements of Clause 5.8.

For slabs, the FRPs may be increased by the addition of toppings and/or the application of insulating materials to the soffit; for flat slabs and plates, only the application of an insulating material to the soffit may be used to improve the structural adequacy.

For walls, the FRPs may be increased by the application of insulating materials to the face exposed to fire.

In either case, other methods (e.g. addition of insulation materials in hollow cores) may be used. Any increase afforded shall be determined in accordance with Clause 5.8.

## 5.4 FIRE RESISTANCE PERIODS (FRPs) FOR BEAMS

### 5.4.1 Structural adequacy for beams incorporated in roof or floor systems

The FRP for structural adequacy for a beam incorporated in a roof or floor system is given by—

- (a) Table 5.4.1(A) or Figure 5.4.1(A) for simply supported beams; or
- (b) Table 5.4.1(B) or Figure 5.4.1(B) for continuous beams;  
provided the beam—
  - (i) has the upper surface integral with or protected by a slab conforming with Clause 5.5;
  - (ii) has a web of uniform width, or one which tapers uniformly over its depth; and
  - (iii) is proportioned so that—
    - (A) the beam width ( $b$ ), measured at the centroid of the lowest level of longitudinal bottom reinforcement; and
    - (B) the average axis distance to the longitudinal bottom reinforcement,are not less than the values for that period given in the appropriate table or figure.

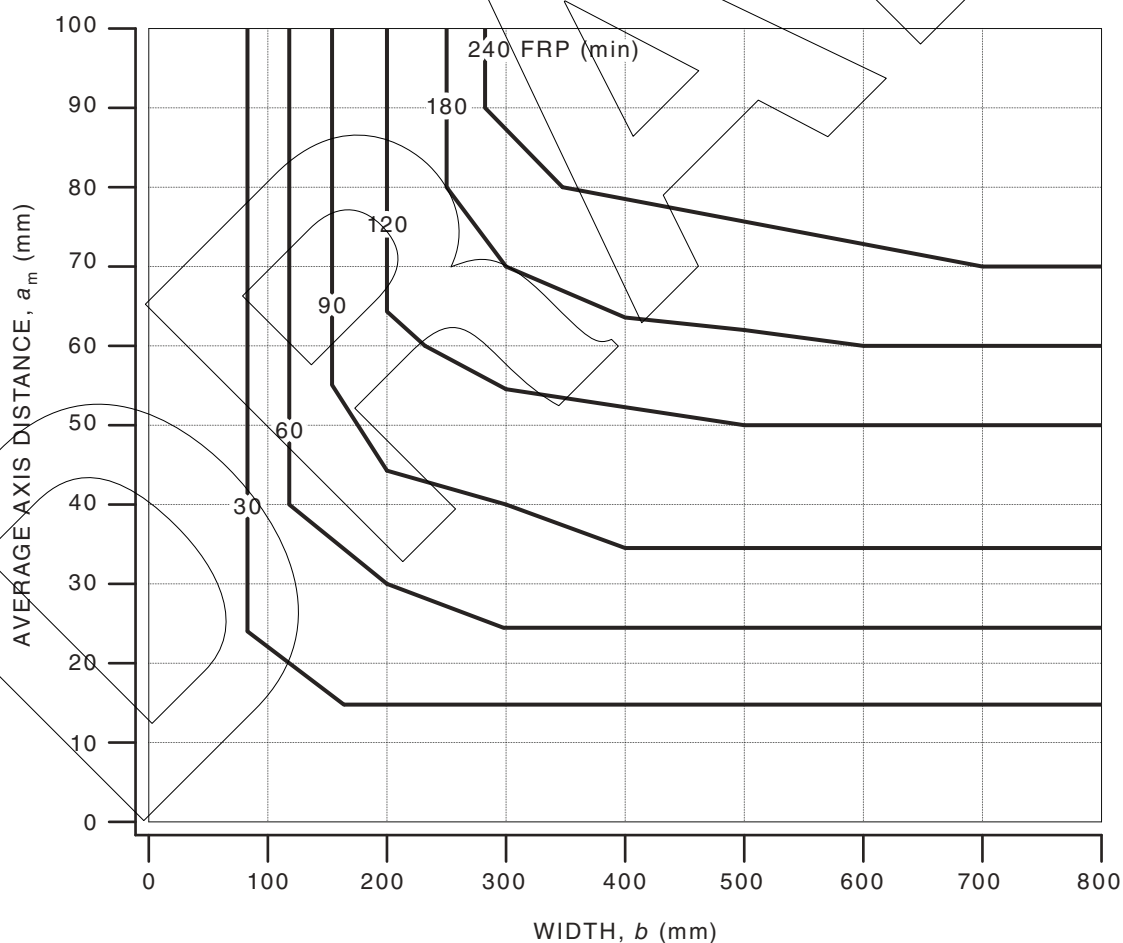
For the purpose of this clause, a beam shall be considered continuous if, under imposed actions, it is designed as flexurally continuous at one or both ends.

**TABLE 5.4.1(A)**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**FOR SIMPLY SUPPORTED BEAMS**

FRP for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of $a_m$ and $b$							
	Combination 1		Combination 2		Combination 3		Combination 4	
	$a_m$	$b$	$a_m$	$b$	$a_m$	$b$	$a_m$	$b$
30	25	80	20	120	15	160	15	200
60	40	120	35	160	30	200	25	300
90	55	150	45	200	40	300	35	400
120	65	200	60	240	55	300	50	500
180	80	240	70	300	65	400	60	600
240	90	280	80	350	75	500	70	700

**LEGEND:** $a_m$  = average axis distance $b$  = width of the beam at the centroid of the bottom reinforcement**NOTES:**

- 1 In beams with only one layer of bottom reinforcement, the axis distance to the side of the beam for the corner bars including tendons or wires, shall be increased by 10 mm, except, where the value of  $b$  is greater than that given in Combination 4, no increase is required.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.



**FIGURE 5.4.1(A) FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**FOR SIMPLY SUPPORTED BEAMS**

**TABLE 5.4.1(B)**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**FOR CONTINUOUS BEAMS**

FRP for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of $a_s$ and $b$							
	Combination 1		Combination 2		Combination 3		Combination 4	
	$a_m$	$b$	$a_m$	$b$	$a_m$	$b$	$a_m$	$b$
30	15	80	12	160	—	—	—	—
60	25	120	12	200	—	—	—	—
90	35	150	25	250	—	—	—	—
120	45	200	35	300	35	450	30	500
180	60	240	50	400	50	550	40	600
240	75	280	60	500	60	650	50	700

**LEGEND:** $a_m$  = average axis distance $b$  = width of the beam at the centroid of the bottom reinforcement**NOTES:**

- 1 In beams with only one layer of bottom reinforcement, the axis distance to the side of the beam for the corner bars including tendons or wires, shall be increased by 10 mm, except, where the value of  $b$  is greater than that given in Combination 4 no increase is required.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.



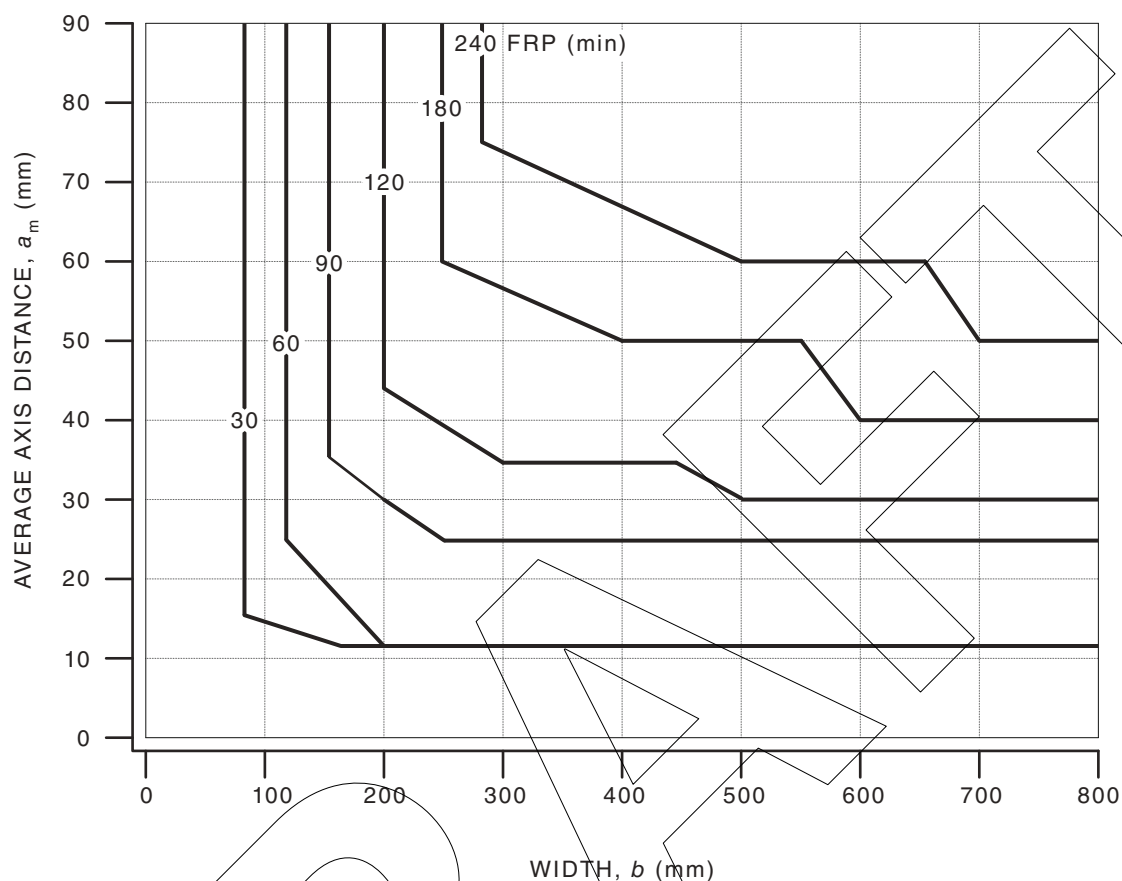


FIGURE 5.4.1(B) FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY FOR CONTINUOUS BEAMS

#### 5.4.2 Structural adequacy for beams exposed to fire on all sides

The FRP for structural adequacy for a beam of approximately rectangular cross-section, which can be exposed to fire on all four sides, is given by—

- (a) Table 5.4.1(A) or Figure 5.4.1(A) for simply supported beams; or
- (b) Table 5.4.1(B) or Figure 5.4.1(B) for continuous beams, provided in each case the beam is proportioned so that—
  - (i) the total depth of the beam is not less than the least value of  $b$  for that period;
  - (ii) the cross-sectional area of the beam is not less than twice the area of a square with a side equal to  $b$  determined as for Item (a); and
  - (iii) the average axis distance is not less than the value for that period determined using the minimum dimension of the beam for  $b$  in the relevant table and applies to all longitudinal reinforcement or tendons.

## 5.5 FIRE RESISTANCE PERIODS (FRPs) FOR SLABS

### 5.5.1 Insulation for slabs

The FRP for insulation for a slab is given in Table 5.5.1 provided the effective thickness of the slab is not less than the corresponding value given in the table.

The effective thickness of the slab to be used in Table 5.5.1 shall be taken as—

- (a) for solid slabs, the actual thickness; or
- (b) for hollow-core slabs, the net cross-sectional area divided by the width of the cross-section; or
- (c) for ribbed slabs, the thickness of the solid slab between the webs of adjacent ribs.

**TABLE 5.5.1**  
**FIRE RESISTANCE PERIODS (FRPs)**  
**FOR INSULATION FOR SLABS**

FRP for insulation min	Effective thickness mm
30	60
60	80
90	100
120	120
180	150
240	175

### 5.5.2 Structural adequacy for slabs

The FRP for structural adequacy for a slab shall be deemed to be satisfied if the following conditions are met:

- (a) For solid or hollow-core slabs supported on beams or walls [see Table 5.5.2(A)], provided the slab is proportioned such that, for the appropriate support conditions, the average axis distance to the bottom reinforcement and tendons is not less than the value for that period given in the table.
- (b) For flat slabs, including flat plates [see Table 5.5.2(B)], provided the average axis distance to the bottom layer of reinforcement and tendons is not less than the value in the table.
- (c) The minimum slab thickness defined in Table 5.5.2(B) is required over a rectangular area extending 0.16 of the larger span length either side of a column in each direction from the column centreline. The depths in this table cannot be reduced by calculation methods in accordance with Clause 5.3.1(b). Outside this area, the minimum slab depths for insulation in Table 5.5.1 for insulation apply. For one-way ribbed slabs, see Table 5.5.2(A) for the appropriate support conditions, if the slab is proportioned, so that—
  - (i) the width of the ribs and the axis distance to the lowest layer of the longitudinal bottom reinforcement in the slabs conform with the requirements for beams given in Clause 5.4.1; and
  - (ii) the axis distance to the bottom reinforcement in the slab between the ribs is not less than that given in Table 5.5.2(A).

- (d) For two-way ribbed slabs, see Table 5.5.2(C) or Table 5.5.2(D) as appropriate for the support conditions. The slabs shall be proportioned so the width and the average axis distance to the longitudinal bottom reinforcement in the ribs, and the axis distance to the bottom reinforcement in the slab between the ribs, and the axis distance of the corner bar to the side face of the rib, is not less than that value plus 10 mm.

For the purpose of this clause, a slab shall be considered continuous if, under imposed actions, it is designed as flexurally continuous at one or both ends.

**TABLE 5.5.2(A)**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**FOR SOLID AND HOLLOW-CORE SLABS SUPPORTED ON BEAMS**  
**OR WALLS AND FOR ONE-WAY RIBBED SLABS**

FRP for structural adequacy (min)	Axis distance ( $a_s$ ) to lowest layer of reinforcement (mm)			
	Simply supported slabs			Continuous slabs (one-way and two-way)
	One-way	Two-way		
		$l_y/l_x \leq 1.5$	$1.5 < l_y/l_x \leq 2$	
30	10	10	10	10
60	20	10	15	10
90	30	15	20	15
120	40	20	25	20
180	55	30	40	30
240	65	40	50	40

NOTES:

- $l_y$  = longer span of a two-way slab.  
 $l_x$  = shorter span of a two-way slab.
- The axis distance for simply supported two-way slabs applies only if the slabs are supported at all four edges. In other cases, the slab shall be treated as a one-way slab.
- For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

**TABLE 5.5.2(B)**  
**FIRE RESISTANCE PERIODS (FRPs) FOR**  
**STRUCTURAL ADEQUACY FOR FLAT**  
**SLABS INCLUDING FLAT PLATES**

FRP for structural adequacy (min)	Minimum dimensions (mm)	
	Slab thickness	Axis distance ( $a_s$ )
30	150	10
60	180	15
90	200	25
120	200	35
180	200	45
240	200	50

NOTES:

- $a_s$  = axis distance to the reinforcement in the lowest layer.
- For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

**TABLE 5.5.2(C)**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**FOR TWO-WAY SIMPLY SUPPORTED RIBBED SLABS**

FRP for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of axis distance ( $a_s$ ) and width of ribs ( $b$ )						Flange thickness ( $h_s$ ) and axis distance ( $a_s$ ) in flange	
	Combination 1		Combination 2		Combination 3			
	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$	$a_s$	$h_s$
30	15	80	—	—	—	—	10	80
60	35	100	25	120	15	$\geq 200$	10	80
90	45	120	40	160	30	$\geq 250$	15	100
120	60	160	55	190	40	$\geq 300$	20	120
180	75	220	70	260	60	$\geq 410$	30	150
240	90	280	75	350	70	$\geq 500$	40	175

NOTES:

- 1 The axis distance is measured to the lowest layer of the longitudinal reinforcement.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

**TABLE 5.5.2(D)**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**FOR TWO-WAY CONTINUOUS RIBBED SLABS**

FRP for structural adequacy (min)	Minimum dimensions (mm)						Flange thickness ( $h_s$ ) and axis distance ( $a_s$ ) in flange	
	Possible combinations of axis distance ( $a_s$ ) and width of ribs ( $b$ )							
	Combination 1		Combination 2		Combination 3		$a_s$	$h_s$
	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$		
30	10	80	—	—	—	—	10	80
60	25	100	15	120	10	$\geq 200$	10	80
90	35	120	25	160	15	$\geq 250$	15	100
120	45	160	40	190	30	$\geq 300$	20	120
180	60	310	50	600	—	—	30	150
240	70	450	60	700	—	—	40	175

NOTES:

- 1 The axis distance is measured to the lowest layer of the longitudinal reinforcement.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

## 5.6 FIRE RESISTANCE PERIODS (FRPs) FOR COLUMNS

### 5.6.1 Insulation and integrity for columns

FRPs for insulation and integrity are required for columns only where columns form part of a wall required to have a separating function. In this situation the column shall conform with the appropriate criteria for walls given in Clause 5.7.1.

### 5.6.2 Structural adequacy for columns

The FRP for structural adequacy for braced columns shall be determined using either Clause 5.6.3 or Clause 5.6.4. Where the ratio of the longer cross-section dimension of the column is equal to or greater than four times the shorter cross-section dimension, Clause 5.7.2 may be used.

NOTE: Clauses 5.6.3 and 5.6.4 cover only braced columns that conform with a series of restrictions. For unbraced or sway columns and braced columns outside these restrictions, see Clause 5.3.1 or the BCA and use an alternative solution.

Where columns are to be designed as walls using Clause 5.7.2, the case of a wall exposed on two faces shall be adopted and the column shall be reinforced with two layers of longitudinal reinforcement (one layer located adjacent to each face), and the two layers shall be structurally restrained together.

### 5.6.3 Restricted tabular method to determine structural adequacy for columns

The FRP for structural adequacy for a column may be determined from Table 5.6.3, provided the following criteria are met:

- (a) The column is proportioned so that the value for the smaller cross-sectional dimension and the axis distance to the longitudinal reinforcement are not less than the values for that period.
- (b) The value of the load level is taken as 0.7 or calculated as follows:

$$\mu_{fi} = \frac{N_f^*}{\phi N_u} \quad \dots 5.6.3(1)$$

where

$N_f^*$  = design axial load in the fire situation

$N_u$  = ultimate strength in compression, or tension, at a cross-section of an eccentrically loaded compression or tension member respectively

- (c) Where  $A_s \geq 0.02A_c$  and the required FRP is greater than 90 min, the bars are distributed along all the faces of the column.
- (d) The effective length of the column under fire conditions is less than 3 m.
- (e) The maximum eccentricity is limited to  $0.15b$ .

For columns that fall outside these limits, reference shall be made to alternative design approaches, as specified in Clause 5.3.1.

NOTE: The effective length of a column under fire conditions ( $l_{0,fi}$ ) may be assumed to be equal to the effective length at normal temperature in all cases. For braced building structures where the required FRP is greater than 30 minutes, the effective length may be taken as  $0.5L_u$  for all cases.

**TABLE 5.6.3**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**OF COLUMNS**

FRP for structural adequacy	Minimum dimensions, mm							
	Combinations for column exposed on more than one side						Column exposed on one side	
	$\mu_{fi} = 0.2$		$\mu_{fi} = 0.5$		$\mu_{fi} = 0.7$		$\mu_{fi} = 0.7$	
	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$
min								
30	25	200	25	200	32 27	200 300	25	155
60	25	200	36 31	200 300	46 40	250 350	25	155
90	31 25	200 300	45 38	300 400	53 40 <sup>(1)</sup>	350 450 <sup>(1)</sup>	25	155
120	40 35	250 350	45 <sup>(1)</sup> 40 <sup>(1)</sup>	350 <sup>(1)</sup> 450 <sup>(1)</sup>	57 <sup>(1)</sup> 51 <sup>(1)</sup>	350 <sup>(1)</sup> 450 <sup>(1)</sup>	35	175
180	45 <sup>(1)</sup>	350 <sup>(1)</sup>	63 <sup>(1)</sup>	350 <sup>(1)</sup>	70 <sup>(1)</sup>	450 <sup>(1)</sup>	55	230
240	61 <sup>(1)</sup>	350 <sup>(1)</sup>	75 <sup>(1)</sup>	450 <sup>(1)</sup>			70	295

**LEGEND:** $a_s$  = axis distance $b$  = smaller cross-sectional dimension of a rectangular column or the diameter of a circular column.**NOTES:**

- 1 These combinations are for columns with a minimum of eight bars.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.
- 3 Dimension  $b$  in table for columns exposed on one side applies only to columns that lie flush with a wall having the same FRP as the column or to columns protruding from the wall, provided the part within the wall is able to carry the whole load. Openings in the wall shall not be closer to the column than the minimum dimension  $b$  for the column for the FRP. In all other cases, the column shall be treated as a column exposed on more than one side.

The FRP for structural adequacy for columns outside the limits defined for Table 5.6.3 and within the limits defined in the variables below may be assessed using Equation 5.6.3(2):

$$FRP = 120 ((R_{\eta,fi} + R_a + R_l + R_b + R_n)/120)^{1.8} \quad \dots 5.6.3(2)$$

where

$$R_{\eta,fi} = 83(1.0 - \mu_{fi}(1 + \omega)/(.945 + \omega))$$

$$R_a = 1.60(a_s - 30)$$

$$R_l = 9.60(5 - l_{o,fi})$$

$$R_b = 0.09 b'$$

$$R_n = 0 \text{ for 4 (corner bars only)}$$

$$= 12 \text{ for greater than 4 bars}$$

$a_s$  = the axis distance to the longitudinal steel bars (mm);  $25 \text{ mm} \leq a_s \leq 80 \text{ mm}$

$l_{o,fi}$  = the effective length of the column under fire conditions;  $2 \text{ metres} \leq l_{o,fi} \leq 6 \text{ metres}$

Values for  $l_{o,fi} < 2 \text{ metres}$  are conservative for  $l_{o,fi} + 2 \text{ metres}$

$$b' = 2A_g/(\gamma + x) \text{ for rectangular cross-sections}$$

$$= \text{the diameter of circular cross sections}$$

within the limits  $200 \text{ mm} \leq b' \leq 450 \text{ mm}$  and  $\gamma \leq 1.5x$

$\omega = 1.3 A_s f_{sy} / A_c f'_c$  denotes the mechanical reinforcement ratio at normal temperature conditions

#### 5.6.4 General tabular method to determine structural adequacy for columns

The FRP for structural adequacy for a column may be determined from Table 5.6.4, provided the following criteria are met:

- The column is proportioned so that the value for the smaller cross-sectional dimension and the axial distance to the longitudinal reinforcement are not less than the values for that period.
- The eccentricity under fire conditions is  $e = \frac{M_f^*}{N_f^*}$ .
- $e/b < 0.25$  with  $e_{\max.} \leq 100 \text{ mm}$ .
- The slenderness of the column under fire conditions is  $\leq 30$ .

For columns that fall outside these limits, reference shall be made to alternative design approaches, as specified in Clause 5.3.1.

**TABLE 5.6.4**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**OF BRACED COLUMNS**

FRP for structural adequacy	$1.3 A_s f_{sy} / A_c f'_c$	Minimum dimensions, mm							
		Combinations of $(a_s)$ and $(b)$							
		$\eta = 0.15$		$\eta = 0.3$		$\eta = 0.5$		$\eta = 0.7$	
		$a_s$	$b$	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$
min	0.1	25	150	25	150	30	200	30	300
						25	250	25	350
	0.5	25	150	25	150	25	150	30	200
								25	250
30	1.0	25	150	25	150	25	150	30	200
								25	300
	0.1	30	150	40	200	40	300	25	500
		25	200	25	300	25	500		
60	0.5	25	150	35	150	35	250	40	350
				25	200	25	350	25	550
	1.0	25	150	30	150	40	200	50	300
				25	200	25	400	30	600
90	0.1	40	200	40	300	50	500	40	550
		25	250	25	400	25	550	25	600
	0.5	35	150	45	200	45	300	50	500
		25	200	25	300	25	550	40	600
	1.0	25	200	40	200	40	250	50	500
				25	300	25	550	45	600

TABLE 5.6.4 (continued)

FRP for structural adequacy	$1.3 A_s f_{sy} / A_c f'_c$	Minimum dimensions, mm								
		Combinations of ( $a_s$ ) and ( $b$ )								
		$\eta = 0.15$		$\eta = 0.3$		$\eta = 0.5$		$\eta = 0.7$		
		$a_s$	$b$	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$	
min	0.1	50	250	50	400	25	550	60	550	
		25	350	25	550			45	600	
		0.5	45	200	45	300	50	450	60	500
			25	300	25	550	25	600	50	600
		1.0	40	200	50	250	45	450	60	600
			25	250	25	400	30	600		
120	0.1	50	400	60	500	60	550	(Note 1)		
		25	500	25	550	30	600			
	0.5	45	300	50	450	60	500	75	600	
		25	450	25	600	50	600			
	1.0	35	300	50	450	60	500	(Note 1)		
		25	400	25	550	45	600			
(continued)										
240	0.1	60	500	40	550	75	600	(Note 1)		
		25	550	25	600					
	0.5	45	450	55	550	70	600	(Note 1)		
		25	500	25	600					
	1.0	45	400	40	500	60	600	(Note 1)		
		25	500	30	600					

## LEGEND:

 $a_s$  = axis distance $b$  = smaller cross-sectional dimension of a rectangular column or the diameter of a circular column

$$\eta = \frac{N_f^*}{0.7 \left[ \left( \frac{A_c f'_c}{1.5} \right) + \left( \frac{A_s f_{sy}}{1.15} \right) \right]}$$

## NOTES:

- 1 Requires a width greater than 600 mm and assessment for buckling.
- 2 The slenderness limit of  $\leq 30$  is applicable to the majority of columns in normal buildings.
- 3 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.
- 4 Dimension  $b$  in table for columns exposed on one side applies only to columns that lie flush with a wall having the same FRP as the column or to columns protruding from the wall, provided the part within the wall is able to carry the whole load. Openings in the wall shall not be closer to the column than the minimum dimension  $b$  for the column for the FRP. In all other cases, the column shall be treated as a column exposed on more than one side.

## 5.7 FIRE RESISTANCE PERIODS (FRPs) FOR WALLS

## 5.7.1 Insulation for walls

The FRP for insulation for a wall given in Table 5.7.1 may be used, provided the effective thickness of the wall is not less than the corresponding value given in the table.

The effective thickness of the wall to be used in Table 5.7.1 shall be taken as follows:



- (a) For solid walls, the actual thickness.
- (b) For hollow-core walls, the net cross-sectional area divided by the length of the cross-section.

**TABLE 5.7.1**  
**FIRE RESISTANCE PERIODS (FRPs)**  
**FOR WALLS FOR INSULATION**

FRP for insulation min	Effective thickness mm
30	60
60	80
90	100
120	120
180	150
240	175

### 5.7.2 Structural adequacy for walls

The FRP for structural adequacy for a wall shall be in accordance with Table 5.7.2, provided the effective thickness of the wall and axis distance are not less than the corresponding values given in the table.

For walls where the lateral support at the top of the wall is provided on one side only by a member not required by the relevant authority to have an FRL, the structural adequacy shall be deemed to have been achieved by satisfying the requirements of Clause 5.7.1.

**TABLE 5.7.2**  
**FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY**  
**FOR WALLS**

FRP for structural adequacy  min	Minimum dimensions, mm Combinations of $a_s$ and $b$							
	$\frac{N_f^*}{\phi N_u} = 0.35$				$\frac{N_f^*}{\phi N_u} = 0.7$			
	Wall exposed on one side		Wall exposed on two sides		Wall exposed on one side		Wall exposed on two sides	
	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$	$a_s$	$b$
30	10	100	10	120	10	120	10	120
60	10	110	10	120	10	130	10	140
90	20	120	10	140	25	140	25	170
120	25	150	25	160	35	160	35	220
180	40	180	45	200	50	210	55	270
240	55	230	55	250	60	270	60	350

**LEGEND:**

$a_s$  = axis distance

$b$  = wall thickness

**NOTES:**

1 For  $N_f^* / \phi N_u$  see Clause 5.6.3.

2 For prestressing tendons, the axis shall be increased, as specified in Clause 5.3.3.

### 5.7.3 Effective height limitations for walls

For walls required to have a FRL, the ratio of the effective height to thickness shall not exceed 40, where the effective height is determined from Clause 11.4. This latter restriction shall not apply to walls where the lateral support at the top of the wall is provided by an element not required by the relevant authority to have a FRL.

### 5.7.4 Other requirements for walls

#### 5.7.4.1 Recesses for services in walls

The effect of recesses for services on the FRP for structural adequacy, integrity and insulation of a wall shall be ignored if the thickness of wall remaining under the bottom of the recess is not less than half the wall thickness and the total recessed area, within any 5 m<sup>2</sup> of wall face, is not more than 10 000 mm<sup>2</sup> on one or both faces of the wall.

If the above limits are exceeded, the wall thickness (*b*) used to determine FRP shall be taken as the overall thickness less the depth of the deepest recess.

#### 5.7.4.2 Effect of chases on structural adequacy of walls

The effect of chases on the FRP for structural adequacy of walls shall be taken into account as follows:

- (a) For walls spanning one way, where—
  - (i) the chase direction is parallel to the span direction—ignored;
  - (ii) the chase direction is perpendicular to the span direction and of a length not greater than four times the wall thickness or 0.4 times the overall length of the wall, whichever is greater—ignored; or
  - (iii) the chase is perpendicular to the span direction and of a length greater than four times the wall thickness or 0.4 times the overall length of the wall—accounted for by using a slenderness ratio for the wall based on the reduced wall thickness.
- (b) For walls spanning two ways (panel action), where—
  - (i) there is either a vertical chase with a length not greater than half the wall height ( $H_w$ ), or a horizontal chase with a length not greater than half the wall length ( $L_w$ )—ignored; and
  - (ii) the length of a vertical chase is greater than half the wall height ( $H_w$ ), or the length of a horizontal chase is greater than half the wall length ( $L_w$ )—accounted for by using a slenderness ratio for the wall based on the reduced wall thickness, or the chase may be regarded as an unsupported edge and the panel designed as two sub-panels.

#### 5.7.4.3 Effect of chases on integrity and insulation of walls

The effect of chases on the FRP for integrity and insulation of walls shall be taken into account as follows:

- (a) Where—
  - (i) the depth of the chase is not greater than 30 mm; and
  - (ii) the cross-sectional area of the chase, on a plane perpendicular to the plane of the wall face and at right angles to the centre-line of the chase, is not greater than 1000 mm<sup>2</sup>; and
  - (iii) the total face area of chases within any 5 m<sup>2</sup> of wall face is not greater than 100 000 mm<sup>2</sup> on one or both wall faces,
 the effect shall be ignored.

- (b) For cases other than those in Item (a) above, the effects shall be taken into account in accordance with the normal rules for insulation and integrity of walls, except that slenderness ratios shall be based on the reduced wall thickness.

## **5.8 INCREASE OF FIRE RESISTANCE PERIODS (FRPs) BY USE OF INSULATING MATERIALS**

### **5.8.1 Increase of FRP by the addition of insulating materials**

#### **5.8.1.1 General**

The FRP for insulation and structural adequacy of a concrete member may be increased by the addition to the surface of an insulating material, to provide increased thickness to the member, or greater insulation to the longitudinal reinforcement or tendons, or both.

#### **5.8.1.2 Acceptable forms of insulation**

Acceptable forms of insulation include the following:

- (a) Slabs of one part cement to four parts vermiculite (by volume) concrete or of one part cement to four parts perlite (by volume) concrete, appropriately bonded to the concrete.
- (b) Gypsum-vermiculite plaster or gypsum-perlite plaster, both mixed in the proportion of 0.16 m<sup>3</sup> of aggregate to 100 kg of gypsum, in the form of either slabs appropriately bonded to the concrete, or as a sprayed or trowelled application applied in situ.
- (c) Any other fire-protective building board or material, which has been demonstrated to be suitable for the purpose in a standard fire-resistance test.

#### **5.8.1.3 Thickness of insulating material**

The minimum thickness of insulating material added to attain the required FRL shall be determined by testing in accordance with AS 1530.4.

In the absence of such testing and only for the materials specified in Clause 5.8.1.2, the minimum thickness of insulating material to be added may be taken as the difference between the required cover or effective thickness specified in this section and the actual cover or effective thickness, whichever governs, multiplied by—

- (a) 0.75, for materials specified in Clauses 5.8.1.2(a) and (b); or
- (b) an appropriate factor for materials specified in Clause 5.8.1.2(c), where the factor is derived from tests in which the difference calculated above lies within the range of insulation thickness tested; and
- (c) the thickness thus calculated rounded to the nearest 5 mm above.

#### **5.8.1.4 Reinforcement in sprayed or trowelled insulating materials**

Where the thickness of sprayed or trowelled insulating materials exceeds 10 mm, it shall be reinforced to prevent detachment during exposure to fire.

### **5.8.2 Increase of insulation period of slabs by application of toppings**

The FRP for insulation of a slab may be increased by incorporating an integral or a separately applied topping of thickness given by the following equation:

$$t_{\text{nom}} = kt_d + 10 \quad \dots 5.8.2$$

where

$$t_{\text{nom}} = \text{nominal thickness of topping applied}$$

- $k$  = 1.0 for a topping of plain concrete
- = 0.8 for a topping of concrete made from lightweight aggregate conforming with AS 2758.1
- = 0.6 for a topping of gypsum (including jointed gypsum block) having a wearing overlay
- $t_d$  = difference between the actual effective thickness of the slab and the effective thickness specified in Table 5.5.1, for the required FRP

## SECTION 6 METHODS OF STRUCTURAL ANALYSIS

### 6.1 GENERAL

#### 6.1.1 Basis for structural analysis

Methods of analysis for concrete structures shall take into account the following:

- (a) The strength and deformational properties of the member materials.
- (b) The equilibrium requirements for all forces acting on and within the structure.
- (c) The requirements of compatibility of deformations within the structure.
- (d) The support conditions and, where appropriate, interaction of the structure with the foundation and other connecting or adjacent structures.

#### 6.1.2 Interpretation of the results of analysis

Irrespective of the method chosen for the structural analysis, the simplifications, idealizations and assumptions implied in the analysis shall be considered in relation to the real, three-dimensional nature of the structure when the results of the analysis are interpreted.

NOTE: Users of software packages for analysis should ensure the package is appropriate for the analysis being undertaken.

#### 6.1.3 Methods of analysis

For the purpose of conforming with the requirements for strength, serviceability and robustness specified in Section 2, it shall be permissible to determine the action effects and deformations in a reinforced or prestressed structure and its component members using the following methods, as appropriate:

- (a) Static analysis for determinate structures.
- (b) Linear elastic analysis, in accordance with Clause 6.2.
- (c) Linear elastic frame analysis incorporating secondary bending moments due to lateral joint displacement, in accordance with Clause 6.3.
- (d) Linear elastic stress analysis of members and structures, in accordance with Clause 6.4.
- (e) Non-linear frame analysis, in accordance with Clause 6.5.
- (f) Non-linear stress analysis of members and structures, in accordance with Clause 6.6.
- (g) Plastic methods of analysis for slabs and frames, in accordance with Clause 6.7.
- (h) Strut-and-tie method of analysis, in accordance with Clause 6.8.
- (i) Structural model tests designed and evaluated in accordance with the principles of mechanics.
- (j) The following simplified methods of analysis:
  - (i) The idealized frame method given in Clause 6.9.
  - (ii) The simplified methods given in Clause 6.10.

NOTE: Clause 2.2 allows different strength check procedures and different methods of analysis to be used for different members in a structure and for the structure.

### 6.1.4 Geometrical properties

For the purpose of this section, the definitions below apply.

#### 6.1.4.1 Column strip

That portion of the design strip extending transversely from the centre-line of the supports—

- (a) for an interior column strip, one quarter of the distance to the centre-line of each adjacent and parallel row of supports; or
- (b) for an edge column strip, to the edge of the slab and one quarter of the distance to the centre-line of the next interior and parallel row of supports,

but of total width not greater than  $L/2$  [see Figure 6.1.4(A)].

#### 6.1.4.2 Design strip

That part of a two-way slab system, which is supported, in the direction of bending being considered, by a single row of supports and which in each span extends transversely from the centre-line of the supports—

- (a) for an interior design strip, halfway to the centre-line of each adjacent and parallel row of supports; or
- (b) for an edge design strip, to the edge of the slab and halfway to the centre-line of the next interior and parallel row of supports [see Figure 6.1.4(A)].

#### 6.1.4.3 Middle strip

The portion of the slab between two column strips or between a column strip and a parallel supporting wall [see Figure 6.1.4(A)].

#### 6.1.4.4 Span support

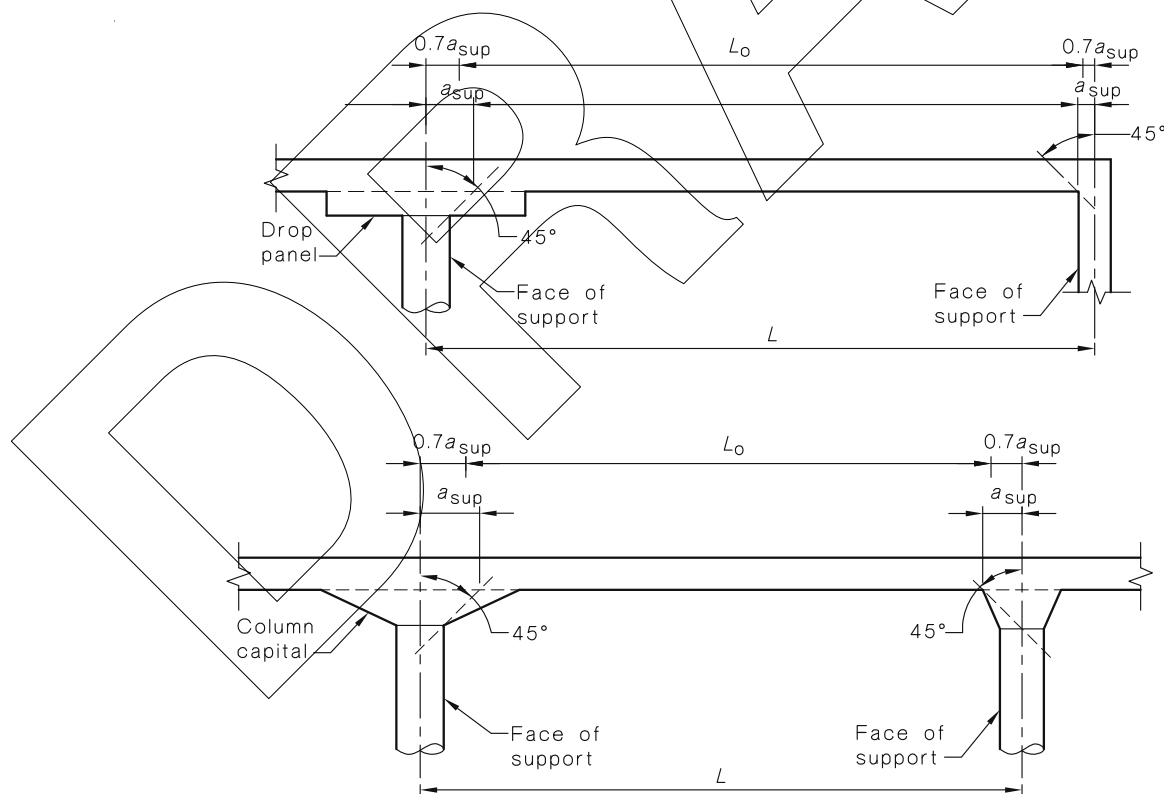
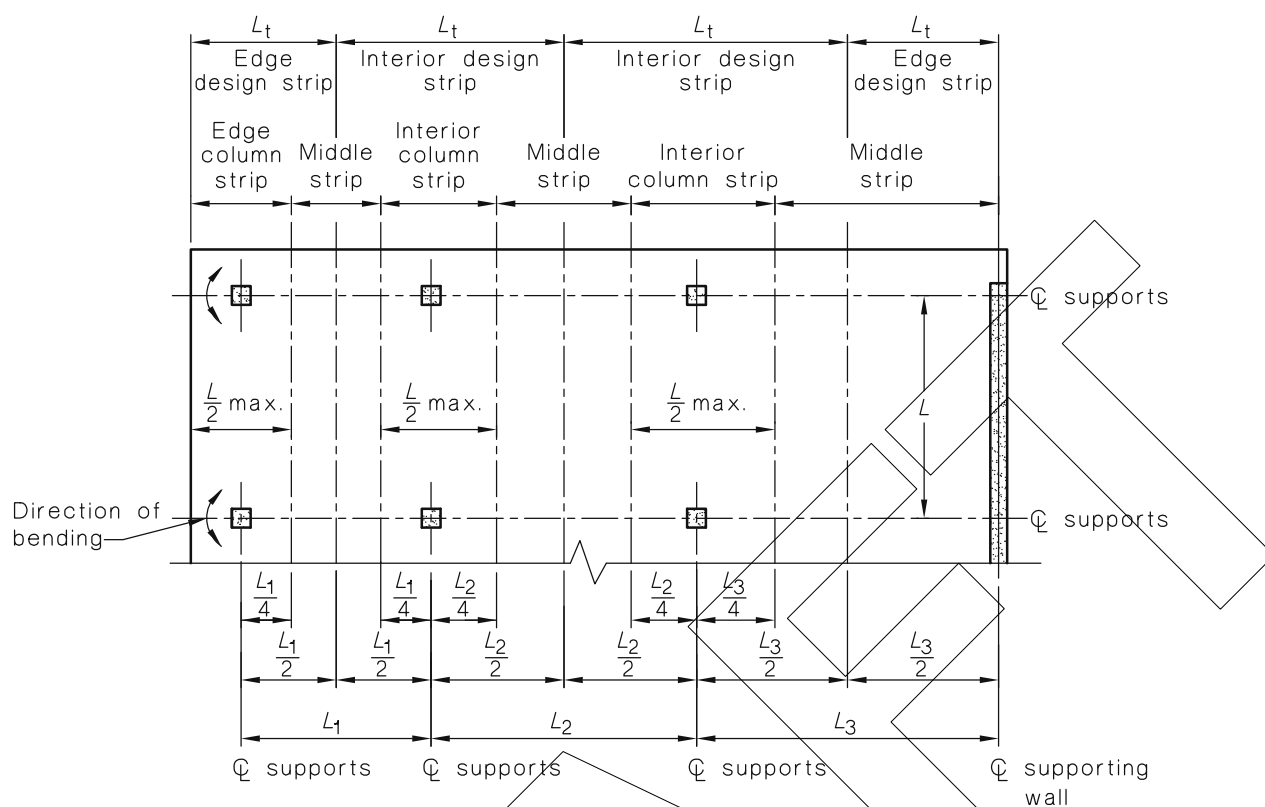
The length of a support in the direction of the span ( $a_{sup}$ ) taken as—

- (a) for beams or for flat slabs without either drop panels or column capitals, the distance from the centre-line of the support to the face of the support; or
- (b) for flat slabs with drop panels or column capitals or both, the distance from the centre-line of the support to the intersection with the plane of the slab soffit of the longest line, inclined at an angle of  $45^\circ$  to the centre-line of the support, which lies entirely within the surfaces of the slab and the support, as shown in Figure 6.1.4(B).

NOTE: For the purpose of Item (b), circular or polygonal columns may be regarded as square columns with the same cross-sectional area.

#### 6.1.4.5 Transverse width

The width of the design strip ( $L_t$ ) measured perpendicular to the direction of bending being considered [see Figure 6.1.4(A)].



## 6.2 LINEAR ELASTIC ANALYSIS

### 6.2.1 General

This clause applies to the analysis of indeterminate continuous beams and framed structures in which secondary geometric effects are insignificant.

### 6.2.2 Span length

The span length of flexural members shall be taken as the distance centre-to-centre of supports.

### 6.2.3 Critical sections for negative moments

The critical section for a maximum negative bending moment may be taken at 0.7 times the length of a support in the direction of the span ( $a_{sup}$ ) from the centre-line of the support.

### 6.2.4 Stiffness

#### 6.2.4.1 General

The stiffness of members shall be chosen to represent the conditions at the limit state being analysed. The stiffness assumptions chosen shall be consistent with all loading conditions, and shall generate critical worst-case actions under all failure modes to be considered. Where multiple degrees of stiffness are possible, the stiffness assumption chosen for the failure mode under consideration shall induce worst-case actions in the element being designed.

The effect of haunching and other variations of cross-section along the axis of a member shall be considered and, where significant, taken into account in the determination of the member stiffness.

Any assumptions regarding the relative stiffness of members shall be applied consistently throughout the analysis.

#### 6.2.4.2 Stiffness of lateral force resisting elements

In the absence of refined analysis conducted in accordance with Section 6, the assessment of inter-storey drift, periods of vibration and distribution of internal actions, the moment of inertia for flexural members, columns and walls shall be determined as follows:

- (a) For uncracked sections, the full gross uncracked cross section of the member; or
- (b) For cracked sections, in accordance with Table 6.2.4.

A section shall be assumed to be uncracked unless the tensile stresses due to combined flexure and axial load at the limit state under consideration are less than the mean characteristic flexural tensile strength of the concrete determined in accordance with Clause 3.1.1.3.



**TABLE 6.2.4**  
**EFFECTIVE SECTION PROPERTIES,**  
 **$I_{eff}$ , AS A PROPORTION OF  $I_g$**

Type of member	Ultimate limit state
Beams and slabs	$0.4I_g$
Columns	
$N^*/Agf^*c > 0.5$	$0.8I_g$
$N^*/Agf^*c = 0.2$	$0.5I_g$
$N^*/Agf^*c = 0.0$	$0.3I_g$
Walls	
$N^*/Agf^*c = 0.2$	$0.42I_g$
$N^*/Agf^*c = 0.1$	$0.33I_g$
$N^*/Agf^*c = 0.0$	$0.25I_g$

### 6.2.5 Deflections

Deflection calculations shall take into account the effects of cracking, tension stiffening, shrinkage, creep, and relaxation of tendons. Calculations in accordance with the requirements of Clauses 8.5 and 9.3 shall be deemed to satisfy this requirement. Where appropriate, consideration shall be given to deformations that may result from deflection of the formwork or settlement of the supporting props during construction.

### 6.2.6 Secondary bending moments and shears resulting from prestress

The secondary bending moments and shears and the associated deformations that are produced in an indeterminate structure by prestressing shall be taken into account in the design calculations for serviceability.

The secondary bending moments and shears due to the effects of prestress may be determined by elastic analysis of the unloaded uncracked structure.

In design calculations for strength, the secondary bending moments and shears due to prestress shall be included with a load factor of 1.0 when the design moments and shears for the load combinations given in Clause 2.4 are calculated. For the special case of permanent action (dead load) plus prestress at transfer, the load factors given by Clause 2.4 shall apply.

### 6.2.7 Moment redistribution in reinforced and prestressed members for strength design

#### 6.2.7.1 General requirements

In design calculations for strength of statically indeterminate members, the elastically determined bending moments at any interior support may be reduced or increased by redistribution, provided an analysis is undertaken to show there is adequate rotation capacity in critical moment regions to allow the assumed distribution of bending moments to be achieved.

The analysis shall take into account—

- (a) the stress-strain curves of the steel reinforcement and tendons as defined in Clauses 3.2.3 and 3.3.3, respectively, assuming that fracture of the reinforcement and tendon occur at  $\epsilon_{su}$  and  $\epsilon_{pu}$ ;
- (b) static equilibrium of the structure after redistribution of the moments; and
- (c) the properties of the concrete as defined in Clause 3.1.

### 6.2.7.2 *Deemed-to-conform approach for reinforced and prestressed members*

The requirement of Clause 6.2.7.1 shall be deemed to be met if the following requirements are satisfied:

- (a) All of the main reinforcement in the member shall be Ductility Class N.
- (b) The bending moment distribution before redistribution shall be determined in accordance with elastic analysis.
- (c) Where the neutral axis parameter ( $k_u$ ) is less than or equal to 0.2 in all peak moment regions, the redistribution of the moment at a support shall not exceed 30%.
- (d) Where  $k_u$  exceeds 0.2 in one or more peak moment regions, but does not exceed 0.4, the redistribution shall not exceed 75 ( $0.4 - k_u$ )%.
- (e) The positive bending moment shall be adjusted to maintain equilibrium.
- (f) Where  $k_u$  exceeds 0.4 in any peak moment region, no redistribution shall be made.
- (g) Static equilibrium of the structure after redistribution of the moments shall be used to evaluate all action effects for strength design.

#### NOTES:

- 1 The values of  $k_u$  are calculated for cross-sections that have been designed on the basis of the redistributed moment diagram.
- 2 The amount of redistribution is measured as a percentage of the bending moment before redistribution.
- 3 Extra checks should be made on ductility and the possibility of punching shear failures.

## 6.3 ELASTIC ANALYSIS OF FRAMES INCORPORATING SECONDARY BENDING MOMENTS

### 6.3.1 General

This clause applies to the elastic analysis of frames not restrained by bracing or shear walls, or both, for which the relative displacement at the ends of compressive members is less than  $L_u/250$  under the design load for strength.

### 6.3.2 Analysis

An elastic analysis incorporating secondary bending moments shall conform with the requirements of Clause 6.2 and the following:

- (a) The effect of lateral joint displacements shall be taken into account.
- (b) For strength design of a regular rectangular framed structure, the cross-sectional stiffness of the flexural members and columns shall be determined in accordance with Clause 6.2.4.2.
- (c) For very slender members, the change in bending stiffness of a member due to axial compression shall be considered.

## 6.4 LINEAR ELASTIC STRESS ANALYSIS

### 6.4.1 General

This clause applies to the linear elastic stress analysis of structures and parts of structures using numerical methods, including finite element analysis.

### 6.4.2 Analysis

The analysis shall conform with the requirements of Clause 6.1.1. The results of the analysis shall be interpreted in accordance with the requirements of Clause 6.1.2.

### **6.4.3 Sensitivity of analysis to input data and modelling parameters**

Checks shall be made to investigate the sensitivity of the results of a linear elastic stress analysis to variations in input data and modelling parameters.

## **6.5 NON-LINEAR FRAME ANALYSIS**

### **6.5.1 General**

This clause applies to the non-linear analysis of framed structures at service load, at overload, and at collapse. Non-linear analysis shall be carried out in accordance with the requirements of Clauses 6.1.1, 6.1.2 and 6.1.4.

### **6.5.2 Non-linear material effects**

The analysis shall take into account all relevant non-linear and inelastic effects in the materials, such as—

- (a) non-linear relationship between stress and strain for the reinforcement, the tendons and the concrete;
- (b) cracking of the concrete;
- (c) the tension stiffening effect in the concrete between adjacent tensile cracks;
- (d) creep and shrinkage of the concrete; and
- (e) relaxation of tendons.

### **6.5.3 Non-linear geometric effects**

Equilibrium of the structure in the deformed condition shall be considered whenever joint displacements or lateral deflections within the length of members significantly affect the action effects or overall structural behaviour.

### **6.5.4 Values of material properties**

When non-linear frame analysis is used as the basis for design, the calculations shall be undertaken using the mean values of all relevant material properties, such as concrete strength, initial elastic moduli and yield stress, and yield strain of the steel reinforcement and tendons.

Additional analysis shall be considered using other values of material properties to allow for variability of material properties and the effects of non-proportionality in non-linear analysis.

### **6.5.5 Sensitivity of analysis to input data and modelling parameters**

Checks shall be made to investigate the sensitivity of the results of a non-linear frame analysis to variations in input data and modelling parameters.

## **6.6 NON-LINEAR STRESS ANALYSIS**

### **6.6.1 General**

This clause applies to the non-linear stress analysis of structures and parts of structures using numerical methods, including finite element analysis.

### **6.6.2 Analysis**

The analysis shall conform with the requirements of Clause 6.1.1. The results of the analysis shall be interpreted in accordance with the requirements of Clause 6.1.2.

### 6.6.3 Non-linear material and geometric effects

The analysis shall take into account all relevant non-linear and inelastic effects, such as—

- (a) non-linear relation between stress and strain for the reinforcement, the tendons and the concrete;
- (b) cracking of the concrete;
- (c) the tension stiffening effect in the concrete between adjacent tensile cracks;
- (d) creep and shrinkage of the concrete;
- (e) relaxation of tendons; and
- (f) geometric non-linear effects.

### 6.6.4 Values of material properties

When non-linear stress analysis is used as the basis for design, the calculations shall be undertaken using the mean values of all relevant material properties chosen taking account of the effect of non-proportionality of the results, such as concrete strength, initial elastic moduli, and yield stress and yield strain of the steel reinforcement. Additional analysis shall be considered using other values of material properties to allow for variability.

### 6.6.5 Sensitivity of analysis to input data and modelling parameters

Checks shall be made to investigate the sensitivity of the results of a non-linear stress analysis to variations in input data and modelling parameters.

## 6.7 PLASTIC METHODS OF ANALYSIS

### 6.7.1 General

This clause applies to the plastic collapse analysis of frames and one-way and two-way slabs. The method may be used provided Ductility Class N reinforcement is used throughout for flexural reinforcement.

Where plastic methods are used in the strength design of structures, the reinforcement shall be arranged with due regard to the serviceability requirements of the structure.

### 6.7.2 Methods for beams and frames

Plastic methods of analysis may be used for the strength design of continuous beams and frames in accordance with Clause 2.2.2, provided it is shown that the high-moment regions possess sufficient moment-rotation capacity to achieve the plastic redistribution implied in the analysis.

### 6.7.3 Methods for slabs

#### 6.7.3.1 Lower-bound method for slabs

The design bending moments obtained using lower-bound theory shall satisfy the requirements of equilibrium and the boundary conditions applicable to the slab.

#### 6.7.3.2 Yield line method for slabs

A yield line analysis for strength design of a slab shall satisfy the following requirements:

- (a) The design bending moments shall be obtained from calculations based on the need for a mechanism to form over the whole or part of the slab at collapse.
- (b) The mechanism that gives rise to the most severe design bending moments shall be used for the design of the slab.

## 6.8 ANALYSIS USING STRUT-AND-TIE MODELS

### 6.8.1 General

When strut-and-tie modelling is used in the analysis of a concrete structure or local region, the relevant requirements of Section 7 shall be satisfied.

### 6.8.2 Sensitivity of analysis to input data and modelling parameters

Checks shall be made to investigate the sensitivity of the results of a strut-and-tie analysis to variations in geometry and modelling parameters.

## 6.9 IDEALIZED FRAME METHOD OF ANALYSIS

### 6.9.1 General

This clause applies to the analysis of multistorey buildings of reinforced concrete and prestressed concrete that can be represented as a framework of line members with a regular layout. The clause also applies to the analysis of framed structures with a regular layout incorporating two-way slab systems as specified in Clause 6.9.5.

### 6.9.2 Idealized frames

The building framework may be analysed as a series of idealized, approximately parallel, two-dimensional frames running in one main direction, and a second series of such frames running in the transverse direction.

Each idealized frame shall consist of the footings, the rows of vertical (or near-vertical) members and the horizontal (or near-horizontal) members they support at each floor level.

The analyses for vertical, horizontal and other loads shall be carried out for each idealized frame in accordance with Clause 6.2, 6.3 or 6.4 and the general requirements of Clauses 6.1.1 and 6.1.2.

The critical section for maximum negative bending moment in a floor of the idealized frame may be taken at 0.7 times the span support ( $a_{sup}$ ) from the centre-line of the support.

### 6.9.3 Analysis for vertical loads

The arrangement of vertical loads to be considered in the analysis of an idealized frame shall be in accordance with Clause 2.5.4. In the analysis of a frame for vertical loads, the frame may be analysed in its entirety. Alternatively, it shall be permissible to deal with one storey at a time, in accordance with the following:

- (a) To determine the moments and shears in a floor due to vertical loading, the floor together with the columns above and below may be isolated and analysed, the columns being assumed fixed at the remote ends. The bending moment and shear at a given support may be determined on the assumption the floor is fixed at the support one span away, provided that the floor continues beyond that point.
- (b) To determine the forces and moments in columns due to vertical loading, each level of columns may be considered together with the floors and columns above and below, the columns being assumed fixed against rotation and translation at their remote ends and the floors being assumed fixed at the adjacent supports.

Any change in length of the beams and slabs due to axial force and any deflection due to shear force may be neglected.

The effect of any change in length of columns, due to axial shortening on the actions in the floor system, shall be considered in the analysis.

In order to provide for imposed actions (live loads) acting on part of a span—

- (i) when the live load reduction factor  $\psi_a = 1$ , the minimum shear force due to imposed actions (live load) in any section of a member shall be taken as at least one quarter of the maximum shear force due to imposed actions (live load) in the member when subjected to uniformly distributed imposed actions (live loads); and
- (ii) when  $\psi_a < 1$ , the designer shall assess the maximum and minimum imposed actions (live loads) due to partial loading of a span.

#### **6.9.4 Analysis for horizontal loads**

In situ concrete floor slabs may be assumed to act as horizontal diaphragms that distribute lateral forces among the frames and walls. They shall be designed in accordance with Section 15.

The full idealized frame shall be considered in the analysis for horizontal loads, unless adequate restraint is provided (for example by bracing or shear walls).

#### **6.9.5 Idealized frame method for structures incorporating two-way slab systems**

##### **6.9.5.1 General**

Provided Ductility Class L reinforcement is not used as the main flexural reinforcement, this clause applies to the analysis of reinforced and prestressed framed structures incorporating two-way slab systems having multiple spans including—

- (a) solid slabs with or without drop panels;
- (b) slabs incorporating ribs in two directions, including waffle-slabs;
- (c) slabs having recessed soffits, if the portion of reduced thickness lies entirely within both middle strips;
- (d) slabs having openings conforming with the requirements of Clause 6.9.5.5; and
- (e) beam-and-slab systems, including thickened slab bands.

##### **6.9.5.2 Effective width**

The idealized frame consists of the footings, the columns and the slab floors acting as wide beams.

The effective width of the beams to be used in the analysis varies depending on span length and column size, and may be different for vertical and lateral loads. In the absence of more accurate calculations, the stiffness of horizontal flexural members at each floor level for a vertical load analysis may be based on a width—

- (a) for flat slabs, equal to the width of the design strip ( $L_1$ ); or
- (b) for T-beams and L-beams, calculated in accordance with Clause 8.8.2.

##### **6.9.5.3 Distribution of bending moments between column and middle strips**

In the idealized frame each beam (design strip) shall be divided into column strips and middle strips.

The column strip shall be designed to resist the total negative or positive bending moment at the critical cross-sections multiplied by an appropriate factor within the ranges given in Table 6.9.5.3.

That part of the design strip bending moment not resisted by the column strip shall be proportionally assigned to the half-middle strips on either side of it.

Each middle strip shall be designed to resist the sum of the moments assigned to its two adjoining halves, except that a middle strip adjacent to and parallel with an edge supported by a wall shall be designed to resist twice the bending moment assigned to the adjoining half-middle strip from the next interior design strip parallel to the wall.

**TABLE 6.9.5.3**  
**DISTRIBUTION OF BENDING MOMENTS TO THE COLUMN STRIP**

Bending moment under consideration	Strength limit state	Serviceability limit state
Negative moment at an interior support	0.60 to 1.00	0.75
Negative moment at an exterior support with spandrel beam	0.75 to 1.00	0.75
Negative moment at an exterior support without spandrel beam	0.75 to 1.00	1.0
Positive moment at all spans	0.50 to 0.70	0.6

#### **6.9.5.4 Torsional moments**

Where moment is transferred to the column by torsional moment in the slab or spandrel beams, the slab or spandrel beams shall be designed in accordance with Clause 9.2 or Clause 8.3, as applicable.

In beam-and-slab construction, the spandrel beams shall be reinforced with at least the minimum torsional reinforcement required by Clause 8.3.3.

#### **6.9.5.5 Openings in slabs**

Slabs containing openings may be analysed in accordance with all of Clause 6.9.5 without the need for further calculation provided the amount of reinforcement interrupted by the opening is distributed to each side of the opening and the plan dimensions of the opening are no larger than the following:

- (a) The width of each middle strip, in the area common to two middle strips.
- (b) One quarter of the width of each strip, in the area common to a column strip and a middle strip.
- (c) One eighth of the width of each column strip, in the area common to two column strips, provided the reduced section is capable of transferring the moment and shear forces to the support. The slab shall also conform with the shear requirements of Clause 9.2.

### **6.10 SIMPLIFIED METHODS OF FLEXURAL ANALYSIS**

#### **6.10.1 General**

In lieu of more detailed structural analysis, it is permissible to design reinforced concrete beams and slabs for strength in accordance with the requirements of Clause 6.10.2, 6.10.3 or 6.10.4, as appropriate.

#### **6.10.2 Simplified method for reinforced continuous beams and one-way slabs**

##### **6.10.2.1 Application**

Clauses 6.10.2.2 to 6.10.2.4 may be used for the calculation of design bending moments and shear forces for strength in continuous beams and one-way slabs of reinforced concrete construction, provided—

- (a) the ratio of the longer to the shorter length of any two adjacent spans does not exceed 1.2;
- (b) the loads are essentially uniformly distributed;

- (c) the imposed action (live load) ( $q$ ) does not exceed twice the permanent action (dead load) ( $g$ );
- (d) members are of uniform cross-section;
- (e) the reinforcement is arranged in accordance with the requirements of Clause 8.1.10.6 or Clause 9.1.3.2; and
- (f) bending moments at supports are caused only by the action of loads applied to the beam or slab.

#### 6.10.2.2 Negative design moment

The negative design moment at the critical section, taken for the purpose of this clause at the face of the support, shall be as follows (where  $F_d$  is the uniformly distributed design load per unit length, factored for strength):

- (a) At the first interior support:
  - (i) Two spans only for Ductility Class N .....  $F_d L_n^2 / 9$ ; or  
for Ductility Class L .....  $F_d L_n^2 / 8$ .
  - (ii) More than two spans .....  $F_d L_n^2 / 10$ .
- (b) At other interior supports .....  $F_d L_n^2 / 11$ .
- (c) At interior faces of exterior supports for members built integrally with their supports:
  - (i) For beams where the support is a column .....  $F_d L_n^2 / 16$ .
  - (ii) For slabs and beams where the support is a beam .....  $F_d L_n^2 / 24$ .

#### 6.10.2.3 Positive design moment

The positive design moment shall be taken as follows (where  $F_d$  is the uniformly distributed design load per unit length, factored for strength):

- (a) In an end span .....  $F_d L_n^2 / 11$ .
- (b) In interior spans for Ductility Class N .....  $F_d L_n^2 / 16$ ; or  
for Ductility Class L .....  $F_d L_n^2 / 14$ .

#### 6.10.2.4 Transverse design shear force

The transverse design shear force in a member shall be taken as follows (where  $F_d$  is the uniformly distributed design load per unit length, factored for strength):

- (a) In an end span:
  - (i) At the face of the interior support .....  $1.15 F_d L_n / 2$ .
  - (ii) At midspan .....  $F_d L_n / 7$ .
  - (iii) At the face of the end support .....  $F_d L_n / 2$ .
- (b) In interior spans:
  - (i) At the face of supports .....  $F_d L_n / 2$ .
  - (ii) At midspan .....  $F_d L_n / 8$ .



### 6.10.3 Simplified method for reinforced two-way slabs supported on four sides

#### 6.10.3.1 General

The design bending moments and shear forces for strength in reinforced two-way simply supported or continuous rectangular slabs, which are supported by walls or beams on four sides and have corners that are prevented from lifting, may be determined from Clauses 6.10.3.2 to 6.10.3.4 provided—

- (a) the loads are essentially uniformly distributed;
- (b) the reinforcement is arranged in accordance with the requirements of Clause 9.1.3.3;
- (c) bending moments at supports are caused only by the action of loads applied to the beam or slab;
- (d) any opening or penetration through the thickness of a slab does not adversely affect strength or stiffness; and
- (e) slabs incorporating Ductility Class L reinforcement are continuously supported on walls.

#### 6.10.3.2 Design bending moments

The design bending moments in a slab shall be determined as follows:

- (a) The positive design bending moments at midspan,  $M_x^*$  and  $M_y^*$  on strips of unit width spanning  $L_x$  and  $L_y$ , respectively, shall be calculated from the following equations:

$$M_x^* = \beta_x F_d L_x^2 \quad \dots 6.10.3.2(1)$$

$$M_y^* = \beta_y F_d L_y^2 \quad \dots 6.10.3.2(2)$$

where  $F_d$  is the uniformly distributed design load per unit area factored for strength, and  $\beta_x$  and  $\beta_y$  are given in—

- (i) Table 6.10.3.2(A) for slabs with Ductility Class N reinforcement as the main flexural reinforcement; and
- (ii) Table 6.10.3.2(B) for slabs with Ductility Class N reinforcement or Ductility Class L mesh as the main flexural reinforcement, no moment redistribution can be accommodated at either the serviceability or strength limit states.

The moments, so calculated, shall apply over a central region of the slab equal to three-quarters of  $L_x$  and  $L_y$  respectively. Outside of this region, the requirement for strength shall be deemed to conform with the minimum strength requirement of Clause 9.1.1.

- (b) The negative design bending moments at a continuous edge shall be taken as—

- (i) 1.33 times the midspan values in the direction considered, when they are taken from Table 6.10.3.2(A); or
- (ii)  $\alpha_x$  or  $\alpha_y$  times the midspan values in the direction considered when they are taken from Table 6.10.3.2(B), where the value of  $\alpha_x$  or  $\alpha_y$  is also taken from Table 6.10.3.2(B) for the appropriate case.

If the negative moment on one side of a common support is different from that on the other side—

- (A) the unbalanced moment may be redistributed (in proportion to the stiffness of span  $L_x$  in the adjacent panels) if Ductility Class N reinforcement is the main flexural reinforcement; or
  - (B) the slab shall be reinforced on both sides of the support for the larger support moment.
- (c) The negative design bending moment at a discontinuous edge, where there is a likelihood of restraint, may be taken as—
- (i) 0.5 times the midspan values in the direction considered when they are taken from Table 6.10.3.2(A); or
  - (ii) 0.8 times the midspan values in the direction considered when they are taken from Table 6.10.3.2(B).

**TABLE 6.10.3.2(A)**  
**BENDING MOMENT COEFFICIENTS FOR RECTANGULAR SLABS**  
**SUPPORTED ON FOUR SIDES (DUCTILITY CLASS N REINFORCEMENT)**

Edge condition		Short span coefficients ( $\beta_x$ )								Long span coefficients ( $\beta_y$ ) for all values of $L_y/L_x$
		Values of $L_y/L_x$								
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	$\geq 2.0$	
1	Four edges continuous	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
2	One short edge discontinuous	0.028	0.032	0.036	0.038	0.041	0.043	0.047	0.050	0.028
3	One long edge discontinuous	0.028	0.035	0.041	0.046	0.050	0.054	0.061	0.066	0.028
4	Two short edges discontinuous	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
5	Two long edges discontinuous	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
6	Two adjacent edges discontinuous	0.035	0.041	0.046	0.051	0.055	0.058	0.065	0.070	0.035
7	Three edges discontinuous (one long edge continuous)	0.043	0.049	0.053	0.057	0.061	0.064	0.069	0.074	0.043
8	Three edges discontinuous (one short edge continuous)	0.043	0.054	0.064	0.072	0.078	0.084	0.096	0.105	0.043
9	Four edges discontinuous	0.056	0.066	0.074	0.081	0.087	0.093	0.103	0.111	0.056

**TABLE 6.10.3.2(B)**  
**BENDING MOMENT COEFFICIENTS FOR RECTANGULAR SLABS**  
**SUPPORTED ON FOUR SIDES (DUCTILITY CLASS N**  
**OR DUCTILITY CLASS L REINFORCEMENT)**

Edge condition			Short span coefficients ( $\beta_x$ and $\alpha_x$ )									Long span coefficients ( $\beta_y$ and $\alpha_y$ ) for all values of $L_y/L_x$
			Values of $L_y/L_x$									
			1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	>2.0	
1	Four edges continuous	$\beta$	0.021	0.025	0.029	0.032	0.034	0.036	0.039	0.041	0.042	0.020
		$\alpha$	2.31	2.22	2.14	2.10	2.06	2.03	2.00	2.00	2.00	2.69
2	One short edge discontinuous	$\beta$	0.027	0.030	0.033	0.035	0.037	0.039	0.041	0.042	0.042	0.024
		$\alpha$	2.20	2.14	2.10	2.06	2.04	2.02	2.00	2.00	2.00	2.29
3	One long edge discontinuous	$\beta$	0.024	0.028	0.034	0.038	0.043	0.047	0.056	0.061	0.070	0.028
		$\alpha$	2.22	2.17	2.09	2.03	1.97	1.93	1.86	1.81	1.80	2.46
4	Two short edges discontinuous	$\beta$	0.032	0.035	0.037	0.038	0.039	0.040	0.042	0.042	0.042	0.024
		$\alpha$	2.09	2.05	2.03	2.01	2.00	2.00	2.00	2.00	2.00	—
5	Two long edges discontinuous	$\beta$	0.024	0.028	0.035	0.042	0.049	0.056	0.071	0.085	0.125	0.039
		$\alpha$	—	—	—	—	—	—	—	—	—	2.31
6	Two adjacent edges discontinuous	$\beta$	0.031	0.036	0.041	0.046	0.050	0.053	0.060	0.064	0.070	0.034
		$\alpha$	2.13	2.07	2.01	1.96	1.92	1.89	1.83	1.80	1.80	2.13
7	Three edges discontinuous (one long edge continuous)	$\beta$	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.066	0.070	0.035
		$\alpha$	2.04	1.97	1.93	1.89	1.86	1.84	1.80	1.80	1.80	—
8	Three edges discontinuous (one short edge continuous)	$\beta$	0.033	0.039	0.047	0.054	0.061	0.067	0.082	0.093	0.125	0.046
		$\alpha$	—	—	—	—	—	—	—	—	—	2.12
9	Four edges discontinuous	$\beta$	0.044	0.052	0.059	0.066	0.073	0.079	0.091	0.100	0.125	0.049
		$\alpha$	—	—	—	—	—	—	—	—	—	—

### 6.10.3.3 Torsional moment at exterior corners

The torsional moment at the exterior corners of a slab shall be deemed to be resisted by conforming with the requirements of Clause 9.1.3.3, Item (e).

### 6.10.3.4 Load allocation

For calculating shear forces in the slab or the forces applied to the supporting walls or beams in the absence of more accurate calculations, it may be assumed that the uniformly distributed load on the slab is allocated to the supporting beams or walls as shown in Figure 6.10.3.4, provided—

- the reactions apply directly when all edges are continuous;
- when one edge is discontinuous, the reactions on all continuous edges are increased by 10% and the reaction on the discontinuous edge can be reduced by 20%; and
- when adjacent edges are discontinuous, the reactions are adjusted for elastic shear considering each span separately.

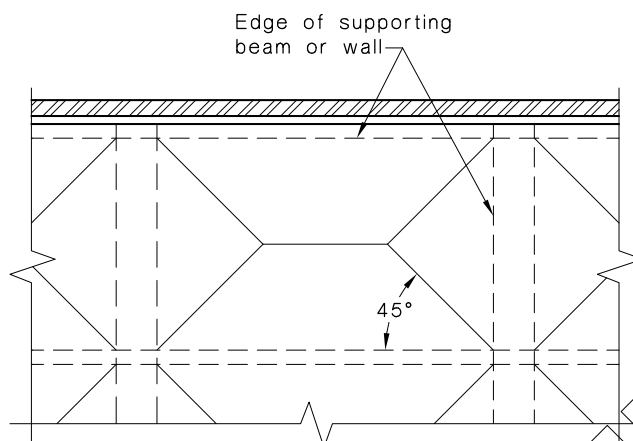


FIGURE 6.10.3.4 ALLOCATION OF LOAD

#### 6.10.4 Simplified method for reinforced two-way slab systems having multiple spans

##### 6.10.4.1 General

For multiple-span reinforced two-way slab systems, including solid slabs with or without drop panels, slabs incorporating ribs in two directions (waffle slabs) and beam-and-slab systems including thickened-slab bands, bending moments and shear forces in both directions may be determined in accordance with this clause provided—

- (a) there are at least two continuous spans in each direction;
- (b) the support grid is rectangular, except that individual supports may be offset up to a maximum of 10% of the span in the direction of the offset;
- (c) in any portion of the slab enclosed by the centre-lines of its supporting members, the ratio of the longer span to the shorter span is not greater than 2.0;
- (d) in the design strips in each direction, successive span lengths do not differ by more than one third of the longer span and in no case is an end-span longer than the adjacent interior span;
- (e) lateral forces on the structure are resisted by shear walls or braced frames;
- (f) vertical loads are essentially uniformly distributed;
- (g) the imposed action (live load) ( $q$ ) does not exceed twice the permanent action (dead load) ( $g$ );
- (h) the reinforcement is arranged in accordance with Clause 9.1.3.4 or Clause 8.1.11.6, as applicable; and
- (i) Ductility Class L reinforcement is not used as the flexural reinforcement.

##### 6.10.4.2 Total static moment for a span

The total static moment ( $M_o$ ) for a span of the design strip shall be taken as not less than—

$$M_o = \frac{F_d L_t L_o^2}{8} \quad \dots 6.10.4.2$$

where

$F_d$  = uniformly distributed design load per unit area, factored for strength

$L_t$  = width of the design strip

$L_o = L$  minus 0.7 times the sum of the values of  $a_{sup}$  at each end of the span  
[see Figure 6.1.4(B)]

### 6.10.4.3 Design moments

The design moments in a span shall be determined by multiplying the total static moment ( $M_o$ ) by the relevant factor given in Tables 6.10.4.3(A) or 6.10.4.3(B), as appropriate.

These design moments may be modified by up to 10% provided the total static moment ( $M_o$ ) for the span in the direction considered is not reduced.

The section under negative moment shall be designed to resist the larger of the two interior negative design moments determined for the spans framing into a common support, unless an analysis is made to distribute the unbalanced moment in accordance with the stiffness of the adjoining members.

**TABLE 6.10.4.3(A)**  
**DESIGN MOMENT FACTORS FOR AN END-SPAN**

Type of slab system and edge rotation restraint	Exterior negative moment factor	Positive moment factor	Interior negative moment factor
Flat slabs with exterior edge unrestrained	0.0	0.60	0.80
Flat slabs with exterior edge restrained by columns only	0.25	0.50	0.75
Flat slabs with exterior edge restrained by spandrel beams and columns	0.30	0.50	0.70
Flat slabs with exterior edge fully restrained	0.65	0.35	0.65
Beam-and-slab construction	0.15	0.55	0.75

**TABLE 6.10.4.3(B)**  
**DESIGN MOMENT FACTORS FOR AN INTERIOR SPAN**

Type of slab system	Negative moment factor	Positive moment factor
All types	0.65	0.35

### 6.10.4.4 Transverse distribution of the design bending moment

The design negative and positive bending moments shall be distributed to the column strip and middle strip in accordance with Clause 6.9.5.3.

### 6.10.4.5 Moment transfer for shear in flat slabs

For the purpose of shear design, the bending moment, transferred from the slab to the support ( $M_v^*$ ), shall be taken as the unbalanced bending moment at that support.

At an interior support,  $M_v^*$  shall be taken as not less than—

$$0.06 [(1.2g + 0.75q) L_t (L_o)^2 - 1.2g L_t (L_o')^2] \quad \dots 6.10.4.5$$

where

$L_o' =$  smaller value of  $L_o$  for the adjoining spans

At an exterior support, the actual moment shall be taken.

**6.10.4.6** *Shear forces in beam-and-slab construction*

In beam-and-slab construction, the shear forces in the supporting beams may be determined by using the allocation of load given in Clause 6.10.3.4.

**6.10.4.7** *Openings in slabs*

Only openings that conform with the requirements of Clause 6.9.5.5(a) and Clause 6.9.5.5(b) shall be permitted in slabs.

DRAFT

## SECTION 7 STRUT-AND-TIE MODELLING

### 7.1 GENERAL

It shall be permissible to use strut-and-tie models to represent the conditions at overload and at failure in non-flexural members and in non-flexural regions of members, as a basis both for strength design and for evaluating strength.

A strut-and-tie model shall consist of compression elements (struts) and tension elements (ties) that are connected together at nodes to form a load-resisting structural system.

Strut-and-tie models shall satisfy the following requirements:

- Loads shall be applied at nodes, and the struts and ties shall be subjected only to axial force.
- The model shall provide load paths to carry the loads and other actions to the supports or into adjacent regions.
- The model shall be in equilibrium with the applied loads and the reactions.
- In determining the geometry of the model, the dimensions of the struts, ties, and nodal zones shall be taken into account.
- Ties shall be permitted to cross struts.
- Struts shall cross or intersect only at nodes.
- For reinforced concrete members at a node point, the angle between the axes of any strut and any tie shall be not less than 30°.
- For prestressed concrete members at a node point, the angle between the axes of any strut and any tie with a tendon acting as the reinforcement shall be not less than 20°.

### 7.2 CONCRETE STRUTS

#### 7.2.1 Types of struts

Struts shall be of prismatic, fan or bottle shape, depending on the geometry of the compression field, as shown in Figure 7.2.1. Prismatic struts shall be used only where the compressive stress field cannot diverge.

#### 7.2.2 Strut efficiency factor

For prismatic struts, the strut efficiency factor ( $\beta_s$ ) that is used to determine the design strength shall be taken as 1.0.

For fan-shaped and bottle-shaped compression fields that are unconfined, the strut efficiency factor shall be taken as—

$$\beta_s = \frac{1}{1.0 + 0.66 \cot^2 \theta} \quad (\text{within the limits } 0.3 \leq \beta_s \leq 1.0) \quad \dots 7.2.2$$

The angle ( $\theta$ ) is measured between the axis of the strut and the axis of a tie passing through a common node (see Figure 7.2.2). Where more than one tie passes through a node, or where the angle ( $\theta$ ) is different for nodes at each end of a strut, the smallest value of  $\theta$  shall be used in determining  $\beta_s$ .

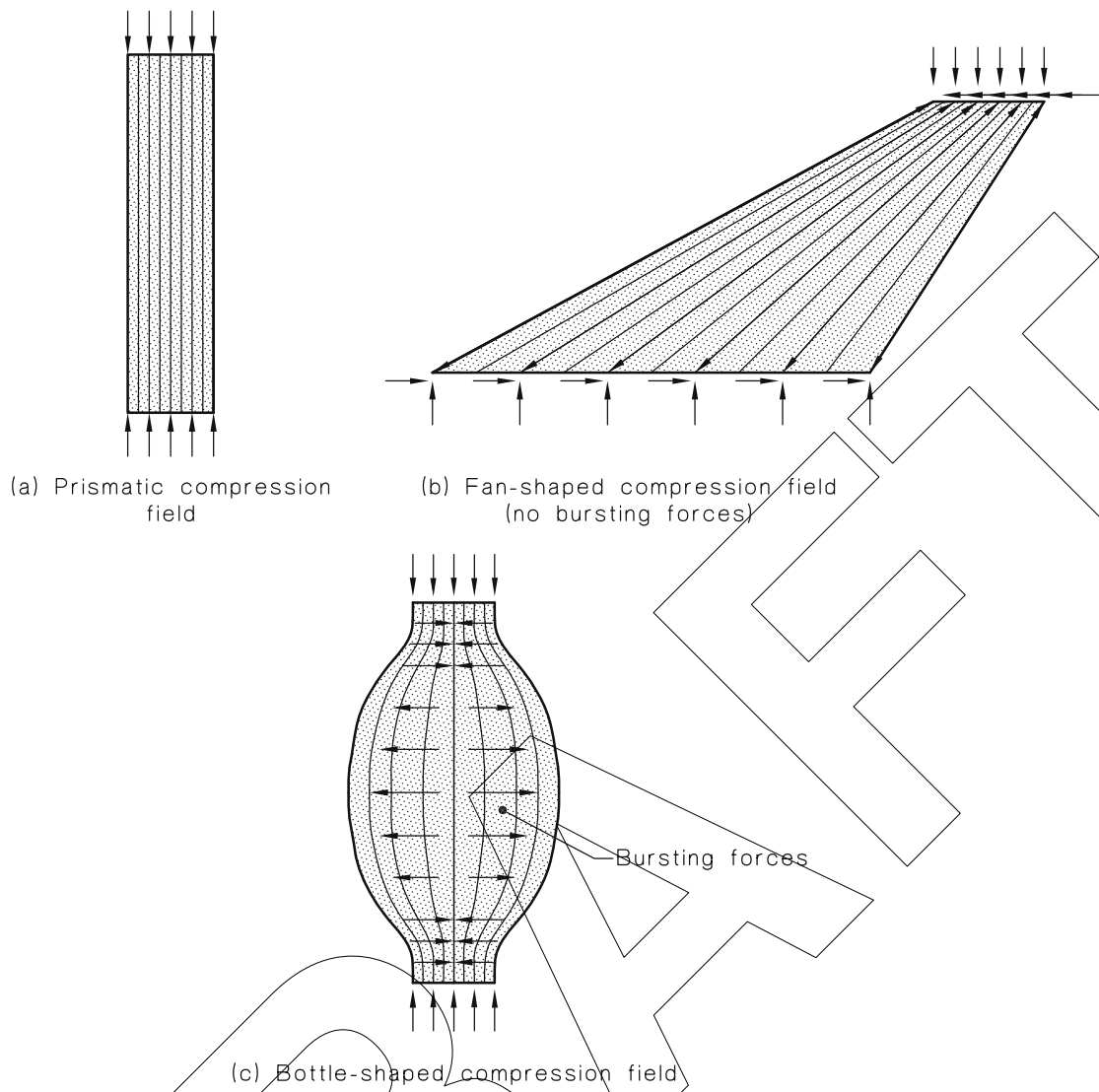
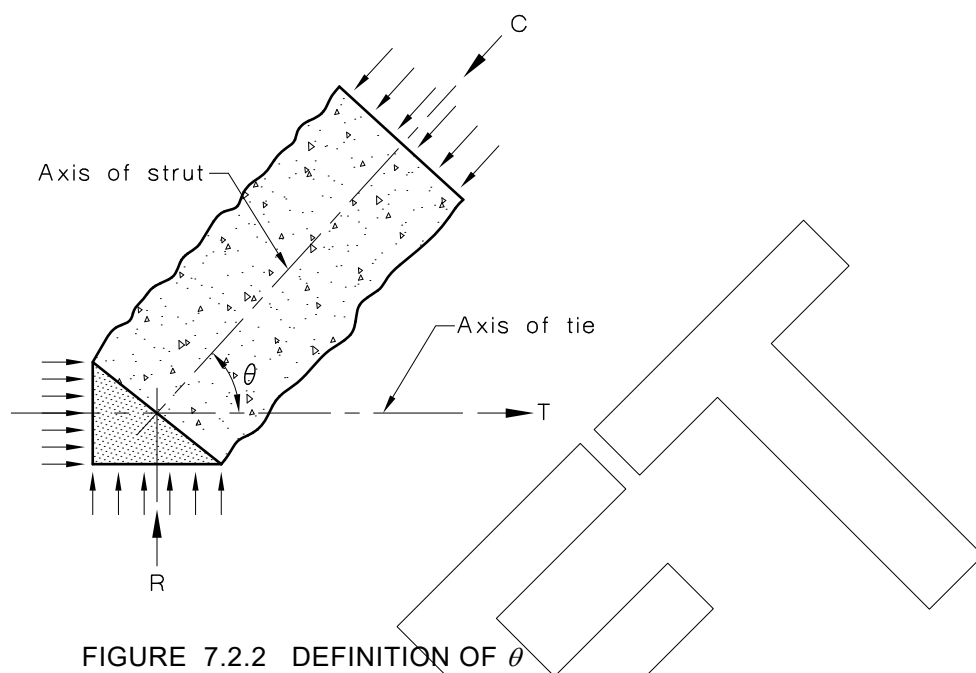


FIGURE 7.2.1 TYPES OF STRUTS



FIGURE 7.2.2 DEFINITION OF  $\theta$ 

### 7.2.3 Design strength of struts

The design strength of a concrete strut shall be taken as—

$$\phi_{st} \beta_s 0.9 f'_c A_c$$

... 7.2.3

where

$A_c$  = smallest cross-sectional area of the concrete strut at any point along its length and measured normal to the line of action of the strut

$\beta_s$  = an efficiency factor given in Clause 7.2.2

The value of the strength reduction factor ( $\phi_{st}$ ) shall be obtained from Table 2.2.4.

Longitudinal reinforcement may be used to increase the strength of a strut. Such reinforcement shall be placed parallel to the axis of the strut, located within the strut and enclosed in ties or spirals satisfying Clause 10.7. The longitudinal reinforcement shall be properly anchored. The strength of a longitudinally reinforced strut may be calculated as for a prismatic, pin-ended short column of similar geometry.

### 7.2.4 Bursting reinforcement in bottle-shaped struts

The design bursting force at both the serviceability limit state ( $T_{b,s}^*$ ) and ultimate limit state ( $T_b^*$ ) shall be calculated using an equilibrium model consistent with the bottle shape shown in Figure 7.2.4(A). The divergence angle ( $\alpha$ ) for the bottle-shaped strut shall be assessed for each situation but shall be not less than—

- (a)  $\tan \alpha = 1/2$  ..... for serviceability; and
- (b)  $\tan \alpha = 1/5$  ..... for strength.

The bursting force across the strut at cracking shall be taken as—

$$T_{b,cr} = 0.7 b l_b f'_{ct} \quad \dots 7.2.4(1)$$

where

$b$  = width of rectangular cross section or member

$l_b$  = length of the bursting zone [see Figure 7.2.4(A)]

If the calculated bursting force ( $T_b^*$ ) is greater than  $0.5T_{b,cr}$ , with  $\tan \alpha$  taken as  $\frac{1}{2}$ , then transverse reinforcement shall be provided in either—

- (i) two orthogonal directions at angles  $\gamma_1$  and  $\gamma_2$  to the axis of the strut [see Figure 7.2.4(B)]; or
- (ii) one direction at an angle  $\gamma_1$  to the axis of the strut, where  $\gamma_1$  shall be not less than  $40^\circ$  and shall satisfy the following—

(A) for serviceability

$$\Sigma A_{si} f_{si} \sin \gamma_i \geq \max(T_{b,s}^*, T_{b,cr}^*) \quad \dots 7.2.4(2)$$

(B) for strength

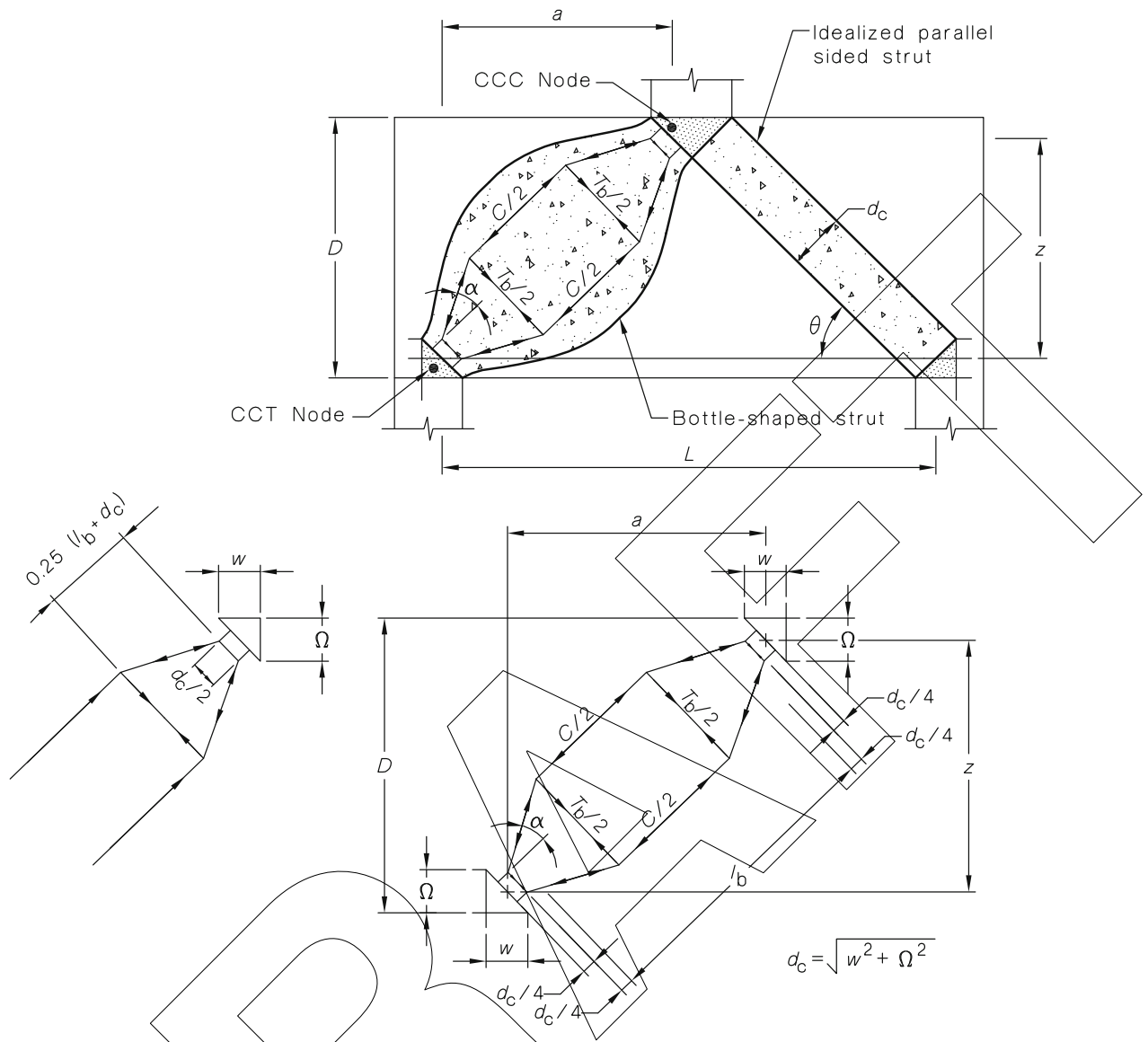
$$\phi_{st} \Sigma A_{si} f_{sy} \sin \gamma_i \geq T_b^* \quad \dots 7.2.4(3)$$

In the above expressions,  $A_{si}$  is the area of reinforcement in directions 1 and 2 crossing a strut at an angle  $\gamma_1$  to the axis of the strut [see Figure 7.2.4(B)] and  $f_{si}$  is the serviceability limit stress in the reinforcement as specified in Clause 12.7.

The transverse reinforcement shall be evenly distributed throughout the length of the bursting zone ( $l_b$ ), which is given by—

$$l_b = \sqrt{z^2 + a^2} - d_c \quad \dots 7.2.4(4)$$

and  $a$ ,  $d_c$  and  $z$  are the shear span, the width of the idealized strut, and the projection of the inclined compressive strut normal to the shear span respectively [see Figure 7.2.4(A)].



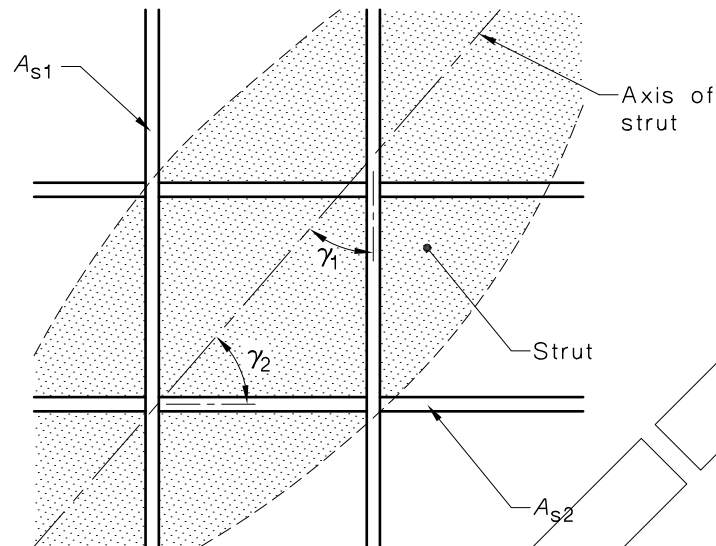


FIGURE 7.2.4(B) BURSTING REINFORCEMENT

### 7.3 TIES

#### 7.3.1 Arrangement of ties

Ties shall consist of reinforcing steel and/or prestressing tendons. The reinforcement and/or tendons shall be evenly distributed across the nodal regions at each end of the tie, and arranged such that the resultant tensile force coincides with the axis of the tie in the strut-and-tie model.

#### 7.3.2 Design strength of ties

The design strength of a tie shall be taken as  $\phi_{st} [A_{st}f_{sy} + A_p(\sigma_{p,ef} + \Delta\sigma_p)]$  where  $(\sigma_{p,ef} + \Delta\sigma_p)$  shall not exceed  $f_{py}$ . The value of  $\phi_{st}$  shall be obtained from Table 2.2.4.

#### 7.3.3 Anchorage of ties

To provide adequate anchorage at each end of the tie, the reinforcement or tendon shall be extended beyond the node to achieve the design strength of the tie at the node and anchored in accordance with Clause 13.1. At least 50% of the development length shall extend beyond the nodal zone.

Alternatively, anchorage of reinforcement may be achieved by a welded or mechanical anchorage, located entirely beyond the nodal zone.

### 7.4 NODES

#### 7.4.1 Types of nodes

Three types of node are distinguished by the arrangement of the entering struts and ties, and the confinement thus provided, as follows:

- (a) CCC—there are only struts entering the node.
- (b) CCT—there are two or more struts and a single tension tie entering the node.
- (c) CTT—there are two or more tension ties entering the node.

### 7.4.2 Design strength of nodes

Where confinement is not provided to the nodal region, the design strength of the node shall be such that the principal compressive stress on any nodal face, determined from the normal and shear stresses on that face, is not greater than  $\phi_{st}\beta_n 0.9f'_c$  where—

- (a) for CCC nodes  $\beta_n = 1.0$ ; or
- (b) for CCT nodes  $\beta_n = 0.8$ ; or
- (c) for CTT nodes  $\beta_n = 0.6$ .

The value of the strength reduction factor ( $\phi_{st}$ ) shall be taken from Table 2.2.4.

Where confinement is provided to the nodal region, the design strength of the node may be determined by tests or calculation, considering the confinement, but shall not exceed a value corresponding to a maximum compressive principal stress on any face of  $\phi_{st}1.8f'_c$ .

## 7.5 ANALYSIS OF STRUT-AND-TIE MODELS

In the analysis of a strut-and-tie model to determine the internal forces in the struts and ties, the requirements of Clause 6.1.1 shall be satisfied, and Clauses 6.1.2 and 6.8.2 shall be complied with.

## 7.6 DESIGN BASED ON STRUT-AND-TIE MODELLING

### 7.6.1 Design for strength

When strut-and-tie modelling is used for strength design, the requirements of Clause 2.2.4 shall be satisfied.

### 7.6.2 Serviceability checks

When design for strength is based on strut-and-tie modelling, separate checks shall be undertaken to ensure that the design requirements for serviceability are satisfied.

## SECTION 8 DESIGN OF BEAMS FOR STRENGTH AND SERVICEABILITY

### 8.1 STRENGTH OF BEAMS IN BENDING

#### 8.1.1 General

The strength of a beam cross-section under bending shall be determined using Clauses 8.1.2 to 8.1.10 and the material properties given in Section 3. Beam properties for T-beams and L-beams shall be as given in Clause 8.8.

This clause does not apply to non-flexural members covered by Section 12.

#### 8.1.2 Basis of strength calculations

Calculations for strength of cross-sections in bending shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following assumptions:

- (a) Plane sections normal to the axis remain plane after bending, except for unbonded tendons (see Clause 8.1.8).
- (b) The concrete has no tensile strength.
- (c) The distribution of compressive stress is determined from a stress-strain relationship for the concrete in accordance with Clause 3.1.4 (see Note 1).
- (d) The strain in compressive reinforcement does not exceed 0.003.

NOTES:

- 1 If a curvilinear stress-strain relationship is used, then—
  - (a) Clause 3.1.4 places a limit on the value of the maximum concrete stress; and
  - (b) the strain in the extreme compression fibre may be adjusted to obtain the maximum bending strength.
- 2 These rules apply to reinforced and bonded prestressed concrete members.

#### 8.1.3 Rectangular stress block

Clause 8.1.2 shall be deemed to be satisfied for the concrete assuming that—

- (a) the maximum strain in the extreme compression fibre is taken as 0.003; and
- (b) a uniform compressive stress of  $\alpha_2 f'_c$  acts on an area bounded by—
  - (i) the edges of the cross-section; and
  - (ii) a line parallel to the neutral axis under the loading concerned, and located at a distance  $\gamma d$  from the extreme compressive fibre, where—
 

$$\alpha_2 = 0.85 - 0.0015 f'_c \quad \dots \text{within the limit of } \alpha_2 \geq 0.67 \quad \dots 8.1.3(1)$$

$$\gamma = 0.97 - 0.0025 f'_c \quad \dots \text{within the limit of } \gamma \geq 0.67 \quad \dots 8.1.3(2)$$

NOTE: The modification of  $0.9 f'_c$  given in Clause 3.1.4 is included in the rectangular stress block assumptions.

#### 8.1.4 Dispersion angle of prestress

In the absence of a more exact calculation, the dispersion angle of the prestressing force from the anchorage shall be assumed to be  $60^\circ$  (i.e.  $30^\circ$  either side of the centre-line).

### 8.1.5 Design strength in bending

The design strength in bending of a section shall be taken as not greater than  $\phi M_{uo}$ , where  $\phi$  is determined from Item (b) of Table 2.2.2.

Sections with  $k_{uo}$  greater than 0.36 and where  $M^* > 0.8\phi M_{uo}$  shall be used only when—

- the structural analysis is carried out in accordance with Clauses 6.2 to 6.6; and
- compressive reinforcement of at least 0.01 times the area of concrete in compression is used and restrained as specified in Clauses 8.1.11.7 and 8.1.11.8, as appropriate.

### 8.1.6 Minimum strength requirements

#### 8.1.6.1 General

The ultimate strength in bending ( $M_{uo}$ ) at critical sections shall be not less than  $(M_{uo})_{min}$ , the minimum required strength in bending at a critical cross-section, and is given by—

$$(M_{uo})_{min} = 1.2 [Z(f'_{ct.f} + P_e / A_g) + P_e e] \quad \dots 8.1.6.1(1)$$

where

$Z$  = section modulus of the uncracked cross-section, referred to the extreme fibre at which flexural cracking occurs

$f'_{ct.f}$  = characteristic flexural tensile strength of the concrete

$P_e$  = total effective prestress force allowing for all losses of prestress

$e$  = eccentricity of the prestressing force ( $P_e$ ), measured from the centroidal axis of the uncracked section

This requirement may be waived at critical sections of a statically indeterminate member, provided it can be demonstrated this will not lead to sudden collapse of a span or a reduced collapse load.

For reinforced concrete cross-sections, this requirement shall be deemed to be satisfied for the direction of bending being considered if tensile reinforcement ( $A_{st}$ ) is provided such that—

$$A_{st} \geq [\alpha_b (D/d)^2 f'_{ct.f} / f_{sy}] b_w d \quad \dots 8.1.6.1(2)$$

where

For rectangular sections:

$$\alpha_b = 0.20$$

For T-sections and L-sections with the web in tension:

$$\alpha_b = 0.20 + \left( \frac{b_{ef}}{b_w} - 1 \right) \left( 0.4 \frac{D_s}{D} - 0.18 \right) \geq 0.20 \left( \frac{b_{ef}}{b_w} \right)^{1/4}$$

For T-sections and L-sections with the flange in tension:

$$\alpha_b = 0.20 + \left( \frac{b_{ef}}{b_w} - 1 \right) \left( 0.25 \frac{D_s}{D} - 0.08 \right) \geq 0.20 \left( \frac{b_{ef}}{b_w} \right)^{2/3}$$

### 8.1.6.2 Prestressed beams at transfer

The strength of a prestressed beam at transfer shall be checked using the load combinations specified in Clause 2.4 and a strength reduction factor ( $\phi$ ) for the section of 0.6.

This requirement shall be deemed to be satisfied if the maximum compressive stress in the concrete, under the loads at transfer, does not exceed—

- (a) for a section that is rectangular in cross section and where the stress distribution is triangular in shape,  $0.6f_{cp}$ ; otherwise
- (b)  $0.5f_{cp}$ .

### 8.1.7 Stress in reinforcement and bonded tendons at ultimate strength

The stress in the reinforcement at ultimate strength shall be taken as not greater than  $f_{sy}$ .

In the absence of a more accurate calculation and provided the minimum effective stress in the tendons is not less than  $0.5f_{pb}$ , the maximum stress that would be reached in bonded tendons at ultimate strength ( $\sigma_{pu}$ ) shall be taken as—

$$\sigma_{pu} = f_{pb} \left( 1 - \frac{k_1 k_2}{\gamma} \right) \quad \dots 8.1.7(1)$$

where

$k_1 = 0.4$  generally; or

if  $f_{py}/f_{pb} \geq 0.9$ ,

$k_1 = 0.28$ ; and

$$k_2 = \frac{1}{b_{ef} d_p f'_c} [A_{pt} f_{pb} + (A_{st} - A_{sc}) f_{sy}]$$

Compressive reinforcement may be taken into account only if  $d_{sc}$ , the distance from the extreme compressive fibre of the concrete to the centroid of compressive reinforcement, is not greater than  $0.15d_p$ , in which case  $k_2$  shall be taken as not less than 0.17.

### 8.1.8 Stress in tendons not yet bonded

Where the tendon is not yet bonded, the stress in the tendon at ultimate strength ( $\sigma_{pu}$ ) shall be determined from the formula given in Item (a) below if the span-to-depth ratio is 35 or less, or from the formula given in Item (b) below if the span-to-depth ratio is greater than 35, but in no case shall  $\sigma_{pu}$  be taken greater than  $f_{py}$ :

$$(a) \quad \sigma_{pu} = \sigma_{p,ef} + 70 + \frac{f'_c b_{ef} d_p}{100 A_{pt}} \leq \sigma_{p,ef} + 400 \quad \dots 8.1.8(1)$$

$$(b) \quad \sigma_{pu} = \sigma_{p,ef} + 70 + \frac{f'_c b_{ef} d_p}{300 A_{pt}} \leq \sigma_{p,ef} + 200 \quad \dots 8.1.8(2)$$

where  $\sigma_{p,ef}$  is the effective stress in the tendon after allowing for all losses.

### 8.1.9 Spacing of reinforcement and tendons

The minimum clear distance between parallel bars (including bundles of bars), ducts and tendons shall be such that the concrete can be properly placed and compacted in accordance with Clause 17.1.3. The maximum spacing of longitudinal reinforcement and tendons shall be determined in accordance with Clause 8.6.1(b).



### 8.1.10 Maximum diameter of longitudinal beam bars in internal beam/column joint zones

For nominally ductile structures the maximum diameter of longitudinal beam bars passing through beam column joint zones shall not exceed the appropriate requirement given below for internal beam column joints:

- (a) Where the critical load combination for flexure in a beam at the face of an internal column includes earthquake actions the ratio of bar diameter to column depth,  $d_b/h_c$ , shall be limited to:

$$d_b / h_c \leq 4 \alpha_f \sqrt{f'_c / f_{sy}} \quad \dots 8.1.10(1)$$

where  $\alpha_f$  is taken as 0.85 where the beam bar passes through a joint in a two-way frame and as 1.0 for a joint in a one-way frame.

- (b) Where the critical load combination for flexure in a beam at the face of a column either, does not include earthquake actions, or, plastic regions cannot develop adjacent to the face of the column, the ratio of bar diameter to column depth shall be limited to:

$$d_b / h_c \leq 6 \alpha_f \sqrt{f'_c / (f_{sy} + f'_s)} \quad \dots 8.1.10(2)$$

The value of  $f'_s$  is the compression stress in the bar on one side of the joint zone, but need not be taken as greater than  $0.5f_{sy}$ , and  $\alpha_f$  is as defined in (a) above where  $h_c$  is the column depth parallel to the longitudinal beam bars being considered.

### 8.1.11 Detailing of flexural reinforcement and tendons

#### 8.1.11.1 General procedure for detailing reinforcement and tendons

Where flexural reinforcement and pretensioned tendons are to be terminated, the bars or tendons shall be extended from the theoretical cut-off point, or theoretical debonding point, by a length of at least  $1.0D + L_{st}$ , or  $1.0D + L_{pt}$ , respectively, where  $D$  is the member depth at the theoretical cut-off point or theoretical debonding point.

Clauses 8.1.11.2 to 8.1.11.8 do not apply to tendons. Clause 8.1.11.9 applies to tendons only.

NOTE: When detailing flexural reinforcement, designers should be cautious in D-regions and design accordingly.

#### 8.1.11.2 Distribution of reinforcement and integrity reinforcement

Tensile reinforcement shall be well distributed in zones of maximum concrete tension, including those portions of flanges of T-beams, L-beams and I-beams over a support.

For in situ construction, the following shall constitute minimum requirements:

- (a) At least one bottom bar but not less than an N20 bar shall be continuous or shall be spliced with a tension lap splice or a mechanical or welded splice satisfying Section 1.
- (b) At non continuous supports shall be anchored to develop  $f_{sy}$  at the face of the support using a standard hook satisfying Clause 13.1.2.7 or headed deformed bar satisfying Clause 13.1.4.

Beams along the perimeter of the structure shall have continuous reinforcement over the span length passing through the region bounded by the longitudinal reinforcement of the column consisting of the following:

- (i) At least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars.
- (ii) At least one-quarter of the tension reinforcement required for positive moment at the midspan, but not less than two bars.

At non-continuous supports, the reinforcement shall be anchored to develop  $f_{sy}$  at the face of the support using a standard hook satisfying Clause 13.1.2.7 or headed deformed bar satisfying Clause 13.1.4.

The continuous reinforcement required shall be enclosed by closed fitments anchored around a longitudinal bar. The transverse reinforcement need not be extended through the column.

Where splices are required, the top reinforcement shall be spliced at or near midspan and bottom reinforcement shall be spliced at or near the support. Splices shall be tension lap splices, or mechanical or welded splices.

#### **8.1.11.3 Continuation of negative moment reinforcement**

Not less than one third of the total negative moment tensile reinforcement required at a support shall be extended a distance equal to the overall depth of the cross-section ( $D$ ) beyond the point of contra-flexure.

#### **8.1.11.4 Anchorage of positive moment reinforcement**

Anchorage of positive moment reinforcement shall conform with the following requirements:

- (a) At a simple support:
  - (i) Sufficient positive moment reinforcement shall be anchored for a length ( $L_{st}$ ) such that the anchored reinforcement shall develop a tensile force of—
    - (A)  $V^* \cot \theta_v / \phi$ ; plus
    - (B) the longitudinal torsion tensile force calculated in accordance with Clause 8.2.9, where  $V^*$  is the design shear force at a distance,  $d \cot \theta_v$  from the anchor point; plus
    - (C) any other longitudinal tensile forces in the reinforcement.

The anchor point shall be taken either halfway along the length of the bearing, or determined by calculating the width of the compressive strut in accordance with Clause 7.2, taking account of both shear and torsion, and allowing for the truss angle being used. The truss angle ( $\theta_v$ ) is as defined in Clause 8.2.4, and  $L_{sy,t}$  is determined from Clause 13.1.2.

- (ii) Not less than one half of the tensile reinforcement required at midspan shall extend past the face of the support for a length of  $12d_b$  plus a cog or an equivalent anchorage; or not less than one third of the tensile reinforcement required at midspan shall extend past the face of the support for a length of  $8d_b$  plus  $D/2$ .
- (b) At a continuous or flexurally restrained support not less than one quarter of the total positive moment reinforcement required at midspan shall continue past the near face of the support

#### **8.1.11.5 Shear strength requirements near terminated flexural reinforcement**

If tensile reinforcement is terminated, the effect on the shear strength shall be assessed in accordance with the principles of 'strut-and-tie modelling'.

This requirement shall be deemed to be satisfied if any one of the following conditions is met:

- (a) Not more than a quarter of the maximum tensile reinforcement is terminated within any distance  $2D$ .
- (b) At the cut-off point,  $\phi V_u \geq 1.5V^*$ .
- (c) Stirrups are provided to give an area of shear reinforcement of  $A_{sv} + A_{sv.min}$  for a distance equal to the overall depth of the cross-section ( $D$ ) along the terminated bar from the cut-off point, where  $A_{sv}$  and  $A_{sv.min}$  are determined in accordance with Clause 8.2.

#### 8.1.11.6 Deemed-to-conform arrangement of flexural reinforcement

For continuous reinforced beams analysed using simplified methods of analysis, as detailed in Clause 6.10, the following deemed to conform arrangement shall be used and shall be deemed to satisfy the requirements of Clauses 8.1.11.3 to 8.1.11.5:

- (a) Of the negative moment tensile reinforcement provided at the support—
  - (i) not less than one quarter shall extend over the whole span;
  - (ii) not less than one half shall extend  $0.3L_n$  or more beyond the face of the support; and
  - (iii) the remainder, if any, shall extend  $0.2L_n$  or more beyond the face of the support.

Where adjacent spans are unequal, the extension of negative reinforcement beyond each face of the common support shall be based on the longer span.
- (b) Of the positive moment tensile reinforcement provided at midspan—
  - (i) not less than one half shall extend into a simple support for a length of  $12d_b$  plus a cog or an equivalent anchorage;
  - (ii) not less than one quarter shall extend into a support where the beam is continuous or flexurally restrained; and
  - (iii) the remainder, if any, shall extend to within  $0.1L_n$  from the face of the support.
- (c) To conform with shear requirements, not more than a quarter of the maximum tensile reinforcement shall be terminated within any distance  $2D$ .

#### 8.1.11.7 Restraint of compressive reinforcement

Compressive reinforcement required for strength in beams shall be adequately restrained by fitments in accordance with Clause 10.7.4.

#### 8.1.11.8 Bundled bars

Groups of parallel longitudinal bars bundled to act as a unit shall—

- (a) have not more than four bars in any one bundle;
- (b) be tied together in contact; and
- (c) be enclosed within stirrups or fitments.

Within the span of a flexural member, individual bars in a bundle shall be terminated so that the points of termination are staggered by a distance greater than or equal to 40 times the diameter of the larger bar within the bundle.

The unit of bundled bars shall be treated as an equivalent single bar of diameter derived from the total area of the bars in the bundle.

### 8.1.11.9 Detailing of tendons

In prestressed members—

- (a) detailing of tendons for termination, anchorage and debonding shall be based on a hypothetical bending-moment diagram formed by uniformly displacing the calculated positive and negative bending moment envelopes a distance ( $D$ ) along the beam from each side of the relevant section of maximum moment;
- (b) anchorages and stress development, as appropriate, shall be provided for all tendons in accordance with Clause 12.5 and Section 13;
- (c) at a simple support of a pretensioned member, at least one third of the tendons required at the section of maximum positive moment shall be continued to the end of the member without debonding; and
- (d) for horizontal curvature of tendons, the designer shall assess the bursting and/or splitting capacity of the beam.

## 8.2 STRENGTH OF BEAMS IN SHEAR

### 8.2.1 General

#### 8.2.1.1 Combined flexure, torsion and shear

Clause 8.2 applies to reinforced and prestressed beams subjected to any combination of torsion, flexure, and shear and axial loads. It does not apply to non-flexural members covered by Sections 7 and 12.

#### 8.2.1.2 Consideration of torsion

Where torsional strength is not required for the equilibrium of the structure and the torsion in a member is induced solely by the angular rotation of adjoining members, the torsional stiffness in the analysis and torsion in the member may be disregarded, provided the minimum torsion reinforcement requirements of Clause 8.2.5.4 and the detailing requirements of Clause 8.3.3 are satisfied.

Torsional effects shall be considered in regions where—

$$T^* > 0.25\phi T_{cr} \quad \dots 8.2.1.2(1)$$

where

$$T_{cr} = 0.33\sqrt{f'_c} \frac{A_{cp}^2}{P_c} \sqrt{1 + \frac{\sigma_{cp}}{0.33\sqrt{f'_c}}}$$

and

$A_{cp}$  = total area enclosed by outside perimeter of concrete section

$T_{cr}$  = torsional cracking moment

$P_c$  = the length of the outside perimeter of concrete cross-section

$\sigma_{cp}$  = average intensity of effective prestress in concrete at the centroid, or at the junction of the web and flange when the centroid lies inside the flange

For cellular structures—

$$\frac{A_{cp}^2}{P_c} \leq 2A_o b_v \quad \dots 8.2.1.2(2)$$

where

$A_o$  = area enclosed by shear flow path, including any area of holes therein

$b_v$  = effective width of the web (see Clause 8.2.1.4)

The equivalent factored shear force ( $V_{eq}^*$ ) at any section for co-existing applied shear ( $V^*$ ) and applied torsion ( $T^*$ ) shall be taken equal to the following—

(a) For solid sections:

$$V_{eq}^* = \sqrt{(V^*)^2 + \left[ \frac{0.9T^*u_h}{2A_o} \right]^2} \quad \dots 8.2.1.2(3)$$

(b) For box sections:

$$V_{eq}^* = V^* + \frac{T^*d_s}{2A_o} \quad \dots 8.2.1.2(4)$$

where

$u_h$  = perimeter of the centre-line of the closed transverse torsion reinforcement

$d_s$  = distance from the extreme compression fibre to the centroid of non-prestressed tensile reinforcement

### 8.2.1.3 Vertical component of prestress

$P_v$  is the vertical component of the prestressing force at the section under consideration. Where  $P_v$  is greater than the minimum design shear force  $V_{min}^*$ , the additional design action of Clause 2.4.4(ii) shall be considered and  $P_v$  shall be taken as zero for the determination of the shear capacity in Clauses 8.2.7, 8.2.1.5, 8.2.3.1, 8.2.3.3 and 8.2.4.5.

### 8.2.1.4 Tapered members

In members that are tapered along their length, the components of inclined tension or compressive forces shall be taken into consideration in the calculation of shear strength.

### 8.2.1.5 Effective web width

The effective web width ( $b_v$ ) shall be taken as the minimum web width within the effective shear depth ( $d_v$ ). In determining  $b_v$  at a particular level, for prestressing ducts with  $d_b \geq b_w/8$ :

$$b_v = (b_w - k_d \sum d_d) \quad \dots 8.2.1.5$$

where

$\sum d_d$  = sum of the diameters of prestressing ducts, if any, in a horizontal plane across the web

$k_d$  = 0.5 for grouted steel duct

= 0.8 for grouted plastic duct

= 1.2 for ungrouted duct

NOTES:

- 1  $k_d$  may be reduced in the presence of reinforcement transverse to the plane of the web.
- 2 For solid circular sections,  $b_w$  may be taken as the diameter.

### 8.2.1.6 Requirements for transverse shear reinforcement

Except for slabs less than 300 mm in depth and walls, transverse shear reinforcement shall be provided in all regions where—

- (a)  $V^* > 0.5\phi(V_{uc} + P_v)$ ; or
- (b)  $T^* > 0.25\phi T_{cr}$ ; or
- (c) the overall depth of the member  $D \geq 750$  mm.

### 8.2.1.7 Minimum transverse shear reinforcement

The minimum area of transverse shear reinforcement ( $A_{sv,min}$ ) provided in a beam shall be calculated from the following equation—

$$A_{sv,min} = 0.06\sqrt{f'_c} b_v s / f_{sy,f} \geq 0.35b_v s / f_{sy,f} \quad \dots 8.2.1.7$$

where

$s$  = centre-to-centre spacing of shear reinforcement, measured parallel to the longitudinal axis of the member

### 8.2.1.8 Design yield strength of tendons as transverse shear reinforcement

The design yield strength of tendons used as transverse shear reinforcement shall be taken as the effective prestress plus 500 MPa, but shall not be taken greater than  $f_{py}$ .

### 8.2.1.9 Effective shear depth

The effective shear depth ( $d_v$ ) shall be taken as the greater of  $0.72D$  or  $0.9d$ , where  $d$  is the distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement in the half-depth of the section containing the flexural tension zone.

## 8.2.2 Design procedures

### 8.2.2.1 Flexural regions

Where it is reasonable to assume that plane sections remain plane after loading, regions of members shall be designed for shear and torsion using either the sectional model in Clause 8.2.3 or by the strut and tie model in Section 7. The requirements of Clause 8.2.1 shall apply.

Members in which the distance from the point of zero shear to the face of the support is less than  $2d$ , or members in which a concentrated load causing more than 50% (or 33% for segmental box girders) of the design shear at a support is closer than  $2d$  from the face of the support, may be considered to be deep components for which the provisions of Section 12 apply.

### 8.2.2.2 Regions near discontinuities

Where the plane sections assumption of flexural theory is not valid, regions of members shall be designed for shear and torsion using the strut-and-tie model as specified in Section 7, and Clause 12.2 shall apply.

### 8.2.2.3 Interface regions

Interfaces between elements shall be designed for shear transfer in accordance with Clause 8.4.

#### 8.2.2.4 Detailed analysis

In lieu of the methods specified in Clauses 8.2.2.1 to 8.2.2.3, the resistance of members in shear or in shear combined with torsion may be determined by satisfying the applicable conditions of equilibrium and compatibility of strains, using appropriate stress-strain relationships for reinforcement and for diagonally cracked concrete.

### 8.2.3 Sectional design of a beam

#### 8.2.3.1 Design shear strength of a beam

The design shear strength of a beam ( $\phi V_u$ ) shall satisfy—

$$\phi V_u \geq V_{eq}^*$$

... 8.2.3.1

where

$$V_u \geq V_{uc} + V_{us} + P_v$$

and

$V_{uc}$  = determined from Clause 8.2.4

$V_{us}$  = determined from Clause 8.2.5

$P_v$  = vertical component of prestress crossing the section under consideration (see Clause 8.2.1.2)

#### 8.2.3.2 Maximum transverse shear near a support

The maximum transverse shear near a support shall be taken as the shear at—

- (a) the face of the support; or
- (b) a distance of  $d_o$  from the face of the support, provided—
  - (i) the member is directly supported and diagonal cracking cannot take place at the support or extend into it; and
  - (ii) the transverse shear reinforcement required at  $d_o$  from the support is continued unchanged to the face of the support.

For concentrated loads near a support, either the maximum transverse shear shall be taken at the face of the support or more refined modelling techniques used to consider the enhanced effect of loads taken directly to a support, such as strut-and-tie action (see Clause 8.2.2.1).

#### 8.2.3.3 Shear strength limited by web crushing

In no case shall the ultimate shear strength ( $V_u$ ) at any section be taken as greater than—

$$V_{u,max} = k_c \left[ f'_c b_v d_v \left( \frac{\cot(\theta_v) + \cot(\alpha_v)}{1 + \cot^2(\theta_v)} \right) \right] + P_v; \text{ or} \quad \dots 8.2.3.3(1)$$

$$V_{u,max} = k_c \left[ 0.85 f'_{cp} b_v d_v \left( \frac{\cot(\theta_v) + \cot(\alpha_v)}{1 + \cot^2(\theta_v)} \right) \right] + P_v \text{ at transfer} \quad \dots 8.2.3.3(2)$$

where

$V_{u,max}$  = ultimate shear strength limited by web crushing failure

$$k_c = 0.55 \left( \frac{30}{f'_c} \right)^{1/3} \leq 0.55; \text{ or}$$

$$0.55 \left( \frac{30}{0.85 f'_{cp}} \right)^{1/3} \leq 0.55 \text{ at transfer}$$

$d_v$  = effective shear depth (see Clause 8.2.1.8)

$\theta_v$  = angle of inclination of the compression strut, (see Clause 8.2.4.2)

$\alpha_v$  = angle between the inclined shear reinforcement and the longitudinal tensile reinforcement

## 8.2.4 Shear strength of a beam excluding transverse shear reinforcement ( $V_{uc}$ )

### 8.2.4.1 General

The shear strength may be determined by any of the following, provided all requirements for use of the chosen model are satisfied:

(a)  $V_u = k_v b_v d_v \sqrt{f'_c}$  ... 8.2.4.1

where

where  $\sqrt{f'_c}$  shall not exceed 8.0 MPa, and  $k_v$  shall be determined in accordance with either—

- (i) Clause 8.2.4.2 modified compression field theory (general); or
- (ii) Clause 8.2.4.6 modified compression field theory (simplified).

- (b) Simplified procedure for uncracked prestressed sections in accordance with Clause 8.2.4.7.

### 8.2.4.2 Determination of $k_v$ and $\theta_v$ (general method)

The value of  $k_v$  shall be calculated as follows—

$$k_v = \left[ \frac{0.4}{1 + 1500 \epsilon_x} \right] \left[ \frac{1300}{1000 + k_{dg} d_v} \right] \quad \dots 8.2.4.2$$

where for normal weight concrete with  $f'_c \leq 70$  MPa—

$$k_{dg} = 32 / (16 + d_g) \text{ but not less than } 0.80$$

$d_g$  = maximum nominal aggregate size

else for light weight concrete of concrete with  $f'_c > 70$  MPa  $k_{dg} = 2.0$ .

Provided the maximum nominal aggregate size ( $d_g$ ) is not less than 16 mm,  $k_{dg}$  may be taken as 1.0.

The angle of inclination of the concrete compression strut to the longitudinal axis of the member ( $\theta_v$ ) shall be calculated as follows:

$$\theta_v = (29 + 7000 \epsilon_x) \text{ degrees} \quad \begin{aligned} &\geq 30^\circ \text{ for reinforced concrete members} \\ &\geq 20^\circ \text{ for prestressed concrete members} \end{aligned}$$

### 8.2.4.3 Determination of the longitudinal strain in concrete $\epsilon_x$ for shear

In lieu of more accurate calculations, the longitudinal strain ( $\epsilon_x$ ) in the concrete at the mid-depth of the section  $\epsilon_x$  shall be calculated as follows:

$$\epsilon_x = \frac{|M^* / d_v| + |V^*| - P_v + 0.5 N^* - A_{pt} f_{po}}{2(E_s A_{st} + E_p A_{pt})} \leq 3.0 \times 10^{-3} \quad \dots 8.2.4.3(1)$$



Where  $\varepsilon_x$  as calculated above is less than zero,  $\varepsilon_x$  may either be taken as zero or calculated as follows:

$$\varepsilon_x = \frac{\left| \frac{M^*}{d_v} + |V^*| - P_v + 0.5N^* - A_{pt}f_{po} \right|}{2(E_s A_{st} + E_p A_{pt})} \text{ within the limits } -0.2 \times 10^{-3} \leq \varepsilon_x \leq 0 \quad \dots 8.2.4.3(2)$$

where

- (a)  $M^*$  and  $V^*$  are absolute values and  $M^* \geq (V^* - P_v)d_v$ .
- (b)  $N^*$  is taken as positive for tension and negative for compression.

NOTE: For rigid frames and rectangular culverts, the value of  $N^*$  used to determine  $\varepsilon_x$  may be taken as twice the compressive axial thrust calculated by elastic analysis.

- (c)  $A_{st}$  and  $A_{pt}$  are the areas of reinforcing bars and prestressing tendons, respectively, located in the half depth of the section containing the flexural tensile zone.

NOTES:

- 1  $f_{po}$  may be taken as  $0.7f_{pu}$  for bonded tendons outside the transfer length and  $f_{pe}$  for unbonded tendons.
- 2 In calculating  $A_{st}$ , the area of bars that terminate less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.

For sections closer than  $d_o$  to the face of the support, the value of  $\varepsilon_x$  calculated at  $d_o$  from the face of the support may be used in evaluating  $k_v$  and  $\theta_v$ .

If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in  $\varepsilon_x$  shall be considered. In lieu of more accurate calculations,  $\varepsilon_x$  calculated from the equation shall be doubled.

$k_v$  and  $\theta_v$  may be determined from Clause 8.2.4.2 using a value of  $\varepsilon_x$  that is greater than that calculated from the equation in this clause. The mid-depth strain parameter  $\varepsilon_x$  shall not be greater than  $3.0 \times 10^{-3}$ .

#### 8.2.4.4 Determination of $\varepsilon_x$ for combined shear and torsion

In lieu of more accurate calculations, the longitudinal strain ( $\varepsilon_x$ ) in the concrete at the mid-depth of the section subjected to shear and torsion shall be calculated as follows—

$$\varepsilon_x = \frac{\left| \frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left[ \frac{0.9T^* u_h}{2A_o} \right]^2} + 0.5N^* - A_{pt}f_{po} \right|}{2(E_s A_{st} + E_p A_{pt})} \leq 3.0 \times 10^{-3} \quad \dots 8.2.4.4(1)$$

Where  $\varepsilon_x$  as calculated above is less than zero  $\varepsilon_x$  shall either be taken as zero or recalculated as follows—

$$\varepsilon_x = \frac{\left| \frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left[ \frac{0.9T^* u_h}{2A_o} \right]^2} + 0.5N^* - A_{pt}f_{po} \right|}{2(E_s A_{st} + E_p A_{pt} + E_c A_{ct})} \quad \dots 8.2.4.4(2)$$

within the limits  $-0.2 \times 10^{-3} \leq \varepsilon_x \leq 0$  and  $M^*$  and  $V^*$  are absolute values and where—

$$M^* \geq d_v \sqrt{(V^* - P_v)^2 + \left[ \frac{0.9T^* u_h}{2A_o} \right]^2} \quad \dots 8.2.4.4(3)$$

$N^*$  is taken as positive for tension and negative for compression.

For rigid frames and rectangular culverts, the value of  $N^*$  used to determine  $\varepsilon_x$  may be taken as twice the compressive axial thrust calculated by elastic analysis.

$A_{st}$  and  $A_{pt}$  are the areas of reinforcing bars and prestressing tendons respectively in the half depth of the section containing the flexural tension zone.

NOTES:

- 1  $f_{po}$  may be taken as  $0.7f_{pu}$  for bonded tendons outside the transfer length and  $f_{pe}$  for unbonded tendons.
- 2 In calculating  $A_{st}$ , the area of bars that terminate less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.

For sections closer than  $d_o$  to the face of the support, the value of  $\varepsilon_x$  calculated at  $d_o$  from the face of the support may be used in evaluating  $k_v$  and  $\theta_v$ .

If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in  $\varepsilon_x$  shall be considered. In lieu of more accurate calculations,  $\varepsilon_x$  calculated from the equation shall be doubled.

$k_v$  and  $\theta_v$  may be determined from Clause 8.2.4.4 using a value of  $\varepsilon_x$  that is greater than that calculated from the equation in this clause. The mid-depth strain parameter  $\varepsilon_x$  shall not be greater than  $3.0 \times 10^{-3}$ .

#### 8.2.4.5 Web crushing due to combined shear and torsion

The minimum wall thickness ( $t_w$ ) to avoid crushing from combined shear and torsion shall be calculated as follows—

(a) For box sections—

(i) Where wall thickness  $t_w > A_{oh}/u_h$

$$\frac{V^* - P_v}{b_v d_v} + \frac{T^* u_h}{1.7 A_{oh}^2} \leq \phi 0.2 f'_c \quad \dots 8.2.4.5(1)$$

(ii) Where wall thickness  $t_w \leq A_{oh}/u_h$

$$\frac{V^* - P_v}{b_v d_v} + \frac{T^*}{1.7 t_w A_{oh}} \leq \phi 0.2 f'_c \quad \dots 8.2.4.5(2)$$

(b) For other sections:

$$\sqrt{\left[ \frac{V^* - P_v}{b_v d_v} \right]^2 + \left[ \frac{T^* u_h}{1.7 A_{oh}^2} \right]^2} \leq \phi 0.2 f'_c \quad \dots 8.2.4.4(3)$$

where

$A_{oh}$  = area enclosed by centre-line of exterior closed transverse torsion reinforcement, including area of holes (if any)

$u_h$  = perimeter of the centre-line of the closed transverse torsion reinforcement

#### 8.2.4.6 Determination of $k_v$ and $\theta_v$ for non-prestressed components (simplified method)

For non-prestressed components not subjected to axial tension, and provided the specified yield strength of the longitudinal reinforcement does not exceed 500 MPa, the design concrete strength does not exceed 65 MPa and the size of maximum aggregate particle is not less than 10 mm, the angle of inclination of the compression strut ( $\theta_v$ ) shall be taken as  $36^\circ$ , and the value of  $k_v$  shall be determined as follows—

$$(a) \quad \text{For } A_{sv} < A_{sv.min} \quad k_v = 200/(1000 + 1.3d_v) \leq 0.10 \quad \dots 8.2.4.6(1)$$

$$(b) \quad \text{For } A_{sv} \geq A_{sv.min} \quad k_v = 0.15 \quad \dots 8.2.4.6(2)$$

#### 8.2.4.7 Simplified method for uncracked prestressed concrete beams

The ultimate shear strength ( $V_{uc}$ ) of a reinforced beam, excluding the contribution of shear reinforcement, shall be calculated from the following equation—

$$V_{uc} = \frac{I_c b_v(y)}{S_c(y)} + \left[ \sqrt{f_{ct}^2 + \alpha_1 f_{ct} \sigma_{cp}(y)} - \tau_{cp}(y) \right] \quad \dots 8.2.4.7$$

where

$I_c$  = second moment of area of concrete section

$y$  = distance from the centroidal axis to the section under consideration

$S_c(y)$  = first moment area above  $y$  and about the centroidal axis

$b_v(y)$  = width of the cross-section at a  $y$  from the centroidal axis

$\sigma_{cp}(y)$  = concrete compressive stress at  $y$  from centroidal axis due to prestressing,

$$= \left[ \frac{1}{A_c} + \frac{Y_c - y}{I_c} \right] F_p l_x$$

$Y_c$  = height of concrete centroidal axis,

$$\tau_{cp}(y) = \frac{1}{b_v(y)} \left[ \frac{A_c(y)}{A_c} + \frac{(Y_c - Y_{pt}) S_c(y)}{I_c} \right] \frac{dP_t(l_x)}{dx}$$

$a_1$  =  $l_x / (1.2 L_{pt})$

$l_x$  = distance from end of beam to location shown on Figure 8.2.4.7

$L_{pt}$  = transmission length

$Y_{pt}$  = height of centroidal axis of prestressing steel

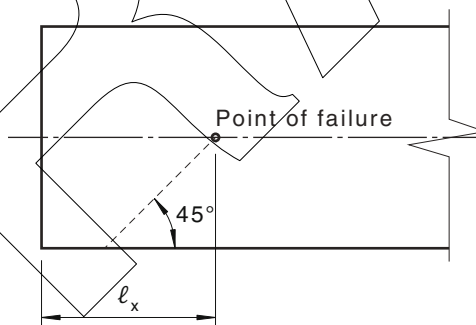


FIGURE 8.2.4.7 LOCATION OF POINT OF FAILURE ( $l_x$ ) AT HEIGHT  $y$  FROM CENTROID

#### 8.2.4.8 Secondary effects on $V_{uc}$

Where stresses due to secondary effects such as creep, shrinkage and differential temperature are significant, they shall be taken into account in the calculation of  $V_{uc}$  both for reinforced and prestressed concrete beams.

#### 8.2.4.9 Reversal of loads

Where loading cases occur which result in cracking in a zone usually in compression, the value of  $V_{uc}$  obtained from Clause 8.2.4 may not apply. In this case,  $V_{uc}$  shall be assessed or be taken as zero.

#### 8.2.4.10 Shear strength of a beam with minimum reinforcement

The minimum ultimate shear strength ( $V_{u,min}$ ) of a beam provided with minimum shear reinforcement ( $A_{sv,min}$ ) shall be taken as the greater of—

$$(a) \quad V_{uc} + 0.1b_v d_o \sqrt{f'_c} \quad \dots 8.2.4.10(1)$$

$$(b) \quad V_{uc} + 0.6b_v d_o \quad \dots 8.2.4.10(2)$$

### 8.2.5 Transverse shear reinforcement

#### 8.2.5.1 General

Where the spacing, ( $s$ ), of the transverse shear reinforcement changes, the quantity ( $A_{sv}/s$ ) may be assumed to vary linearly over a length,  $D$ , centred on the location where the spacing changes.

#### 8.2.5.2 Transverse reinforcement for shear

The contribution to the design shear strength ( $V_u$ ) by shear reinforcement in a beam ( $V_{us}$ ) shall be determined from the following equations—

(a) For perpendicular shear reinforcement:

$$V_{us} = (A_{sv} f_{sy.f} d_o / s) \cot(\theta_v) \quad \dots 8.2.5.2(1)$$

(b) For inclined shear reinforcement:

$$V_{us} = (A_{sv} f_{sy.f} d_o / s) (\sin(\alpha_v) \cot(\theta_v) + \cos(\alpha_v)) \quad \dots 8.2.5.2(2)$$

where

$\alpha_v$  = angle between the inclined shear reinforcement and the longitudinal tensile reinforcement

#### 8.2.5.3 Transverse reinforcement for combined shear and torsion

For sections subjected to combined shear and torsion, the transverse reinforcement that is provided shall be at least equal to the sum of that required for shear and that required for the coexisting torsion.

#### 8.2.5.4 Transverse reinforcement for torsion

The amount of transverse reinforcement required for torsion shall be such that  $T^* \leq \phi T_{us}$ .

#### 8.2.5.5 Minimum torsional reinforcement

Minimum torsional reinforcement shall consist of the following—

(a) Sufficient reinforcement in the form of closed ties such that:

$$(i) \quad T_{us} \geq T_{cr}; \text{ or} \quad \dots 8.2.5.5(1)$$

$$(ii) \quad \frac{A_{sw}}{s} \geq \frac{0.2y_1}{f_{sy.f}} \quad \dots 8.2.5.5(2)$$

where  $y_1$  is the larger overall dimension of the closed tie.

NOTE: Reinforcement provided for this requirement may be included in the requirement for minimum shear reinforcement (Clause 8.2.1.6).

and

(b) Longitudinal reinforcement in accordance with Clauses 8.2.7, 8.2.8 and 8.2.9.

### 8.2.5.6 Torsional resistance

The value of  $T_{us}$  shall be determined from the following equation:

$$T_{us} = 2A_o \frac{A_{sw} f_{sy.f}}{s} \cot(\theta_v) \quad \dots 8.2.5.6$$

where

$$A_o = 0.85 A_{oh}$$

$$\theta_v = \text{as calculated in Clause 8.2.4.2}$$

### 8.2.6 Hanging reinforcement

Loads applied to a member other than at the top chord of the member shall be transferred to the top chord, within the load application region, by the provision of hanging reinforcement of area consistent with strut-and-tie modelling.

### 8.2.7 Proportioning longitudinal reinforcement on the flexural tension side

Additional longitudinal reinforcement and tendons on the flexural tension side of a beam section shall be proportioned such that the additional force is greater than or equal to  $\Delta F_{td}$ , calculated as follows—

(a) Shear with torsion:

$$\Delta F_{td} = 0.5N^* + \sqrt{(V^* - 0.5V_{us} - P_v)^2 + \left[ \frac{0.45T^* u_h}{2A_o} \right]^2} \cot(\theta_v) \text{ but not less than zero} \quad \dots 8.2.7(1)$$

or

(b) Shear without torsion:

$$\Delta F_{td} = 0.5N^* + (V^* - 0.5V_{us} - P_v) \cot(\theta_v) \quad \dots 8.2.7(2)$$

where  $M^*$  and  $V^*$  are absolute values and  $N^*$ , is positive for axial tension and negative for axial compression.

Additional reinforcement and tendons shall be fully anchored and proportioned such that the following is satisfied—

$$A_{st} f_{sy} + A_{pt} f_{py} \geq \Delta F_{td} / \phi \quad \dots 8.2.7(3)$$

where

$$\phi = 0.7 [\text{see Table 2.3.2(c)}]$$

NOTE: For reinforcement and tendons that are not fully anchored, see Clauses 13.1.2.3 and 13.3.

### 8.2.8 Proportioning longitudinal reinforcement on the flexural compression side

Additional longitudinal reinforcement on the flexural compression side of the beam section shall be proportioned such that the additional force is greater than or equal to the force  $\Delta F_{cd}$ , calculated as follows:

(a) Shear with torsion:

$$\Delta F_{cd} = 0.5N^* + \sqrt{(V^* - 0.5V_{us} - P_v)^2 + \left[ \frac{0.45T^* u_h}{2A_o} \right]^2} \cot(\theta_v), \text{ but not} \quad \dots 8.2.8(1)$$

less than zero

or

(b) Shear with torsion:

$$\Delta F_{cd} = 0.5N^* + (V^* - 0.5V_{us} - P_v)\cot(\theta_v), \quad \dots 8.2.8(2)$$

where  $M^*$  and  $V^*$  are absolute values and  $N^*$ , is positive for axial tension and negative for axial compression.

Additional reinforcement and tendons shall be fully anchored and proportioned such that the following is satisfied—

$$A_{sc}f_{sy} \geq \Delta F_{cd} / \phi \quad \dots 8.2.8(3)$$

NOTE: Where the reinforcement is not fully anchored, see Clause 13.1.2.3.

## 8.2.9 Extension of longitudinal reinforcement and tendons

### 8.2.9.1 General

At every section, the longitudinal reinforcement and tendons shall be designed to resist the flexural design tension force determined in Clause 8.1.5, and additional tensile forces caused by shear and torsion as specified in Clause 8.2.7, Clause 8.2.8 and Figure 8.2.9.1. For members not subjected to significant direct tension or torsion, these requirements may be satisfied by extending the flexural tension reinforcement and tendons to develop the flexural tensile force beyond the location required by flexure alone as follows:

- (a) Where transverse reinforcement is not required, a distance  $D$ .
- (b) Where transverse reinforcement is required, a distance  $d_o\cot(\theta_v)$ .

where  $D$  and  $d_o$  are taken at the section under consideration.  $v$ )

$d_o\cot(\theta_v)$

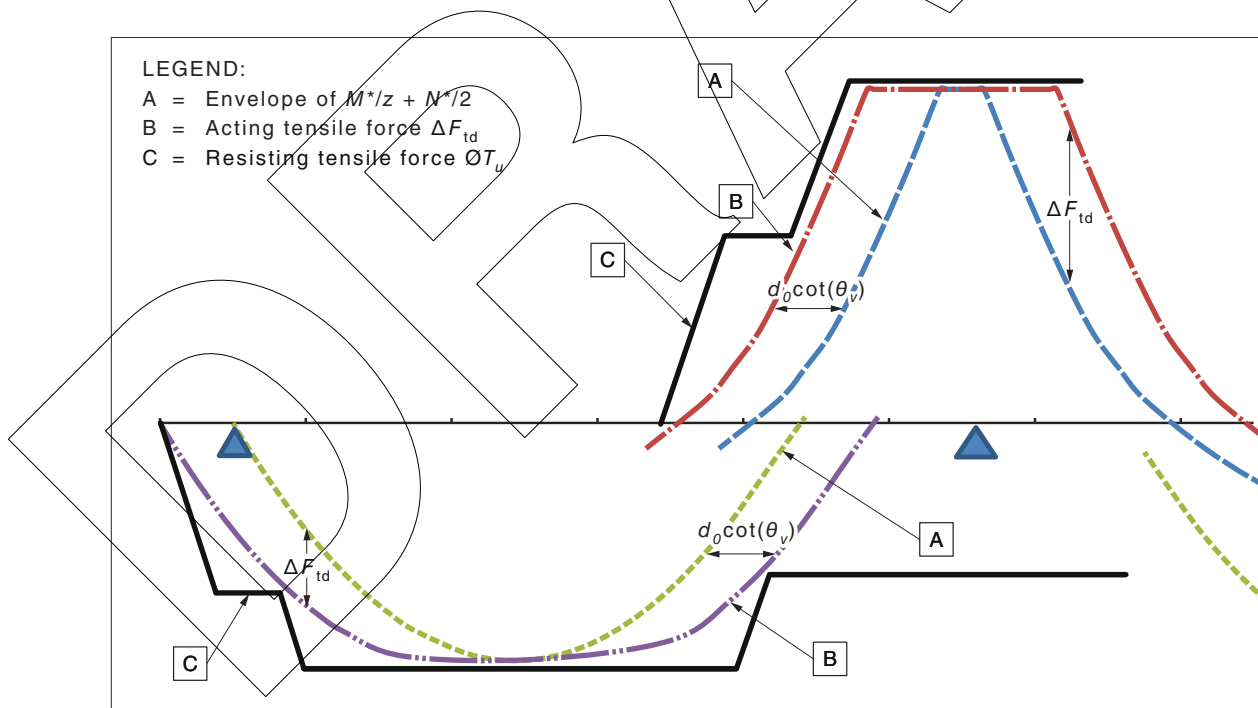


FIGURE 8.2.9.1 ILLUSTRATION OF THE CURTAILMENT OF LONGITUDINAL REINFORCEMENT AND TENDONS, TAKING INTO ACCOUNT THE EFFECT OF INCLINED CRACKS AND THE RESISTANCE OF REINFORCEMENT AND TENDONS WITHIN ANCHORAGE LENGTHS

### 8.2.9.2 *Compression fan regions*

In regions adjacent to maximum moment locations, the cross-sectional area of longitudinal reinforcement on the flexural tension side of the member shall not exceed the cross-sectional area required to resist the maximum moment acting alone, where the support or the load at the maximum moment location introduces direct compression into the flexural compression face of the member and the member is not subject to significant torsion.

### 8.2.9.3 *Anchorage of longitudinal reinforcement at exterior supports*

At exterior direct-bearing supports, the longitudinal reinforcement on the flexural tension side of a beam shall be capable of resisting a tensile force of—

$$\phi f_{sy} A_{st} \geq 0.5N^* + (V^* - 0.5V_{us} - P_v) \cot(\theta_v)$$

where  $V_{us}$  is the contribution by the transverse reinforcement provided within a length of  $d_v \cot(\theta_v)$  from the face of the support where  $V_{us}$  shall not exceed  $V^*$ .

The tension force in the reinforcement shall be developed at the point where a line inclined at angle  $\theta_v$  (to the longitudinal axis and extending from the inside edge of the bearing area) intersects the centroid of the reinforcement.

NOTES:

- 1 For minimum extent of reinforcement, see Clause 8.2.3.4.
- 2 When detailing flexural reinforcement, designers should be cautious in D-regions and design accordingly.

## 8.3 GENERAL DETAILS

### 8.3.1 *Detailing of flexural reinforcement*

#### 8.3.1.1 *Distribution of reinforcement and tendons*

Tensile reinforcement and tendons shall be uniformly distributed in zones of maximum concrete tension, including those portions of flanges of T-beams, L-beams and I-beams over a support.

#### 8.3.1.2 *Continuation of negative moment reinforcement and tendons*

Not less than one-third of the total negative moment tensile reinforcement and tendons required at a support shall be extended a distance into the span equal to the overall depth of the cross-section ( $D$ ) beyond the point of contra-flexure.

#### 8.3.1.3 *Shear strength requirements near terminated flexural reinforcement*

Where tensile reinforcement is terminated, the shear strength shall be assessed by strut-and-tie modelling or other suitable model.

This requirement shall be deemed to be satisfied if any one of the following conditions is met:

- (a) Not more than a quarter of the maximum tensile reinforcement or tendons is terminated within any distance  $2D$ ; or
- (b) At the cut-off point,  $\phi V_u \geq V_{eq}^*$ ; or
- (c) Stirrups are provided to give an area of shear reinforcement of  $A_{sv} + A_{sv,min}$  for a distance equal to the overall depth of the cross-section ( $D$ ) along the terminated bar beyond the cut-off point, where  $A_{sv,min}$  and  $A_{sv}$  are determined in accordance with Clause 8.2.1.6 and Clause 8.2.5.1, respectively.

#### **8.3.1.4 Anchorage of flexural reinforcement**

Notwithstanding the requirements of Clause 8.2.9, anchorage of positive moment longitudinal flexural reinforcement provided at mid-span shall meet the following requirements—

- (a) not less than one half of the bar area shall extend into a simple support for a length of  $12 d_b$ ; and
- (b) not less than one quarter of the bar area shall extend into a support where the beam is continuous or flexurally restrained.

#### **8.3.1.5 Restraint of compressive reinforcement**

Compressive reinforcement required for strength in beams shall be restrained by fitments in accordance with Clause 10.7.4.

#### **8.3.1.6 Bundled bars**

Groups of parallel longitudinal bars bundled to act as a unit shall—

- (a) have not more than four bars in any one bundle;
- (b) be tied together in contact; and
- (c) be enclosed within stirrups or fitments.

Within the span of a flexural member, individual bars in a bundle shall be terminated so that the points of termination are staggered by a distance greater than or equal to 40 times the diameter of the largest bar within the bundle.

Bundled bars shall be treated as an equivalent single bar of diameter equal to the total area of the bars in the bundle.

#### **8.3.1.7 Detailing of tendons**

Anchorage and stress development shall be provided for tendons in accordance with Clause 12.7 and Section 13, respectively.

Notwithstanding the requirements of Clause 8.2.9, at a simple support, at least 50% of the tendons required at the section of maximum positive moment shall be continued to the end of the member without debonding.

The designer shall assess the bursting and/or splitting capacity of the beam due to forces generated by the horizontal curvature of tendons.

### **8.3.2 Detailing of shear and torsional reinforcement**

#### **8.3.2.1 General**

Shear reinforcement shall comprise of—

- (a) stirrups or fitments making an angle of between  $45^\circ$  and  $90^\circ$  with the longitudinal bars; or
- (b) welded wire mesh; or
- (c) in circular or oval-sectioned members, by helices.

NOTE: Straight reinforcement or tendons is permissible provided they are fully anchored at both the top and bottom of a member such that the design stress can be fully developed. Where stress in the shear reinforcement are greater than 500 MPa at the ultimate limit state, crack widths under service conditions should be considered. [Requires BD002 discussion]



### 8.3.2.2 Spacing

In members not greater than 1.2 metres in depth, the maximum longitudinally spacing shall not exceed the lesser of 300 mm and  $0.5D$ ; otherwise, the longitudinally spacing shall not exceed 600 mm.

The maximum transverse spacing across the width of the member shall not exceed the lesser of 600 mm and  $D$ .

### 8.3.2.3 Extent

The shear reinforcement required at the critical cross-section shall be carried to the face of the support.

Shear reinforcement, of area not less than that calculated as being necessary at any cross-section, shall be provided for a distance ( $D$ ) from that cross-section in the direction of decreasing shear. The first fitment at each end of a span shall be positioned not more than 50 mm from the face of the adjacent support.

Shear reinforcement shall extend as close to the compression face and the tension face of the member as cover requirements and the proximity of other reinforcement and tendons will permit.

### 8.3.2.4 Anchorage of shear reinforcement

The anchorage of shear reinforcement transverse to the longitudinal flexural reinforcement may be achieved by a hook or cog conforming with Clause 13.1.2.7 or by welding of the fitment to a longitudinal bar or by a welded splice or by lapped splices. Where lapped splices are used, the lap length shall be as calculated in Clause 13.1.2, except that for stirrups or fitments adjacent to the cover concrete, the lap length calculated in Clause 13.1.2 shall be multiplied by 1.3.

Shear reinforcement shall be deemed to be anchored provided the following criteria are met:

- (a) Bends in bars used as fitments shall enclose a longitudinal bar with a diameter not less than the diameter of the fitment bar. The enclosed bar shall be in contact with the fitment bend.
- (b) The stirrup spacing shall conform with Clause 8.3.2.2. Provided the hook complies with Clause 13.1.2.7, anchorage shall be deemed to be satisfied.
- (c) Fitment cogs shall not be used when the anchorage of the fitment is solely in the outer layer of reinforcement. In this case fitment hooks shall be used.

NOTE: The type of anchorage used should not induce splitting or spalling of the concrete cover. Notwithstanding the above, fitment cogs shall not to be used when the fitment cog is located within 50 mm of any concrete surface.

### 8.3.2.5 End anchorage of mesh

Where mesh is used as shear reinforcement, the ends shall be anchored—

- (a) in accordance with Clause 8.3.2.4, if the wires are bent at least to the dimensions of a standard fitment hook; or
- (b) by embedding two or more transverse wires at least 25 mm within the compressive zone.

### 8.3.3 Detailing of torsional reinforcement

Torsional reinforcement shall be detailed in accordance with Clause 8.3.2.4 and the following:

- (a) Torsional reinforcement shall consist of both closed fitments and longitudinal reinforcement and tendons.
- (b) The closed ties shall be continuous around all sides of the cross -section and anchored so as to develop full strength at any point, unless a more refined analysis shows that full anchorage is not required over part of the tie at each corner of the torsion cell to develop full strength. The spacing of the closed ties shall be not greater than the lesser of  $0.12u_t$  and 300 mm. In large members where a single closed loop of reinforcement is not possible, bars shall extend in one length over the full depth of the web, or width of the flange, with adequate anchorage by means of hooks or cogs at the intersection of webs and flanges.

Alternatively, stirrups or fitments may be lapped. Lap lengths shall be as calculated in Clause 13.1.2, except that for stirrups or fitments adjacent to the cover concrete the lap length calculated in Clause 13.1.2 shall be multiplied by 1.3.

- (c) Additional longitudinal reinforcement shall be placed as close as practicable to the corners of the cross section, and in all cases at least one longitudinal bar shall be provided at each corner of the closed ties.
- (d) The section shall be capable of distributing axial torsional tensile force equally to the corners of the torsion cell.

## 8.4 LONGITUDINAL SHEAR IN COMPOSITE AND MONOLITHIC BEAMS

### 8.4.1 General

This clause applies to the transfer of longitudinal shear forces, across interface shear planes through webs and flanges of—

- (a) composite beams constructed of precast concrete sections and cast in situ toppings or flanges; and
- (b) beams constructed monolithically.

### 8.4.2 Design shear stress

The design shear stress ( $\tau^*$ ) acting on the interface shall be taken as follows:

$$\tau^* = \beta V^* / (z b_f)$$

... 8.4.2

where

$z$  = internal moment lever arm of the section

For a shear plane that passes through a region in compression—

$\beta$  = ratio of the compressive force in the member (calculated between the extreme compressive fibre and the shear plane) and the total compression force in the section

For a shear plane that passes through a region in tension—

$\beta$  = ratio of the tensile force in the longitudinal reinforcement (calculated between the extreme tensile fibre and the shear plane) and the total tension force in the section

### 8.4.3 Shear stress capacity

The design shear stress at the shear interface shall not exceed  $\tau\beta_u$  where—

$$\tau_u = \mu \left( \frac{A_{sf} f_{sy}}{s b_f} + \frac{g_p}{b_f} \right) + k_{co} f'_{ct} \quad \dots 8.4.3$$

$$\leq \text{lesser of } (0.2 f'_c, 10 \text{ MPa})$$

where

$\tau_u$  = unit shear strength

$g_p$  = permanent distributed load normal to the shear interface per unit length, newtons per millimetre (N/mm)

$\mu$  = coefficient of friction given in Table 8.4.3

$k_{co}$  = cohesion coefficient given in Table 8.4.3

$b_f$  = width of the shear plane, in millimetres (mm)

$A_{sf}$  = area of fully anchored shear reinforcement crossing the interface (mm<sup>2</sup>)

$f_{sy}$  = yield strength of shear reinforcement not exceeding 500 MPa

$s$  = spacing of anchored shear reinforcement crossing interface

**TABLE 8.4.3**  
**SHEAR PLANE SURFACE COEFFICIENTS**

Surface condition of the shear plane	Coefficients	
	$\mu$	$k_{co}$
A smooth surface, as obtained by casting against a form, or finished to a similar standard	0.6	0.1
A surface trowelled or tamped, so that the fines have been brought to the top, but where some small ridges, indentations or undulations have been left; slip-formed and vibro-beam screeded; or produced by some form of extrusion technique	0.6	0.2
A surface deliberately roughened— (a) by texturing the concrete to give a pronounced profile; (b) by compacting but leaving a rough surface with coarse aggregate protruding but firmly fixed in the matrix; (c) by spraying when wet, to expose the coarse aggregate without disturbing it; or (d) by providing mechanical shear keys.	0.9	0.4
Monolithic construction	0.9	0.5

NOTE: Where a beam is subjected to high levels of differential shrinkage, temperature effects, tensile stress or fatigue effects across the shear plane, the values of  $\mu$  and  $k_{co}$  in the above table do not apply.

### 8.4.4 Shear plane reinforcement

Where reinforcement is required to increase the longitudinal shear strength, the reinforcement shall consist of shear reinforcement anchored to develop its full strength at the shear plane. Shear and torsional reinforcement already provided, and which crosses the shear plane, may be taken into account for this purpose.

The centre-to-centre spacing ( $s$ ) of the shear reinforcement shall not exceed the maximum spacing—

$$s_{\max} = 3.5t_f \quad \dots 8.4.4$$

where

$t_f$  = thickness of topping or flange anchored by the shear reinforcement

#### 8.4.5 Minimum thickness of structural components

The average thickness of structural components subject to interface shear shall be not less than 50 mm with a minimum local thickness not less than 30 mm.

### 8.5 DEFLECTION OF BEAMS

#### 8.5.1 General

The deflection of a beam shall be determined in accordance with Clause 8.5.2 or Clause 8.5.3.

Alternatively, for reinforced beams, the effective-span to effective-depth ratio shall conform with Clause 8.5.4.

#### 8.5.2 Beam deflection by refined calculation

The calculation of the deflection of a beam by refined calculation shall make allowance for the following:

- (a) Cracking and tension stiffening.
- (b) Shrinkage and creep properties of the concrete.
- (c) Expected load history.
- (d) Expected construction procedure.
- (e) Deflection of formwork or settlement of props during construction, particularly when the beam formwork is supported on suspended floors or beams below.

#### 8.5.3 Beam deflection by simplified calculation

##### 8.5.3.1 Short-term deflection

The short-term deflections due to external loads and prestressing, which occur immediately on their application, shall be calculated using the value of  $E_{cj}$  determined in accordance with Clause 3.1.2 and the value of the effective second moment of area of the member ( $I_{ef}$ ). This value of  $I_{ef}$  may be determined from the values of  $I_{ef}$  at nominated cross-sections as follows:

- (a) For a simply supported span, the value at midspan.
- (b) In a continuous beam—
  - (i) for an interior span, half the midspan value plus one quarter of each support value; or
  - (ii) for an end span, half the midspan value plus half the value at the continuous support.
- (c) For a cantilever, the value at the support.

For the purpose of the above determinations, the value of  $I_{ef}$  at each of the cross-sections nominated in Items (a) to (c) above is given by—

$$I_{ef} = \frac{I_{cr}}{1 - \left(1 - \frac{I_{cr}}{I}\right) \left(\frac{M_{cr,t}}{M_s^*}\right)^2} \leq I_{ef,max} \quad \dots 8.5.3.1(1)$$

where

$I_{ef,max}$  = maximum effective second moment of area and is taken as  $I$ , for reinforced sections when  $p = A_{st}/bd \geq 0.005$  for prestressed sections

=  $0.6 I$ , for reinforced sections when  $p = A_{st}/bd < 0.005$

$b$  = width of the cross-section at the compression face

$M_s^*$  = maximum bending moment at the section, based on the short-term serviceability load or the construction load

$$M_{cr,t} = Z(f'_{ct,f} - \sigma_{cs} + P/A_g) + Pe \geq 0$$

$Z$  = section modulus of the uncracked section, referred to the extreme fibre at which cracking occurs

$f'_{ct,f}$  = characteristic flexural tensile strength of concrete

$\sigma_{cs}$  = maximum shrinkage-induced tensile stress on the uncracked section at the extreme fibre at which cracking occurs. In the absence of more refined calculation, the value of  $\sigma_{cs}$  that accounts for the restraint provided by the steel reinforcement may be taken as—

$$= \frac{2.5p_w - 0.8p_{cw}}{1 + 50p_w} E_s \varepsilon_{cs}$$

$p_w$  = web reinforcement ratio for tensile reinforcement =  $(A_{st} + A_{pt})/b_w d$

$p_{cw}$  = web reinforcement ratio for compressive reinforcement =  $A_{sc}/b_w d$

$\varepsilon_{cs}$  = final design shrinkage strain of concrete at the time when deflection is to be calculated

If the short-term portion of the final long-term deflection is to be calculated  $\varepsilon_{cs}$  is the final long-term shrinkage strain determined in accordance with Clause 3.1.7. In the case of indeterminate members,  $\sigma_{cs}$  shall also account of the tension caused by the redundant actions induced by restraint.

Alternatively, as a further simplification but only for reinforced members,  $I_{ef}$  may be taken as—

$$I_{ef} = [(5 - 0.04f'_c)p + 0.002] b_{ef} d^3 \leq [0.1 / \beta^{2/3}] b_{ef} d^3 \text{ when } p \geq 0.001(f'_c)^{1/3} / \beta^{2/3} \quad \dots 8.5.3.1(2)$$

$$I_{ef} = [(0.055(f'_c)^{1/3}) / \beta^{2/3} - 50p] b_{ef} d^3 \leq [0.06 / \beta^{2/3}] b_{ef} d^3 \text{ when } p < 0.001(f'_c)^{1/3} / \beta^{2/3} \quad \dots 8.5.3.1(3)$$

where

$\beta = b_{ef}/b_w \geq 1$

$p = A_{st}/(b_{ef} d)$  at midspan

### 8.5.3.2 Long-term deflection

For reinforced and prestressed beams, that part of the deflection that occurs after the short term deflection shall be calculated as the sum of—

- the shrinkage component of the long-term deflection, determined from the design shrinkage strain of concrete ( $\varepsilon_{cs}$ ) (see Clause 3.1.7) and the principles of mechanics; and
- the additional long-term creep deflections, determined from the design creep coefficient of concrete ( $\varphi_{cc}$ ) (see Clause 3.1.8) and the principles of mechanics.

In the absence of more accurate calculations, the additional long-term deflection of a reinforced beam due to creep and shrinkage may be estimated by multiplying the short-term deflection caused by the sustained loads (obtained using the final long-term shrinkage strain in the estimate of  $M_{cr,t}$ ) by a multiplier,  $k_{cs}$ , given by—

$$k_{cs} = [2 - 1.2(A_{sc} / A_{st})] \geq 0.8 \quad \dots 8.5.3.2$$

where  $A_{sc}$  is the area of steel in the compressive zone of the cracked section between the neutral axis at service loads and the extreme concrete compressive fibre and  $A_{sc}/A_{st}$  is taken at midspan, for a simply supported or continuous beam and at the support, for a cantilever beam.

### 8.5.4 Deemed to conform span-to-depth ratios for reinforced beams

For reinforced beams of uniform cross-section, fully propped during construction, subject to uniformly distributed loads only and where the imposed action (live load) ( $q$ ) does not exceed the permanent action (dead load) ( $g$ ), beam deflections shall be deemed to conform with the requirements of Clause 2.3.2 if the ratio of effective span to effective depth satisfies the following:

$$L_{ef} / d \leq \left[ \frac{k_1 (\Delta / L_{ef}) b_{ef} E_c}{k_2 F_{d,ef}} \right] \quad \dots 8.5.4$$

where

$\Delta / L_{ef}$  = deflection limit selected in accordance with Clause 2.3.2(a)

$L_{ef}$  = effective span

$F_{d,ef}$  = effective design load per unit length, taken as—

- $(1.0 + k_{cs})g + (\psi_s + k_{cs} \psi_1)q$  for total deflection; or
- $k_{cs} g + (\psi_s + k_{cs} \psi_1)q$  for the deflection that occurs after the addition or attachment of the brittle partitions or finishes

where  $k_{cs}$  is determined in accordance with Clause 8.5.3.2 and  $\psi_s$  and  $\psi_1$  are given in AS/NZS 1170.0

$k_1 = L_{ef} / b_{ef} d^3$ , which may be taken as—

$$(5 - 0.04 f'_c) p + 0.002 \leq 0.1 / \beta^{2/3} \quad \text{when } p \geq 0.001 (f'_c)^{1/3} / \beta^{2/3}$$

$$0.055 (f'_c)^{1/3} / \beta^{2/3} - 50 p \leq 0.06 / \beta^{2/3} \quad \text{when } p < 0.001 (f'_c)^{1/3} / \beta^{2/3}$$

where  $\beta = b_{ef} / b_w \geq 1$  and  $p = A_{st} / b_{ef} d$  at midspan

$k_2$  = deflection constant, taken as—

- for simply supported beams,  $5/384$ ; or

- (b) for continuous beams, where the ratio of the longer to the shorter of two adjacent spans does not exceed 1.2 and where no end span is longer than an interior span—
- (i) 2.4/384 in an end span; or
  - (ii) 1.5/384 in interior spans.

NOTE:  $E_c$  is in megapascals.

## 8.6 CRACK CONTROL OF BEAMS

### 8.6.1 General requirements for reinforced concrete beams

Cracking is normal in reinforced concrete structures subjected to bending, shear, torsion or axial tension caused by either direct loading or restraint or imposed deformation. Cracking shall be limited to an extent that does not impair the durability or serviceability of the structure, both in terms of function and appearance. A characteristic maximum crack width  $w'_{\max}$  shall be selected for each surface of a structure taking into account the function and exposure of the surface. The characteristic maximum crack width is the maximum limit imposed on the calculated crack width.

For regions of beams fully enclosed within a building except for a brief period of weather exposure during construction, and where it is assessed that cracking will not impair the functioning of the structure, only Items (a) and (b) need be satisfied:

- (a) The minimum area of reinforcement in a tensile zone of a beam shall conform with Clause 8.1.6.1.
- (b) The distance from the side or soffit of a beam to the centre of the nearest longitudinal bar shall not exceed 100 mm. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored. The centre-to-centre spacing of bars near a tension face of the beam shall not exceed 300 mm. For T-beams and L-beams, the reinforcement required in the flange shall be distributed across the effective width.

In other situations, in addition to (a) and (b) above, crack widths may be calculated and limited to the selected value of  $w'_{\max}$  in accordance with Clause 8.6.2.2. A simplified alternative approach is to limit the size and spacing of reinforcement crossing the cracks in accordance with Clause 8.6.2.1.

#### NOTES:

- 1 For prestressed elements with unbonded tendons, the requirements for reinforced concrete elements apply. For prestressed elements with bonded tendons, the requirement of Clause 8.6.3 apply.
- 2 It is possible that the width of some cracks in the actual structure may exceed the maximum characteristic value specified in design.

### 8.6.2 Crack control for tension and flexure in reinforced beams

#### 8.6.2.1 General

For the purpose of this clause the resultant action is considered to be *primarily tension* when the whole of the section is in tension, or *primarily flexure* when the tensile stress distribution within the section prior to cracking is triangular with some part of the section in compression.

### 8.6.2.2 Crack control without direct calculation of crack widths

The calculated steel stress ( $\sigma_{scr}$ ) shall comply with:

- (a) For beams primarily subject to tension, the calculated steel stress ( $\sigma_{scr}$ ) on the cracked section shall not exceed the maximum steel stress given in Table 8.6.2.2(A) for the largest nominal diameter ( $d_b$ ) of the bars in the section, and under direct loading the calculated tensile steel stress ( $\sigma_{scr,1}$ ) shall not exceed  $0.8f_{sy}$ .
- (b) For beams primarily subject to flexure, the calculated tensile steel stress ( $\sigma_{scr}$ ) shall not exceed the larger of the maximum steel stresses given in—
  - (i) Table 8.6.2.2(A) for the largest nominal diameter ( $d_b$ ) of the bars in the tensile zone; and
  - (ii) Table 8.6.2.2(B) for the largest centre-to-centre spacing of adjacent parallel bars in the tensile zone.

Under direct loading the calculated tensile steel stress ( $\sigma_{scr,1}$ ) shall not exceed  $0.8f_{sy}$ . Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored when determining spacing.

NOTE: Design bending moments at the serviceability limit state will normally be estimated using elastic analysis. Substantial errors may result where the actual in-service moments are likely to have redistributed significantly from the elastic distribution.

**TABLE 8.6.2.2(A)**  
**MAXIMUM STEEL STRESS FOR TENSION**  
**OR FLEXURE IN REINFORCED BEAMS**

Nominal bar diameter ( $d_b$ ) mm	Maximum steel stress MPa		
	$w'_{max} = 0.2 \text{ mm}$	$w'_{max} = 0.3 \text{ mm}$	$w'_{max} = 0.4 \text{ mm}$
10	190	265	335
12	175	245	305
16	255	215	270
20	140	195	240
24	125	175	215
28	115	160	200
32	105	150	185
36	100	140	175
40	90	130	165

**TABLE 8.6.2.2(B)**  
**MAXIMUM STEEL STRESS FOR FLEXURE**  
**IN REINFORCED BEAMS**

Centre-to-centre spacing mm	Maximum steel stress MPa		
	$w'_{max} = 0.2 \text{ mm}$	$w'_{max} = 0.3 \text{ mm}$	$w'_{max} = 0.4 \text{ mm}$
50	200	300	400
100	170	270	360
150	155	245	330
200	145	225	300
250	135	210	280
300	125	200	260



### 8.6.2.3 Crack control by calculation of crack widths

The calculated maximum crack width in a reinforced concrete member at any time after cracking is given by:

$$w = s_{r,\max} (\varepsilon_{sm} - \varepsilon_{cm}) \leq w'_{\max} \quad \dots 8.6.2.3(1)$$

where

$s_{r,\max}$  = the maximum crack spacing;

$\varepsilon_{sm}$  = the mean strain in the reinforcement at the design loads for the serviceability limit states, including the effects of tension stiffening and any imposed deformations; and

$\varepsilon_{cm}$  = the mean strain in the concrete between the cracks.

The difference between the mean strain in the reinforcement and the mean strain in the concrete may be taken as:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{scr}}{E_s} - \frac{0.6f_{ct}}{E_s \rho_{pe,ff}} (1 + n_e \rho_{pe,ff}) + \varepsilon_{cs} \geq 0.6 \frac{\sigma_{scr}}{E_s} \quad \dots 8.6.2.3(2)$$

where

$\sigma_{scr}$  = the stress in the tensile reinforcement assuming a cracked section

$\varepsilon_{cs}$  = the absolute value of the final long-term shrinkage strain determined in accordance with Clause 3.1.7

$n_e$  = the effective modular ratio  $(1 + \phi_{cc})E_s/E_c$

$\phi_{cc}$  = the creep coefficient associated with the time interval after cracking determined in accordance with Clause 3.1.8

$f_{ct}$  = the mean value of the axial tensile strength of concrete at the time cracking is expected

$\rho_{p,eff}$  = the reinforcement ratio given by  $A_{st}/A_{c,eff}$

$A_{c,eff}$  = the effective area of concrete in tension surrounding the bars with depth  $h_{c,ef}$  equal to the lesser of  $2.5(D-d)$ ,  $(D-kd)/3$  or  $D/2$

$D$  = the overall depth of the cross-section

$d$  = the effective depth to the centroid of the tensile steel

$kd$  = the depth of the neutral axis on the cracked section

For cross-sections with bonded reinforcement fixed at reasonably close centres, i.e. bar spacing  $\leq 5(c + 0.5d_b)$ , the maximum final crack width may be calculated from:

$$s_{r,\max} = 3.4c + 0.3k_1k_2d_b / \rho_{p,eff} \quad \dots 8.6.2.3(3)$$

where

$d_b$  = the tensile bar diameter

$c$  = the clear cover to the longitudinal reinforcement

$k_1$  = a coefficient that accounts for the bond properties of the bonded reinforcement, with  $k_1 = 0.8$  for deformed and  $k_1 = 1.6$  for plain bars

$k_2$  = a coefficient that accounts for the longitudinal strain distribution, with  $k_2 = 0.5$  for bending and  $k_2 = 1.0$  for pure tension. For cases in combined tension and bending,  $k_2 = (\varepsilon_1 + \varepsilon_2)/(2\varepsilon_1)$  and  $\varepsilon_1$  is the greater and  $\varepsilon_2$  is the lesser of the tensile strains at the boundaries of the cross-section (assessed on the basis of a cracked section)

### 8.6.3 Crack control for flexure in prestressed beams

Flexural cracking in a prestressed beam shall be deemed to be controlled if, under the short-term service loads, the resulting maximum tensile stress in the concrete does not exceed  $0.25\sqrt{f'_c}$  or, if this stress is exceeded, by providing reinforcement or bonded tendons, or both, near the tensile face with a centre-to-centre spacing not exceeding 300 mm and by either of the following—

- limiting the calculated maximum flexural tensile stress in the concrete under short-term service loads to  $0.6\sqrt{f'_c}$ ; or
- limiting the increment in steel stress near the tension face to that given in Table 8.6.3, as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the short-term service load value; or
- limiting the calculated maximum crack width in accordance with Clause 8.6.2.3.

**TABLE 8.6.3**  
**MAXIMUM INCREMENT OF STEEL STRESS**  
**FOR FLEXURE IN PRESTRESSED BEAMS**

Nominal reinforcement bar diameter ( $d_b$ ) mm	Maximum increment of steel stress MPa
≤12	320
16	280
20	240
24	210
≥28	200
All bonded tendons	200

### 8.6.4 Crack control in the side face of beams

For crack control in the side face of beams where the overall depth exceeds 750 mm, longitudinal reinforcement, consisting of 12 mm bars at 200 mm centres or 16 mm bars at 300 mm centres, shall be placed in each side face.

### 8.6.5 Crack control at openings and discontinuities

Reinforcement shall be provided for crack control at openings and discontinuities in a beam.

## 8.7 VIBRATION OF BEAMS

Vibration of beams shall be considered and appropriate action taken, where necessary, to ensure that vibrations induced by machinery, or vehicular or pedestrian traffic, will not adversely affect the serviceability of the structure.

## 8.8 T-BEAMS AND L-BEAMS

### 8.8.1 General

Where a slab is assumed to provide the flange of a T-beam or L-beam, the longitudinal shear capacity of the flange-web connection shall be checked in accordance with Clause 8.4.

For isolated T-beams or L-beams, the shear strength of the slab flange on vertical sections parallel to the beam shall also be checked in accordance with Clause 8.2.

## 8.8.2 Effective width of flange for strength and serviceability

In the absence of a more accurate determination, the effective width of the flange of a T beam or L-beam for strength and serviceability shall be taken as—

- (a) T-beams.....  $b_{ef} = b_w + 0.2a$ ; and
- (b) L-beams.....  $b_{ef} = b_w + 0.1a$ ,

where  $a$  is the distance between points of zero bending moment, which, for continuous beams, may be taken as  $0.7L$ .

In both Items (a) and (b) above, the overhanging part of the flange considered effective shall not exceed half the clear distance to the next member. The effective width so determined may be taken as constant over the entire span.

## 8.9 SLENDERNESS LIMITS FOR BEAMS

### 8.9.1 General

Unless a stability analysis is carried out, beams shall conform with the limits specified in Clauses 8.9.2 to 8.9.4, as appropriate.

### 8.9.2 Simply supported and continuous beams

For a simply supported or continuous beam, the distance  $L_1$  between points at which lateral restraint is provided shall be such that  $L_1/b_{ef}$  does not exceed the lesser of  $180 b_{ef}/D$  and 60.

### 8.9.3 Cantilever beams

For a cantilever beam having lateral restraint only at the support, the ratio of the clear projection ( $L_n$ ) to the width ( $b_{ef}$ ) at the support shall be such that  $L_n/b_{ef}$  does not exceed the lesser of  $100 b_{ef}/D$  and 25.

### 8.9.4 Reinforcement for slender prestressed beams

For a prestressed beam in which  $L_1/b_{ef}$  exceeds 30, or for a prestressed cantilever beam in which  $L_n/b_{ef}$  exceeds 12, the following reinforcement shall be provided:

- (a) Stirrups providing a steel area,  $A_{sv.min}$  in accordance with Clause 8.2.8.
- (b) Additional longitudinal reinforcement, consisting of at least one bar in each corner of the compression face, such that—

$$A_{sc} \geq 0.35 A_{pt} f_{pb} / f_{sy}$$

... 8.9.4

## SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY

### 9.1 STRENGTH OF SLABS IN BENDING

#### 9.1.1 General

The strength of a slab in bending shall be determined in accordance with Clauses 8.1.1 to 8.1.8, except that for two-way reinforced slabs, the minimum strength requirements of Clause 8.1.6.1 shall be deemed to be satisfied by providing tensile reinforcement such that  $A_{st}/bd$  is not less than the following in each direction:

- |  |                                  |
|--|----------------------------------|
| (a) Slabs supported by columns at their corners.....     | $0.24(D/d)^2 f'_{ct.f} / f_{sy}$ |
| (b) Slabs supported by beams or walls on four sides..... | $0.19(D/d)^2 f'_{ct.f} / f_{sy}$ |

#### 9.1.2 Reinforcement and tendon distribution in two-way flat slabs

In two-way flat slabs, at least 25% of the total of the design negative moment in a column-strip and adjacent half middle-strips shall be resisted by reinforcement or tendons or both, located in a cross-section of slab centred on the column and of a width equal to twice the overall depth of the slab or drop panel plus the width of the column.

#### 9.1.3 Detailing of tensile reinforcement in slabs

##### 9.1.3.1 General procedure for arrangement

Tensile reinforcement shall be arranged in accordance with the following, as appropriate:

- (a) Where the bending moment envelope has been calculated, the termination and anchorage of flexural reinforcement shall be based on a hypothetical bending-moment diagram formed by displacing the calculated positive and negative bending-moment envelopes a distance  $D$  along the slab from each side of the relevant sections of maximum moment. Additionally, the following shall apply:
  - (i) Not less than one third of the total negative moment reinforcement required at a support shall be extended a distance  $12d_b$  or  $D$ , whichever is greater, beyond the point of contraflexure.
  - (ii) At a simply supported discontinuous end of a slab, not less than one half of the total positive moment reinforcement required at midspan shall be anchored by extension past the face of the support for a distance of  $12d_b$  or  $D$ , whichever is greater, or by an equivalent anchorage.  
  
Where no shear reinforcement is required in accordance with Clause 8.2.5 or Clause 9.2, the extension of the midspan positive moment reinforcement past the face of the support may be reduced to  $8d_b$  if at least one half of the reinforcement is so extended, or to  $4d_b$  if all the reinforcement is so extended.
  - (iii) At a support where the slab is continuous or flexurally restrained, not less than one quarter of the total positive moment reinforcement required at midspan shall continue past the near face of the support.
  - (iv) Where frames incorporating slabs are intended to resist lateral loading, the effects of such loading on the arrangement of the slab reinforcement shall be taken into account but in no case shall the lengths of reinforcement be made less than those shown in Figures 9.1.3.2 and 9.1.3.4, as appropriate.

- (b) Where the bending moment envelope has not been calculated, the requirements of Clauses 9.1.3.2, 9.1.3.3 or 9.1.3.4, as appropriate to the type of slab, shall be satisfied.

**9.1.3.2 Deemed-to-conform arrangement for one-way slabs**

For one-way slabs continuous over two or more spans analysed using simplified elastic analysis, as detailed in Clause 6.10.2, where—

- (a) the ratio of the longer to the shorter of any two adjacent spans does not exceed 1.2; and
- (b) the imposed actions (live loads) may be assumed to be uniformly distributed and the imposed action (live load) ( $q$ ) is not greater than twice the permanent action (dead load) ( $g$ ),

the arrangement of tensile reinforcement shown in Figure 9.1.3.2 shall be deemed to conform with Clause 9.1.3.1(a).

Where adjacent spans are unequal, the extension of negative moment reinforcement beyond each face of the common support shall be based on the longer span.

For one-way slabs of single span, the arrangement of tensile reinforcement shown in Figure 9.1.3.2, for the appropriate end support conditions, shall be deemed to conform with Clause 9.1.3.1(a).

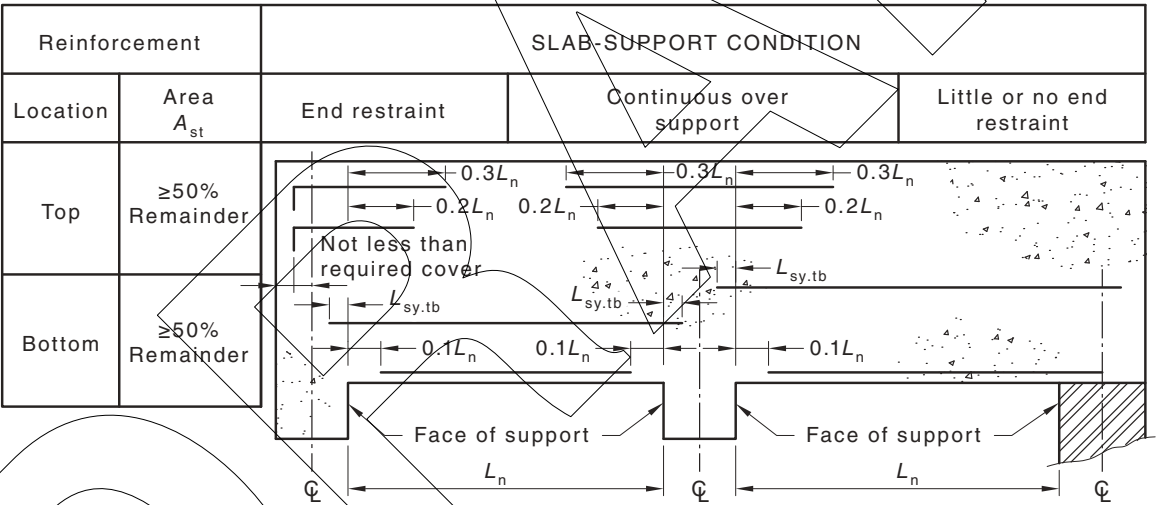


FIGURE 9.1.3.2 ARRANGEMENT OF REINFORCEMENT

**9.1.3.3 Deemed-to-conform arrangement for two-way slabs supported on beams or walls**

For two-way simply supported or continuous rectangular slabs supported by walls or beams on four sides analysed using simplified elastic analysis, as detailed in Clause 6.10.3, the following deemed-to-conform arrangement shall be used and the arrangement of tensile reinforcement, shown in Figure 9.1.3.2 and further prescribed herein, shall be deemed to conform with Clause 9.1.3.1(a):

- (a) The arrangement shall apply to each direction.
- (b) Where a simply supported or continuous slab is not square, the arrangement shall be based on the span ( $L_n$ ) taken as the shorter span.
- (c) Where adjacent continuous rectangular slabs have unequal shorter spans, the extension of negative moment reinforcement beyond each face of a common support shall be based on the span ( $L_n$ ) taken as the longer of the shorter spans.

- (d) Negative moment reinforcement provided at a discontinuous edge shall extend from the face of the support into the span for a distance of 0.15 times the shorter span.
- (e) At an exterior corner of a two-way rectangular slab supported on four sides and restrained against uplift, reinforcement shall be provided in both the top and the bottom of the slab. This reinforcement shall consist of two layers perpendicular to the edges of the slab and extend from each edge for a distance not less than 0.2 times the shorter span. The area of the reinforcement in each of the four layers shall be not less than—
  - (i) for corners where neither edge is continuous .....  $0.75A_{st}$ ; and
  - (ii) for corners where one edge is continuous .....  $0.5A_{st}$ ,where  $A_{st}$  is the area of the maximum positive moment reinforcement required at midspan.

Any reinforcement provided may be considered as part of this reinforcement.

**9.1.3.4 Deemed-to-conform arrangement for two-way flat slabs**

For multispans, reinforced, two-way flat slabs analysed using simplified elastic analysis, as detailed in Clause 6.10.4, the following deemed to conform arrangement shall be used and the arrangement of tensile reinforcement, shown in Figure 9.1.3.4 and further prescribed herein, shall be deemed-to-conform with Clause 9.1.3.1(a).

Where adjacent spans are unequal, the extension of negative moment reinforcement beyond each face of the common support shall be based on the longer span.

All slab reinforcement perpendicular to a discontinuous edge shall be extended (straight, bent or otherwise) past the internal face of the spandrel, wall or column for a length—

- (a) for positive moment reinforcement, not less than 150 mm except that it shall extend as close as permitted to the edge of the slab if there is no spandrel beam or wall; and
- (b) for negative moment reinforcement, such that the calculated force is developed at the internal face in accordance with Clause 13.1.

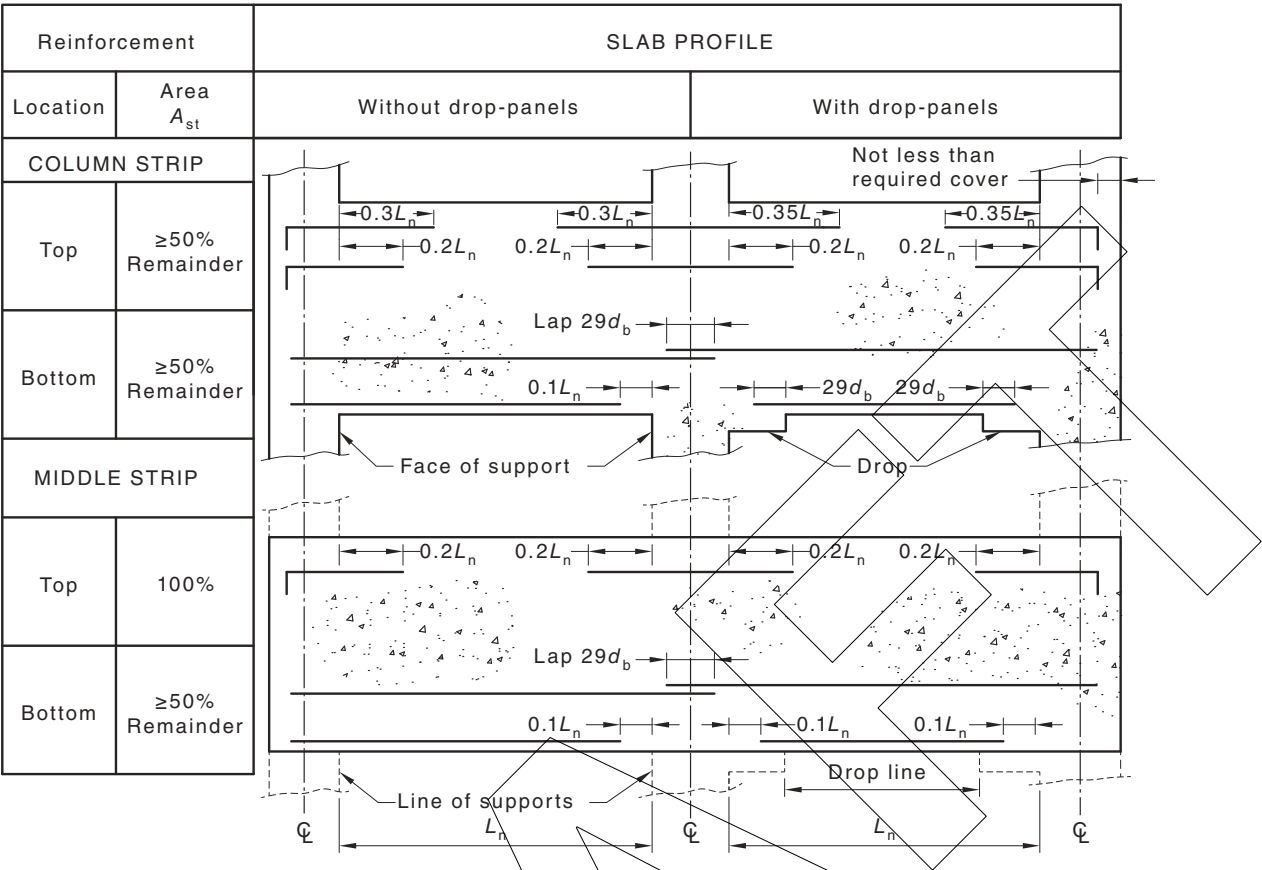


FIGURE 9.1.3.4 ARRANGEMENT OF REINFORCEMENT

9.2 STRUCTURAL INTEGRITY REINFORCEMENT

9.2.1 General

Reinforcement at connections shall be provided to increase the resistance of the structural system to progressive collapse at walls and columns.

At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

9.2.2 Minimum structural integrity reinforcement

The summation of the area of bottom reinforcement connecting the slab, drop panel, or slab band to the column or column capital on all faces of the periphery of a column or column capital shall be not less than,

$$\frac{A_{s,min}}{\phi f_{sy}} = 2N^*$$

... 9.2.2

in which  $\phi = 0.8$  and  $N^*$  is the column reaction from the floor slab.

Integrity reinforcement shall not be required if there are beams containing shear reinforcement and with at least two bottom bars continuous through the joint in all spans framing into the column.

This reinforcement should be placed within the column core and may consist of the following:

- (a) Bottom reinforcement extended to lap over the column in accordance with Clause 13.2.

- (b) Extra bars over the column extending a minimum distance of  $2 L_{sy.tb}$  from the face of the column or column capital and lapping with any existing bottom reinforcement. These bars should have hooked or cogged ends where terminating at discontinuous edges.

### 9.2.3 Minimum reinforcement for distributing loads

Minimum reinforcement in a secondary direction shall be provided for the purpose of distributing loads.

NOTE: For shrinkage and temperature effects see Clause 9.4.3.

### 9.2.4 Spacing of reinforcement and tendons

The minimum clear distance between parallel bars (including bundled bars), ducts and tendons shall be such that the concrete can be properly placed and compacted in accordance with Clause 17.1.3.

The maximum spacing of reinforcement and tendons for crack control shall be determined in accordance with Clause 9.4.

Where the plain concrete between tendons is unable to safely distribute the applied loads to the tendons, adequate reinforcement for this purpose shall be supplied transverse to the tendons in a prestressed slab. Unless calculations show that a wider spacing is possible, the maximum spacing of tendons in an unreinforced slab subjected to uniformly distributed loads shall be the smaller of 10 times the slab thickness and 1500 mm.

NOTE: The tendon spacing adjacent to a supporting column will be controlled by Clause 9.1.2 and may need to be significantly less than the above limits.

## 9.3 STRENGTH OF SLABS IN SHEAR

### 9.3.1 Definitions and symbols

For the purpose of this clause, the definitions and symbols below apply to flat slabs.

#### 9.3.1.1 Critical shear perimeter

The perimeter defined by a line geometrically similar to the boundary of the effective area of a support or concentrated load and located at a distance of  $d_{om}/2$  therefrom [see Figure 9.3(A)].

#### 9.3.1.2 Critical opening

Any opening through the thickness of a slab where an edge, or part of the edge, of the opening is located at a clear distance of less than  $2.5b_o$  from the critical shear perimeter [see Figure 9.3(A)].

#### 9.3.1.3 Effective area of a support or concentrated load

The area totally enclosing the actual support or load and for which the perimeter is a minimum [see Figure 9.3(A)].

#### 9.3.1.4 Torsion strip

A strip of slab of width  $a$ , whose longitudinal axis is perpendicular to the direction of  $M_v^*$  [see Figure 9.3(B)].



### 9.3.1.5 Symbols

The following symbols apply:

- $a$  = dimension of the critical shear perimeter measured parallel to the direction of  $M_v^*$  [see Figure 9.3(B)]
- $b_o$  = dimension of an opening [see Figure 9.3(A)]
- $b_w$  = width of the web of a spandrel beam [see Figure 9.3(B)]
- $D_b$  = overall depth of a spandrel beam (see Figure 9.2.6)
- $D_s$  = overall depth of a slab or drop panel as appropriate
- $d_{om}$  = mean value of  $d_o$ , averaged around the critical shear perimeter
- $M_v^*$  = bending moment transferred from the slab to a support in the direction being considered [see Figure 9.3(B)]
- $u$  = length of the critical shear perimeter [see Figure 9.3(A)]
- $y_1$  = larger overall dimension of a closed fitment (see Figure 9.3.6)
- $\beta_h$  = ratio of the longest overall dimension of the effective loaded area,  $Y$ , to the overall dimension,  $X$ , measured perpendicular to  $Y$  [see Figure 9.3(A)]

### 9.3.2 Strength

The strength of a slab in shear shall be determined in accordance with the following:

- Where shear failure can occur across the width of the slab, the design shear strength of the slab shall be calculated in accordance with Clause 8.2.
- Where shear failure can occur locally around a support or concentrated load, the design shear strength of the slab shall be taken as  $\phi V_u$ , where  $V_u$  is calculated in accordance with one of the following, as appropriate:
  - Where  $M_v^*$  is zero,  $V_u$  is taken as equal to  $V_{uo}$  calculated in accordance with Clause 9.2.3.
  - Where  $M_v^*$  is not zero,  $V_u$  is calculated in accordance with Clause 9.3.4.

NOTE: For types of shear reinforcement other than those covered in Clauses 9.3.3 and 9.3.4 strength may be determined by tests, in accordance with Appendix B.

### 9.3.3 Ultimate shear strength where $M_v^*$ is zero

The ultimate shear strength of a slab where  $M_v^*$  is zero,  $V_{uo}$  is given by either—

- where there is no shear head—

$$V_{uo} = u d_{om} (f_{cv} + 0.3 \sigma_{cp}) \quad \dots 9.3.3(1)$$

where

$$f_{cv} = 0.17 \left( 1 + \frac{2}{\beta_h} \right) \sqrt{f'_c} \leq 0.34 \sqrt{f'_c}; \text{ or}$$

NOTE: The value of  $\sigma_{cp}$  should be evaluated separately for the case of corner, edge and internal columns.

- where there is a shear head—

$$V_{uo} = u d_{om} (0.5 \sqrt{f'_c} + 0.3 \sigma_{cp}) \leq 0.2 u d_{om} f'_c \quad \dots 9.3.3(2)$$

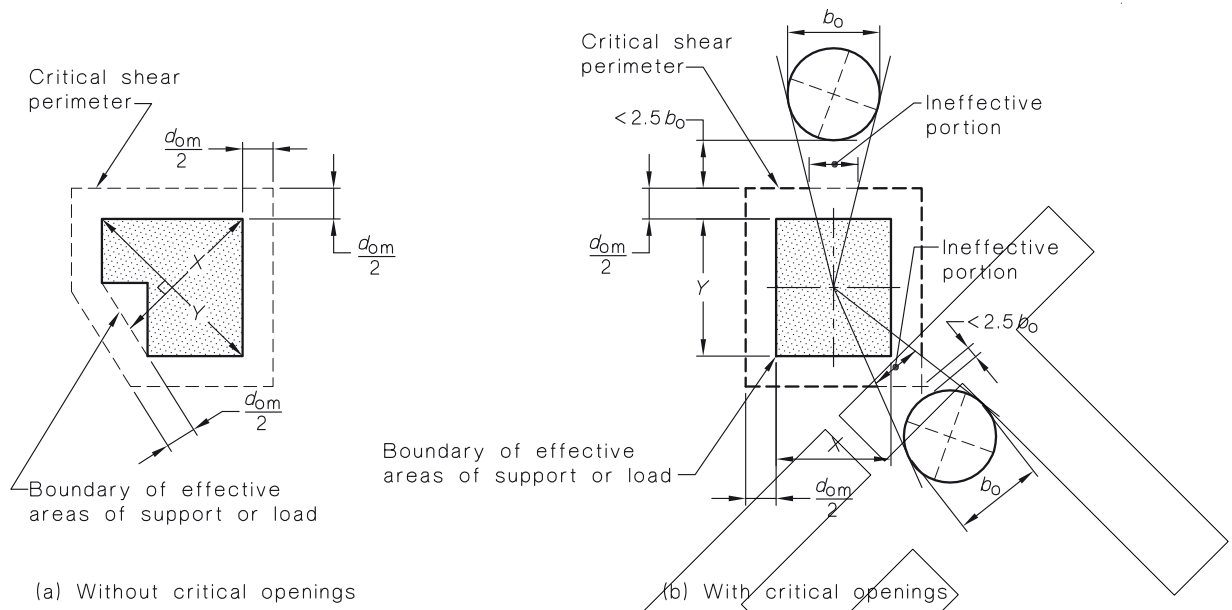


FIGURE 9.3(A) CRITICAL SHEAR PERIMETER

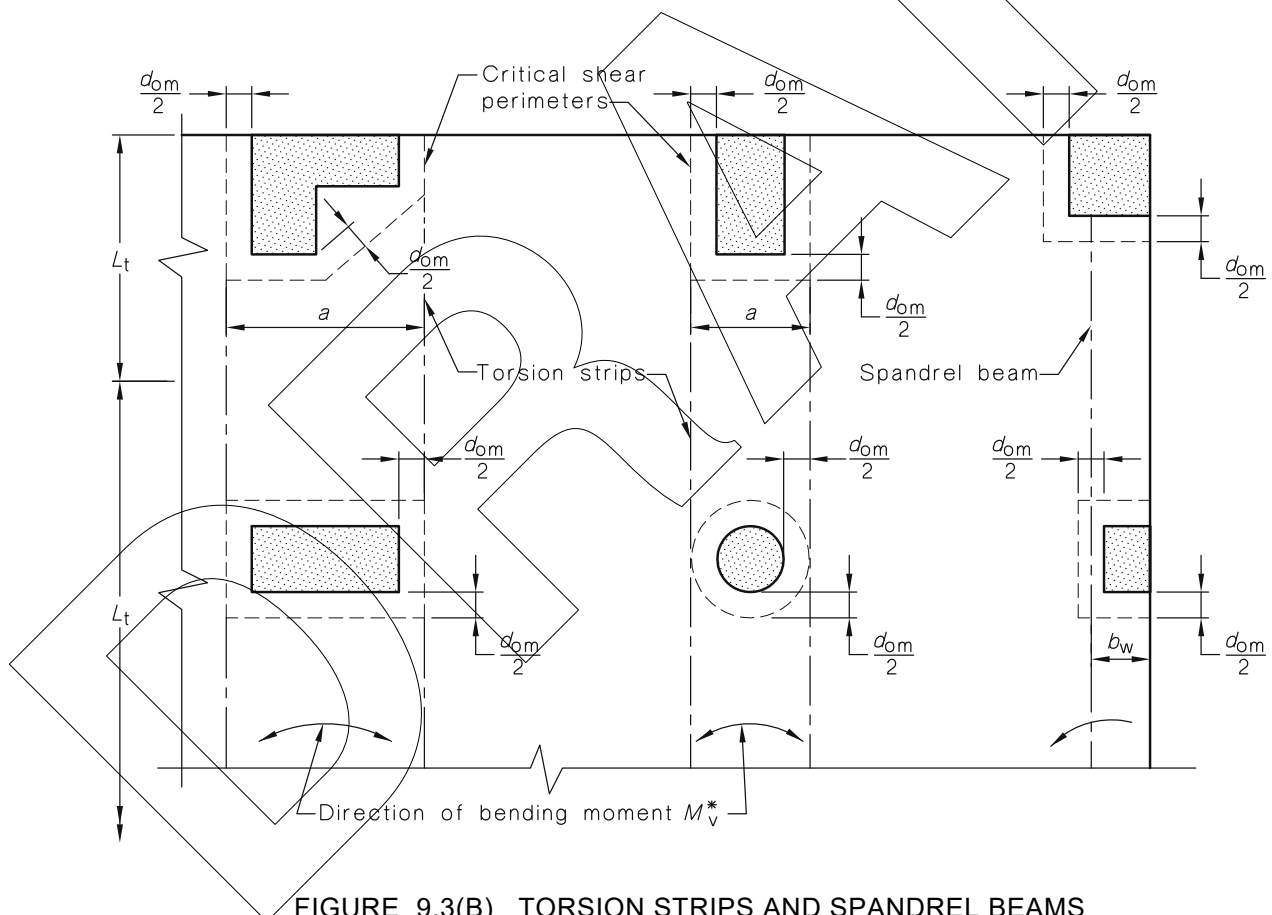


FIGURE 9.3(B) TORSION STRIPS AND SPANDREL BEAMS

### 9.3.4 Ultimate shear strength where $M_v^*$ is not zero

Where  $M_v^*$  is not zero and shear reinforcement, if provided, complies with Clauses 9.3.5 and 9.2.6, then  $V_u$  shall be determined from one of the following:

- (a) If there are no closed fitments in the torsion strip or spandrel beams,  $V_u$  is given by—

$$V_u = V_{uo} / [1.0 + uM_v^* / (8V^* ad_{om})] \quad \dots 9.3.4(1)$$

- (b) If the torsion strip contains the minimum quantity of closed fitments,  $V_u$  shall be taken as  $V_{u,min}$  given by—

$$V_{u,min} = 1.2 V_{uo} / [1.0 + uM_v^* / (2V^* a^2)] \quad \dots 9.3.4(2)$$

- (c) If there are spandrel beams perpendicular to the direction of  $M_v^*$  which contain the minimum quantity of closed fitments,  $V_u$  shall be taken as  $V_{u,min}$  given by—

$$V_{u,min} = 1.2 V_{uo} (D_b / D_s) / [1.0 + uM_v^* / (2V^* ab_w)] \quad \dots 9.3.4(3)$$

- (d) If the torsion strip or spandrel beam contains more than the minimum quantity of closed fitments,  $V_u$  is given by—

$$V_u = V_{u,min} \sqrt{[(A_{sw} / s) / (0.2y_1 / f_{sy,f})]} \quad \dots 9.3.4(4)$$

where  $V_{u,min}$  is calculated in accordance with Item (b) or (c), as appropriate.

In no case shall  $V_u$  be taken greater than  $V_{u,max}$  given by—

$$V_{u,max} = 3V_{u,min} \sqrt{(x/y)} \quad \dots 9.3.4(5)$$

where  $x$  and  $y$  are the shorter and longer dimensions respectively of the cross-section of the torsion strip or spandrel beam.

### 9.3.5 Minimum area of closed fitments

The minimum cross-sectional area of the reinforcement forming the closed fitments shall satisfy the following inequality:

$$A_{sw} / s \geq 0.2y_1 / f_{sy,f} \quad \dots 9.3.5$$

### 9.3.6 Detailing of shear reinforcement

Reinforcement for slab shear in torsion strips and spandrel beams shall be in the form of closed fitments arranged and detailed in accordance with the following:

- The fitments shall extend along the torsion strip or spandrel beam for a distance not less than  $L_d/4$  from the face of the support or concentrated load, on one or both sides of the centroid axis, as applicable. The first fitment shall be located at not more than  $0.5s$  from the face of the support.
- The centre-to-centre spacing ( $s$ ) of the fitments shall not exceed the greater of 300 mm and  $D_b$  or  $D_s$ , as applicable.
- At least one longitudinal bar shall be provided at each corner of the fitment.
- The dimensions of the fitments shall be in accordance with Figure 9.3.6.

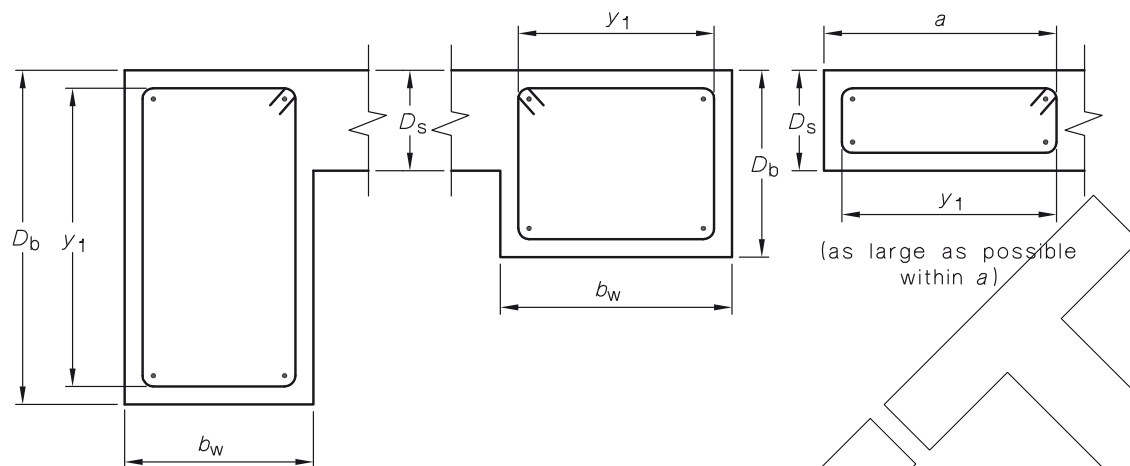


FIGURE 9.3.6 PARAMETERS AND DETAILS OF SHEAR REINFORCEMENT FOR SLABS

## 9.4 DEFLECTION OF SLABS

### 9.4.1 General

The deflection of a slab shall be determined in accordance with Clause 9.3.2 or Clause 9.3.3.

Alternatively, for reinforced slabs, the effective span-to-effective depth ratio of the slab shall conform with Clause 9.3.4.

### 9.4.2 Slab deflection by refined calculation

The calculation of the deflection of a slab by refined calculation shall make allowance for the following:

- Two-way action.
- Cracking and tension stiffening.
- Shrinkage and creep properties of the concrete.
- Expected load history.
- Expected construction procedure.
- Deflection of formwork or settlement of props during construction, particularly when the slab formwork is supported off suspended floors below.

### 9.4.3 Slab deflection by simplified calculation

The deflection of a slab subject to uniformly distributed loads shall be calculated in accordance with Clause 8.5.3 on the basis of an equivalent beam taken as follows:

- For a one-way slab, a prismatic beam of unit width.
- For a rectangular slab supported on four sides, a prismatic beam of unit width through the centre of the slab, spanning in the short direction  $L_x$ , with the same conditions of continuity as the slab in that direction and with the load distributed so that the proportion of load carried by the beam is given by—

$$L_y^4 / (\alpha L_x^4 + L_y^4) \quad \dots 9.4.3$$

where  $\alpha$  is given in Table 9.4.3 for the appropriate slab-edge condition.

- (c) For a two-way flat slab having multiple spans (for deflections on the column lines or midway between the supports), the column strips of the idealized frame described in Clause 6.9.

**TABLE 9.4.3**  
**COEFFICIENT OF PROPORTIONALITY ( $\alpha$ )**

Edge condition		Coefficient ( $\alpha$ )
1	Four edges continuous	1.0
2	One short edge discontinuous	0.5
3	One long edge discontinuous	2.0
4	Two short edges discontinuous	0.2
5	Two long edges discontinuous	5.0
6	Two adjacent edges discontinuous	1.0
7	Three edges discontinuous (one long edge continuous)	0.4
8	Three edges discontinuous (one short edge continuous)	2.5
9	Four edges discontinuous	1.0

#### 9.4.4 Deemed to conform span-to-depth ratio for reinforced slabs

##### 9.4.4.1 One-way slabs and two-way flat slabs

For a reinforced one-way slab, or a multiple-span reinforced two-way flat slab of essentially uniform depth, fully propped during construction, subject to uniformly distributed loads and where the imposed action (live load) ( $q$ ) does not exceed the permanent action (dead load) ( $g$ ), slab deflections shall be deemed to conform with the requirements of Clause 2.3.2 if the ratio of the effective span to the effective depth satisfies the following:

$$L_{\text{ef}} / d \leq k_3 k_4 \left[ \frac{(\Delta / L_{\text{ef}}) 1000 E_c}{F_{\text{d.ef}}} \right]^{1/3} \quad \dots 9.4.4.1$$

NOTE:  $E_c$  is in megapascals.

where

$\Delta / L_{\text{ef}}$  = deflection limit selected in accordance with Clause 2.3.2 and the deflection ( $\Delta$ ) is taken on the centre-line between the supports used to calculate  $L_{\text{ef}}$

$L_{\text{ef}}$  = effective span

$F_{\text{d.ef}}$  = effective design service load, per unit area, in kilopascals, taken as—

(a)  $(1.0 + k_{\text{cs}})g + (\psi_s + k_{\text{cs}}\psi_l)q$  for total deflection; or

(b)  $k_{\text{cs}} g + (\psi_s + k_{\text{cs}}\psi_l)q$  for the deflection that occurs after the addition or attachment of the brittle partitions or finishes

$k_{\text{cs}}$  is determined in accordance with Clause 8.5.3.2 and  $\psi_s$  and  $\psi_l$  are given in AS/NZS 1170.0

$k_3$  = 1.0 for a one-way slab

= 0.95 for a two-way flat slab without drop panels

= 1.05 for a two-way flat slab with drop panels, which extend at least  $L/6$  in each direction on each side of a support centre-line and have an overall depth not less than  $1.3D$ , where  $D$  is the slab thickness beyond the drops

$k_4$  = deflection constant, which may be taken as—

- (a) for simply supported slabs, 1.4; or
- (b) for continuous slabs, where in adjoining spans the ratio of the longer span to the shorter span does not exceed 1.2 and where no end span is longer than an interior span—
  - (i) 1.75 in an end span; or
  - (ii) 2.1 in interior spans

#### 9.4.4.2 Rectangular slabs supported on four sides

For a reinforced concrete slab, supported on four sides by walls or beams, subject to uniformly distributed loads and where the imposed action (live load) ( $q$ ) does not exceed the permanent action (dead load) ( $g$ ), the slab deflection shall be deemed to conform with the requirements of Clause 2.3.2 if the ratio of the shorter effective span to the effective depth satisfies the requirements given in Clause 9.3.4.1, except that—

- (a)  $k_3$  shall be taken as 1.0; and
- (b) the appropriate value of  $k_4$  shall be taken from Table 9.4.4.2.

**TABLE 9.4.4.2**  
**SLAB-SYSTEM MULTIPLIER ( $k_4$ ) FOR RECTANGULAR SLABS**  
**SUPPORTED ON FOUR SIDES**

Edge condition	Deflection constant ( $k_4$ )			
	Ratio of long to short side ( $L_y/L_x$ )			
	1.0	1.25	1.5	2.0
1 Four edges continuous	3.60	3.10	2.80	2.50
2 One short edge discontinuous	3.40	2.90	2.70	2.40
3 One long edge discontinuous	3.40	2.65	2.40	2.10
4 Two short edges discontinuous	3.20	2.80	2.60	2.40
5 Two long edges discontinuous	3.20	2.50	2.00	1.60
6 Two adjacent edges discontinuous	2.95	2.50	2.25	2.00
7 Three edges discontinuous (one long edge continuous)	2.70	2.30	2.20	1.95
8 Three edges discontinuous (one short edge continuous)	2.70	2.10	1.90	1.60
9 Four edges discontinuous	2.25	1.90	1.70	1.50

## 9.5 CRACK CONTROL OF SLABS

### 9.5.1 General requirements for reinforced concrete slabs

Cracking shall be limited to an extent that does not impair the durability or serviceability of the slab, both in terms of function and appearance. A characteristic maximum crack width  $w'_{\max}$  shall be selected for each surface of a slab taking into account the function and exposure of the surface.

For regions of slabs fully enclosed within a building except for a brief period of weather exposure during construction, and where it is assessed that cracking will not impair the functioning of the structure, only Items (a) and (b) need be satisfied:

- (a) The minimum area of reinforcement in a tensile zone of a beam shall conform with Clause 9.1.1.
- (b) The centre-to-centre spacing of bars in each direction shall not exceed the lesser of  $2.0D_s$  or 300 mm. Bars with a diameter less than half the diameter of the largest bar in the cross-section shall be ignored.

In other situations, in addition to (a) and (b) above, crack widths may be calculated and limited to the selected value of  $w'_{\max}$  in accordance with Clause 9.5.2.2. A simplified alternative approach is to limit the size and spacing of reinforcement crossing the cracks in accordance with Clause 9.5.2.1.

NOTES:

- 1 For prestressed elements with unbonded tendons, the requirements for reinforced concrete elements apply. For prestressed elements with bonded tendons, the requirements of Clause 9.5.2.3 apply.
- 2 It is possible that the width of some cracks in the actual structure may exceed the maximum characteristic value specified in design.

## 9.5.2 Crack control for flexure in reinforced slabs

### 9.5.2.1 Crack control without direct calculation of crack widths

The calculated tensile steel stress ( $\sigma_{scr}$ ) shall not exceed the larger of the maximum steel stresses given in—

- (a) Table 9.5.2.1(A) for the largest nominal diameter ( $d_b$ ) of the bars in the tensile zone; and
- (b) Table 9.5.2.1(B) for the largest centre-to-centre spacing of adjacent parallel bars in the tensile zone and, when determining spacing, bars with a diameter less than half the diameter of the largest bar in the section shall be ignored.

NOTE: Design bending moments at the serviceability limit state will normally be estimated using elastic analysis. Substantial errors may result where the actual in-service moments are likely to have redistributed significantly from the elastic distribution.

**TABLE 9.5.2.1(A)**  
**MAXIMUM STEEL STRESS FOR FLEXURE IN REINFORCED SLABS**

Nominal bar diameter ( $d_b$ ) mm	Maximum steel stress (MPa) for overall depth, $D_s$ (mm)					
	$w'_{\max} = 0.2 \text{ mm}$		$w'_{\max} = 0.3 \text{ mm}$		$w'_{\max} = 0.4 \text{ mm}$	
	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$
6	210	230	290	315	365	390
8	195	210	260	290	340	360
10	180	190	240	265	315	335
12	165	175	225	245	290	305
16	150	155	205	215	260	270
20	140	140	190	195	235	240
24	125	125	175	175	215	215

**TABLE 9.5.2.1(B)**  
**MAXIMUM STEEL STRESS FOR FLEXURE IN REINFORCED SLABS**

Centre-to-centre spacing mm	Maximum steel stress (MPa)		
	$w'_{\max} = 0.2 \text{ mm}$	$w'_{\max} = 0.3 \text{ mm}$	$w'_{\max} = 0.4 \text{ mm}$
50	200	300	400
100	170	270	360
150	155	245	330
200	145	225	300
250	135	210	280
300	125	200	260

### 9.5.2.2 Crack control by calculation of crack widths

The calculated maximum crack width in a reinforced concrete member at any time after cracking shall be determined in accordance with Clause 8.6.2.3.

### 9.5.2.3 Crack control for flexure in prestressed slabs

Flexural cracking, in a prestressed slab shall be deemed to be controlled if, under the short-term service loads, the resulting maximum tensile stress in the concrete does not exceed  $0.25\sqrt{f'_c}$  or, if this stress is exceeded, by providing reinforcement or bonded tendons, or both, near the tensile face with a centre-to-centre spacing not exceeding the lesser of 300 mm or  $2.0D_s$  and by one of the following

- limiting the calculated maximum flexural tensile stress in the concrete under short-term service loads to  $0.6\sqrt{f'_c}$ ; or
- limiting the increment in steel stress near the tension face to that given in Table 9.5.2.3, as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the short-term service load value; or
- limiting the calculated maximum crack width in accordance with Clause 8.6.2.2.

**TABLE 9.5.2.3**  
**MAXIMUM INCREMENT OF STEEL STRESS FOR FLEXURE**  
**IN PRESTRESSED SLABS**

Nominal bar diameter ( $d_b$ ) mm	Maximum steel stress (MPa) for overall depth, $D_s$ (mm)					
	$w'_{\max} = 0.2 \text{ mm}$		$w'_{\max} = 0.3 \text{ mm}$		$w'_{\max} = 0.4 \text{ mm}$	
	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$
$\leq 10$	245	280	300	340	350	390
12	210	240	270	300	310	340
16	180	200	240	260	280	300
20	160	170	220	220	255	260
24	145	145	195	195	225	225
$\geq 28$	120	120	185	185	210	210
All bonded tendons	120	120	185	185	210	210



### 9.5.3 Crack control for shrinkage and temperature effects

#### 9.5.3.1 General

The area of reinforcement required to control cracking due to shrinkage and temperature effects shall take into account the influence of flexural action, the degree of restraint against in-plane movements and the exposure classification, in accordance with Clauses 9.5.3.2 to 9.5.3.5.

For members greater than 500 mm thick, the reinforcement required near each surface may be calculated using 250 mm for  $D$ .

#### 9.5.3.2 Reinforcement in the primary direction

No additional reinforcement is required to control expansion or contraction cracking if the area of reinforcement in the direction of the span of a one-way slab, or in each direction of a two-way slab, is not less than—

- (a) the area required by Clause 9.1.1; and
- (b) 75% of the area required by one of Clauses 9.5.3.3 to 9.5.3.5, as appropriate. The maximum centre-to-centre spacing of bars required to control cracking due to shrinkage and temperature effects in the primary direction shall conform with Clause 9.5.1(b).

#### 9.5.3.3 Reinforcement in the secondary direction in unrestrained slabs

Where the slab is free to expand or contract in the secondary direction, the minimum area of reinforcement in that direction shall be  $(1.75-2.5 \sigma_{cp}) bD \times 10^{-3}$ .

#### 9.5.3.4 Reinforcement in the secondary direction in restrained slabs

Where a slab is restrained from expanding or contracting in the secondary direction, the area of reinforcement in that direction shall be not less than the following, as appropriate:

- (a) For a slab fully enclosed within a building except for a brief period of weather exposure during construction:
  - (i) Where a minor degree of control over cracking is required .....  $(1.75-2.5 \sigma_{cp}) bD \times 10^{-3}$ .
  - (ii) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view .....  $(3.5-2.5 \sigma_{cp}) bD \times 10^{-3}$ .
  - (iii) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes .....  $(6.0-2.5 \sigma_{cp}) bD \times 10^{-3}$ .
- (b) For all other surface and exposure environments in classification A1, and for exposure classification A2:
  - (i) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view .....  $(3.5-2.5 \sigma_{cp}) bD \times 10^{-3}$ .
  - (ii) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes .....  $(6.0-2.5 \sigma_{cp}) bD \times 10^{-3}$ .

- (c) For exposure classifications B1, B2, C1 and C2 .....  $(6.0 - 2.5\sigma_{cp})bD \times 10^{-3}$ .

The centre-to-centre spacing of bars required to provide a moderate degree of control over cracking shall not exceed the lesser of  $2.0D_s$  or 300 mm. For a strong degree of control over cracking, the centre-to-centre spacing of bars shall not exceed the lesser of  $1.5D_s$  or 200 mm.

#### 9.5.3.5 Reinforcement in the secondary direction in partially restrained slabs

Where a slab is partially restrained from expanding or contracting in the secondary direction, the minimum area of reinforcement in that direction shall be assessed taking into account the requirements of Clauses 9.5.3.3 and 9.5.3.4.

#### 9.5.4 Crack control in the vicinity of restraints

In the vicinity of restraints, special attention shall be paid to the internal forces and cracks that may be induced by prestressing, shrinkage or temperature.

#### 9.5.5 Crack control at openings and discontinuities

For crack control at openings and discontinuities in a slab, additional properly anchored reinforcement shall be provided if necessary.

### 9.6 VIBRATION OF SLABS

Vibration in slabs shall be considered and appropriate action taken, where necessary, to ensure that the vibrations induced by machinery, or vehicular or pedestrian traffic, will not adversely affect the serviceability of the structure.

### 9.7 MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS

The width of a solid one-way simply supported or continuous slab deemed to resist the moments caused by a concentrated load, may be taken as follows:

- (a) Where the load is not near an unsupported edge—

$$b_{cr} = \text{the load width} + 2.4a[1.0 - (a/L_n)] \quad \dots 9.7$$

where

$a$  = perpendicular distance from the nearer support to the section under consideration

- (b) Where the load is near an unsupported edge, not greater than the lesser of—

- (i) the value given in Item (a) above; and
- (ii) half the value given in Item (a) above plus the distance from the centre of the load to the unsupported edge.

### 9.8 LONGITUDINAL SHEAR IN COMPOSITE SLABS

Composite slab systems shall be checked for longitudinal shear at the interfaces between components, in accordance with Clause 8.4.

## SECTION 10 DESIGN OF COLUMNS FOR STRENGTH AND SERVICEABILITY

### 10.1 GENERAL

#### 10.1.1 Design strength

The design strength of a column shall be determined by its ability to resist the axial forces, shear forces and bending moments caused by the design loading for strength and any additional bending moments produced by slenderness effects.

#### 10.1.2 Minimum bending moment

At any cross-section of a column, the design bending moment about each principal axis shall be taken to be not less than  $N^*$  times  $0.05D$ , where  $D$  is the overall depth of the column in the plane of the bending moment.

#### 10.1.3 Definitions

For the purpose of this section the definitions below apply.

##### 10.1.3.1 Braced column

Column in a structure for which the lateral actions applied at the ends in the direction under consideration are resisted by components such as masonry infill panels, shear walls or lateral bracing.

##### 10.1.3.2 Short column

Column in which the additional bending moments due to slenderness can be taken as zero.

##### 10.1.3.3 Slender column

Column that does not satisfy the requirements for a short column.

### 10.2 DESIGN PROCEDURES

#### 10.2.1 Design procedure using linear elastic analysis

Where the axial forces and bending moments are determined by a linear elastic analysis, as provided in Clause 6.2, a column shall be designed as follows:

- (a) For a short column, in accordance with Clauses 10.3, 10.6 and 10.7.
- (b) For a slender column, in accordance with Clauses 10.4 to 10.7.

The value of  $\phi$  shall be determined from Table 2.2.2.

#### 10.2.2 Design procedure incorporating secondary bending moments

Where the axial forces and bending moments are determined by an elastic analysis incorporating secondary bending moments due to lateral joint displacements, as provided in Clause 6.3, a column shall be designed in accordance with Clauses 10.6 and 10.7. The bending moments in slender columns shall be further increased by applying the moment magnifier for a braced column ( $\delta_b$ ) calculated in accordance with Clause 10.4.2 with  $L_e$  taken as  $L_u$  in the determination of  $N_c$ .

The value of  $\phi$  shall be determined from Table 2.2.2.

#### 10.2.3 Design procedure using rigorous analysis

Where the axial forces and bending moments are determined by a rigorous analysis, as provided in Clause 6.5 and 6.6, a column shall be designed in accordance with Clauses 10.6 and 10.7 without further consideration of additional moments due to slenderness. The value of  $\phi_{sys}$  shall be determined from Table 2.2.5, as appropriate.

### 10.2.4 Design for shear

Design for shear shall be undertaken in accordance with Section 8.2.

Shear reinforcement shall not be less than the fitments required in accordance with Clause 10.7.2 and Section 14 as applicable.

## 10.3 DESIGN OF SHORT COLUMNS

### 10.3.1 General

Short columns shall be designed in accordance with this clause and Clauses 10.6 and 10.7, with additional bending moments due to slenderness taken to be zero. Alternatively, for short columns with small axial forces or small bending moments, the design may be in accordance with Clauses 10.3.2 and 10.3.3 respectively.

A column shall be deemed to be short where—

- (a) for a braced column—

$$L_e/r \leq 25; \text{ or}$$

$$\leq \alpha_c (38 - f'_c/15) (1 + M_1^*/M_2^*)$$

whichever is the greater

where

$$\alpha_c = \sqrt{2.25 - 2.5N^*/0.6N_{uo}} \quad \text{for } N^*/0.6N_{uo} \geq 0.15; \text{ or}$$

$$\alpha_c = \sqrt{1/(3.5N^*/0.6N_{uo})} \quad \text{for } N^*/0.6N_{uo} < 0.15$$

- (b) for an unbraced column—

$$L_e/r \leq 22$$

where, for Items (a) and (b) above—

$r$  = radius of gyration of the cross-sections determined in accordance with Clause 10.5.2

$M_1^*/M_2^*$  = ratio of the smaller to the larger of the design bending moments at the ends of the column.

The ratio is taken to be negative when the column is bent in single curvature and positive when the column is bent in double curvature. When the absolute value of  $M_2^*$  is less than or equal to  $0.05DN^*$ , the ratio shall be taken as  $-1.0$

$L_e$  = effective length determined in accordance with Clause 10.5.3; or alternatively may be taken as—

(i) for a braced column restrained by a flat slab floor,  $L_u$

(ii) for a braced column restrained by beams,  $0.9L_u$

### 10.3.2 Short column with small compressive axial force

Where the design compressive axial force ( $N^*$ ) in a short column is less than  $0.1f'_cA_g$ , the cross-section may be designed for bending only.

### 10.3.3 Short braced column with small bending moments

The bending moments in a short interior column of a braced rectangular framed building structure may be disregarded if—

- (a) the ratio of the longer to the shorter length of any two adjacent spans does not exceed 1.2;
- (b) the loads are essentially uniformly distributed;
- (c) the imposed action (live load) ( $q$ ) does not exceed twice the permanent action (dead load) ( $g$ );
- (d) members are of uniform cross-section; and
- (e) the cross-section of the column is symmetrically reinforced,

in which case the design axial strength ( $\phi N_u$ ) is taken as not greater than  $0.75\phi N_{uo}$ , where  $N_{uo}$  is determined in accordance with Clause 10.6.2.2.

## 10.4 DESIGN OF SLENDER COLUMNS

### 10.4.1 General

Slender columns shall be designed in accordance with this clause and Clauses 10.5, 10.6 and 10.7, with additional bending moments due to slenderness effects taken into account by multiplying the largest design bending moment by the moment magnifier ( $\delta$ ).

The moment magnifier ( $\delta$ ) shall be calculated in accordance with Clause 10.4.2 for a braced column and Clause 10.4.3 for an unbraced column.

NOTE: The moment magnification factors also apply to the case of minimum moments.

For columns subject to bending about both principal axes, the bending moment about each axis shall be magnified by  $\delta$ , using the restraint conditions applicable to each plane of bending.

The additional end moments calculated from moment magnification may be distributed to the members of the joint in proportion to their stiffness.

### 10.4.2 Moment magnifier for a braced column

The moment magnifier ( $\delta$ ) for a braced column shall be taken to be equal to  $\delta_b$  given by—

$$\delta_b = k_m / (1 - N^* / N_c) \geq 1 \quad \dots 10.4.2$$

where

$N_c$  = buckling load given in Clause 10.4.4

$k_m = (0.6 - 0.4M_1^* / M_2^*)$  but shall be taken as not less than 0.4, except that if the column is subjected to significant transverse loading between its ends and in the absence of more exact calculations,  $k_m$  shall be taken as 1.0

### 10.4.3 Moment magnifier for an unbraced column

The moment magnifier ( $\delta$ ) for an unbraced column shall be taken as the larger value of  $\delta_b$  or  $\delta_s$  where—

- (a)  $\delta_b$  for an individual column is calculated in accordance with Clause 10.4.2 assuming the column is braced; and
- (b)  $\delta_s$  for each column in the storey is calculated as—

$$1 / (1 - \Sigma N^* / \Sigma N_c) \quad \dots 10.4.3(1)$$

where the summations include all columns within the storey and  $N_c$  is calculated for each column in accordance with Clause 10.4.4.

As an alternative to Item (b),  $\delta_s$  may be calculated from a linear elastic critical buckling load analysis of the entire frame, where  $\delta_s$  is taken as a constant value for all columns given by—

$$\delta_s = 1/[1 - (1 + \beta_d)/(\alpha_s \lambda_{uc})] \quad \dots 10.4.3(2)$$

where

$\beta_d$  =  $G/(G + Q)$  taken as zero when  $L_e/r \leq 40$  and  $N^* \leq M^*/2D$ , and  $G$  and  $Q$  are the design axial load components due to permanent action (dead load) and imposed action (live load) respectively

$\alpha_s$  = a correlation factor taken as 0.6

$\lambda_{uc}$  = ratio of the elastic critical buckling load of the entire frame to the design load for strength, calculated by taking the cross-sectional stiffness of the flexural members and columns as  $0.4E_c I_f$  and  $0.8E_c I_c$  respectively

The frame shall be proportioned so that  $\delta_s$  for any column is not greater than 1.5.

#### 10.4.4 Buckling load

The buckling load ( $N_c$ ) shall be taken as—

$$N_c = (\pi^2 / L_e^2) [182d_o (\phi M_c) / (1 + \beta_d)] \quad \dots 10.4.4$$

where

$M_c$  =  $M_{ub}$  with  $k_u = 0.545$  and  $\phi = 0.6$

### 10.5 SLENDERNESS

#### 10.5.1 General

The slenderness ratio ( $L_e/r$ ) of a column shall not exceed 120, unless a rigorous analysis has been carried out in accordance with Clauses 6.4, 6.5 or 6.6 and the column is designed in accordance with Clause 10.2.3.

Where the forces and moments acting on a column have been obtained from a linear elastic analysis, as specified in Clause 6.2, the influence of slenderness shall be taken into account using a radius of gyration ( $r$ ) specified in Clause 10.5.2 and an effective length ( $L_e$ ), in accordance with Clause 10.5.3.

#### 10.5.2 Radius of gyration

The radius of gyration ( $r$ ) shall be calculated for the gross concrete cross-section. For a rectangular cross-section,  $r$  may be taken as  $0.3D$ , where  $D$  is the overall dimension in the direction in which stability is being considered and for a circular cross-section,  $r$  may be taken as  $0.25D$ .

#### 10.5.3 Effective length of a column

The effective length of a column ( $L_e$ ) shall be taken as  $kL_u$  where the effective length factor ( $k$ ) is determined from Figure 10.5.3(A) for columns with simple end restraints, or more generally from Figure 10.5.3(B) or 10.5.3(C), as appropriate.

The end restraint coefficients ( $\gamma_1$  and  $\gamma_2$ ) shall be determined—

- for regular rectangular framed structures where the axial forces in the beams are generally small, in accordance with Clause 10.5.4;
- for all structures, including non-rectangular framed structures or structures where the axial forces in the restraining members are large, in accordance with Clause 10.5.5; and
- where the column ends at a footing, in accordance with Clause 10.5.6.

Alternatively, the effective length of a column may be determined from the elastic critical buckling load of the frame, as calculated by analysis.

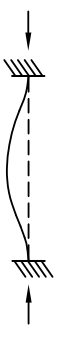

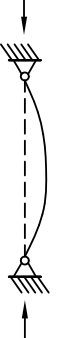







	Braced column			Unbraced column		
Buckled shape						
Effective length factor ( $k$ )	0.70	0.85	1.00	1.20	2.20	2.20
Symbols for end restraint conditions	 = Rotation fixed, translation fixed  = Rotation free, translation fixed			 = Rotation fixed, translation free  = Rotation free, translation free		

FIGURE 10.5.3(A) EFFECTIVE LENGTH FACTOR ( $k$ ) FOR COLUMNS WITH SIMPLE END RESTRAINTS

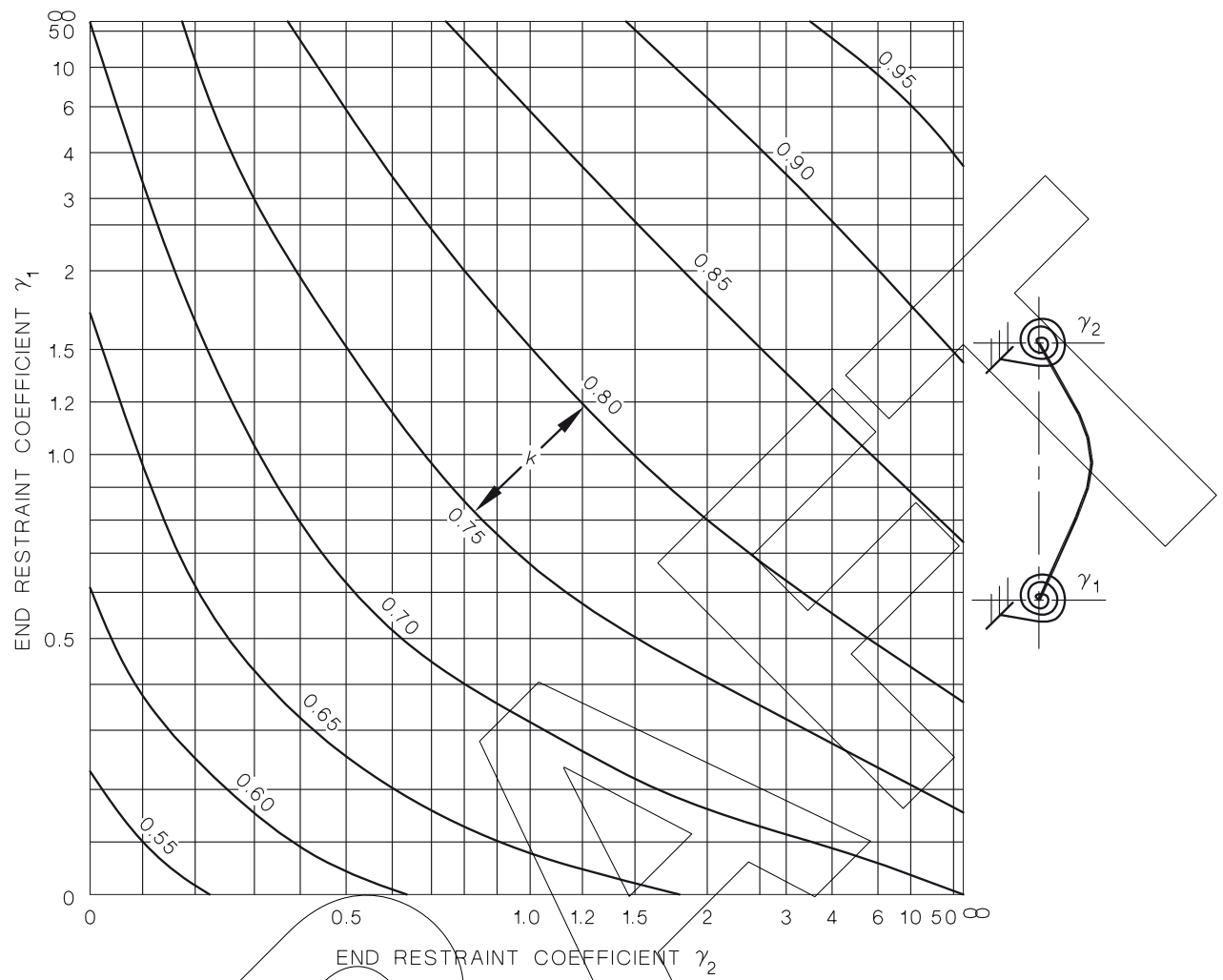


FIGURE 10.5.3(B) EFFECTIVE LENGTH FACTOR ( $k$ ) FOR BRACED COLUMNS



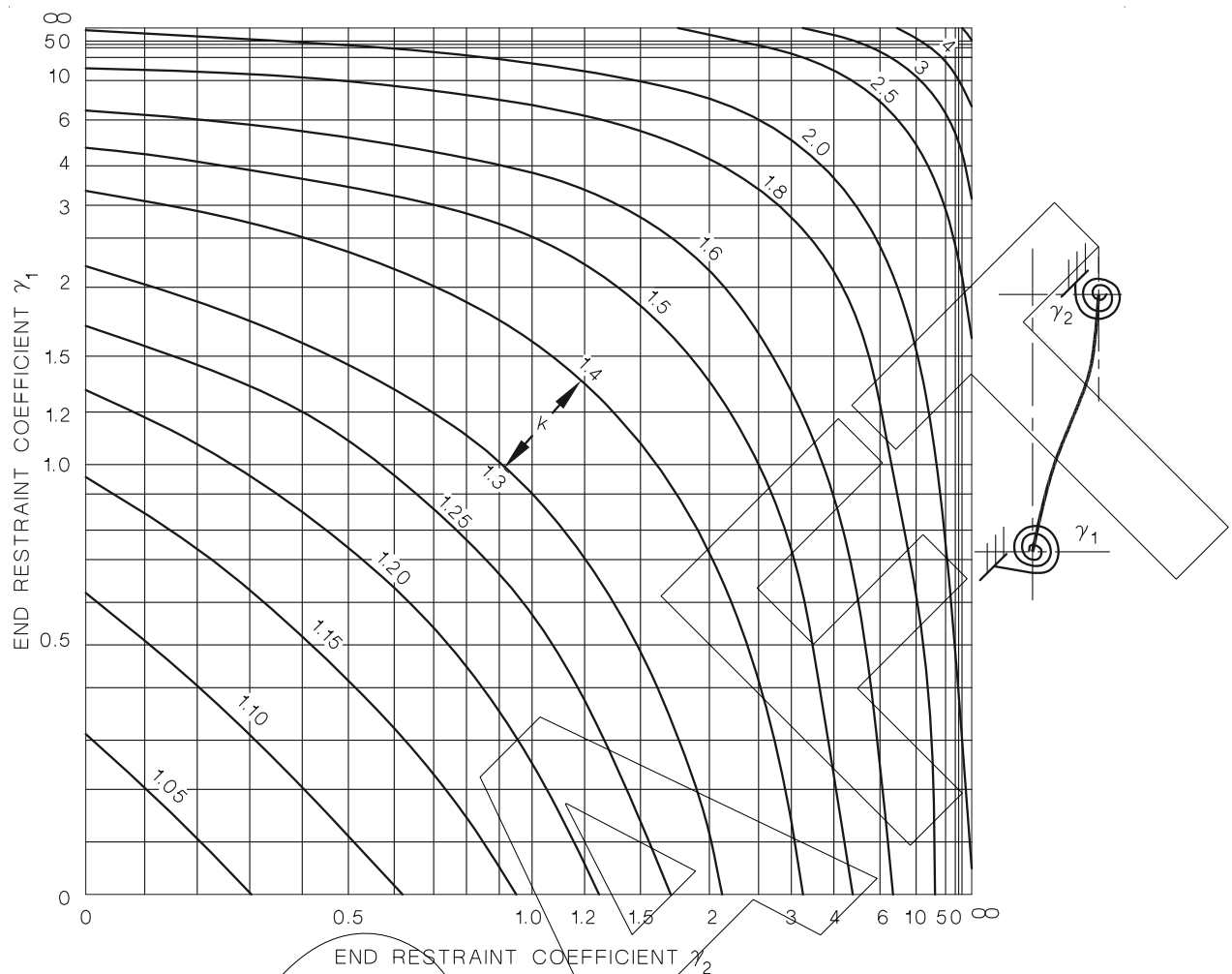


FIGURE 10.5.3(C) EFFECTIVE LENGTH FACTOR ( $k$ ) FOR UNBRACED COLUMNS

#### 10.5.4 End restraint coefficients for regular rectangular framed structures

For regular rectangular framed structures, the end restraint coefficient ( $\gamma_1$ ) at one end of a column and the end restraint coefficient ( $\gamma_2$ ) at the other end may each be calculated as—

$$\frac{\sum(I/L)_c}{\sum(\beta I/L)_b} \quad \dots 10.5.4$$

where

$\sum(I/L)_c$  = sum of the stiffness in the plane of bending of all the columns meeting at and rigidly connected to the end of the column under consideration

$\sum(\beta I/L)_b$  = sum of the stiffness in the plane of bending of all the beams or slabs, or both, meeting at and rigidly connected to the same end of the column under consideration

$\beta$  = a fixity factor, given in Table 10.5.4, for fixity conditions at the end of each beam or slab, or both, opposite to the end connected to the column under consideration

**TABLE 10.5.4**  
**FIXITY FACTOR ( $\beta$ )**

Fixity conditions at far end of a beam or slab, or both	Fixity factor ( $\beta$ )	
	Beam or slab or both, in a braced frame	Beam or slab or both, in an unbraced frame
Pinned	1.5	0.5
Rigidly connected to a column	1.0	1.0
Fixed	2.0	0.67

### 10.5.5 End restraint coefficients for any framed structure

For any framed structure, the end restraint coefficient ( $\gamma_1$ ) at one end of a column and the end restraint coefficient ( $\gamma_2$ ) at the other end may be calculated as the ratio of the column stiffness to the sum of the stiffnesses of all the members, except the column, meeting at the end under consideration. In the calculation of the stiffness of members, other than the column, due account shall be taken of the fixity conditions of each member at the end remote from the column-end being considered as well as any reduction in member stiffness due to axial compression.

### 10.5.6 End restraint provided by footings

Where a footing provides negligible restraint to the rotation of the end of a column,  $\gamma$  is theoretically infinite but may be taken as 10.

Where a footing is specifically designed to prevent rotation of the end of a column,  $\gamma$  is theoretically zero but shall be taken as 1.0 unless analysis would justify a smaller value.

## 10.6 STRENGTH OF COLUMNS IN COMBINED BENDING AND COMPRESSION

### 10.6.1 Basis of strength calculations

Calculations for the strength of cross-sections in bending, combined with axial forces, shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following assumptions:

- Plane sections normal to the axis remain plane after bending.
- The concrete has no tensile strength.
- The distribution of stress in the concrete and the steel is determined using a stress-strain relationship determined from Clauses 3.1.4 and 3.2.3 respectively (see Note 1).
- The strain in compressive reinforcement does not exceed 0.003.
- Where the neutral axis lies outside of the cross-section, consideration shall be given to the effect on strength of spalling of the cover concrete.

#### NOTES:

- If a curvilinear stress-strain relationship is used then—
  - Clause 3.1.4 places a limit on the value of the maximum concrete stress; and
  - the strain in the extreme fibre may be adjusted to obtain the maximum bending strength for a given axial load.
- The effect of confinement on the strength of a section may be taken into account, provided secondary effects such as concrete spalling, for example, are also considered.

Columns subject to axial force with bending moments about each principal axis may take into account the concessions given in Clauses 10.6.3 and 10.6.4.

## 10.6.2 Strength of cross-sections calculated using the rectangular stress block

### 10.6.2.1 General

It shall be permissible to represent the strength of a cross-section in combined bending and compression using a strength interaction diagram as shown in Figure 10.6.2.1 defined as given in Clauses 10.6.2.2 to 10.6.2.5.

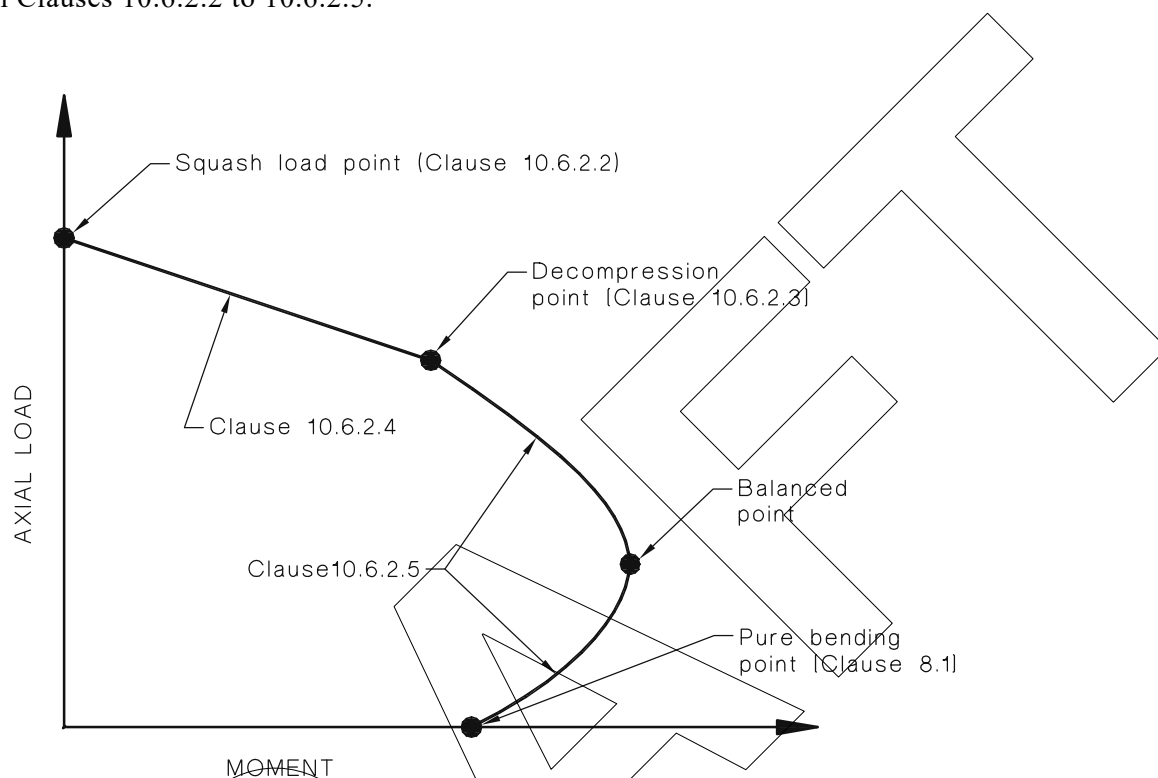


FIGURE 10.6.2.1 AXIAL LOAD—MOMENT DIAGRAM

### 10.6.2.2 Squash load ( $N_{uo}$ )

The ultimate strength in compression without bending ( $N_{uo}$ ) shall be calculated by assuming—

- (a) a uniform concrete compressive stress of  $\alpha_1 f'_c$ , where—

$$\alpha_1 = 1.0 - 0.003 f'_c \text{ with the limits 0.72 to 0.85; and}$$

... 10.6.2.2

NOTE: The modification of  $0.9 f'_c$  given in Clause 3.1.4 is included in the calculation of  $\alpha_1$ .

- (b) a maximum strain in the reinforcement of 0.0025.

### 10.6.2.3 Decompression point

The decompression point is calculated taking the strain in the extreme compressive fibre equal to 0.003, the strain in the extreme tensile fibre equal to zero and using the rectangular stress block given in Clause 10.6.2.5.

### 10.6.2.4 Transition from decompression point to squash load

Where the neutral axis lies outside of the section, the section strength may be calculated using a linear relationship between the decompression point given by Clause 10.6.2.3 and the squash load ( $N_{uo}$ ) calculated using Clause 10.6.2.2.

### 10.6.2.5 Transition from decompression point to bending strength

Where the neutral axis lies within the cross-section and provided the maximum strain in the extreme compression fibre of the concrete is taken as 0.003, Clause 10.6.1(c) shall be deemed to be satisfied for the concrete by assuming a uniform concrete compressive stress of  $\alpha_2 f'_c$  acts on an area bounded by—

- (a) the edges of the cross-section; and
- (b) a line parallel to the neutral axis under the loading concerned, and located at a distance  $\gamma k_u d$  from the extreme compressive fibre, where—

$$\alpha_2 = 1.0 - 0.003 f'_c \text{ (within the limits } 0.67 \leq \alpha_2 \leq 0.85 \text{)} \quad \dots 10.6.2.5(1)$$

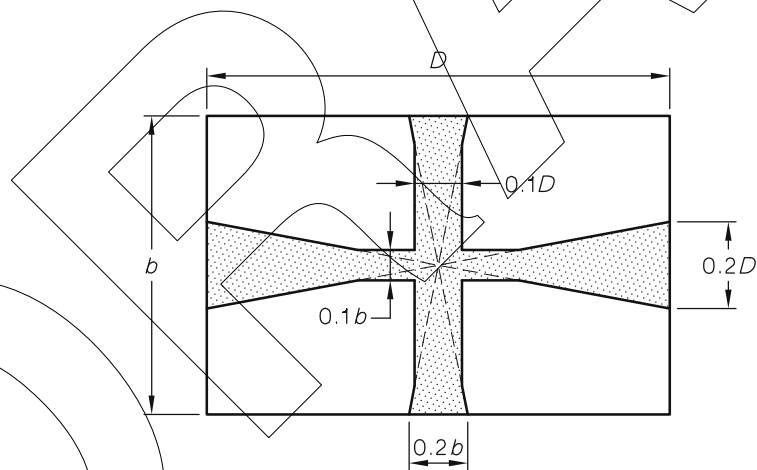
$$\gamma = 1.05 - 0.007 f'_c \text{ (within the limits } 0.67 \leq \gamma \leq 0.97 \text{)} \quad \dots 10.6.2.5(2)$$

#### NOTES:

- 1 The modification of  $0.9 f'_c$  given in Clause 3.1.4 is included in the rectangular stress block assumptions.
- 2 Cover spalling may be a problem in columns cast with high-strength concrete. The effect of cover spalling on strength given in Clause 10.6.1(e) is included in the parameters developed for the calculation of the interaction diagram.

### 10.6.3 Design based on each bending moment acting separately

For a rectangular cross-section, where the ratio of the larger to the smaller cross-sectional dimension does not exceed 3.0, which is subjected simultaneously to an axial force and bending moments about each principal axis, the cross-section may be designed for the axial force with each bending moment considered separately, provided the line of action of the resultant force falls within the shaded area of the cross-section shown in Figure 10.6.3.



Shaded areas symmetrical about column centre-lines

FIGURE 10.6.3 LIMITATION FOR LINE OF ACTION OF THE RESULTANT AXIAL FORCE

### 10.6.4 Design for biaxial bending and compression

A rectangular cross-section, subject to axial force and bending moment acting simultaneously about each principal axis, may be designed such that—

$$\left( \frac{M_x^*}{\phi M_{ux}} \right)^{a_n} + \left( \frac{M_y^*}{\phi M_{uy}} \right)^{a_n} \leq 1.0 \quad \dots 10.6.4$$

where

$\phi M_{ux}, \phi M_{uy}$  = design strength in bending, calculated separately, about the major and minor axis respectively under the design axial force ( $N^*$ )

$M_x^*, M_y^*$  = design bending moment about the major and minor axis respectively, magnified, if applicable

$\alpha_n$  =  $0.7 + 1.7N^*/0.6N_{uo}$ , within the limits  $1 \leq \alpha_n \leq 2$

## 10.7 REINFORCEMENT REQUIREMENTS FOR COLUMNS

### 10.7.1 Limitations on longitudinal steel

The cross-sectional area of the longitudinal reinforcement in a column shall—

- be not less than  $0.01A_g$  except that, in a column that has a larger area than that required for strength, a reduced value of  $A_{sc}$  may be used if  $A_{sc}f_{sy} > 0.15N^*$ ; and
- not exceed  $0.04A_g$  unless the amount and disposition of the reinforcement will not prevent the proper placing and compaction of the concrete at splices and at junctions of the members.

Groups of parallel longitudinal bars, that are bundled to act as a unit, shall have not more than four bars in any one bundle and shall be tied together in contact.

### 10.7.2 Functions of fitments

Fitments shall satisfy the requirements of confinement of concrete (Clause 10.7.3) and lateral restraint of longitudinal bars against premature buckling (Clause 10.7.4), in addition to shear and torsion in accordance with Clauses 8.2 and 8.3. The maximum area required for shear combined with torsion, confinement or control of buckling of bars shall be used.

### 10.7.3 Confinement to the core

#### 10.7.3.1 General requirements

Fitments (including helical reinforcement) shall be detailed to provide confinement to the core of the column—

- for columns where  $f'_c \leq 50$  MPa confinement shall be deemed to be provided if the requirements of Clause 10.7.4 are satisfied; and
- for columns where  $f'_c > 50$  MPa confinement shall be provided—
  - in special confinement regions using fitments detailed to provide a minimum effective confining pressure to the core of  $0.01 f'_c$ , calculated in accordance with Clause 10.7.3.2, 10.7.3.3 or 10.7.3.4; and
  - outside of the special confinement regions, confinement shall be deemed to be provided if the maximum spacing of the fitments does not exceed the lesser of  $0.8D_c$ , 300 mm and that of Clause 10.7.4.

In the special confinement regions, the spacing or pitch of the fitments shall not exceed the lesser of  $0.6 D_c$ , 300 mm and that of Clause 10.7.4.

Special confinement regions are regions where the design action effects satisfy one of the following [see Figure 10.7.3.1(A)]:

(A)  $N^* \geq 0.75 \phi N_{uo}$  ... 10.7.3.1(1)

(B)  $N^* \geq \phi 0.3 f'_c A_g$  and  $M^* \geq 0.6 \phi M_u$  ... 10.7.3.1(2)

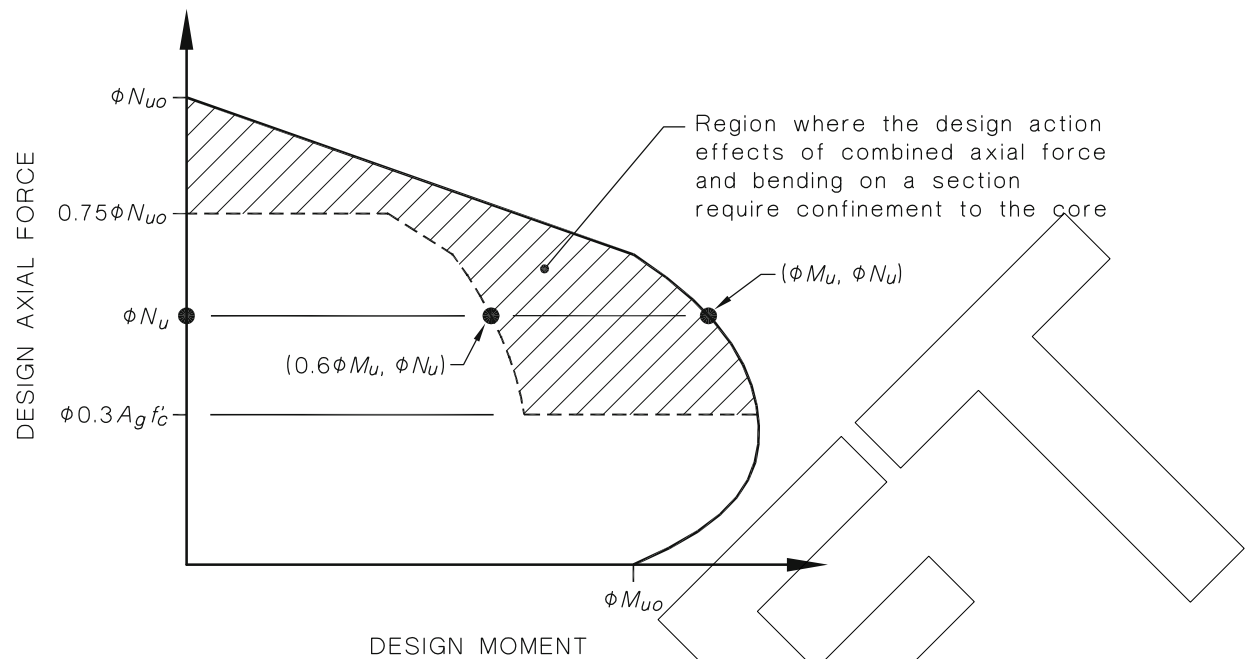
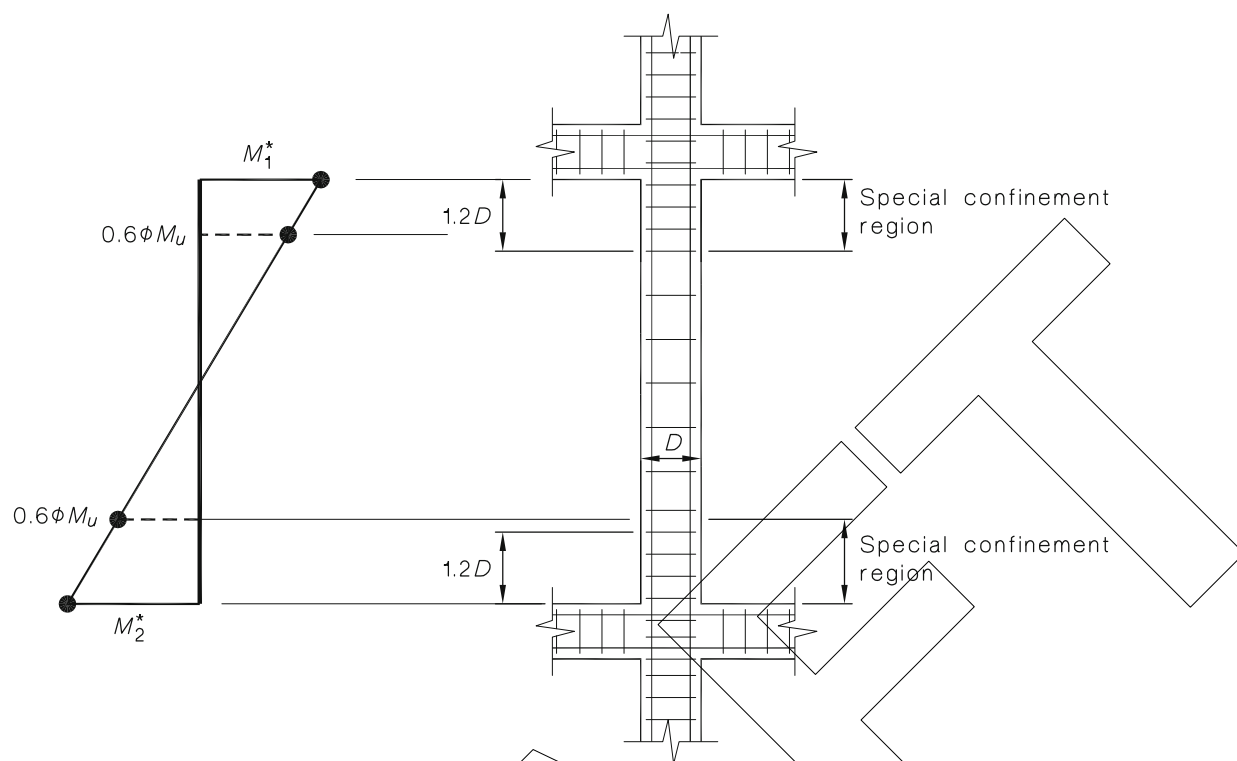


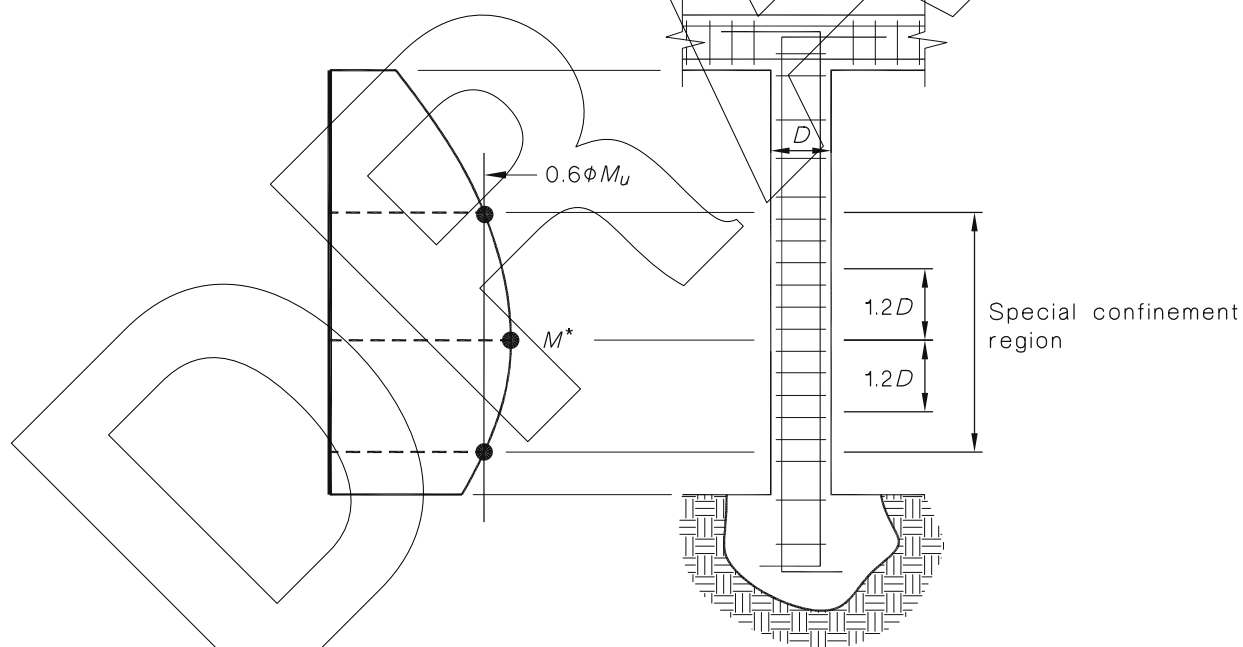
FIGURE 10.7.3.1(A) CONFINEMENT TO THE CORE

Fitments in special confinement regions shall be provided over the limits defined by the special confinement regions [see Figure 10.7.3.1(B)], but not less than a length measured each side of the maximum moment and bounded by the lesser of—

- (1) 1.2 times the dimension of the cross-section measured normal to the axis of bending being considered; and
- (2) the distance to the end of the member.



(a) Column with confinement regions at member ends for  $N^* \leq 0.75\phi N_{uo}$



(b) Column with confinement region within member length for  $N^* \leq 0.75\phi N_{uo}$

FIGURE 10.7.3.1(B) SPECIAL CONFINEMENT REGIONS

For columns in moment resisting frame structures that are in double curvature where  $0.3\phi f'_c A_g < N^* < 0.75\phi N_{uo}$ , the special confinement regions may be taken as the length from each end of the member equal to the larger of—

$$(i) \quad 0.4L \frac{|M_2^*|}{|M_1^*| + |M_2^*|} \quad \dots 10.7.3.1(3)$$

(ii) 1.2 times that of the larger cross-sectional dimension.

### 10.7.3.2 Calculation of core confinement by rational calculation

The confining pressure applied to the core of a column may be obtained with consideration of triaxial stresses across all sections and the effectiveness of fitments in providing the specified confinement.

### 10.7.3.3 Calculation of core confinement by simplified calculation

The effective confining pressure applied to the core of a column is calculated as—

$$f_{r,eff} = k_e f_r \quad \dots 10.7.3.3(1)$$

where

$k_e$  = an effectiveness factor accounting for the arrangement of the fitments

$f_r$  = average confining pressure on the core cross-section taken at the level of the fitments (see Figure 10.7.3.3)

NOTE: For non-circular sections,  $f_r$  is taken as the smaller of the confining pressures calculated for each of the major directions [XX and YY shown in Figure 10.7.3.3(d)].

The average confining pressure on the core at the level of the fitments shall be calculated as follows:

$$f_r = \frac{\sum_{i=1}^m A_{b,fit} f_{sy,f} \sin \theta}{d_s s} \quad \dots 10.7.3.3(2)$$

where

$A_{b,fit}$  = cross-sectional area of one leg of the fitment

$f_{sy,f}$  = yield stress of the reinforcement used as fitment (not greater than 800 MPa)

$\theta$  = angle between the fitment leg and the confinement plane

$m$  = number of fitment legs crossing the confinement plane

$d_s$  = overall dimension measured between centre-lines of the outermost fitments

$s$  = centre to centre spacing of fitments along the column

The effectiveness factor shall be calculated as follows:

(a) For rectangular sections

$$k_e = \left(1 - \frac{nw^2}{6A_c}\right) \left(1 - \frac{s}{2b_c}\right) \left(1 - \frac{s}{2d_c}\right) \quad \dots 10.7.3.3(3)$$

where

$A_c$  = cross-sectional area bounded by the centre-line of the outermost fitments

$n$  = number of laterally restrained longitudinal bars [see Clause 10.7.4.2(a)]

$w$  = average clear spacing between adjacent restrained longitudinal bars



$b_c$  = core dimension measured between the centre-lines of the outermost fitments measured across the width of the section

$d_c$  = core dimension measured between the centre-lines of the outermost fitments measured through the depth of the section

(b) For circular sections

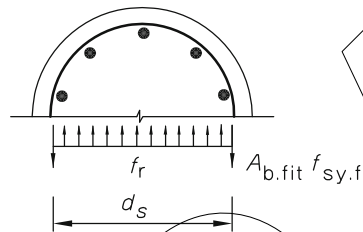
$$k_e = \left(1 - \frac{s}{2d_s}\right)^2 \quad \dots 10.7.3.3(4)$$

Alternatively, for rectangular or circular columns, the effective confining pressure may be calculated as follows:

$$f_{r,eff} = 0.5k_e\rho_s f_{sy,f} \quad \dots 10.7.3.3(5)$$

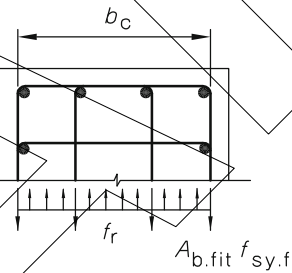
where

$\rho_s$  = the volumetric ratio of the fitments relative to the volume of the core  
calculated as  $\rho_s = \frac{A_{b,fit} \times \text{total perimeter of fitments crossing the section}}{A_c \times s}$



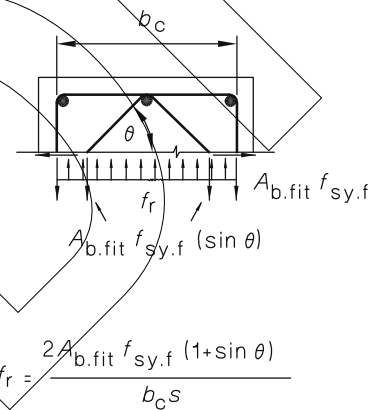
$$f_r = \frac{2A_{b,fit} f_{sy,f}}{d_s s}$$

(a)



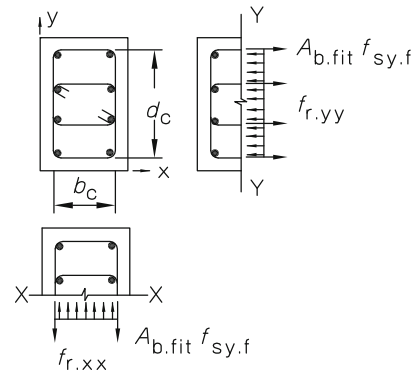
$$f_r = \frac{4A_{b,fit} f_{sy,f}}{b_c s}$$

(b)



$$f_r = \frac{2A_{b,fit} f_{sy,f} (1 + \sin \theta)}{b_c s}$$

(c)



$$f_r = \min(f_{r,xx}, f_{r,yy})$$

(d)

FIGURE 10.7.3.3 CALCULATION OF CONFINING PRESSURES

### 10.7.3.4 Deemed to conform core confinement

The requirement of an effective confining pressure of  $0.01f'_c$  is deemed to be satisfied provided the fitment spacings do not exceed the following:

- (a) For rectangular sections:

$$\frac{15nA_{b,fit}f_{sy,f}}{f'_c\sqrt{A_c}} \quad \dots 10.7.3.4(1)$$

where

$n$  = number of laterally restrained longitudinal bars [see Clause 10.7.4.2(a)]

- (b) For circular sections:

$$\frac{100A_{b,fit}f_{sy,f}}{d_s f'_c} \quad \dots 10.7.3.4(2)$$

## 10.7.4 Restraint of longitudinal reinforcement

### 10.7.4.1 General requirements

The following longitudinal bars in columns shall be laterally restrained in accordance with Clause 10.7.4.2:

- (a) Single bars—
- (i) each corner bar;
  - (ii) all bars, where bars are spaced at centres of more than 150 mm or where the design axial force  $> 0.3A_g f'_c$ ; and
  - (iii) at least every alternate bar, where bars are spaced at 150 mm or less.
- (b) Bundled bars—each bundle.

### 10.7.4.2 Lateral restraint

Lateral restraint shall be deemed to be provided if the longitudinal reinforcement is placed within and in contact with—

- (a) a non-circular fitment (see Figure 10.7.4.2)—
- (i) at a bend in the fitment, where the bend has an included angle of  $135^\circ$  or less; or
  - (ii) between two  $135^\circ$  fitment hooks; or
  - (iii) inside a single  $135^\circ$  fitment hook of a fitment that is approximately perpendicular to the column face; or
  - (iv) for internal fitments with one leg, at a  $90^\circ$  fitment hook of a fitment that is approximately perpendicular to the column face, provided—
    - (A) the other end of the fitment has a  $135^\circ$  angle hook around a longitudinal reinforcing bar;
    - (B) the end types on adjacent fitments in plan are alternated;
    - (C) the end types on consecutive internal fitments along the longitudinal reinforcement are alternated;
    - (D) the design axial force is  $\leq 0.3 A_g f'_c$ ; and
    - (E)  $f'_c \leq 50$  MPa

- (b) a circular fitment or helical reinforcement and the longitudinal reinforcing bars are equally spaced around the circumference.

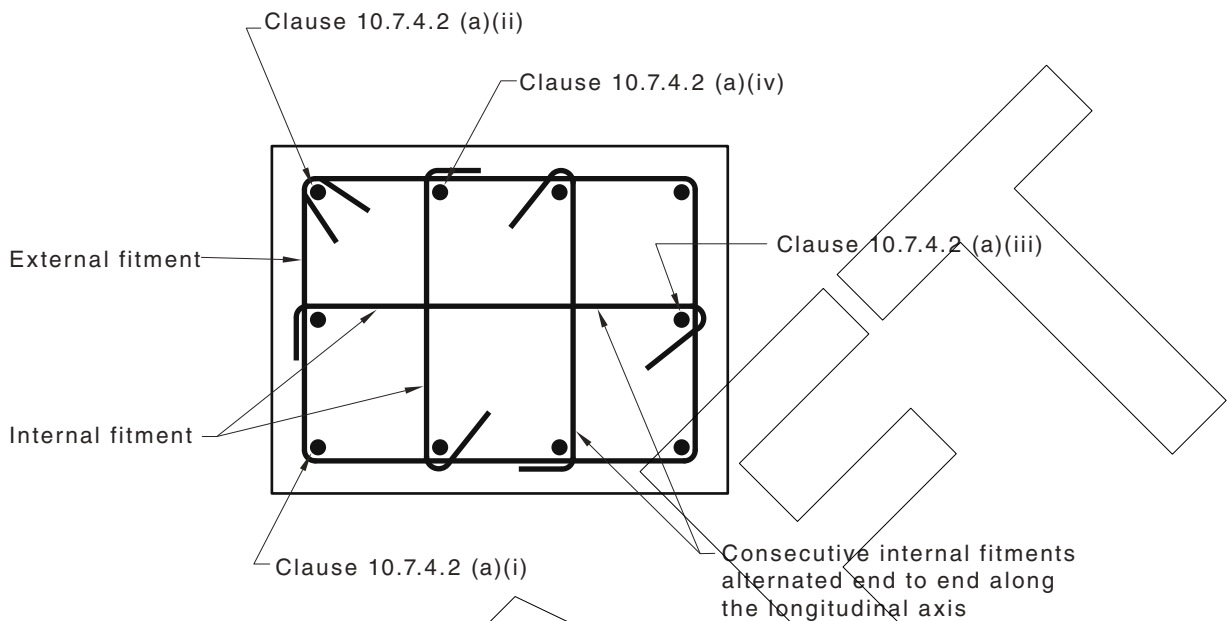


FIGURE 10.7.4.2 LATERAL RESTRAINT TO LONGITUDINAL BARS

**10.7.4.3 Diameter and spacing of fitments and helices**

The diameter and spacing of fitments and helices shall conform with the following:

- (a) The bar diameter of the fitment and helical reinforcement shall be not less than that given in Table 10.7.4.3.

**TABLE 10.7.4.3**  
**BAR DIAMETERS FOR FITMENTS AND HELICES**

Longitudinal bar diameter mm	Minimum bar diameter of fitment and helix mm
Single bars up to 20	6
Single bars 24 to 28	10
Single bars 32 to 36	12
Single bars > 40	16
Bundled bars	12

- (b) The spacing of fitments, or the pitch of a helix, shall not exceed the smaller of—
- (i)  $D_c$  and  $15d_b$  for single bars; or
- (ii)  $0.5D_c$  and  $7.5d_b$  for bundled bars,

where

$D_c$  = smaller column cross-sectional dimension if rectangular or the column diameter if circular

$d_b$  = diameter of the smallest bar in the column

- (iii) Where  $L_u \leq 5D$  the spacing of the fitments shall not exceed the requirements of Section 14.5.

- (c) One fitment, or the first turn of a helix, shall be located not more than 50 mm vertically above the top of a footing, or the top of a slab in any storey. Another fitment, or the final turn of a helix, shall be located not more than 50 mm vertically below the soffit of the slab except that in a column with a capital, the fitment or turn of the helical reinforcement shall be located at a level at which the area of the cross-section of the capital is not less than twice that of the column.

Where beams or brackets frame from four directions into a column and adequately restrain the column in all directions, the fitments or helical reinforcement may be terminated 50 mm below the highest soffit of such beams or brackets.

NOTE: Welded wire mesh, having strength and anchorage equivalent to that required for bars, may be used.

#### 10.7.4.4 Detailing of fitments and helices

Detailing of fitments and helices shall be as follows:

- (a) A rectangular fitment shall be spliced by welding, or by fixing two 135° fitment hooks around a bar or a bundle at a fitment corner. Internal fitments may be spliced by lapping within the column core.
- (b) A circular shaped fitment shall be spliced either by welding, or by overlapping and fixing two 135° fitment hooks around adjacent longitudinal bars or bundles.
- (c) A helical reinforcement shall be anchored at its end by one and one half extra turns of the helix. It may be spliced within its length either by welding or mechanical means.
- (d) Where hooks or cogs are specified in combination with bundled bars, the internal diameter of the bend shall be increased sufficiently to readily accommodate the bundle.

#### 10.7.4.5 Column joint reinforcement

Where bending moments from a floor system are transferred to a column, lateral shear reinforcement, of area  $A_{sv} \geq 0.35 bs/f_{sy.f}$ , shall be provided through the joint except that where a slab or beams exist on all four sides of the column, the fitments may be omitted over the depth of the shallowest slab or beam. The spacing of fitments shall be in accordance with Clause 10.7.4.3.

### 10.7.5 Splicing of longitudinal reinforcement

#### 10.7.5.1 General

Longitudinal reinforcement in columns shall be spliced in accordance with Clauses 10.7.5.2 to 10.7.5.5. The splices shall conform with Clause 13.2.

#### 10.7.5.2 Minimum tensile strength

At any splice in a column, a tensile strength in each face of the column of not less than  $0.25f_{sy}A_s$  shall be provided, where  $A_s$  is the cross-sectional area of longitudinal reinforcement in that face.

#### 10.7.5.3 Where tensile force exceeds the minimum tensile strength

At any splice in a column where tensile stress exists and the tensile force in the longitudinal bars at any face of the column, due to strength design load effects, exceeds the minimum strength requirements given in Clause 10.7.5.2, the force in the bars shall be transmitted by—

- (a) a welded or mechanical splice in accordance with Clause 13.2.6; or
- (b) a lap-splice in tension in accordance with Clause 13.2.2 or 13.2.5, as appropriate.

#### 10.7.5.4 End-bearing splice in compression

Where the splice is always in compression, the force in the longitudinal bar may be transmitted by the bearing of square-cut mating ends held in concentric contact by a sleeve, provided an additional fitment, which complies with Clause 10.7.4, is placed above and below each sleeve. The bars shall be rotated to achieve the maximum possible area of contact between the ends of the bars and the requirements of Clause 10.7.5.2 shall be met.

#### 10.7.5.5 Offset bars

Where a longitudinal bar is offset to form a lap splice—

- (a) the slope of the inclined part of the bar in relation to the axis shall not exceed one in six;
- (b) the portions of the bar on either side of the offset shall be parallel; and
- (c) adequate lateral support shall be provided at the offset.

Where a column face is offset 75 mm or greater, longitudinal bars shall not be offset by bending but shall be lap-spliced with separate splicing bars placed adjacent to the offset column faces.

### 10.8 TRANSMISSION OF AXIAL FORCE THROUGH FLOOR SYSTEMS

The transmission of axial force through floor systems shall be designed in accordance with the following:

- (a) If the concrete strength specified for the floor system is greater than or equal to 0.75 times that specified for the column and the longitudinal reinforcement is continuous through the joint, transmission of forces through the joint is deemed to be provided.
- (b) If the concrete strength specified for the floor system is less than 0.75 times that specified for the column, transmission of force through the floor concrete shall be provided by adding longitudinal reinforcement through the joint, in accordance calculations made using the effective compressive strength of the concrete in the joint ( $f'_{ce}$ ) as follows:
  - (i) For columns restrained on four sides by beams of approximately equal depth or by a slab,  $f'_{ce}$  shall be taken as—

$$f'_{ce} = \left[ 1.33 - \frac{0.33}{h/D_c} \right] f'_{cs} + \frac{0.25}{h/D_c} f'_{cc} \quad \text{within the limits} \quad \dots 10.8(1)$$

$$\min(f'_{cc}, 1.33f'_{cs}) \leq f'_{ce} \leq \min(f'_{cc}, 2.5f'_{cs})$$

where

$h$  = overall depth of the joint

$D_c$  = smaller column cross-sectional dimension

$f'_{cc}$  = compressive strength of the concrete in the column

$f'_{cs}$  = compressive strength of the concrete in the slab or beams

The value of  $h/D_c$  shall not be taken as less than 0.33.

- (ii) For columns restrained on two opposing sides by beams of approximately equal depth or by a slab,  $f'_{ce}$  shall be taken as—

$$f'_{ce} = \left[ 1.1 - \frac{0.3}{h/D_c} \right] f'_{cs} + \frac{0.2}{h/D_c} f'_{cc} \quad \text{within the limits} \quad \dots 10.8.(2)$$

$$\min(f'_{cc}, 1.33f'_{cs}) \leq f'_{ce} \leq \min(f'_{cc}, 2.0f'_{cs})$$

The value of  $h/D_c$  shall not be taken as less than 0.33.

- (iii) For columns restrained on two adjacent sides by beams of approximately equal depth or by a slab,  $f'_{ce}$  shall be taken as—

$$f'_{ce} = 1.33f'_{cs} \leq f'_{cc} \quad \dots 10.8.(3)$$

NOTES:

- 1 Confining reinforcement may be used to increase the effective strength of the concrete in the joint.
- 2 Beams and slabs should provide restraint over the width of the column joint.

## SECTION 11 DESIGN OF WALLS

### 11.1 GENERAL

This section applies to the following:

- (a) Braced walls (as defined in Clause 11.3) that are subject to in-plane load effects, which shall be designed in accordance with Clauses 11.2 to 11.7.
- (b) Braced walls that are subject to simultaneous in-plane and out-of-plane load effects and unbraced walls, which shall be designed in accordance with Section 9 and Section 10, as appropriate.

Where the stress at the mid-height section of a wall due to factored in-plane bending and axial forces does not exceed the lesser of  $0.03f'_c$  and 2 MPa, the wall may be designed as a slab in accordance with Section 9, provided—

- (i) second-order deflections due to in-plane loads and long-term effects are considered in the calculation of bending moments; and
- (ii) the ratio of effective height to thickness does not exceed 50.

NOTE: The ratio of effective height to thickness may be restricted by fire considerations (see Section 5).

In this section yield stress  $f_{sy}$  shall not be taken as greater than 500 MPa.

### 11.2 DESIGN PROCEDURES

#### 11.2.1 General

Braced walls where in-plane horizontal forces, acting in conjunction with the axial forces, are such that where a horizontal cross-section of the wall—

- (a) is subject to compression over the entire section, the wall designed for horizontal shear forces in accordance with Clause 11.6 and for the vertical compressive forces either—
  - (i) in accordance with the simplified method of Clause 11.5; or
  - (ii) as columns in accordance with Section 10 where vertical reinforcement is provided in each face, except that Clause 11.7.4 may override the requirements of Clause 10.7.4; or
- (b) is subject to tension on part of the section, the wall shall be designed for in-plane bending in accordance with Section 8 and for horizontal shear in accordance with Clause 11.6 or, for in-plane bending and shear, in accordance with Section 12, if appropriate.

When considering design actions due to earthquake, the determination of whether a wall horizontal cross-section is subject to compression over the entire section shall be based on a lateral analysis undertaken using a structural ductility factor ( $\mu$ ) of 1.00 and a structural performance factor ( $S_p$ ) of 0.77.

#### 11.2.2 Groups of walls

When two or more walls are directly inter-connected, or linked to each other by coupling beams or the like, the distribution of in-plane load effects between the walls shall be obtained by linear elastic analysis of the overall structure under the global vertical and lateral loads. The analysis shall distribute in-plane load effects to the individual walls in proportion to their relative stiffness, as determined by their gross cross-sectional properties.

The interconnected vertical edges of walls shall be designed to withstand the transmitted vertical shear forces.

### 11.3 BRACED WALLS

A wall may be considered to be braced if it forms part of a structure that does not rely on out-of-plane strength and stiffness of the wall and the connection of the wall to the rest of the structure can transmit—

- (a) any calculated load effects; and
- (b) 2.5% of the total vertical load the wall is designed to carry at the level of lateral support, but not less than 2 kN per metre length of wall.

### 11.4 EFFECTIVE HEIGHT

For a wall with a floor-to-floor unsupported height of  $H_w$  and a horizontal length of  $L_1$  between the centres of lateral restraint or from the lateral restraint to the free edge, as applicable, the effective height ( $H_{we}$ ) shall be taken as  $H_{we} = kH_w$ , where the factor  $k$  is given as follows:

- (a) *One-way buckling* With floors providing lateral support at both ends—
  - $k = 0.75$  where restraint against rotation is provided at both ends; and
  - $k = 1.0$  where no restraint against rotation is provided at one or both ends.
- (b) *Two-way buckling* With lateral support on three sides provided by floors and intersecting walls—

$$k = \left[ \frac{1}{1 + \left( \frac{H_w}{3L_1} \right)^2} \right] \quad \text{but not less than 0.3 or greater than what is obtained from Item (a)} \quad \dots 11.4(1)$$

- (c) *Two-way buckling* With lateral support on four sides provided by floors and intersecting walls—

$$k = \frac{1}{1 + (H_w / L_1)^2} \quad \text{where } H_w \leq L_1 \quad \dots 11.4(2)$$

$$k = \frac{L_1}{2H_w} \quad \text{where } H_w > L_1 \quad \dots 11.4(3)$$

Walls supported laterally on four sides that contain one or more openings shall be designed as follows:

- (i) If the total area of the openings is less than 1/10 of the area of the wall and the height of any opening, not vertically one above the other, is less than 1/3 of the height of the wall, then the effect of the openings may be ignored.
- (ii) In other cases—
  - (A) the area of the wall between the support and the opening shall be designed as supported on three sides; and
  - (B) the area between the openings shall be designed as supported on two sides.

NOTE: An intersecting wall with a minimum length of  $0.2H_w$  may be considered a lateral restraint.



## 11.5 SIMPLIFIED DESIGN METHOD FOR WALLS SUBJECT TO VERTICAL COMPRESSION FORCES

### 11.5.1 General

Where a wall is subject to in-plane bending, the wall shall be divided into one or more design segments and each segment shall be designed for the highest stress in that segment.

### 11.5.2 limitation on use of method

Structural walls designed using Clause 11.5.2 shall—

- not be used in a building exceeding 12 metres in overall building height as defined in AS 1170.4;
- not be constructed on sites with soil classifications of either  $D_e$  or  $E_e$  and where subjected to earthquake design actions; and
- have a ratio of effective height to thickness that does not exceed 20 for singly reinforced walls or 30 for doubly reinforced walls.

Otherwise, the wall shall be designed as a column in accordance with Section 10.

### 11.5.3 Design axial strength of a wall

The design axial strength per unit length of a braced wall in compression can be taken as—

### 11.5.4 Design axial strength of a wall

Provided the ratio of effective height to thickness ( $H_{we}/t_w$ ) does not exceed 30, the design axial strength per unit length of a braced wall in compression can be taken as—

$$\phi N_u \quad \dots 11.5.4$$

where

$$\phi = 0.6$$

$$N_u = (t_w - 1.2e - 2e_a)0.6f'_c$$

$N_u$  = ultimate strength per unit length of wall

$t_w$  = thickness of the wall

$e_a$  = an additional eccentricity taken as  $(H_{we})^2/2500t_w$

### 11.5.5 Eccentricity of vertical load

The eccentricity of a vertical load applied to the top of a wall shall be determined as follows:

- The eccentricity of the load applied by the floor (or roof) supported by the top of the wall being designed shall be taken as—
  - for a discontinuous floor, one third of the depth of the bearing area measured from the span face of the wall;
  - for a cast in situ concrete floor continuous over the wall, zero.
- The eccentricity of the aggregated load from all floors above the floor at the top of the wall being designed may be taken to be zero.

The resultant eccentricity of the total load from (a) and (b) shall be calculated but taken as not less than  $0.05t_w$ .

## 11.6 DESIGN OF WALLS FOR IN-PLANE SHEAR FORCES

### 11.6.1 Critical section for shear

The critical section for maximum shear shall be taken at a distance from the base of  $0.5L_w$  or  $0.5H_w$ , whichever is less.

### 11.6.2 Strength in shear

The design strength of a wall subject to in-plane shear shall be taken as—

$$\phi V_u$$

... 11.6.2

where

$$V_u = V_{uc} + V_{us}$$

and  $V_{uc}$  and  $V_{us}$  are determined from Clause 11.6.3 and Clause 11.6.4 respectively, but in no case shall  $V_u$  be taken as greater than—

$$V_{u,max.} = 0.2f'_c (0.8L_w t_w)$$

The value of  $\phi$  shall be determined from Table 2.2.2.

### 11.6.3 Shear strength excluding wall reinforcement

The ultimate shear strength of a wall excluding wall reinforcement ( $V_{uc}$ ) shall be taken as follows:

- (a) For  $H_w/L_w \leq 1$ —

$$V_{uc} = \left( 0.66\sqrt{f'_c} - 0.21 \frac{H_w}{L_w} \sqrt{f'_c} \right) 0.8L_w t_w; \text{ or} \quad \dots 11.6.3(1)$$

- (b) For  $H_w/L_w > 1$ , the lesser of the values calculated from Item (a) above; and—

$$V_{uc} = \left[ 0.05\sqrt{f'_c} + \frac{0.1\sqrt{f'_c}}{\left( \frac{H_w}{L_w} - 1 \right)} \right] 0.8L_w t_w \quad \dots 11.6.3(2)$$

but not less than  $0.17\sqrt{f'_c} (0.8L_w t_w)$ .

### 11.6.4 Contribution to shear strength by wall reinforcement

The contribution to the ultimate shear strength of a wall by wall reinforcement ( $V_{us}$ ) shall be determined from the following equation:

$$V_{us} = p_w f_{sy} (0.8L_w t_w), \quad \dots 11.6.4$$

where  $p_w$  is determined as follows:

- (a) For walls where  $H_w/L_w \leq 1$ ,  $p_w$  shall be the lesser of the ratios of either the vertical reinforcement area or the horizontal reinforcement area to the cross-sectional area of wall in the respective direction.
- (b) For walls where  $H_w/L_w > 1$ ,  $p_w$  shall be the ratio of the horizontal reinforcement area to the cross-sectional area of wall per vertical metre.

## 11.7 REINFORCEMENT REQUIREMENTS FOR WALLS

### 11.7.1 Minimum reinforcement

Walls shall have a reinforcement ratio ( $p_w$ )—

- (a) in the vertical direction, of not less than the larger of 0.0025 and the value required for strength unless the design axial force does not exceed the lesser of  $0.03 f'_c$  and 2 MPa where the limit may be reduced to 0.0015; and
- (b) in the horizontal direction, of not less than 0.0025, except that for a wall designed for one-way buckling [using Clause 11.4(a)] and where there is no restraint against horizontal shrinkage or thermal movements, this may be reduced to zero if the wall is less than 2.5 m wide, or to 0.0015 otherwise.

NOTE: For walls greater than 500 mm thick, the minimum reinforcement required near each surface may be calculated using 250 mm for  $t_w$ .

### 11.7.2 Horizontal reinforcement for crack control

Where a wall is restrained from expanding or contracting horizontally due to shrinkage or temperature, the horizontal reinforcement ratio shall be not less than the following, as appropriate:

- (a) For exposure classifications A1 and A2—
  - (i) where a minor degree of control over cracking is required ..... 0.0025;
  - (ii) where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view ..... 0.0035; and
  - (iii) where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes ..... 0.006.
- (b) For exposure classifications B1, B2, C1 and C2 ..... 0.006.

NOTE: For walls longer than 8 m, additional horizontal crack control reinforcement may be needed at the base of the wall to control thermal cracking during hydration.

### 11.7.3 Spacing of reinforcement

The minimum clear distance between parallel bars, ducts and tendons shall be sufficient to ensure the concrete can be placed and compacted to conform with Clause 17.1.3 but shall be not less than  $3d_b$ .

The maximum centre-to-centre spacing of parallel bars shall be the lesser of  $2.5t_w$  and 350 mm.

The vertical and horizontal reinforcement shall be provided in two grids, one near each face of the wall under any of the following conditions:

- (a) Walls greater than 200 mm thick.
- (b) Any part of a wall structure where tension exceeds the tensile capacity of the concrete under the design ultimate loads. When considering design actions due to earthquake, the determination of whether or not a wall horizontal cross-section is subject to tension shall be based on a lateral analysis undertaken using a structural ductility factor ( $\mu$ ) of 1.00 and a structural performance factor ( $S_p$ ) of 0.77.
- (c) Walls designed for two-way buckling [based on Clauses 11.4(b) or 11.4(c)].
- (d) Walls exceeding 12 metres in unsupported height.

#### 11.7.4 Restraint of vertical reinforcement

For walls designed as columns in accordance with Section 10, the restraint provisions of Clause 10.7.4 are not required if either one of the following conditions is met:

- (a)  $N^* \leq 0.5 \phi N_u$  ... 11.7.4
- (b) The concrete strength is  $\leq 50$  MPa and either—
  - (i) the vertical reinforcement is not used as compressive reinforcement; or
  - (ii) the vertical reinforcement ratio is not greater than 0.01 and a minimum horizontal reinforcement ratio of 0.0025 is provided.

For walls with a concrete strength exceeding 50 MPa the vertical reinforcement shall be restrained in accordance with Clause 14.5.4 of this Standard.

#### 11.7.5 Dowelled connections in precast walls

Where a ductility factor  $\mu$  greater than 1.0 has been used in the earthquake analysis and a ductile response has therefore been assumed, dowels shall be capable of transferring the yield force of the vertical wall reinforcement in the wall through the joint and as a minimum  $A_{st,dowel} > A_{st,wall}$ .

## SECTION 12 DESIGN OF NON-FLEXURAL MEMBERS, END ZONES AND BEARING SURFACES

### 12.1 GENERAL

#### 12.1.1 Scope of Section

This section applies to the design of non-flexural members, including deep beams, footings, and pile caps where the ratio of the clear span or projection to the overall depth is less than—

- (a) for cantilevers.....1.5;
- (b) for simply supported members ..... 3; and
- (c) for continuous members.....4.

This section also applies to the design of non-flexural regions, including corbels, continuous nibs, end zones of prestressed members, and surfaces where concentrated forces act.

#### 12.1.2 Design for strength

The design for strength shall be carried out using one of the following:

- (a) Linear elastic stress analysis and the checking procedure given in Clause 2.2.3.
- (b) Strut-and-tie analysis, and the checking procedure given in Clause 2.2.4.
- (c) Non-linear stress analysis and the checking procedure given in Clause 2.2.6.

The value of the capacity reduction factor shall be determined from Clause 2.2, as appropriate for the analysis and checking procedure adopted.

#### 12.1.3 Design for serviceability

Design for serviceability shall be in accordance with Clause 2.3 and Clause 12.7.

### 12.2 STRUT-AND-TIE MODELS FOR THE DESIGN OF NON-FLEXURAL MEMBERS

#### 12.2.1 Design models

Design models are distinguished by the method in which the forces are transferred from the point of loading to the supports. The models are identified as Types I, II and III. These are shown in Figure 12.2.1 for the specific case of deep beams, and are defined as follows:

- (a) *Type I* The load is carried to the supports directly by major struts.
- (b) *Type II* The load is taken to the supports by a combination of primary (major) and secondary (minor) struts. Hanger reinforcement is required to return the vertical components of forces developed in the secondary struts to the top of the member.
- (c) *Type III* The load is carried to the supports via a series of minor struts with hanger reinforcement used to return the vertical components of the strut forces to the top of the member.

For Type II models, the force carried by the secondary struts shall be within the limits  $0 \leq T_w \leq F$ , where  $T_w$  is the vertical component of the force carried by the secondary struts and  $F$  is the total vertical component of the external load carried through the shear span.

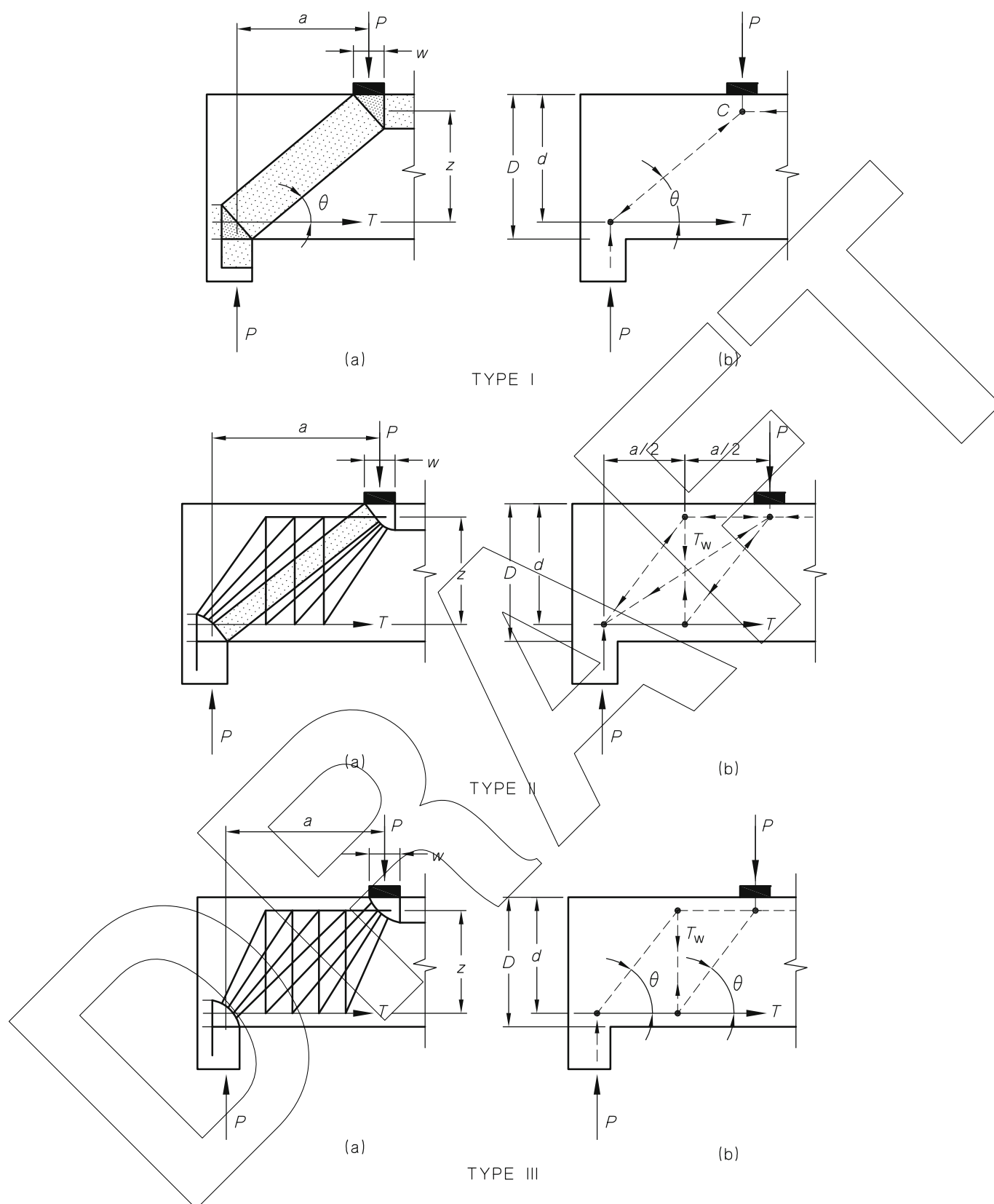


FIGURE 12.2.1 (a) STRUT-AND-TIE MODELS AND (b) SIMPLIFIED DESIGN MODELS

### 12.2.2 Strut bursting reinforcement

Strut bursting reinforcement shall be provided in accordance with Clause 7.2.4.

### 12.3 ADDITIONAL REQUIREMENTS FOR CONTINUOUS CONCRETE NIBS AND CORBELS

Corbels and continuous nibs that support other members shall be also designed to conform with the following:

- (a) The tensile reinforcement shall be anchored at the free end of the nib or corbel, either by a welded or mechanical anchorage, or by a loop in either the vertical or horizontal plane. Where the main reinforcement is looped, the loaded area shall not project beyond the straight portion of this reinforcement.
- (b) Horizontal forces resulting from the supported member, because of factors, such as movement, shrinkage, temperature and prestress, shall be assessed but shall not be taken as less than 20% of the vertical force.
- (c) The line of action of the load shall be taken at the outside edge of a bearing pad for continuous nibs and at one third the width of the bearing from the free end for a corbel. Where no bearing pad is provided, the line of action may be taken at the commencement of any edge chamfer, or at the outside face of the nib or corbel as appropriate.
- (d) Where a flexural member is being supported, the outside face of a nib shall be protected against spalling.

### 12.4 ADDITIONAL REQUIREMENTS FOR STEPPED JOINTS IN BEAMS AND SLABS

The design of stepped joints shall take into account the horizontal forces and movements from the supported members and shall conform with the following:

- (a) Horizontal forces resulting from movement, shrinkage, temperature, prestress and other factors in the supported member shall be assessed but shall not be taken as less than 20% of the vertical force.
- (b) In prestressed members, the vertical component of the force from the prestressing steel shall be ignored.
- (c) The horizontal reinforcement shall extend at least a distance equal to the beam depth ( $D$ ) beyond the step and shall be provided with anchorage beyond the plane of any potential shear crack.
- (d) Hanging reinforcement shall be placed as close as possible to the vertical face of the step.

### 12.5 ANCHORAGE ZONES FOR PRESTRESSING ANCHORAGES

#### 12.5.1 General

This clause applies to the design of prismatic anchorage zones in post-tensioned concrete members but is limited to cases having no more than two anchorages in any elevation or plan.

NOTE: Where there are more than two anchorages in any elevation or plan, the design may be undertaken in accordance with Section 7.

#### 12.5.2 Reinforcement

Reinforcement shall be provided to carry tensile forces that arise from the action and dispersal of the prestressing forces in anchorage zones.

In general, the dispersal occurs through both the depth and the width of the anchorage zone, and reinforcement shall, therefore, be provided in planes parallel to the end faces in two orthogonal directions. A two-dimensional analysis for each loading case shall be carried out in each direction in turn. The tensile forces shall be calculated on longitudinal sections through anchorages and on longitudinal sections where peak values of transverse moments occur.

The transverse moment on a longitudinal section is the equilibrating moment acting on the free body bounded by the longitudinal section, a free surface parallel to it, the loaded face, and a plane parallel to the loaded face at the inner end of the anchorage zone.

### 12.5.3 Loading cases to be considered

Loading cases to be considered shall include—

- (a) all anchorages loaded; and
- (b) critical loadings during the stressing operation.

Where the distance between two anchorages is less than 0.3 times the total depth, or breadth, of the member, consideration shall be given to the effects of the pair acting in a manner similar to a single anchorage subject to the combined forces.

### 12.5.4 Calculation of tensile forces along line of an anchorage force

The force resultant of transverse tensile stresses induced along the line of action of an anchorage force shall be taken as follows:

$$T = 0.25P (1 - k_r) \quad \dots 12.5.4$$

where

$P$  = maximum force occurring at the anchorage during jacking

$k_r$  = ratio of the depth, or breadth, of an anchorage bearing plate to the corresponding depth, or breadth, of the symmetrical prism

The symmetrical prism is defined as a notional prism with an anchorage at the centre of its end face and a depth, or breadth, taken as twice the distance from the centre of an anchorage to the nearer concrete face.

### 12.5.5 Calculation of tensile forces induced near the loaded face

At longitudinal sections remote from a single eccentric anchorage, or between widely spaced anchorages, where the sense of the transverse moment indicates the tensile stress resultant acts near the loaded face, the tensile force shall be calculated as follows:

- (a) For a single eccentric anchorage, by dividing the peak transverse moment by a lever arm assumed to be one half the overall depth of the member.
- (b) Between pairs of anchorages, by dividing the peak transverse moment by a lever arm assumed to be 0.6 times the spacing of the anchorages.

### 12.5.6 Quantity and distribution of reinforcement

The cross-sectional area of reinforcement for each situation shall be calculated by dividing the tensile forces derived in accordance with Clauses 12.5.4 and 12.5.5 by 150 MPa. This reinforcement shall be distributed as follows:

- (a) Reinforcement to resist the forces calculated under Clause 12.5.4 distributed uniformly from 0.2D to 1.0D from the loaded face. Similar reinforcement shall be placed from the plane at 0.2D to as near as practicable to the loaded face.  $D$  shall be equal to the depth or breadth of the symmetrical prism as appropriate.
- (b) Reinforcement to resist the forces calculated under Clause 12.5.5 shall be placed as close to the loaded face as is consistent with cover and compaction requirements.



At any plane parallel to the loaded face, the reinforcement shall be determined from the longitudinal section with the greatest reinforcement requirements at that plane, and shall extend over the full depth or breadth of the end zone.

## 12.6 BEARING SURFACES

Unless special confinement reinforcement is provided, the design bearing stress at a concrete surface shall not exceed

$$\phi 0.9 f'_c \sqrt{(A_2 / A_1)} \text{ or } \phi 1.8 f'_c \quad \dots 12.6$$

whichever is less—

where

$A_2$  = largest area of the supporting surface that is geometrically similar to and concentric with  $A_1$

$A_1$  = a bearing area

In the case of a bearing surface where the supporting structure is sloped or stepped, it shall be permissible to take  $A_2$  as the area of the base of the largest frustum of a right pyramid or cone—

- (a) having for its opposite end the bearing area  $A_1$ ;
- (b) having side slopes of one longitudinally to two transversely, with respect to the direction of the load; and
- (c) contained wholly within the supporting structure.

This clause is not applicable to the design of nodes within a strut-and-tie model.

## 12.7 CRACK CONTROL

The requirements for crack control may be deemed to be satisfied if the stress in the reinforcement under the short-term service loads  $\sigma_{st}$  and the maximum centre-to-centre spacing between the bonded reinforcement crossing the crack  $s$  satisfy the following:

- (a) Where a minor degree of control over cracking is required .....  $\sigma_{st} \leq 350 \text{ MPa}$ .  
 $s \leq 350 \text{ mm}$
- (b) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view .....  $\sigma_{st} \leq 250 \text{ MPa}$ .  
 $s \leq 300 \text{ mm}$ .
- (c) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes .....  $\sigma_{st} \leq 200 \text{ MPa}$ .  
 $s \leq 200 \text{ mm}$ .

For prestressed concrete, the change in stress in the tendons after the point of decompression shall not exceed the limits given by Items (a), (b) or (c), as appropriate.

## SECTION 13 STRESS DEVELOPMENT OF REINFORCEMENT AND TENDONS

### 13.1 STRESS DEVELOPMENT IN REINFORCEMENT

#### 13.1.1 General

The calculated force in reinforcing steel at any cross-section shall be developed on each side of that cross-section in accordance with Clauses 13.1.2 to 13.1.8, as appropriate.

#### 13.1.2 Development length for a deformed bar in tension

##### 13.1.2.1 Development length to develop yield strength

The development length ( $L_{sy,t}$ ) to develop the characteristic yield strength ( $f_{sy}$ ) of a deformed bar in tension shall be calculated from either Clause 13.1.2.2 or Clause 13.1.2.3.

##### 13.1.2.2 Basic development length

The development length ( $L_{sy,t}$ ) shall be taken as the basic development length of a deformed bar in tension ( $L_{sy,tb}$ ), calculated from—

$$L_{sy,tb} = \frac{0.5k_1k_3f_{sy}d_b}{k_2\sqrt{f'_c}} \geq 0.058f_{sy}k_1d_b \quad \dots 13.1.2.2$$

where

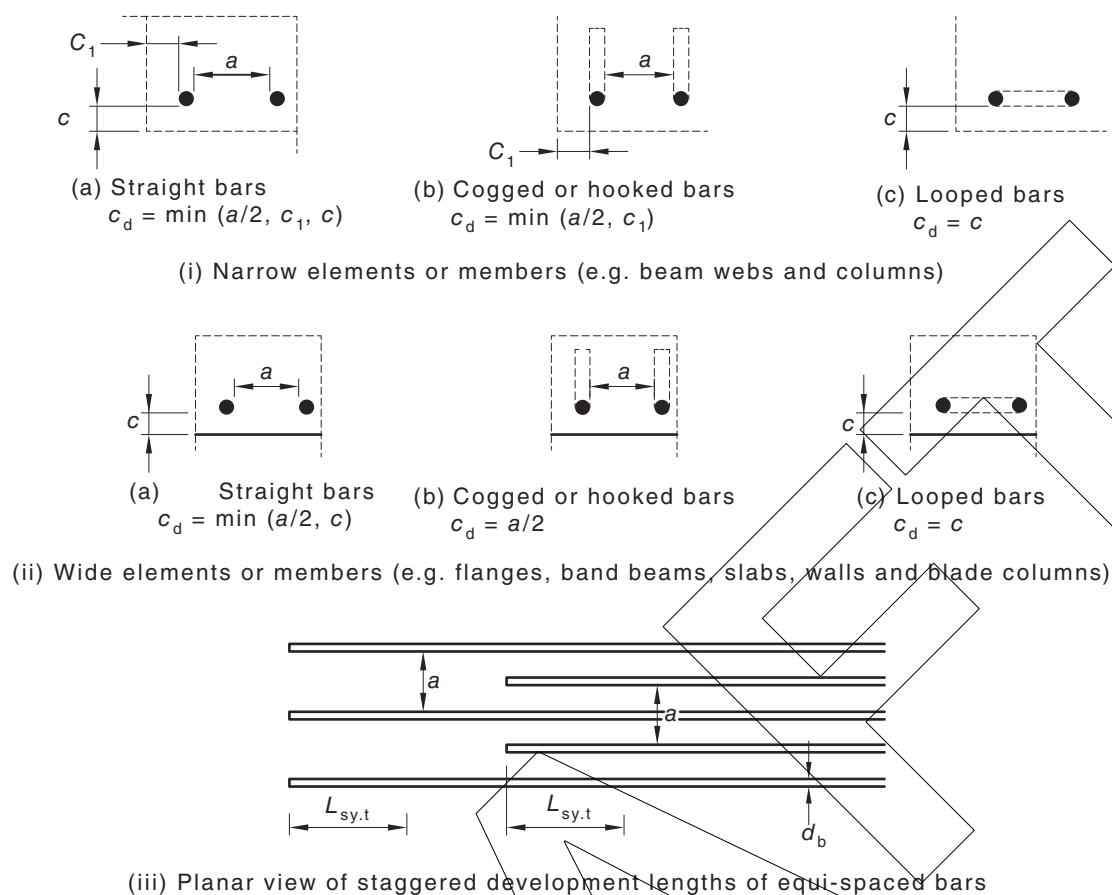
- $k_1 = 1.3$  for a horizontal bar with more than 300 mm of concrete cast below the bar; or
- $= 1.0$  otherwise
- $k_2 = (132 - d_b)/100$ , and
- $k_3 = 1.0 - 0.15(c_d - d_b)/d_b$  (within the limits  $0.7 \leq k_3 \leq 1.0$ ); where  
 $c_d$  = a dimension (in millimetres), as shown in Figure 13.1.2.2

The value of  $f'_c$  used in Equation 13.1.2.2 shall not be taken to exceed 65 MPa; and the bar diameter ( $d_b$ ) is in millimetres.

The value of  $L_{sy,tb}$  calculated as above shall be—

- (a) multiplied by 1.5 for epoxy-coated bars; and
- (b) multiplied by 1.3 when lightweight concrete is used; and
- (c) multiplied by 1.3 for all structural elements built with slip forms.

NOTE: A smaller value of  $L_{sy,t}$  may be possible using the provisions of Clause 13.1.2.3.

FIGURE 13.1.2.2 VALUES OF  $c_d$

### 13.1.2.3 Refined development length

Where a refined development length is required, the development length in tension ( $L_{sy,t}$ ) shall be calculated from—

$$L_{sy,t} = k_4 k_5 L_{sy,tb} \quad \dots 13.1.2.3$$

where

$$k_4 = 1.0 - K\lambda \quad (\text{within the limits } 0.7 \leq k_4 \leq 1.0)$$

$$k_5 = 1.0 - 0.04\rho_p \quad (\text{within the limits } 0.7 \leq k_5 \leq 1.0)$$

$K$  = a factor that accounts for the weighted average effectiveness of transverse reinforcement in controlling potential splitting cracks along a development or lap splice length;

$$= 0.05 \times (1 + n_f/n_{bs}) \leq 0.10$$

= with values of  $n_f$  and  $n_{bs}$  shown in Table 13.1.2.3 for typical arrangements of transverse reinforcement for different member types; and

= 0 if the transverse reinforcement is not located between the longitudinal bars and the concrete tensile face

$$\lambda = (\Sigma A_{tr} - \Sigma A_{tr,min})/A_s \geq 0$$

$\Sigma A_{tr}$  = sum of cross-sectional area of the transverse bars along a development or lap length

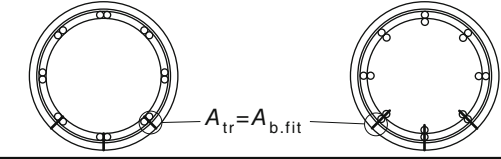
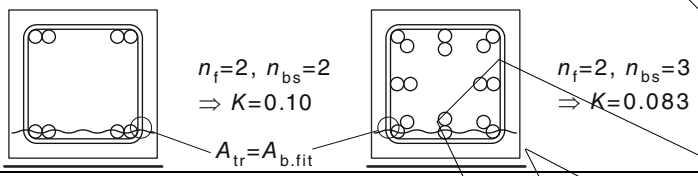
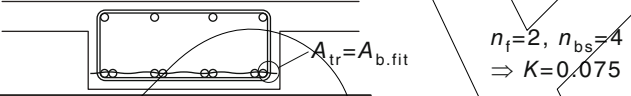
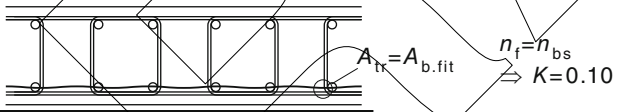
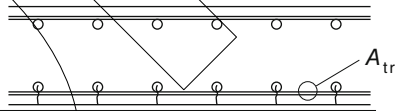
$\Sigma A_{tr,min}$  = sum of cross-sectional area of the minimum transverse reinforcement, which may be taken as  $0.25A_s$  for beams or columns and 0 for slabs or walls

$A_s$  = cross-sectional area of a single bar of diameter ( $d_b$ ) being anchored

$\rho_p$  = transverse compressive pressure (in MPa), at the ultimate limit state along the development length and perpendicular to the plane of splitting

The product  $k_3 k_4 k_5$  shall be not taken as less than 0.7.

**TABLE 13.1.2.3**  
**VALUES OF K FOR TYPICAL ARRANGEMENTS OF TRANSVERSE REINFORCEMENT FOR DIFFERENT MEMBER TYPES**

Member type	Examples of potential splitting cracks at a tensile face	$n_f$	$n_{bs}$	$K$ (see Note 2)
Circular column		1	1	0.10
Rectangular column		$\geq 1$	$\geq 1$	$0.05 \leq K \leq 0.10$
Beam		$\geq 1$	$\geq 1$	$0.05 \leq K \leq 0.10$
Slab or wall (with fitments)		$\geq 1$	$\geq 1$	$0.05 \leq K \leq 0.10$
Slab or wall (without fitments) <sup>1</sup>		0	1 per main bar spacing	0.05 (see Note 3)

**NOTES:**

- Fitments are a type of transverse reinforcement.
- The same value of  $K$  applies to all of the longitudinal bars being either anchored or lap spliced, that is, it is a weighted average value.
- To be effective, the transverse reinforcement needs to be located between the longitudinal bars and the concrete tensile face as shown, otherwise  $K = 0$ .

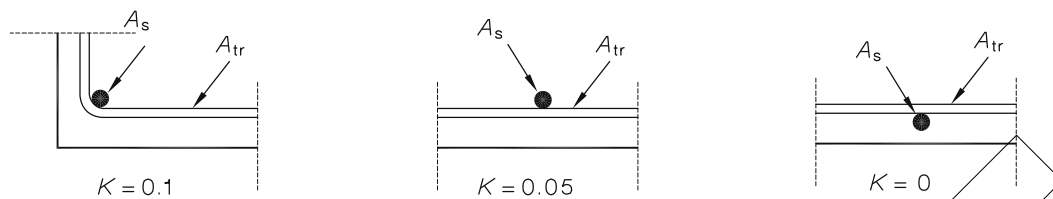


FIGURE 13.1.2.3 VALUES OF K FOR DIFFERENT BAR POSITIONS

**13.1.2.4 Development length to develop less than the yield strength**

The development length ( $L_{st}$ ) to develop a tensile stress ( $\sigma_{st}$ ), less than the yield strength ( $f_{sy}$ ), shall be calculated from—

$$L_{st} = L_{sy,t} \frac{\sigma_{st}}{f_{sy}}$$

... 13.1.2.4

but shall be not less than—

- (a)  $12d_b$ ; or
- (b) for slabs, as permitted by Clause 9.1.3.1(a)(ii).

**13.1.2.5 Development length around a curve**

Tensile stress may be considered to be developed around a curve if the internal diameter of the curve is  $10d_b$  or greater.

**13.1.2.6 Development length of a deformed bar with a standard hook or cog**

Where a deformed bar ends in a standard hook or cog conforming with Clause 13.1.2.7, the tensile development length of that end of the bar, measured from the outside of the hook/cog, shall be taken as  $0.5L_{sy,t}$  or  $0.5L_{st}$  as applicable (as shown in Figure 13.1.2.6).

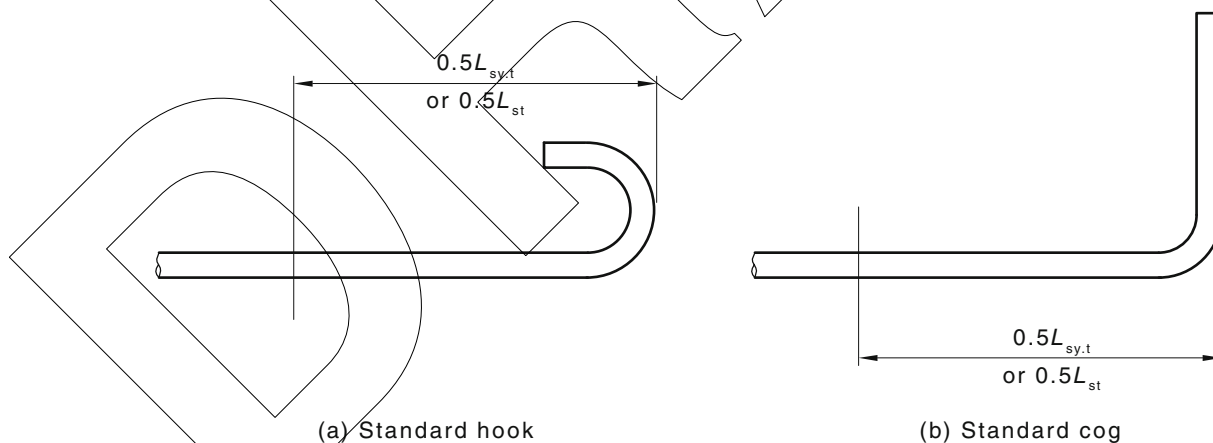


FIGURE 13.1.2.6 DEVELOPMENT LENGTH OF A DEFORMED BAR WITH A STANDARD HOOK OR COG

### 13.1.2.7 *Standard hooks and cogs*

The standard hook or cog referred to in Clause 13.1.2.6 shall be one of the following:

- (a) A hook consisting of a 180° bend with a nominal internal diameter conforming with Clause 17.2.3.2 plus a straight extension of  $4d_b$  or 70 mm, whichever is greater.
- (b) A hook consisting of a 135° bend with the same internal diameter and length as Item (a).
- (c) A cog, consisting of a 90° bend with a nominal internal diameter conforming with Clause 17.2.3.2 but not greater than  $8d_b$  and having the same total length as required for a 180° hook of the same diameter bar.

### 13.1.3 *Development length of plain bars in tension*

The development length ( $L_{sy,t}$ ) to develop the yield strength ( $f_{sy}$ ) of a plain bar in tension shall be taken as the basic development length calculated in accordance with Clause 13.1.2.2 multiplied by 1.5, but  $L_{sy,t}$  shall be not less than 300 mm.

Where a plain bar ends in a standard hook or cog conforming with Clause 13.1.2.7, the tensile development length of that end of the bar, measured from the outside of the hook/cog, shall be taken as  $0.5L_{sy,t}$  or  $0.5L_{st}$  as applicable (as shown in Figure 13.1.2.6).

### 13.1.4 *Development length of headed reinforcement in tension*

A head used to develop a deformed bar in tension shall consist of a nut or plate, having either a round, elliptical or rectangular shape, attached to the end(s) of the bar by welding, threading or swaging of suitable strength to avoid failure of the steel connection at ultimate load.

In addition—

- (a) the bar diameter ( $d_b$ ) shall not exceed 40 mm;
- (b) the density of the concrete shall be greater than 2100 kg/m<sup>3</sup>;
- (c) the net bearing area of head shall be not less than four times the cross-sectional area of the bar;
- (d) the clear cover for the bar shall not be less than  $2d_b$ ; and
- (e) the clear spacing between bars shall be not less than  $4d_b$ .

If the cross-sectional area of the head of the headed reinforcement, or the area of the end plate for deformed bars mechanically anchored with an end plate in the plane perpendicular to the axis of the bar, is at least 10 times the cross-sectional area of the bar, the bar shall be considered to have a development length ( $L_{sy,t}$ ) measured from the inside face of the head equal to  $0.4 L_{sy,t}$  of a bar of the same diameter.

### 13.1.5 *Development length of deformed bars in compression*

#### 13.1.5.1 *Development length to develop yield strength*

The development length ( $L_{sy,c}$ ) to develop the characteristic yield strength ( $f_{sy}$ ) of a deformed bar in compression shall be calculated from either Clause 13.1.5.2 or Clause 13.1.5.3, but shall be not less than 200 mm.

### 13.1.5.2 Basic development length

The development length ( $L_{sy.c}$ ) shall be taken as the basic development length of a deformed bar in compression ( $L_{sy.cb}$ ) calculated from—

$$L_{sy.cb} = \frac{0.22 f_{sy}}{\sqrt{f'_c}} d_b \geq 0.0435 f_{sy} d_b \text{ or } 200 \text{ mm, whichever is the greater} \quad \dots 13.1.5.2$$

NOTE: A smaller value of  $L_{sy.c}$  may be obtained using the provisions of Clause 13.1.5.3.

### 13.1.5.3 Refined development length

Where a refined development length is required, the development length in compression ( $L_{sy.c}$ ) shall be calculated from—

$$L_{sy.c} = k_6 L_{sy.cb} \quad \dots 13.1.5.3$$

Where transverse reinforcement with at least three bars, transverse to and outside the bar being developed is provided within  $L_{sy.cb}$  and when  $\Sigma A_{tr}/s \geq A_s/600$ ,

$$k_6 = 0.75$$

where  $\Sigma A_{tr}$  and  $A_s$  are defined in Clause 13.1.2.3.

In all other cases,  $k_6 = 1.0$ .

### 13.1.5.4 Development length to develop less than the yield strength

The development length ( $L_{sc}$ ) to develop a compressive stress ( $\sigma_{sc}$ ), less than the yield strength ( $f_{sy}$ ), shall be calculated from—

$$L_{sc} = L_{sy.c} \frac{\sigma_{sc}}{f_{sy}} \text{ (but not less than 200 mm)} \quad \dots 13.1.5.4$$

A bend or a standard hook shall not be considered effective in developing stress in reinforcement in compression.

### 13.1.6 Development length of plain bars in compression

The development length for plain bars in compression shall be twice the calculated value of  $L_{sy.c}$  or  $L_{sy.cb}$  for a deformed bar.

### 13.1.7 Development length of bundled bars

The development length of a unit of bundled bars shall be based on the development length required for the largest bar within the bundle increased by—

- (a) for a 3-bar bundle ..... 20%; and
- (b) for a 4-bar bundle ..... 33%.

### 13.1.8 Development length of welded plain or deformed mesh in tension

#### 13.1.8.1 Development length to develop yield strength

The development length ( $L_{sy.t}$ ) of welded plain or deformed mesh, measured from the critical section to the end of the bar or wire, shall be calculated in accordance with Clause 13.1.8.2, Clause 13.1.8.3 or 13.1.8.4, as appropriate.

#### 13.1.8.2 Two or more cross-bars within development length

The yield strength of plain or deformed bars of welded mesh shall be considered to be developed by embedding at least two cross-bars spaced at not less than 100 mm or 50 mm apart within the development length for plain or deformed bars respectively, with the first one not less than 50 mm from the critical section.



### 13.1.8.3 One cross-bar within development length

When only one cross-bar is located within the development length, the minimum length measured from the critical section to the outermost cross-bar shall be not less than  $L_{sy.tb}$  calculated from—

$$L_{sy.tb} = 3.25 \frac{A_b}{s_m} \frac{f_{sy}}{\sqrt{f'_c}} \quad \dots 13.1.8.3$$

but not less than 150 mm for plain mesh and not less than 100 mm for deformed mesh, where

$A_b$  = area of the individual bar being developed in square millimetres

$s_m$  = spacing of bars being developed, in millimetres

### 13.1.8.4 No cross-bars within development length

When no cross-bars are located within the development length, the development length of welded mesh shall be determined by Clauses 13.1.2 and 13.1.3, as appropriate.

### 13.1.8.5 Development length to develop less than the yield strength

The development length ( $L_{st}$ ) to develop a tensile stress ( $\sigma_{st}$ ) less than the yield strength ( $f_{sy}$ ) shall be calculated from the development length of Clauses 13.1.8.3 or 13.1.8.4 using the following equation:

$$L_{st} = L_{sy.tb} \frac{\sigma_{st}}{f_{sy}} \quad \dots 13.1.8.5$$

but not less than 150 mm for plain mesh and not less than 100 mm for deformed mesh.

## 13.2 SPlicing OF REINFORCEMENT

### 13.2.1 General

The following general requirements shall apply to the splicing of reinforcement:

- Splices of reinforcement shall be made only as required or permitted on the design drawings or in specifications.
- The splice shall be made by welding, by mechanical means, by end-bearing, or by lapping.
- Splicing of reinforcement shall take into account the requirements of Clause 17.1.3 regarding the placement of concrete.
- Splices required in bars in tension-tie members shall be made only by welding or mechanical means.
- Lapped splices shall not be used for bars in compression or tension with diameter larger than 40 mm.
- Welding of reinforcing bars shall not be made less than  $3d_b$  from that part of a bar that has been bent and re-straightened.

### 13.2.2 Lapped splices for bars in tension

In wide elements or members (e.g. flanges, band beams, slabs, walls and blade columns), where the bars being lapped are in the plane of the element or member, the tensile lap length ( $L_{sy.t.lap}$ ) for either contact or non-contact splices shall be calculated from—

$$L_{sy.t.lap} = k_7 L_{sy.t} \geq 29k_1 d_b \quad \dots 13.2.2$$

where

$L_{sy,t}$  is calculated in accordance with Clause 13.1.2.1. (In the determination of  $L_{sy,t}$  for use in Equation 13.2.2, the lower limit of  $29k_1d_b$  in Equation 13.1.2.2 does not apply); and

$k_7$  shall be taken as 1.25 unless  $A_s$  provided is at least twice  $A_s$  required and no more than half of the reinforcement at the section is spliced, in which case  $k_7$  may be taken as 1.

In narrow elements or members (such as beam webs and columns), the tensile lap length ( $L_{sy,t.lap}$ ) shall be not less than the larger of  $29k_1d_b$ ,  $k_7 L_{sy,t}$  and  $L_{sy,t} + 1.5s_b$ , where  $s_b$  is the clear distance between bars of the lapped splice as shown in Figure 13.2.2. However, if  $s_b$  does not exceed  $3d_b$ , then  $s_b$  may be taken as zero for calculating  $L_{sy,t.lap}$ .

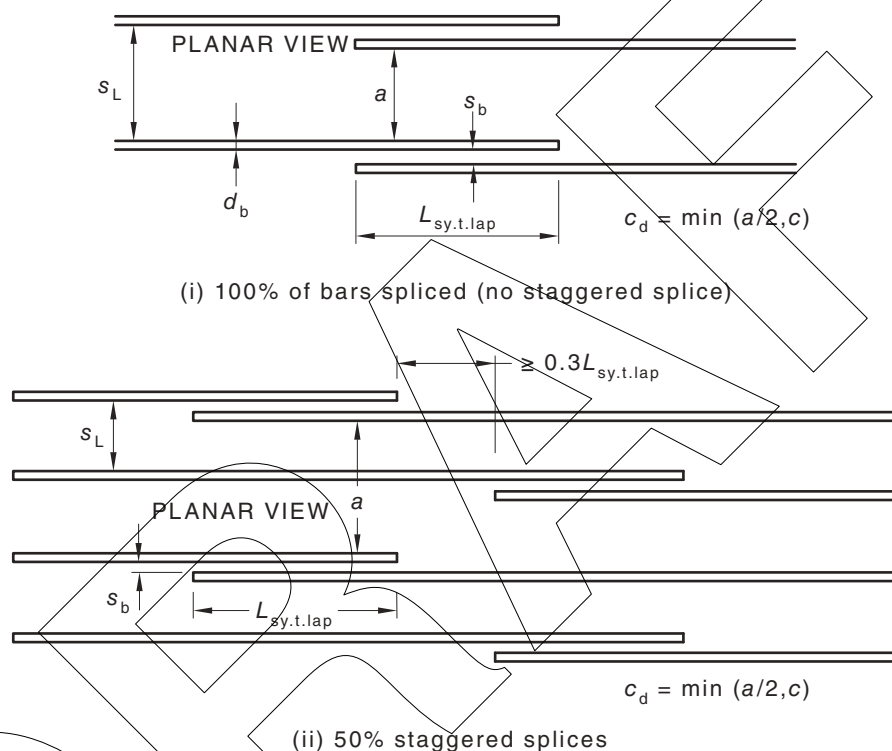


FIGURE 13.2.2 VALUE OF  $c_d$  FOR LAPPED SPLICES

### 13.2.3 Lapped splices for mesh in tension

A lapped splice for welded mesh in tension shall be made so the two outermost cross-bars spaced at not less than 100 mm or 50 mm apart for plain or deformed bars, respectively, of one sheet of mesh overlap the two outermost cross-bars of the sheet being lapped as shown in Figure 13.2.3. The minimum length of the overlap shall equal 100 mm.

A lapped splice for welded deformed and plain meshes, with no cross-bars within the splice length shall be determined in accordance with Clause 13.2.2.



FIGURE 13.2.3 LAPPED SPLICES FOR WELDED MESH

13.2.4 Lapped splices for bars in compression

The minimum length of a lapped splice for deformed bars in compression shall be the development length in compression ( $L_{sy,c}$ ) given in Items (a), (b) or (c), as appropriate, but shall be not less than 300 mm:

- (a) The development length in compression shall be in accordance with Clause 13.1.5 but not less than  $40d_b$ .
- (b) In compressive members with stirrups or fitments where at least three sets of fitments are present over the length of the lap and  $A_{tr}/s \geq A_b/1000$ , a lap length of 0.8 times the value given in Item (a).
- (c) In helically tied compressive members, if at least three turns of helical reinforcement are present over the length of the lap and  $A_{tr}/s \geq n A_b/6000$ , a lap length of 0.8 times the value given in Item (a), where  $n$  = the number of bars uniformly spaced around the helix.

In this clause,  $A_b$  is defined as the area of the bar being spliced.

13.2.5 Lapped splices for bundled bars

Lapped splices for a unit of bundled bars shall be based on the lap splice length required for the largest bar within the bundle increased by—

- (a) for a three bar bundle..... 20%; and
- (b) for a four bar bundle..... 33%.

Individual bar splices within a bundle shall not overlap.

13.2.6 Welded or mechanical splices

Welded or mechanical splices formed between Ductility Class N reinforcing bars shall not fail prematurely in tension or compression before the reinforcing bars, unless it can be shown that the strength and ductility of the concrete member meets the design requirements.

When control of cracking or vertical deflection are relevant serviceability design criteria, the potentially detrimental effects of excessive longitudinal slip between spliced Ductility Class N bars and a proprietary mechanical connector shall be considered if tests show the effective slip in the assemblage could exceed 0.1 mm at a tensile stress of 300 MPa. The effective slip shall be taken as the overall deformation of a spliced pair of reinforcing bars measured over a gauge length of  $12d_b$ , less the elongation of the bars assuming they are unspliced over the same gauge length.

13.3 STRESS DEVELOPMENT IN TENDONS

13.3.1 General

In the absence of substantiated test data, the length to develop the calculated force in a pretensioned tendon shall be taken to be a bi-linear relationship defined by the transmission length ( $L_{pt}$ ) in Clause 13.3.2.1 and the total development length ( $L_p$ ) in Clause 13.3.2.2.

### 13.3.2 Transmission lengths of pretensioned tendons

#### 13.3.2.1 Transmission lengths of pretensioned tendons

The transmission length required to develop the effective prestress in pretensioned tendons shall be taken as the length given in Table 13.3.2, as appropriate to type of tendon and strength of concrete at transfer. The transmission length shall be taken to be independent of the effective prestress in the tendon.

It shall be assumed that no change in the position of the inner end of the transmission length occurs with time but that a completely unstressed zone of length  $0.1L_{pt}$  develops at the end of the tendon.

**TABLE 13.3.2**  
**MINIMUM TRANSMISSION LENGTH**  
**FOR PRETENSIONED TENDONS**

Type of tendon	$L_{pt}$ for gradual release	
	$f_{cp} \geq 32 \text{ MPa}$	$f_{cp} < 32 \text{ MPa}$
Indented wire	$100 d_b$	$175 d_b$
Crimped wire	$70 d_b$	$100 d_b$
Ordinary and compact strand	$60 d_b$	$60 d_b$

#### 13.3.2.2 Development length of pretensioned strand

In absence of test data, the bonded length to develop the stress in seven wire pretensioned strand at ultimate strength shall be taken as not less than—

$$L_p = 0.145(\sigma_{pu} - 0.67\sigma_{p,ef})d_b \geq 60d_b \quad \dots 13.3.2.2$$

where  $\sigma_{p,ef}$  is the effective stress in the tendon after allowing for all losses.

Both  $\sigma_{pu}$  and  $\sigma_{p,ef}$  are in megapascals, and the expression in parenthesis is used without units.

Embedment less than the development length is permitted at a section of a member provided the design stress in the strand at that section does not exceed the values obtained from the bi-linear relationship defined by this clause and Clause 13.3.2.1.

The development length of de-bonded strand shall be taken to be  $2L_p$  where the design includes tension in accordance with Clauses 8.6.2 and 9.4.2 in the development length.

#### 13.3.2.3 Development length of pretensioned wire

Pretensioned indented and crimped wire tendons shall be bonded beyond the critical section for a length sufficient to develop the design stress in the wire but not less than 2.25 times the value for the transmission length in Table 13.3.2 as appropriate.

#### 13.3.2.4 Development length of untensioned strand or wire

Where strand or wire is untensioned, the development length shall be taken as not less than 2.5 times the value of the appropriate transmission length of a stressed tendon given in Table 13.3.2 for a tendon stressed to the tensile strength ( $f_{pb}$ ) in Table 3.3.1.

### 13.3.3 Stress development in post-tensioned tendons by anchorages

Anchorages for tendons shall be capable of developing in the tendon the minimum tensile strength ( $f_{pb}$ ).

In addition, anchorages for unbonded tendons shall be capable of sustaining cyclic loading conditions.

### 13.4 COUPLING OF TENDONS

Coupling of tendons shall conform with the following:

- (a) Couplers shall be capable of developing at least 95% of the tendon characteristic minimum breaking force specified.
- (b) Couplers shall be enclosed in grout-tight housings to facilitate grouting of the duct.

DRAFT

## SECTION 14 REQUIREMENTS FOR STRUCTURES SUBJECT TO EARTHQUAKE ACTIONS

### 14.1 GENERAL

This section applies to concrete structures and structural members that contain reinforcement, or tendons, or both, and which form the whole or part of structures or buildings to which AS 1170.4 applies.

Plain concrete members and structures shall not be used to resist earthquake actions, except plain concrete pedestals, footings and pavements are deemed to satisfy the requirements of Clause 2.1.2 and this section.

Values for the structural ductility factor ( $\mu$ ) and the structural performance factor ( $S_p$ ) for concrete members and structures conforming with this Standard shall be as given in Clause 14.3.

Concrete structures and members shall be designed and detailed depending on the value adopted for the structural ductility factor ( $\mu$ ) as follows:

All members shall be designed and detailed—

- (a) in accordance with Clause 14.4; and
- (b) the required option below depending on the value of  $\mu$ —
  - (i) for  $\mu \leq 2$  designed and detailed in accordance with this Standard (AS 3600);
  - (ii) for  $2 < \mu \leq 3$  designed and detailed in accordance with this Standard (AS 3600) and this section, as appropriate;
  - (iii) for  $\mu > 3$  the structure should be designed and detailed in accordance with NZS 1170.5 and NZS 3101. This will require the use of ductility Class E reinforcement; and
  - (iv) structures with an importance level 4 shall be designed to remain serviceable for immediate use following a design event for importance level 2 structures (refer to AS 1170.4).

### 14.2 DEFINITIONS

For the purpose of this Standard, the definitions given in AS 1170.4 and those below apply. Where the definitions in this Standard differ from those given in AS 1170.4, for the purpose of this Standard, those below apply.

NOTE: A number of definitions given in AS 1170.4 are repeated here to avoid users having to refer to AS 1170.4. Those marked with an asterisk have been modified from those in AS 1170.4 to suit their application in this section.

#### 14.2.1 Connection

Mechanical means that provide a load path for actions between structural elements, non-structural elements and structural and non-structural elements.

#### 14.2.2 Ductility (of a structure)

Ability of a structure to sustain its load-carrying capacity and dissipate energy when responding to cyclic displacements in the inelastic range during an earthquake.

#### 14.2.3 Earthquake actions

Inertia-induced actions arising from the response to earthquake of the structure.

#### **14.2.4 Moment-resisting frame**

Essentially complete space frame that supports the vertical and horizontal actions by both flexural and axial resistance of its members and connections.

#### **14.2.5 Moment-resisting frame, intermediate**

Concrete moment-resisting frame designed and detailed in accordance with this Standard to achieve moderate structural ductility (see Table 14.3) and which complies with the specific earthquake detailing requirements of this section.

#### **14.2.6 Moment-resisting frame, ordinary**

Moment-resisting frame, with no particular earthquake detailing except for columns (see Clause 14.4, as specified in this Standard (see Table 14.3)).

#### **14.2.7 Space frame**

A three-dimensional structural system composed of interconnected members, other than loadbearing walls, which is capable of supporting vertical loads and may also provide horizontal resistance to earthquake forces.

#### **14.2.8 Structural ductility factor ( $\mu$ )**

Numerical assessment of the ability of a structure to sustain cyclic displacements in the inelastic range. Its value depends upon the structural form, the ductility of the materials and structural damping characteristics.

#### **14.2.9 Structural performance factor ( $S_p$ )**

Numerical assessment of the additional ability of the total building (structure and other parts) to survive earthquake motion.

#### **14.2.10 Structural wall**

Wall (either loadbearing or non-loadbearing) connected to floor and/or roof diaphragms that are designed to resist horizontal earthquake forces acting in the plane of the wall.

Structural walls may be considered as Non-Ductile, Limited Ductile, or Moderately Ductile and analysed in accordance with AS 1170.4 using the structural ductility and structural performance factors nominated in Table 14.3.

### **14.3 STRUCTURAL DUCTILITY FACTOR ( $\mu$ ) AND STRUCTURAL PERFORMANCE FACTOR ( $S_p$ )**

The structural ductility factor ( $\mu$ ) and the structural performance factor ( $S_p$ ) for concrete structures and members constructed in accordance with this Standard shall be as given in Table 14.3.

**TABLE 14.3**  
**STRUCTURAL DUCTILITY FACTOR ( $\mu$ ) AND STRUCTURAL**  
**PERFORMANCE FACTOR ( $S_p$ )**

Structural system description	( $\mu$ )	$S_p$	$S_p/\mu$	$\mu/S_p$
Special moment-resisting frames (fully ductile)	4	0.67	0.17	6
Ductile structural walls	4	0.67	0.17	6
Ductile partially or fully coupled walls	4	0.67	0.17	6
Intermediate moment-resisting frames (moderately ductile) designed in accordance with this Standard and Clauses 14.4 and 14.5 of this section	3	0.67	0.22	4.5
Combined systems of intermediate moment-resisting frames and moderately ductile structural walls designed in accordance with this Standard and Clauses 14.4, 14.5 and 14.7 of this section	3	0.67	0.22	4.5
Moderately ductile structural walls designed in accordance with this Standard and Clauses 14.4 and 14.7 of this section	3	0.67	0.22	4.5
Ordinary moment-resisting frames in accordance with this Standard	2	0.77	0.38	2.6
Ordinary moment-resisting frames in combination with limited ductile shear walls designed in accordance with this Standard and Clauses 14.4 and 14.6 of this section	2	0.77	0.38	2.6
Limited ductile structural walls designed in accordance with this Standard and Clauses 14.4 and 14.6 of this section	2	0.77	0.38	2.6
Other concrete structures not listed in this table	1.5	0.77	0.51	1.95
Non-ductile structural walls designed in accordance with this Standard and Clause 14.4 of this section	1	0.77	0.77	1.3

## 14.4 GENERAL EARTHQUAKE DESIGN REQUIREMENTS

### 14.4.1 Design for strength

All members shall be designed for strength for the earthquake loads as defined in AS 1170.4 in accordance with this section and Clause 2.2.

### 14.4.2 Inter-storey drift

All vertical load bearing elements shall be designed for the calculated horizontal drifts from earthquake actions.

Prefabricated concrete members which are attached to, or enclose, the exterior of a structure (e.g. non-structural cladding panels) shall be capable of accommodating movements of the structure resulting from the relevant earthquake actions as follows:

- All connections and panel joints shall allow for the expected relative movement between floors in adjoining storeys.
- Connections shall have sufficient ductility and rotational capacity to preclude non-ductile failure of the connection.

### 14.4.3 Ordinary moment—resistant frames (OMRFs)

For an ordinary moment frame forming part of the seismic-force-resisting system, beams shall have at least two of the longitudinal bars continuous along both the top and bottom faces. These bars shall be fully developed at the face of the supports.

Any column in an Ordinary Moment Resisting Frame (OMRF) which is part of the lateral force resisting system and where the unsupported length  $L_u \leq 5D$  where  $D$  is the dimension of the column in the direction of the span, shall be detailed in accordance with Clause 14.5.



## 14.4.4 STRUCTURAL WALLS

### 14.4.4.1 General

The simplified design method for walls subjected to vertical compression forces provided in Clause 11.5 of this Standard shall only be used for non-ductile walls.

Limited ductile structural walls shall conform with Clause 14.6 and moderately ductile structural walls shall conform with Clause 14.7.

### 14.4.4.2 Groups of walls

When two or more walls are directly inter-connected, or linked to each other by coupling beams or the like, the distribution of in-plane load effects between the walls shall be obtained by linear elastic analysis of the overall structure under the global vertical and lateral loads. The analysis shall distribute in-plane load effects to the individual walls in proportion to their relative stiffness, as determined by their gross cross-sectional properties.

The interconnected vertical edges of walls shall be designed to withstand the transmitted vertical shear forces.

### 14.4.4.3 Axial load limit

All structural walls shall be proportioned to ensure that the vertical stress due to gravity loads acting on the wall at the same time as the earthquake at any horizontal cross section  $N^*/A_g$  is less than  $0.2 f'_c$ .

## 14.4.5 Diaphragms

### 14.4.5.1 General

Diaphragms shall be regarded as non-ductile elements, and shall be designed for seismic actions in accordance with Section 15.

### 14.4.5.2 Calculation of inertia forces

Inertia forces acting on the diaphragm at a given floor level shall be determined by equivalent static analysis in accordance with AS 1170.4, with diaphragm forces calculated based on a structural ductility factor ( $\mu$ ) equal to 1.0, and a structural performance factor ( $S_p$ ) of 0.77.

For lower floors in multi-storey buildings, inertia forces shall incorporate higher mode effects to ensure the maximum inertia force is calculated for the floor under design. Alternatively, the inertia force for any floor may be determined by multiplying the maximum seismic distribution factor for the structure at any floor calculated in accordance with AS 1170.4 by the weight of the floor under calculation.

For seismic design, the mass shall be assumed to be distributed evenly over the diaphragm when undertaking seismic load calculations.

### 14.4.6 Ductility of flexural members with $\mu > 1.25$ and $\mu \leq 3$

In all potential plastic hinge zones:

- (a) Compression face reinforcement equivalent to 1/3 of the ultimate capacity of the tension face reinforcement and tendons shall be provided.
- (b) The neutral axis depth  $k_{uo}$  shall be limited to a maximum of 0.25.

## 14.5 INTERMEDIATE MOMENT-RESISTING FRAMES (IMRFs)

### 14.5.1 General

Reinforced IMRFs and prestressed IMRFs shall be regarded as moderately ductile if, in addition to the detailing requirements of this Standard, they satisfy the detailing requirements of this clause and Clauses 14.5.2 to 14.5.6, and provided only ductility Class N steel or prestressing tendons are used as flexural reinforcement.

Rigid elements may be incorporated into a moment-resisting frame, provided it is shown that the action or failure of these elements will not impair the capacity of the frame to resist horizontal or vertical forces.

In prestressed members, the quantity of tensile steel (reinforcement and tendons) shall be such that the flexural strength of any section is greater than  $1.1(M_{uo})_{min}$  at that section.

### 14.5.2 Beams

#### 14.5.2.1 Longitudinal reinforcement

Beams shall be provided with longitudinal reinforcement and tendons as follows:

- (a) The top and bottom face of the beam shall be continuously reinforced.
- (b) The area of reinforcement and tendons provided in a span shall be such that—
  - (i) the positive-moment strength at a support face is not less than one third of the negative-moment strength provided at that face of the support; and
  - (ii) neither the negative nor the positive-moment strength at any section along the member length is less than one fifth of the maximum moment strength provided at the face of either support.
- (c) Longitudinal reinforcement shall be continuous through intermediate supports. When framing into external columns, the longitudinal reinforcement shall be extended to the far face of the confined region and anchored to develop the yield strength of the reinforcement at the span face of the support.
- (d) Lapped splices in longitudinal reinforcement, located in a region of tension or reversing stress, shall be confined by at least two closed ties at each splice.

#### 14.5.2.2 Shear reinforcement

Beams shall be provided with shear reinforcement conforming with the following requirements:

- (a) The design shear force at any cross-section shall be at least the lesser of:
  - (i) The sum of the shear associated with development of nominal moment strengths ( $\phi = 1.0$ ) of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity loads.
  - (ii) The maximum shear obtained from design load combinations that include earthquake action E, with E taken as twice that prescribed by the AS 1170.4.
- (b) Shear reinforcement shall be perpendicular to the longitudinal reinforcement; be provided throughout the length of the member; have at least two legs; and have a maximum spacing of  $0.5D$ .
- (c) The area of shear reinforcement ( $A_{sv}$ ) shall be not less than  $0.5b_v s / f_{sy.f}$ .

- (d) Over a distance of at least  $2D$  from the face of a support, shear reinforcement shall be closed ties, with the first tie located 50 mm from the support face. These closed ties shall be spaced at centres not greater than  $0.25 d_o$ ,  $8 d_b$ ,  $24 d_f$  or 300 mm, whichever is least,

where

$d_b$  = diameter of the smallest longitudinal bar enclosed by the tie; and

$d_f$  = diameter of the bar forming the tie

### 14.5.3 Slabs

#### 14.5.3.1 General

Slabs shall conform with Paragraph 14.5.2.1, Items (a), (b) and (c). Two-way flat slabs forming part of a moment-resisting frame shall also conform with Paragraph 14.5.3.2.

#### 14.5.3.2 Reinforcement detailing in flat slabs

Reinforcement in flat slabs shall be located and anchored in accordance with the following:

- (a) The top and bottom faces of the column strips shall be continuously reinforced in both directions.
- (b) All reinforcement, which is provided to resist the portion of the slab moment transferred to the support, shall be placed within the column-strip defined in Clause 6.1.4.1.
- (c) A proportion of the reinforcement required by Item (a) above shall be evenly distributed in a width of slab between planes that are 1.5 times the thickness of the slab or drop panel beyond faces of the column or capital.

The proportion to be distributed is given by—

$$1 / \left\{ 1 + (2/3) \left[ \sqrt{(b_l + d_o) / (b_t + d_o)} \right] \right\} \quad \dots 14.10.3.2$$

or 0.5, whichever is greater,

where

$b_l$  = size of rectangular, or equivalent rectangular, column, capital, or bracket, measured in the direction of the span for which moments are being determined

$b_t$  = size of rectangular, or equivalent rectangular, column, capital, or bracket, measured transverse to the direction of the span for which moments are being determined

- (d) Negative moment strength at any section in the column strip along the member length shall not be less than one quarter of the maximum moment strength provided at the face of either support. Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.
- (e) Positive moment strength at any point in the column strip shall be not less than one third of the maximum negative moment strength provided at the face of either support, nor less than half of the maximum positive moment strength provided in the span.
- (f) At discontinuous edges of the slab, all top and bottom reinforcement at a support shall be capable of developing its yield strength at the face of the support.

#### 14.5.4 Columns

At each end of the clear height of a column within a storey, the longitudinal reinforcement shall be restrained by closed ties for a distance from the end equal to the greater of the maximum dimension of the column cross-section, or one sixth of the least clear height between consecutive flexural members framing into it.

The spacing of the closed fitments shall not be greater than required by Clauses 10.7.3 and 10.7.4 and the shear requirements of Clause 14.5.2.2(b), (c) and (d) and the smallest of the following limits:

- (a) 8 times the diameter of the smallest longitudinal bar enclosed.
- (b) 24 times the diameter of the fitment bar.
- (c) One-half of the smallest cross-sectional dimension of the column.
- (d) 300 mm.

The cross-sectional area of the fitments shall be sufficient to satisfy the shear requirements for the column but not less than that required by Clauses 10.7.3 and 10.7.4, except that, where a slab or beams exist on all four sides of the column, the cross-sectional area may be reduced by half this value for the depth of the shallowest slab or beam.

Where

$$P_u > \phi 0.3 A_g f'_c \text{ or } f'_c > 65 \text{ MPa}$$

each longitudinal bar shall be restrained by a closed fitment.

#### 14.5.5 Column joints

Joints between columns and flexural members framing into them shall be confined by closed ties throughout the depth of the joint.

The spacing of the closed ties shall be not greater than required by Clause 14.5.4, except that the cross-sectional area may be reduced to half this value for the depth of the shallowest of those members framing into the column from at least two directions at right angles.

#### 14.5.6 Robustness and structural integrity

Robustness and structural integrity should be checked in accordance with Clause 2.1.3 and the following:

- (a) Stairs and ramps are to be designed to be serviceable under the maximum design earthquake actions.
- (b) In structures where a moment resisting frame is being relied on to act as part of the lateral support system in an earthquake, to reduce the likelihood of column failure under earthquake loading and to promote a strong column/weak beam strength hierarchy at the joint  $M_{nc}$  is required to be  $> 6/5 M_{nb}$ . That is, the sum of the nominal flexural strengths of columns (calculated in conjunction with the applied axial loads due to earthquake design actions) framing into the joint (evaluated at the faces of the joint) shall be greater than or equal to 1.2 times the sum of nominal flexural strengths of the beams framing into the joint (evaluated at the faces of the joint).

If columns cannot conform with these requirements, then they cannot be considered as part of the lateral support system in an earthquake.

## 14.6 LIMITED DUCTILE STRUCTURAL WALLS

### 14.6.1 General

Structural walls shall be provided with boundary elements in accordance with Clause 14.6.2. For structures of not more than four storeys above their structural base and where boundary elements are required, an integrally cast column, or additional edge reinforcement consisting of two N16 or four N12 bars, shall be deemed to satisfy this requirement.

### 14.6.2 Boundary elements

In any storey, boundary elements shall be provided at discontinuous edges of structural walls and around openings through them if—

- (a) the vertical reinforcement within the storey height is not laterally restrained in accordance with Clause 10.7.4; and
- (b) the calculated extreme fibre compressive stress in the wall exceeds  $0.15 f'_c$ .

The stress referred to in Item (b) shall be calculated using the design action effects for the strength limit state, a linear-elastic strength model and the gross cross-section properties of the wall.

Where boundary elements are required, the horizontal cross-section of the wall shall be treated as an I-beam in which the boundary elements are the flanges and the section of wall between them is the web. Restraint of the longitudinal reinforcement in boundary elements shall conform with Clause 10.7.4 of this Standard or, if the extreme fibre compressive stress calculated as above exceeds  $0.2 f'_c$ , with Clause 14.5.4 of this Standard.

### 14.6.3 Confinement of the wall core

For structural walls where  $f'_c > 50$  MPa confinement of the wall core shall be provided throughout by fitments in accordance with Clause 14.5.4.

### 14.6.4 Mean concrete strength

The mean value of the 28 day cylinder strength as tested shall be less than  $1.4 f'_c$ .

PUBLIC COMMENT NOTE: The committee is particularly interested in hearing from the industry on the limit on concrete over-strength.

### 14.6.5 Aspect ratio

The total wall height to length ratio shall be greater than or equal to two. Short or squat walls with aspect ratios less than two shall be designed as non-ductile structural walls using the strut and tie methods in Section 12 of this Standard.

### 14.6.6 Effective height to thickness

The ratio of the effective height to thickness shall be equal to or less than 20.

### 14.6.7 In-plane shear

In determining the required shear strength of a structural wall and supporting foundations under earthquake design actions, allowance for the effects of flexural over-strength and dynamic amplification shall be included. This requirement is satisfied when the wall shear capacity ( $\phi V_u$ ) at a cross-section is not less than:

$$\frac{\phi V_u \geq 1.6 M_u V^*}{M^*} \quad \dots 14.12.7$$

### 14.6.8 Reinforcement

The vertical reinforcement ratio ( $\rho_{wv}$ ) shall not be less than—

- (a)  $0.7(f'_c)^{0.5}/f_{sy}$  in the outermost  $0.15L_w$  region at each end of the wall; and
- (b) 50% of this value in the centre region ( $0.7L_w$ ) of the wall.
- (c) This minimum reinforcement ratio shall extend vertically from the base of the wall for a distance of not less than—
  - (i) the greater of twice the wall length ( $2L_w$ ); or
  - (ii) the height of the lower two storeys.

For interconnected groups of structural walls the longest wall of the group shall be used in determining the height over which the minimum vertical reinforcement ratio shall apply.

The minimum vertical reinforcement ratio may be reduced by 10% per floor above the region of applicability to a minimum value of 0.0025.

The vertical reinforcement ratio ( $\rho_{wv}$ ) shall not exceed  $16/f_{sy}$  except in regions where lapped splices in boundary elements are unavoidable, in which case the total ratio including the area of splices shall not exceed  $21/f_{sy}$ .

The horizontal reinforcement ratio ( $\rho_{wh}$ ) shall not be not less than 0.0025.

Vertical and horizontal reinforcement shall be provided on both wall faces and shall be divided equally between the two wall faces and detailed as follows:

- (A) The ends of all horizontal lapped bars shall be provided with  $135^\circ$  hooks and lapped with a full strength splice.
- (B) At the ends of walls, where boundary elements exist, horizontal bars shall be provided with hooks and anchored into the confined concrete core of the boundary element. Where no boundary element is required, ends of horizontal bars shall be terminated at the end of the wall with full tension laps with U bars of the same diameter.
- (C) Ductility Class L reinforcement shall not be used as structural reinforcement.

Wall reinforcement terminating in footings, columns, slabs, or beams shall be anchored to develop the yield stress in the reinforcement at the junction of the wall with the terminating member.

## 14.7 MODERATELY DUCTILE STRUCTURAL WALLS

### 14.7.1 General

Moderately ductile structural walls shall conform with Clauses 14.6 except Clause 14.6.6. In Clause 14.6.2 all vertical reinforcement in boundary elements shall be restrained in accordance with Clause 14.5.4 irrespective of the calculated compressive stress.

### 14.7.2 Effective height to thickness

The ratio of the effective height to thickness shall be equal to or less than 16.

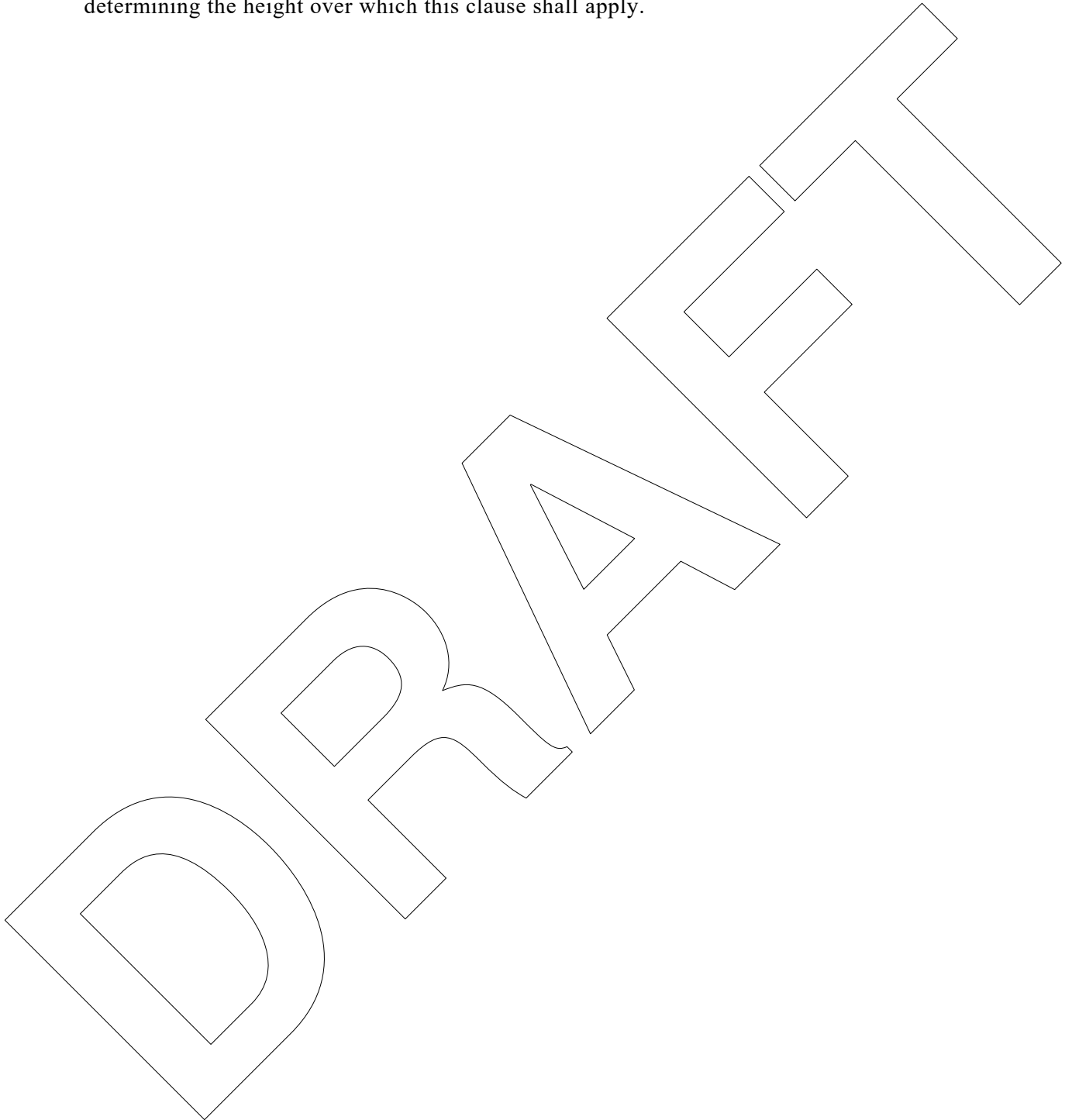
### 14.7.3 Vertical reinforcement laps

In the region extending for a vertically from the base of the wall immediately above the footing to a height equal to—

- (a) the greater of twice the wall length; and
- (b) the height of the lower two storeys.

Laps or mechanical splices to vertical reinforcement shall be evenly staggered along the length of the wall such that, at every horizontal section, not more than 50% of the vertical reinforcement is lapped or spliced. Staggered lapped splices shall conform with the arrangement shown in Figure 13.2.2(ii).

For interconnected groups of structural walls the longest wall of the group shall be used in determining the height over which this clause shall apply.



## SECTION 15 DIAPHRAGMS

### 15.1 GENERAL

This section applies to the design of both conventionally reinforced and prestressed diaphragms, including:

- (a) Cast in situ slabs, with or without beams.
- (b) Cast in situ topping slabs on precast elements.

Diaphragms shall be considered as part of the primary structure with identifiable internal load paths to transfer the required actions between the diaphragms and associated connected elements to the elements of the lateral force resisting structural system.

Elements within diaphragms and connections between diaphragms and elements of the lateral force resisting structural system shall be capable of accommodating both the imposed displacement and force demands imposed on them.

### 15.2 DESIGN ACTIONS

#### 15.2.1 General design actions

Design actions for diaphragms shall include actions arising from all of the following loadings:

- (a) Associated permanent (dead load) and imposed (live load) actions on the floor.
- (b) Diaphragm in-plane forces due to lateral loads acting on the building, including floor design accelerations and associated inertia loads under seismic actions (see Clause 14.4.5).
- (c) Force transfer between the lateral force resisting elements interconnected by the diaphragm.
- (d) Interaction with elements vertically or near vertically supporting the diaphragm.

Load combinations for diaphragms shall be as provided in AS 1170.0. Where required, it shall be assumed that in-plane and out-of-plane loadings occur concurrently on the diaphragm.

Forces induced in the diaphragm by the lateral load resisting system, including at discontinuities in the vertical stiffness of the building and plan irregularity between stories, shall be explicitly considered in design.

#### 15.2.2 Analysis procedure

##### 15.2.2.1 General

Rational analysis shall be used to establish that there is adequate in-plane flexural and shear strength at the ultimate limit state. The analysis may be based on a strut and tie model of the diaphragm in accordance with Section 7. The effect of openings and penetrations shall be considered in analysis.

The effect of cracking and joint opening due to distributed reinforcement throughout the diaphragm resisting in-plane tension forces shall be explicitly determined in analysis of the diaphragm. Alternatively, these effects may be ignored if the diaphragm reinforcement resisting in-plane moments is distributed within one quarter of the width of the diaphragm from the tension edge of the diaphragm.



### 15.2.2.2 Stiffness

Analysis for the internal forces transmitted between diaphragms and their supports shall account for the stiffness of the chosen load path.

Where the maximum lateral deformation of the diaphragm itself is calculated to be more than half the average inter-story deflection of the vertical lateral force resisting elements of the associated storey the diaphragm shall be considered to be flexible, and the displacement of the diaphragm shall be considered in the analysis of the structure.

## 15.3 CAST IN-PLACE TOPPINGS

A topping slab cast in place on a prefabricated concrete floor or roof shall be permitted to be used as a structural diaphragm, provided that—

- (a) the cast in-place topping acting alone is proportioned and detailed to resist the design diaphragm forces, and the topping has a minimum thickness of 75 mm; or
- (b) sufficient reinforcement is provided in accordance with Clause 8.4, is provided to ensure the topping slab and precast elements act as a composite unit; and
  - (i) the surface of the previously hardened concrete on which the topping slab is placed is clean, free of laitance, and intentionally roughened; and
  - (ii) the topping slab has a minimum thickness of 65 mm excluding the thickness of the precast element at all locations within the diaphragm.

## 15.4 DIAPHRAGM REINFORCEMENT

### 15.4.1 General

Where concentrated actions are developed in the diaphragm (such as within collector elements, and at the perimeter of the diaphragm under in-plane bending) Ductility Class N reinforcement or bonded post-tensioning shall be used to resist diaphragm forces.

Reinforcement designed to resist diaphragm in-plane forces shall be in addition to reinforcement provided to resist other load effects, except reinforcement provided to resist shrinkage and temperature effects shall be permitted to also resist diaphragm forces.

### 15.4.2 Minimum reinforcement

Reinforcement shall be placed in both orthogonal directions, with minimum reinforcement ratios provided in accordance with Clause 9.4.3. The maximum spacing of reinforcement shall not exceed the requirements of Clause 9.4.1.

### 15.4.3 Effect of development and laps

The effect of development and laps shall be considered in the capacity of the reinforcement at all locations within the diaphragm in accordance with Section 13. Stresses induced in reinforcement from vertical actions shall be included when calculating reinforcement requirements.

### 15.4.4 Collectors

Collectors shall be provided to transfer concentrated diaphragm forces between the diaphragm and the vertical elements of the lateral load resisting system.

Collectors shall extend from the vertical element of the lateral load resisting system a minimum of one development length beyond the point at which the analysis determines the collector element is no longer in tension.

Collectors shall extend along the vertical element for the larger of either—

- (a) the length required to develop the reinforcement in tension, noting this length shall be increased if the collector is assumed to lap with the horizontal reinforcement in the vertical element; or
- (b) the length required to transmit the design force from the collector into the vertical element through either shear-friction at the face of the wall, mechanical connectors or tension laps with the horizontal reinforcement in the vertical element.

Splices shall only be permitted in collector elements where the splice length is sufficient to develop the full yield strength of the reinforcement in accordance with Section 13.

#### **15.4.5 Construction joints**

Transfer of diaphragm forces across construction joints shall only be assumed where sufficient fully developed reinforcement is placed across the joint to transmit the calculated diaphragm forces.

Mechanical connectors may be used to transfer diaphragm forces across joints, or between precast and in situ elements in a precast topping, provided the connectors are designed to resist the required tension under the anticipated joint opening.

## SECTION 16 STEEL FIBRE REINFORCED CONCRETE

### 16.1 GENERAL

This section shall apply where steel fibres are used to improve the performance and capacity of reinforced and prestressed concrete structures.

The design of steel fibre reinforced concrete (SFRC) at both the ultimate and serviceability limit states shall be based on the stress ( $\sigma$ )-strain ( $\epsilon$ ) relationships for SFRC as specified in Clause 16.3.3.

NOTE: When using brittle fibres that rely on a fibre pullout failure mechanism to obtain member ductility, care is required for cases where a significant number of fibres fracture or where fibres result in local crushing of the concrete due to the local forces imposed on the matrix by the fibres. Fibre fracture may occur where the bond between the fibre and the matrix is high, and this is more likely in a high strength concrete combined with fibres of high bond capacity and of lower strength steels. In assessing the suitability of a given fibre for a given application, bond-strength gain in time and the resulting potential loss of ductility should be considered.

Steel fibres shall not be relied upon at construction joints for either serviceability or strength requirements.

Design procedures in this section are for steel-fibre-reinforced concrete with a softening classification only (see Figure 16.3.3.1). Hardening SFRC and the use of synthetic fibres is beyond the scope of this Standard.

### 16.2 DEFINITIONS

For the purposes of this section the definitions below apply.

#### 16.2.1 Crack mouth opening displacement (CMOD)

The width of a crack measured at its mouth in a flexural tensile test undertaken in accordance with EN 14651.

#### 16.2.2 Crack opening displacement (COD)

The width of a single localized crack, taken as an average on four sides, for a direct tensile test on a strain softening SFRC dog-bone shaped specimen in accordance with Appendix C.

#### 16.2.3 Hardening behaviour

A material that displays an enhanced tensile strength with increasing crack widths beyond that measured at the point of initial cracking of the cementitious matrix.

#### 16.2.4 Softening behaviour

A material that exhibits a loss of tensile strength upon cracking.

#### 16.2.5 Steel fibre reinforced concrete (SFRC)

A mixture of concrete and steel fibres.

#### 16.2.6 Target dosage

The specified quantity of fibres in kilograms per cubic metre of concrete (kg/m<sup>3</sup>).

## 16.3 PROPERTIES OF SFRC

### 16.3.1 General

SFRC shall be classified in terms of both its characteristic compressive (cylinder) strength ( $f'_c$ ) (see Clause 16.3.2) and its characteristic residual tensile strength ( $f'_{1.5}$ ) (see Clause 16.3.3.3).

### 16.3.2 Compressive strength

The characteristic compressive strength of SFRC at 28 days ( $f'_c$ ) shall be determined in accordance with Clause 3.1.1.1.

In the absence of more accurate data, the mean value of the in situ compressive strength ( $f_{cmi}$ ) shall be taken as 90% of the mean value of the cylinder strength ( $f_{cm}$ ).

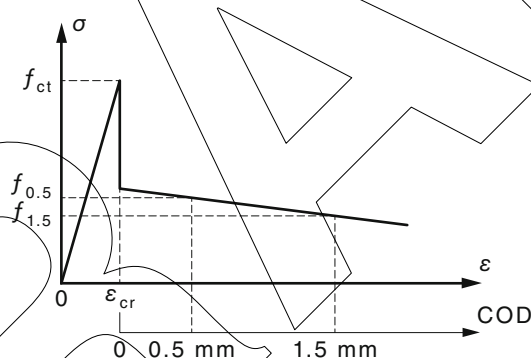
### 16.3.3 Tensile properties

#### 16.3.3.1 Classification

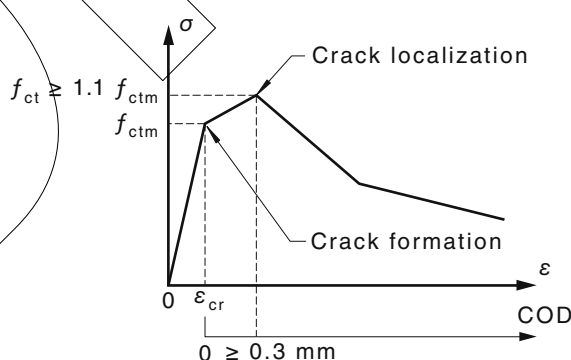
SFRC shall be classified as either softening or hardening as shown in Figure 16.3.3.1.

A hardening SFRC is outside the scope of this section.

NOTE: A hardening material is defined as one with a tensile strength equal to or greater than 1.1 times the strength of the matrix without fibres and taken at a crack opening displacement (COD) of equal to or greater than 0.3 mm (refer to AS 5100.5 Supp. 1).



(a) Strain softening SFRC



(b) Strain hardening SFRC

FIGURE 16.3.3.1 CLASSIFICATION OF SFRC

### 16.3.3.2 Matrix tensile strength

The matrix tensile strength of the softening SFRC ( $f_{ct}$ ) shall be obtained using direct or indirect tensile testing, in accordance with Clause 3.1.1.3.

When only the characteristic compressive (cylinder) strength ( $f'_c$ ) has been determined, the mean and characteristic flexural tensile strength of SFRC shall be calculated in accordance with Clause 3.1.1.3.

### 16.3.3.3 Residual tensile strength

The standard characteristic residual tensile strength grades ( $f'_{1.5}$ ) are 0.4 MPa, 0.6 MPa, 0.8 MPa, 1.2 MPa, 1.6 MPa and 2.0 MPa.

The characteristic residual tensile strengths of concrete at 28 days ( $f'_{1.5}$ ) shall be determined statistically from tests carried out in accordance with Clause 16.3.3.4 or Clause 16.3.3.5.

Higher residual tensile strength grades are permitted if supported by direct tensile testing undertaken in accordance with Clause 16.3.3.4.

NOTE: Where similar SFRC mixes have been tested in accordance with Clause 16.3.3.5 and the difference in fibre content is not more than 20 kg/m<sup>3</sup>, the residual tensile strengths that fall between these limits can be obtained by linear interpolation.

### 16.3.3.4 Determination of strength by direct testing

The characteristic residual tensile strengths  $f'_{0.5}$  and  $f'_{1.5}$  shall be obtained using direct tensile tests as specified in Appendix C.

Alternatively, where matched direct and indirect testing has been undertaken in accordance with Clause 16.3.3.6 for similar SFRC mixtures, the characteristic residual tensile may be determined as—

$$f'_{0.5} = k_{R,2} f'_{R,2} \quad \dots 16.3.3.4(1)$$

$$f'_{1.5} = k_{R,4} f'_{R,4} \quad \dots 16.3.3.4(2)$$

where  $f'_{R,2}$  and  $f'_{R,4}$  are determined in accordance with Clause 16.3.3.8 and calculated statistically, and the factors  $k_{R,2}$  and  $k_{R,4}$  determined from Clause 16.3.3.6.

For the purposes of this clause, similar SFRC mixtures are defined as having the same—

- (a) fibre type and content;
- (b) water to cementitious material ratio;
- (c) maximum aggregate particle size;
- (d) aggregates of same geological origin; and
- (e) compressive strength ( $f'_c$ ).

The characteristic value for the residual tensile strengths shall be determined as follows:

- (i) The strength of the population may be treated as normally distributed and the characteristic strength determined in accordance with ISO 12491.
- (ii) A confidence level of 75% shall be used such that 95% of the population exceeds the characteristic value.
- (iii) The standard deviation of the population shall be determined from the sample but shall not be taken as less than that calculated for a COV = 0.25.
- (iv) The minimum number of test specimens is six.

The residual tensile stress for other CODs may be obtained or linear interpolation or extrapolation from a straight line connecting  $f'_{0.5}$  and  $f'_{1.5}$  but shall not be taken as greater than  $f'_{0.5}$  nor less than zero.

#### 16.3.3.5 Determination of strength by indirect testing

The characteristic residual tensile strengths at 0.5 mm ( $f'_{0.5}$ ) and 1.5 mm COD ( $f'_{1.5}$ ) may be obtained using indirect tests as specified in Clause 16.3.3.8 and calculated as follows:

$$f'_{0.5} = k_{3Db} (-0.04 f'_{R,4} + 0.37 f'_{R,2}) \leq k_{3Db} 0.36 \sqrt{f'_c} \quad \dots 16.3.3.5(1)$$

$$f'_{1.5} = k_{3Db} (0.4 f'_{R,4} - 0.07 f'_{R,2}) \leq k_{3Db} 0.36 \sqrt{f'_c} \quad \dots 16.3.3.5(2)$$

where the three-dimensional orientation factor is—

$$k_{3Db} = \frac{1}{1 + 0.19 l_f / b} \leq 1$$

and  $b$  is the prism sectional width.

The residual tensile stress for other CODs may be obtained or linear interpolation or extrapolation of Equations 16.3.3.5(1) and 16.3.3.5(2) but shall not be taken as greater than  $f'_{0.5}$  nor less than zero.

NOTE: For fibres not greater than 70 mm in length, the three-dimensional orientation factor may conservatively be taken as  $k_{3Db} = 0.92$ .

#### 16.3.3.6 Residual tensile strength—Residual flexural strength relationship

The relationship between residual tensile strength and the residual flexural strength shall be obtained by matched testing using the same SFRC mixture. Residual tensile strength specimens shall be prepared and tested in accordance with Clause 16.3.3.7. Residual flexural tensile tests shall be in accordance with EN 14651 and—

- (a) the concrete mix shall be batched to ensure a uniform distribution of fibres, the SFRC shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres and, the SFRC shall be compacted using lightly applied external vibration; and
- (b) a minimum of six specimens shall be tested.

The reference factors  $k_{R,2}$  and  $k_{R,4}$  shall be determined as—

$$k_{R,2} = f'_{0.5m} / f_{R,2m} \quad \dots 16.3.3.6(1)$$

$$k_{R,4} = f'_{1.5m} / f_{R,4m} \quad \dots 16.3.3.6(2)$$

where

$f'_{0.5m}$  = mean residual tensile strengths corresponding to a COD of 0.5 mm, determined in accordance with Appendix C

$f'_{1.5m}$  = mean residual tensile strengths corresponding to a COD of 1.5 mm, determined in accordance with Appendix C

$f_{R,2m}$  = mean residual flexural tensile strengths corresponding to a CMOD of 1.5 mm, determined in accordance with Clause 16.3.3.7

$f_{R,4m}$  = mean residual flexural tensile strengths corresponding to a CMOD of 3.5 mm, determined in accordance with Clause 16.3.3.7

### 16.3.3.7 Residual flexural tensile strength

The residual flexural tensile strength ( $f_{R,j}$ ) shall be determined from three point notched bending tests on 150 mm square section prisms. The notch depth shall be 25 mm and the test conducted in accordance with EN 14651. The force  $F$  shall be plotted against the crack mouth opening displacement (CMOD), as shown in Figure 16.3.3.7, and the residual flexural stress calculated as—

$$f_{R,j} = \frac{3F_{R,j}L}{2bh_{sp}^2}$$

where

$b$  = width of the specimen in millimetres

$h_{sp}$  = distance between tip of the notch and top of cross section in millimetres

$L$  = span in millimetres

$F_{R,j}$  = load recorded at CMOD<sub>j</sub> (see Figure 16.3.3.8)

NOTE: Testing should be undertaken in a laboratory accredited by the National Association of Testing Laboratories (NATA).

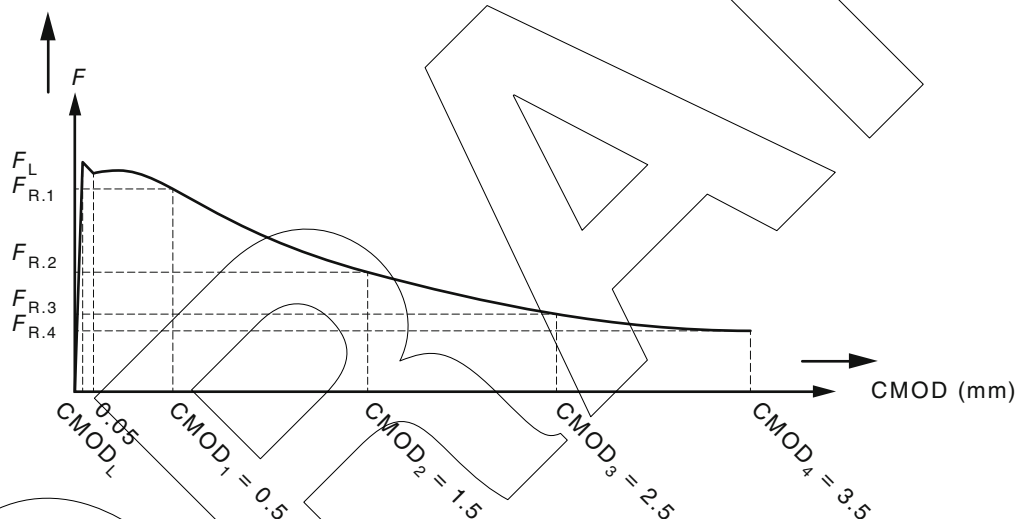


FIGURE 16.3.3.7 LOAD VERSUS CMOD FOR RESIDUAL FLEXURAL TENSION

The characteristic value for the residual flexural tensile strength shall be determined as follows:

- The strength of the population may be treated as normally distributed and the characteristic strength determined in accordance with ISO 12491.
- A confidence level of 75% shall be used such that 95% of the population exceeds the characteristic value.
- The standard deviation of the population shall be determined from the sample but shall not be taken as less than that calculated for a COV = 0.25.
- The minimum number of test specimens is six.

### 16.3.3.8 Minimum fibres dosage

A dosage of fibres shall be provided such that the following minimum material performance criterion is achieved—

- (a)  $f_{R,1m}/f_{Lm} > 0.4$ ; and
- (b)  $f_{R,3m}/f_{R,1m} > 0.5$

where

$f_{Lm}$  = mean flexural tensile strength corresponding to the limit of proportionality, determined in accordance with Clause 16.3.3.8 with  $F_{R,j}$  substituted by  $F_L$

$f_{R,1m}$  = mean residual flexural tensile strength corresponding to a CMOD of 0.5 mm, determined in accordance with Clause 16.3.3.7

$f_{R,3m}$  = mean residual flexural tensile strength corresponding to a CMOD of 2.5 mm, determined in accordance with Clause 16.3.3.7

Alternatively, the dosage of fibres shall not be less than the greater of—

- (i)  $8.6\gamma_s (d_f/l_f)^2$ ; and
- (ii)  $20 \text{ kg/m}^3$

where  $\gamma_s$  is the mass density of steel and taken as  $7850 \text{ kg/m}^3$ .

### 16.3.4 Modulus of elasticity

The mean modulus of elasticity at the appropriate age ( $E_{cj}$ ) shall be determined in accordance with Clause 3.1.2.

## 16.4 DESIGN OF SFRC MEMBERS CONTAINING REINFORCEMENT OR TENDONS

### 16.4.1 General

This clause applies to reinforced and prestressed beams that include steel fibres subjected to any combination of shear force, bending moment and axial force.

NOTE: This clause does not apply when torsion acts in conjunction with shear or to non-flexural members.

### 16.4.2 Strength in bending and combined bending and axial force

Calculations for strength of cross-sections in bending shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following assumptions:

- (a) Plane sections normal to the axis shall remain plane after bending.
- (b) The stress in the SFRC in that part of the cross-section in tension shall be taken to be  $k_g f'_{1.5}$ , where  $f'_{1.5}$  is the characteristic residual tensile stress determined in accordance with Clause 16.3.3.3 and  $k_g$  is a factor to take account of member size.
- (c) The distribution of compressive stress shall be determined from a stress-strain relationship for the concrete in accordance with Clause 3.1.4.

The member size factor is—

$$k_g = 1 + \frac{0.0067 A_{ctu}}{A_o} \quad \dots 16.4.2$$

where

$A_{ctu}$  = area of concrete within the tensile zone ( $\text{mm}^2$ ) at ultimate

$A_o$  = a reference area taken as  $A_o = 15\,600 \text{ mm}^2$ .

The strength of a section in bending, or in combined bending and axial force, shall be determined using rectangular stress blocks for the concrete in compression and concrete in tension, as shown in Figure 16.4.2.



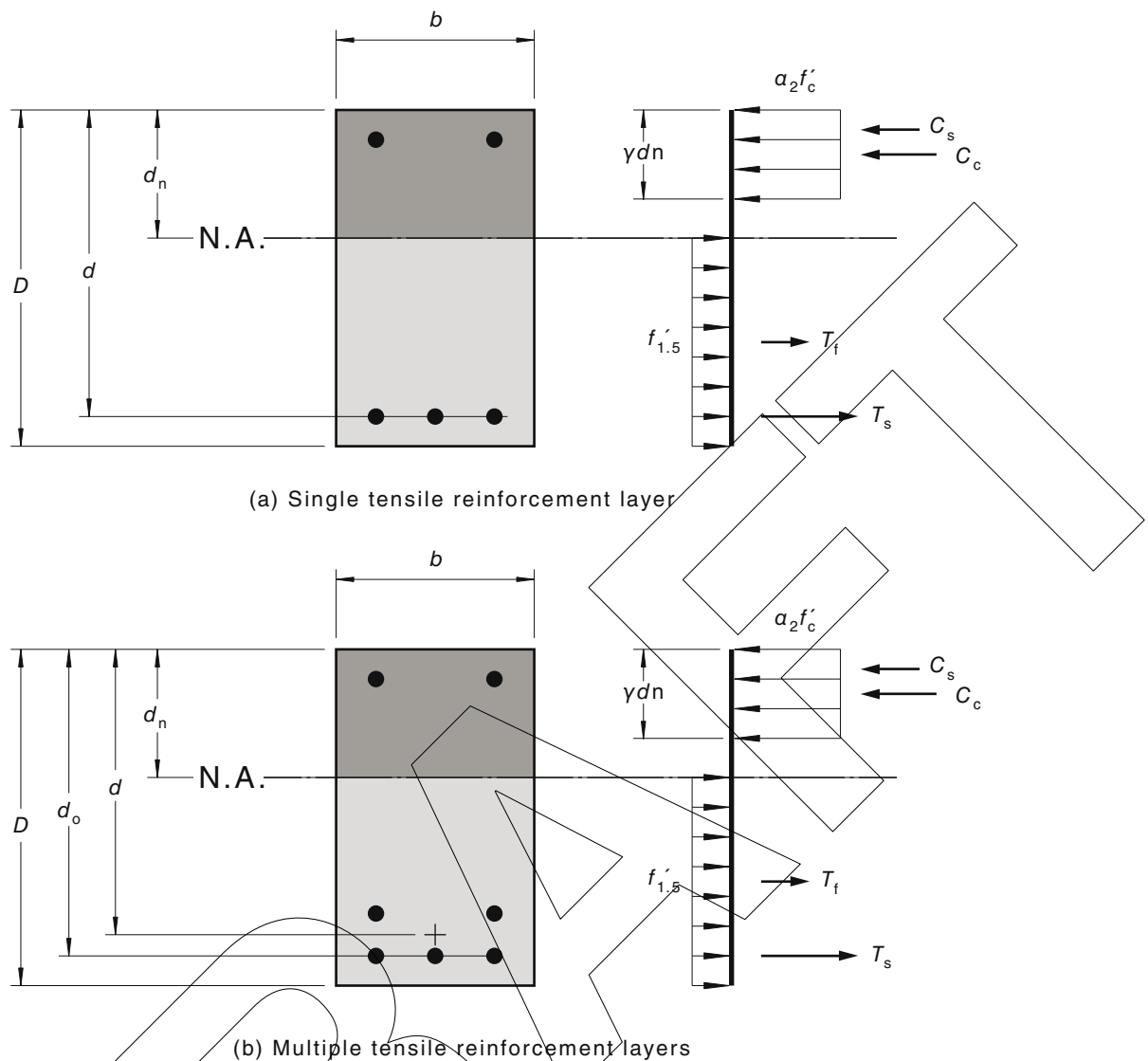


FIGURE 16.4.2 STRESS BLOCK AND FORCES ON REINFORCED SFRC SECTION

### 16.4.3 Minimum reinforcement requirements for bending

The minimum tensile reinforcement shall be not less than that determined using the principles of Clause 8.1.6, excluding fibres.

The minimum reinforcement may be reduced by not greater than 20% where it is shown by calculation that the capacity of the beam is not less than  $1.2M_{cr}$ .

#### NOTES:

- 1 This clause does not apply to foundation or pavement slabs fully or partially supported on soil.
- 2 For statically indeterminate members, this clause may be waived provided that it can be shown that the collapse load of the member is not reduced or that cracking would not cause a sudden collapse.

### 16.4.4 Strength of beams in shear

#### 16.4.4.1 Design shear strength of a beam

The design shear strength of a beam shall be taken as  $\phi V_u$ , where—

$$V_u = V_{uc} + V_{uf} + V_{us} \quad \dots 16.4.4.1$$

where  $V_{uc}$  and  $V_{us}$  are determined from Clause 8.2.3 and  $V_{uf}$  is determined from either Clause 16.4.4.2.1 or Clause 16.4.4.2.2.

Notwithstanding Equation 16.4.4.1, the fibres component to the ultimate shear strength of a beam,  $V_{uf}$ , shall not exceed the greater of  $V_{uc}$  and that determined by Clause 16.4.4.3 with  $V_{us}$  taken as zero.

#### 16.4.4.2 Contribution to shear strength by steel fibres

##### 16.4.4.2.1 Design by refined calculation

The contribution of the fibres to the ultimate shear strength ( $V_{uf}$ ) of an SFRC beam shall be calculated from the following equation:

$$V_{uf} = K_S k_g d_v b_v f'_w \cot \theta_v \quad \dots 16.4.4.2.1(1)$$

where

$K_S$  = fibre orientation casting bias factor and is taken as 0.64

$k_g$  = member size factor determined by Equation 16.4.2, where area of concrete within the tensile zone (mm<sup>2</sup>) at ultimate is calculated as  $A_{ctu} = b_v d_v \cot \theta_v$

$f'_w$  = the characteristic residual tensile strength of SFRC corresponding COD  $w$

$\theta_v$  = angle between the axis of the concrete compression strut and the longitudinal axis of the member calculated in accordance with Clause 8.2.4.2

$d_v$  = the effective shear depth of the member calculated in accordance with Clause 8.2.1.9

The characteristic residual tensile strength  $f'_w$  shall be determined in accordance with either Clauses 16.3.3.4, 16.3.3.5 or 16.3.3.6 where the crack width is determined as—

$$w = (0.2 + 1000 \varepsilon_x) \left( \frac{1000 + k_{dg} z}{1300} \right) \frac{1}{\cos \theta} \geq 0.125 \text{ mm} \quad \dots 16.4.4.2.1(2)$$

where  $\varepsilon_x$  is determined from Clause 8.2.4.3.

Alternatively for beams less than 1000 mm in depth,  $f'_w$  in Equation 16.4.4.2.1(1) may be taken as equal to  $f'_{1.5}$ .

The concrete and steel components shall be determined in accordance with Clauses 8.2.3 with  $k_v$  determined from Clause 8.2.4.2.

NOTE: The components  $V_{uc}$ ,  $V_{us}$  and  $V_{uf}$  are coupled through the strain parameter  $\varepsilon_x$  and the equations are solved iteratively.

##### 16.4.4.2.2 Design by simplified calculation

For non-prestressed components not subjected to axial tension, and provided—

- (a) the specified yield strength of the longitudinal reinforcement does not exceed 500 MPa;
- (b) the design concrete strength does not exceed 65 MPa;
- (c) the size of maximum aggregate particle is not less than 10 mm; and
- (d) the length of the fibres does not exceed 70 mm.

The angle of inclination of the compression strut ( $\theta_v$ ) shall be taken as 36°, the value of  $k_v$  shall be determined from Clause 8.2.4.6 and the contribution of the fibres to the ultimate shear strength ( $V_{uf}$ ) determined from Equation 16.4.4.2.1(1) with  $f'_w$  taken as equal to  $f'_{1.5}$ .

### 16.4.4.3 Minimum shear reinforcement

The minimum contribution from the total of the transverse steel reinforcement and fibres shall satisfy the following:

$$(V_{us} + V_{uf})_{\min} \geq 0.1b_v d_o \sqrt{f'_c} \text{ and} \quad \dots 16.4.4.3$$

$$\geq 0.6b_v d_o$$

### 16.4.5 Design with strut and tie models

Steel fibres may be used in combination with bar reinforcement to resist tension in strut-and-tie models provided that—

- (a) at service loads the crack width is demonstrated to be not greater than 0.5 mm; and
- (b) at the strength limit state fibres do not contribute to more than 30% of the total calculated tension, with bar reinforcement supplied for the remainder.

NOTE: A higher level of crack control may be required for other limit state conditions, e.g. durability.

### 16.4.6 Fatigue

Steel fibres shall not be included in calculations for fatigue resistance unless demonstrated by testing.

### 16.4.7 Design for serviceability limit states

#### 16.4.7.1 General

When an SRFC cross-section is uncracked, the full cross-section shall be assumed to be active and both concrete and steel assumed to be elastic in tension as well as in compression.

When an SRFC cross-section is cracked, the SFRC shall be assumed to be elastic in compression, and capable of sustaining a tensile stress equal to  $1.1f'_{1.5}$ .

NOTE: Where the value of  $f'_{0.5}$  is specified, the value of  $1.1f'_{1.5}$  in Clause 16.4.7.1, Clause 16.4.7.4 and in Figure 16.4.7.3.2 may be replaced with  $1.0f'_{0.5}$ .

#### 16.4.7.2 Stress limits

##### 16.4.7.2.1 Concrete

The maximum compressive stress in the concrete at the serviceability limit states shall not exceed  $0.6f_{cm}(t)$ . Under permanent effect loading, the maximum compressive stress in the concrete shall not exceed  $0.4f_{cm}(t)$ .

NOTE: The satisfaction of limits on the concrete tensile stress at the serviceability limit state is not necessary if the member performance is satisfactory at the ultimate limit state.

##### 16.4.7.2.2 Reinforcing steel

To avoid any inelastic deformation that could lead to large, permanently open cracks, tensile stresses in the reinforcement at the serviceability limit states shall not exceed  $0.8f_{sy}$ .

#### 16.4.7.3 Deflection control

##### 16.4.7.3.1 General

The deflection of an SFRC member shall be calculated using the procedures outlined in Clauses 16.4.7.3.2 and 16.4.7.3.3. Allowance shall be made for the expected load history, the expected construction procedure and any anticipated deflections resulting from deformation of forms or settlement of props.

### 16.4.7.3.2 Short-term deflection

The short-term deflections due to external loads and prestressing, which occur immediately on their application, shall be calculated using the value of  $E_{cj}$  determined in accordance with Clause 3.1.2 and the value of the effective second moment of area of the member ( $I_{ef}$ ). The value of  $I_{ef}$  may be determined from the values of  $I_{ef}$  at nominated cross-sections as follows:

- (a) For a simply supported span, the value at mid-span.
- (b) In a continuous beam—
  - (i) for an interior span, half the mid-span value plus one quarter of each support value; or
  - (ii) for an end span, half the mid-span value plus half the value at the continuous support.
- (c) For a cantilever, the value at the support.

For the purpose of the above determinations, the value of  $I_{ef}$  at each of the cross-sections nominated in Items (a) to (c) above shall be obtained from the instantaneous curvature  $k_i = M_s^* / (E_{cj} I_{ef})$  calculated as the slope of the strain diagram in Figure 16.4.7.3.2(b) and obtained by satisfying the requirements for rotational and horizontal equilibrium of the stress distribution in Figure 16.4.7.3.2(c).

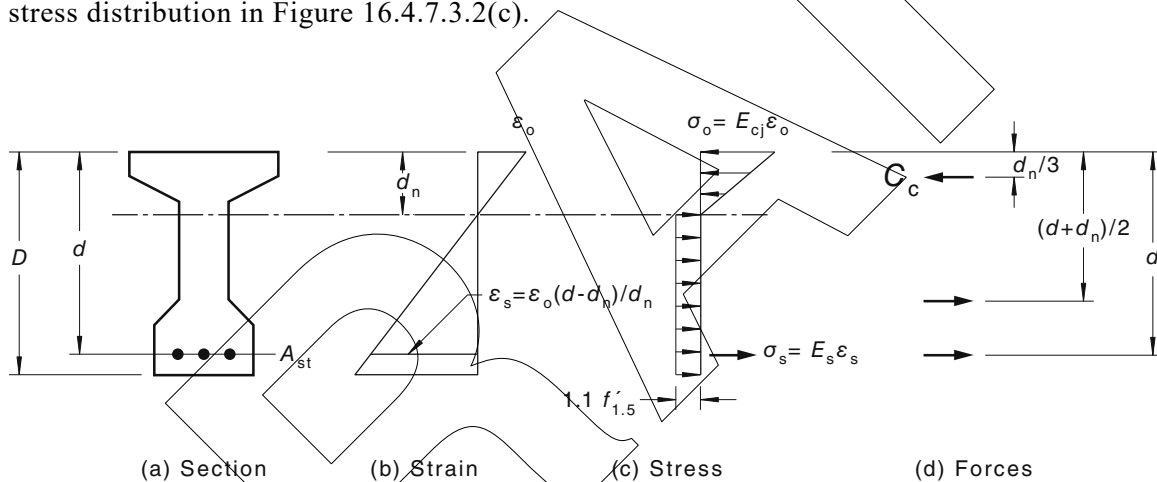


FIGURE 16.4.7.3.2 STRESS AND STRAIN DISTRIBUTION ON A CRACKED SECTION SUBJECTED TO APPLIED IN-SERVICE BENDING MOMENT ( $M_s^*$ )

### 16.4.7.3.3 Long-term deflection

For reinforced and prestressed SFRC flexural members, that part of the deflection that occurs after the short-term deflection shall be calculated as the sum of—

- (a) the shrinkage component of the long-term deflection, determined from the design shrinkage strain of concrete ( $\epsilon_{cs}$ ) (see Clause 3.1.7.1) and the principles of mechanics; and
- (b) the additional long-term creep deflections, determined from the design creep coefficient at any time ( $t$ ) of concrete ( $\phi_{cc}$ ) (see Clause 3.1.8.3) and the principles of mechanics.

#### 16.4.7.4 Flexural crack control

##### 16.4.7.4.1 Minimum reinforcement for crack control

In reinforced concrete members (or prestressed members where calculations indicate that flexural cracking will occur), the minimum amount of longitudinal reinforcement required to obtain controlled crack formation shall be—

$$A_{st,min} = k_c k_{sec} (f_{ct,ef} - 1.1 f'_{1.5}) \frac{A_{ct}}{f_{s,max}} \geq 0.0 \text{ (in mm}^2\text{)} \quad \dots 16.4.7.4.1$$

where

$A_{st,min}$  = the minimum area of reinforcement required within the tensile zone (mm<sup>2</sup>). If  $A_{st,min}$  is zero or negative, only steel fibres are necessary to control cracking

$A_{ct}$  = area of concrete within the tensile zone (mm<sup>2</sup>). The tensile zone is that part of the cross-section calculated to be in tension just before formation of the first crack

$f_{s,max}$  = maximum stress permitted in the reinforcement immediately after formation of the crack given in either Table 8.6.2(A) or Table 9.5.2(A) as appropriate

$f_{ct,ef}$  = tensile strength of the concrete effective at the time when the cracks may first be expected to occur (MPa). Values of  $f_{ct,ef}$  shall be obtained from  $0.6\sqrt{f_{cm}}$  but not less than 3.0 MPa, where  $f_{cm}$  is the mean concrete compressive strength at the time cracking is expected to occur

$k_c$  = a coefficient that takes account of the nature of the stress distribution within the section immediately prior to cracking. The relevant stress distribution is that resulting from the combined effects of loading and restrained imposed deformations

$$k_c = 1.0 \text{ for pure tension } (e = M/N = 0)$$

$$k_c = 0.6 \text{ for pure bending}$$

$k_{ses}$  = a coefficient which allows for the effect of non-uniform self-equilibrating stresses due to non-linear shrinkage or temperature profiles through the member depth. In the absence of more detailed analysis,  $k_{ses}$  may be taken as 1.0 when  $D \leq 300$  mm and 0.65 when  $D \geq 800$  mm. Intermediate values shall be obtained by interpolation

##### 16.4.7.4.2 Crack control by calculation of crack widths

The calculated maximum flexural crack width in a reinforced concrete beam or slab containing fibres in addition to conventional reinforcement shall be determined in accordance with Clause 8.6.2.3, except that the maximum crack spacing  $s_{r,max}$  determined using Equation 8.6.2.3(3) shall be multiplied by  $k_f$ , where—

$$k_f = \frac{(f_{ct,ef} - 1.1 f'_{1.5})}{f_{ct,ef}} \geq 0.25 \quad \dots 16.4.7.4.2$$

## 16.5 DURABILITY

The minimum concrete grade and cover for SFRC in exposure classifications A, B1 and B2 shall be as for concrete without fibres and shall apply to the steel reinforcement only.

In exposure classification C1 or C2 and U, steel fibres, if present, shall be disregarded in calculations of strength limit states.

## NOTES:

- 1 Steel fibres do not require concrete cover as specified for steel reinforcement in Clause 4.10.3.
- 2 SFRC may not be suitable in some exposure classification U environments.

**16.6 FIRE**

The structural performance of SFRC for fire shall be determined in accordance with Section 5 of the Standard.

The material properties for SFRC shall be as specified for concrete in Clause 5.4.1 except that the characteristic residual tensile stress of SFRC at elevated temperatures ( $f'_{1.5\theta}$ ) shall be either—

- (a) taken as  $f'_{1.5\theta} = k_{\theta 1} \times f'_{1.5}$  where  $k_{\theta 1}$  is given in Table 16.6; or
- (b) determined statistically from tests.

**TABLE 16.6**  
**ELEVATED TEMPERATURE COEFFICIENT FOR**  
**RESIDUAL TENSILE STRESS OF SFRC**

Temperature of SFRC °C	0	100	500	700	1200
$k_{\theta 1}$	1.0	1.0	0.6	0.1	0.0

NOTE: Linear interpolation between values.

**16.7 PRODUCTION OF SFRC****16.7.1 Fibres**

Steel fibres shall conform with ISO 13270:2013. The current 'Declaration of Performance' shall be supplied on request to the relevant authority.

NOTE: Steel fibres conforming with the EN 14889-1 "Fibres for concrete – Part 1: Steel fibres" conform with ISO 13270 and this clause.

**16.7.2 Mixing of fibres**

Fibres of the type and quantity specified shall be added in a controlled process ensuring that they are dispersed uniformly through the concrete mix. If added after the main mixing process, the concrete shall be remixed until the fibres have been completely dispersed throughout the batch.

## NOTES:

- 1 Balling of fibres should be avoided.
- 2 A record of fibre content should be recorded for each batch.

**16.7.3 Pre-construction testing of materials**

Pre-construction testing in the form of initial type tests shall be undertaken in accordance with Table 16.7.3.

New initial type tests shall be undertaken when any of the following occur:

- (a) The concrete composition changes.
- (b) At least one source material of the concrete changes.
- (c) The results of compressive tests undertaken to AS 1379 do not meet the specification.
- (d) At least once in a period of 12 months.

The pre-construction tests shall be performed with the same materials and equipment that will be used during production.

**TABLE 16.7.3**  
**PRE-CONSTRUCTION TESTS**

Material	Inspection/Test	Purpose
Steel fibres	Check delivery note	Verify that the delivery is in accordance with the order, and is shipped from the correct source
	Check ISO 13270 conformance	Verify that the fibres have the correct ISO or CE-label for use of steel fibres in concrete
Steel fibre content in the fresh concrete	Testing according to EN 14721 and Clause 16.7.5 on the basis of nine samples	Conformity with the target dosage. Verify homogeneous distribution of the steel fibres in the mix
Steel fibre concrete performance	Check limit of proportionality, and post-crack flexural strength in accordance to EN 14651 on a minimum of six beams	Verify that the performance is in accordance to the specification. The performance level serves as the reference for continuous production control

NOTE: The characteristic post-crack residual flexural strength shall be determined using the procedures given in Clause 16.3.3.7.

#### **16.7.4 Factory production control**

Factory production control in accordance with Table 16.7.4 shall be undertaken to establish a production process for the steel fibre reinforced concrete, and shall include the following:

- (a) Checking the correct concrete constituents are being used in production.
- (b) Steel fibres are checked against ISO or CE labelling.
- (c) Compressive tests undertaken to AS 1379.
- (d) Fibre content and distribution.

NOTE: The fibre type and dosage shall be shown on the delivery docket.

**TABLE 16.7.4**  
**ROUTINE PRODUCTION CONTROL**

Material	Inspection/Test	Purpose	Frequency
<b>Equipment inspection</b>			
Automatic dosing equipment for steel fibres	Visual inspection	Assure correct functioning of dosing device	Once per production day
	Control of accuracy	Avoid improper fibre dosage	On installation Periodically In case of doubt
<b>Materials inspection</b>			
Steel fibres	Check delivery note	Verify that the delivery is in accordance with the order, and is shipped from the correct source	Each delivery
	Check ISO or CE label	Verify that the fibres have the correct ISO or CE label which matches the corresponding Declaration of Conformity	Each delivery
	Visual control, measure fibre dimensions	Compare the fibre geometry with the fibres used for ITT	Each delivery
<b>Production process inspection</b>			
Fibre content in the fresh concrete	Testing according to EN 14721 and Clause 16.7.5	Conformity with the target dosage. Verify homogeneous distribution of the steel fibres in the mix	Beginning of each day and every /50 m <sup>3</sup> (manual dosing) /150 m <sup>3</sup> (auto dosing)
Concrete mix	Visual check	Correct mixing with correct fibre type and even fibre distribution without balling	Daily
<b>Finished product inspection</b>			
Fibre content-record	Record the quantity added	Check the content	Every batch
Steel fibre concrete performance	Check limit of proportionality, and post-crack flexural strength in accordance to EN 14651	Check performance level of the specification	At minimum of six beams over any 12 month period of production

### 16.7.5 Determining the steel fibre content

Steel fibre content shall be measured from samples taken from the production concrete and carried out in accordance with EN 14721.

In addition—

- a sample shall be taken from the batch of concrete at unloading from the first third, middle third and final third of the batch;
- each sample shall be a minimum of 10 litres;
- the sample container shall be filled in one continuous pour and where possible directly from the discharge chute; and



- (d) wash-out, magnetic separation or validated automated dosage equipment shall be used.

NOTE: Automated dosing equipment refers to fibre distribution determined by electronic means through continuous measuring of the concrete as it's discharged from the concrete truck. The method needs to have been demonstrated to be validated for the intended usage and supply conditions and environment.

The steel fibre content shall be deemed to conform provided the criteria in Table 16.7.5 are met.

**TABLE 16.7.5**  
**CRITERIA OF ACCEPTANCE FOR STEEL FIBRE DOSAGE**

Test control	Test control	Criteria
Each sample	Each partial test	$\geq 0.80$ of the specified target dosage
Average of three samples from the batch	Each test	$\geq 0.80$ of the specified target dosage
Continuous control: average of >3 tests	Continuous control: average of >3 tests	$\geq 0.90$ of the specified target dosage

#### **16.7.6 Sampling, testing and assessment for conformance of hardened SFRC**

When concrete is specified by parameters other than strength grade, the method of production control and, if required, project control shall be specified together with the relevant conformance criteria.

Methods of control and assessment shall provide a reliable operating characteristic curve so that—

- (a) concrete with a proportion defective of 0.05 has a probability of acceptance of at least 50%; and
- (b) concrete with a proportion defective of 0.30 has a probability of rejection of at least 98%.

## SECTION 17 MATERIAL AND CONSTRUCTION REQUIREMENTS

### 17.1 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR CONCRETE AND GROUT

#### 17.1.1 Materials and limitations on constituents

Materials for concrete and grout, and limitations on their chemical content, shall conform with the relevant requirements of AS 1379.

#### 17.1.2 Specification and manufacture of concrete

Concrete to which this Standard applies shall be—

- (a) specified as either normal-class or special-class and manufactured and supplied in accordance with AS 1379; and
- (b) handled, placed, compacted, finished and cured in accordance with this Standard, so that the hardened concrete will satisfy the design requirements for strength, serviceability, durability and other limit states.

Project assessment shall be specified for special-class concrete specified by strength grade and may be specified for normal-class concrete and other special-class concrete, all as defined in AS 1379.

#### 17.1.3 Handling, placing and compacting of concrete

Concrete shall be handled, placed and compacted so as to—

- (a) limit segregation or loss of materials;
- (b) limit premature stiffening;
- (c) produce a monolithic mass between planned joints or the extremities of members, or both;
- (d) completely fill the formwork to the intended level, expel entrapped air, and closely surround all reinforcement, tendons, ducts, anchorages, embedments and fixings; and
- (e) provide the specified finish to the formed surfaces of the member.

#### 17.1.4 Finishing of unformed concrete surfaces

Unformed concrete surfaces shall be finished by appropriate methods, to achieve the specified—

- (a) dimensions, falls, tolerances, or similar details relating to the shape and uniformity of the surfaces;
- (b) cover from the surfaces to reinforcement, tendons, ducts and embedments; and
- (c) texture of the surface.

#### 17.1.5 Curing and protection of concrete

##### 17.1.5.1 Curing

Concrete shall be cured continuously for a period of time so the design requirements for strength, serviceability and stripping are satisfied. To satisfy durability, curing requirements shall be not less than those given in Clauses 4.4 and 4.5.

Curing shall be achieved by the application of water to, accelerated curing of, or the retention of water in, the freshly cast concrete. Curing shall commence as soon as practicable after the finishing of any unformed surfaces has been completed. Where retention of water in the fresh concrete relies on the application to exposed surfaces of sprayed membrane-forming curing compounds, the compounds used shall conform with AS 3799.

Curing requirements for the various members of the structure shall be detailed in the project specification.

#### **17.1.5.2 Protection**

Freshly cast concrete shall be protected from the effects of rain, running water and freezing or drying prior to hardening. During the initial curing period the concrete shall be protected from freezing or drying.

### **17.1.6 Sampling and testing for conformance**

#### **17.1.6.1 General**

Concrete, which is intended for use in structures designed in accordance with this Standard, shall be assessed in accordance with AS 1379 for conformance with the specified parameters.

NOTE: When project assessment is required, the project specification should nominate responsibility for carrying out the relevant sampling, testing and assessment and, if these differ from or are not covered by AS 1379, should give details of how the assessment is to be made.

#### **17.1.6.2 Concrete specified by strength grade**

Concrete specified by strength grade shall satisfy the following criteria:

- (a) For each strength grade of concrete supplied to a project, the mean cylinder compressive strength ( $f_{cm}$ ), as defined in AS 1379, shall be maintained within the limits specified in that Standard.
- (b) For concrete subject to project assessment—
  - (i) the slump of the supplied concrete shall be within the tolerance specified in AS 1379 for the relevant specified slump; and
  - (ii) in addition to Item (a), the mean compressive strength of the representative samples taken from the project shall be within the limits specified in AS 1379.

#### **NOTES:**

- 1 'Strength grade' is defined in AS 1379 as 'the specified value of the characteristic compressive strength of the concrete at 28 days ( $f'_c$ )'.
- 2 The compressive strength of the concrete sampled, tested and assessed in accordance with AS 1379 indicates the potential strength of the supplied concrete, when placed, compacted and cured under optimum conditions; the responsibility of demonstrating rests on the supplier. The achievement of that potential on site is dependent upon the handling, placing, compacting and curing techniques actually used; the responsibility for which rests with the construction contractor (see Clauses 17.1.3 and 17.1.5). Information on appropriate site techniques may be found in HB 64 and HB 67.

#### **17.1.6.3 Concrete specified by parameters other than strength grade**

When concrete is specified by parameters other than strength grade, the method of production control and, if required, project control shall be specified together with the relevant conformance criteria.

The specified methods of control and assessment shall provide a reliable operating characteristic curve so that—

- (a) concrete with a proportion defective of 0.05 has a probability of acceptance of at least 50%; and
- (b) concrete with a proportion defective of 0.30 has a probability of rejection of at least 98%.

### **17.1.7 Rejection of concrete**

#### **17.1.7.1 Plastic concrete**

Plastic concrete may be rejected if, after completion of mixing but prior to site handling—

- (a) the slump, determined in accordance with AS 1012.3.1, differs from the specified slump by more than the tolerances permitted in AS 1379;
- (b) the elapsed time from first introduction of the mixing water is outside the time interval allowed in AS 1379; or
- (c) the appearance and cohesiveness of a particular quantity is significantly different from previously supplied quantities of the same specification.

#### **17.1.7.2 Hardened concrete**

Hardened concrete shall be liable to rejection if—

- (a) it does not satisfy the requirements of Clause 17.1.6;
- (b) it is porous, segregated, or honeycombed, or contains surface defects outside the specified limits; or
- (c) it fails to conform with the other requirements of this Standard.

#### **17.1.7.3 Action on hardened concrete liable to rejection**

Where hardened concrete is liable to rejection in terms of Clause 17.1.7.2, the concrete may be accepted if it can be demonstrated, either by calculation or by testing in accordance with the appropriate clauses of Appendix B, that the structural adequacy and intended use of the affected members are not significantly impaired. Otherwise, the concrete shall be rejected.

### **17.1.8 Requirements for grout and grouting**

#### **17.1.8.1 Grout properties**

Grout shall be proportioned to give the desired properties for its intended use. Grout to be used in grouting prestressing ducts shall have sufficient fluidity to enable it to be pumped through the duct, have low sedimentation and shrinkage, and contain no more than 750 mg of chloride ions per litre of grout.

#### **17.1.8.2 Mixing and agitation**

Grout shall be mixed in a mixer capable of producing a uniform grout of the specified fluidity and free from lumps of undispersed cement.

After mixing, grout shall be held in an agitation tank and kept in motion, to prevent settlement or segregation occurring, before it is pumped into its final position.

## **17.2 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR REINFORCING STEEL**

### **17.2.1 Materials**

#### **17.2.1.1 Reinforcement**

Reinforcement shall be deformed Ductility Class N bars, or Ductility Class L or Ductility Class N welded wire mesh (plain or deformed), except that fitments may be manufactured from Ductility Class L wire or bar, or plain Ductility Class N bar.

All reinforcement shall conform with AS/NZS 4671.

Ductility Class L reinforcement shall not be substituted for Ductility Class N reinforcement unless the structure is redesigned.

### 17.2.1.2 Protective coatings

A protective coating may be applied to reinforcement provided such coating does not reduce the properties of the reinforcement below those assumed in the design.

## 17.2.2 Fabrication

Reinforcement shall be fabricated in accordance with the following:

- (a) Reinforcement shall be fabricated to the shape and dimensions shown in the drawings and within the following tolerances:
  - (i) On any overall dimension for bars and mesh except where used as a fitment—
    - (A) for lengths up to 600 mm ..... -25, +0 mm;
    - (B) for lengths over 600 mm ..... -40, +0 mm.
  - (ii) On any overall dimension of bars or mesh used as a fitment—
    - (A) for deformed bars and mesh ..... -15, +0 mm;
    - (B) for plain round bars and wire ..... -10, +0 mm.
  - (iii) On the overall offset dimension of a cranked column bar ..... -0, +10 mm.
  - (iv) For the sawn or machined end of a straight bar intended for use as an end-bearing splice, the angular deviation from square, measured in relation to the end 300 mm, shall be within ..... 2°.
- (b) Bending of reinforcement shall conform with Clause 17.2.3.
- (c) Welding if required shall conform with AS/NZS 1554.3. Locational tack welding shall also conform with AS/NZS 1554.3.

## 17.2.3 Bending

### 17.2.3.1 General

Reinforcement may be bent either—

- (a) cold, by the application of a force, around a pin of diameter conforming with Clause 17.2.3.2, so as to avoid impact loading of the bar and mechanical damage to the bar surface; or
- (b) hot, provided—
  - (i) the steel is heated uniformly through and beyond the portion to be bent;
  - (ii) the temperature of the steel does not exceed 600°C;
  - (iii) the bar is not cooled by quenching; and
  - (iv) if during heating the temperature of the bar exceeds 450°C, the design yield strength of the steel after bending is taken as 250 MPa and the design checked for strength.

Reinforcement that has been bent and subsequently straightened or bent in the reverse direction shall not be bent again within 20 bar diameters of the previous bend.

Bars shall not be bent using impact, such as with hammers.

Reinforcement partially embedded in concrete may be field-bent provided the bending complies with Items (a) or (b) above and the bond of the embedded portion is not impaired thereby.

### 17.2.3.2 Site rebending

Where site rebending is required, the following procedures shall be followed:

- (a) Rebending of bars shall be undertaken using a rebending tool. Bars shall be rebent against a flat surface or a pin with a diameter that is at least equal to or greater than the minimum pin size as specified in Clause 17.2.3.3. Bars shall not be over-bent beyond the original bend, typically 90°.
- (b) If scabbling tools are to be used near bars because of concrete leakage or contamination, extreme care shall be exercised to prevent any impact or damage to the bars.
- (c) The bar shall be positioned with the initial bend of the bar clear of the concrete.
- (d) A bar shall only be rebent once at the same location.

### 17.2.3.3 Internal diameter of bends or hooks

The nominal internal diameter ( $d$ ) of a reinforcement bend or hook shall be taken as the external diameter of the pin around which the reinforcement is bent. The diameter of the pin shall be not less than the value determined from the following as appropriate:

- (a) For fitments of—
  - (i) 500L bars .....  $3d_b$ ;
  - (ii) R250N bars .....  $3d_b$ ; and
  - (iii) D500N bars .....  $4d_b$ .
- (b) For reinforcement, other than that specified in Item (c) and Item (d) of any grade .....  $5d_b$ .
- (c) For reinforcement, in which the bend is intended to be subsequently straightened or rebent, of—
  - (i) 16 mm diameter or less .....  $4d_b$ ;
  - (ii) 20 mm diameter or 24 mm .....  $5d_b$ ; and
  - (iii) 28 mm diameter or greater .....  $6d_b$ .

Any such straightening or rebending shall be clearly specified or shown in the drawings.
- (d) For reinforcement that is epoxy-coated or galvanized, either before or after bending, of—
  - (i) 16 mm diameter or less .....  $5d_b$ ;
  - (ii) 20 mm diameter or greater .....  $8d_b$ .

### 17.2.4 Surface condition

At the time concrete is placed, the surface condition of reinforcement shall be such as not to impair its bond to the concrete or its performance in the member. The presence of millscale or surface rust shall not be cause for rejection of reinforcement under this clause.

### 17.2.5 Fixing

All reinforcement, including secondary reinforcement provided for the purpose of maintaining main reinforcement and tendons in position, shall be supported and maintained in position within the tolerances given in Clause 17.5.3 until the concrete has hardened. Bar chairs and spacers used for this purpose shall conform with AS/NZS 2425 and be specified with a strength grade appropriate to the mass of supported reinforcement and applied construction loads. Tie wire shall be used to maintain position unless the bar chair has an in-built clip. Tie wire shall not be finished within the concrete cover.

NOTE: Off-cuts of tie wire should be removed before placement of concrete.

### **17.2.6 Lightning protection by reinforcement**

Where lightning protection is to be provided by the reinforcement, the reinforcement shall conform with the relevant requirements of AS/NZS 1768.

## **17.3 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR PRESTRESSING DUCTS, ANCHORAGES AND TENDONS**

### **17.3.1 Materials for ducts, anchorages and tendons**

#### **17.3.1.1 Ducts**

Sheaths and removable formers used to form ducts shall be capable of maintaining their required cross-section and profile during construction.

#### **17.3.1.2 Anchorages**

The quality and properties of anchorages shall be established by testing.

#### **17.3.1.3 Tendons**

Prestressing tendons shall conform with AS/NZS 4672.1, as applicable.

Tendons shall not be galvanized.

Hard-drawn, high tensile steel wire, which has not been stress-relieved, shall not be used for wire winding unless its elongation, tested in accordance with AS/NZS 4672.1, is 3.5% or greater.

Plain wire shall not be used for pretensioning.

### **17.3.2 Construction requirements for ducts**

#### **17.3.2.1 Surface condition**

When concrete is placed, the outside surface of sheaths and formers for ducts shall be such as not to impair bond of the concrete to the duct. Immediately before grouting, the inside surfaces of sheaths shall be such as not to impair bond of the grout to the duct.

Where an extractable core is used, a suitable technique shall be chosen to ensure its withdrawal, without damage to the formed duct.

#### **17.3.2.2 Sealing**

Prior to the placing of concrete, ducts shall be sealed at the ends and at all joints, to exclude concrete, or other matter.

#### **17.3.2.3 Fixing**

Ducts shall be supported and fixed at regular intervals so the required tendon profile will be maintained in accordance with Clause 17.5.3.

### **17.3.3 Construction requirements for anchorages**

#### **17.3.3.1 Fixing**

Anchorages shall be fixed strictly in accordance with the supplier's recommendations and the following:

- (a) The anchorage shall be square to the line of the tendon.
- (b) The duct shall be securely attached to the anchorage so it provides a grout-tight joint between the duct and the anchorage.
- (c) Where the anchorage is fixed to the formwork, the joint between the two parts shall be grout-tight.

### **17.3.3.2 Surface condition**

At the time concrete is placed, the surface condition of the anchorage shall be such as not to impair its bond to the concrete.

## **17.3.4 Construction requirements for tendons**

### **17.3.4.1 Fabrication**

Tendons shall be fabricated in accordance with the following:

- (a) Cutting of tendons shall be carried out so damage to tendons, ducts and anchorages is avoided.
- (b) Tendons shall not be welded.
- (c) Prestressing bars shall be within manufacturing tolerances and not bent in the threaded portion.

Small adjustments on site shall be carried out cold. If the bar temperature is lower than 10°C, the bar temperature shall be raised above this value by means of steam or hot water.

### **17.3.4.2 Protection**

Before stressing, tendons shall be protected from stray current arcing and splashes from the cutting operation of an oxy-acetylene torch or an arc-welding process.

The threaded ends of prestressing bars shall be provided with suitable protection, at all times.

If tendons are to have a coating or wrapping, such coating or wrapping shall be inert with respect to both the steel and the concrete.

After stressing and anchoring, all tendons and anchorages shall be protected from physical damage and corrosion.

### **17.3.4.3 Surface condition**

The surface condition of tendons shall be such as not to impair bond to the concrete or grout, or performance in the member.

The presence of surface rust shall not be cause for rejection of ducts under this clause unless the steel is visibly pitted.

### **17.3.4.4 Fixing**

All tendons shall be supported and maintained in position within the permissible tolerances given in Clause 17.5.3 until the concrete has hardened.

### **17.3.4.5 Tensioning**

Tensioning of tendons shall be carried out in a safe manner and in accordance with the following:

- (a) The stressing procedure shall ensure the force in a tendon increases at a uniform time rate and that the force is transferred gradually to the concrete.
- (b) The prestressing force applied to the tendon shall be measured at the jack by measuring the jack pressure. The prestressing force shall be measured to an accuracy of  $\pm 3\%$ .
- (c) The tendon extension shall be measured.
- (d) A check shall be made for each tendon, on the correlation between the measured extension and the calculated extension derived from the prestressing force, using the load-elongation curves for the tendons and assumed friction values for the cable. Any disparity between the two figures greater than 10% of the calculated extension shall be investigated.



- (e) No stressing shall be carried out when the temperature of the surrounding air is lower than 0°C.

#### **17.3.4.6 Maximum jacking forces**

The maximum force to be applied to a tendon during the stressing operation shall not exceed—

- (a) for pretensioned tendons .....  $0.80f_{pb}A_p$ ;  
 (b) for stress-relieved post-tensioned tendons .....  $0.85f_{pb}A_p$ ; or  
 (c) for post-tensioned tendons and bars not stress-relieved .....  $0.75f_{pb}A_p$ .

#### **17.3.4.7 Grouting**

Ducts containing post-tensioned tendons shall be completely filled with grout, conforming with Clause 17.1.8, as soon as practicable after stressing. Grouting shall not be carried out when the temperature of the surrounding air is lower than 5°C.

Precautions shall be taken to prevent corrosion for the tendons if the elapsed period prior to grouting is likely to exceed four weeks.

#### **17.3.5 Construction requirements for unbonded tendons**

Unbonded tendons shall not be permitted except in slabs on the ground. Where so used, the requirements of Clauses 17.3.4.1 to 17.3.4.6 shall apply, and the tendons shall be adequately protected against corrosion.

### **17.4 CONSTRUCTION REQUIREMENTS FOR JOINTS AND EMBEDDED ITEMS**

#### **17.4.1 Location of construction joints**

Construction joints shall be located in accordance with the following:

- (a) Construction joints designed in accordance with Clause 19.1.2 shall be located to facilitate the placement of concrete in accordance with Clause 17.1.3.
- (b) Where an interruption to the placing of concrete occurs such that the requirements of Clause 17.1.3(c) or Clause 17.1.3(d) or Clause 17.1.3(e) cannot be fulfilled, a construction joint conforming with Clause 19.1.2 shall be made at an appropriate location.

#### **17.4.2 Embedded and other items not shown in the drawings**

Where an embedded item, driven fixing device or hole is required, it shall be located so that the behaviour of the members is not impaired (see Clause 14.2).

### **17.5 TOLERANCES FOR STRUCTURES AND MEMBERS**

#### **17.5.1 General**

For the purposes of the strength requirements of this Standard, the position of any point on the surface of a concrete member shall conform with Clause 17.5.2. More stringent tolerances may be required for reasons of serviceability, fit of components, or aesthetics of the structure.

For formed surfaces, the tolerances given in AS 3610 take precedence, unless those in Clause 17.5.2 are more stringent. For unformed plane surfaces, the flatness tolerances and the methods for measuring them shall be detailed in the project specification, and shall be not greater than the relevant values given in Clause 17.5.2.

## 17.5.2 Tolerances for position and size of structures and members

### 17.5.2.1 Absolute position

The deviation from the specified position shall not exceed the following:

- (a) In plan, for a point on the surface of a column or wall at any floor level—
  - (i) in the first 20 storeys of any building..... 40 mm horizontally; and
  - (ii) for subsequent storeys, an increase of 15 mm horizontally for each additional 10 storeys or part thereof.
- (b) In elevation, for a point on the top surface of a floor or the soffit of a beam or slab adjacent to a column or wall ..... 40 mm vertically.

### 17.5.2.2 Floor-to-floor plumb

In any column or wall, the deviation from plumb, measured floor-to-floor, shall not exceed 1/200 times the dimension between the floors or 10 mm, whichever is the greater.

### 17.5.2.3 Deviation from specified dimensions

The deviation from any specified height, plan, or cross-sectional dimension, shall not exceed 1/200 times the specified dimension or 5 mm, whichever is the greater.

### 17.5.2.4 Deviation from surface alignment

The deviation of any point on a surface of a member, from a straight line joining any two points on the surface, shall not exceed 1/250 times the length of the line.

## 17.5.3 Tolerance on position of reinforcement and tendons

The deviation from the specified position of reinforcement and tendons shall not exceed the following:

- (a) For positions controlled by cover—
  - (i) in beams, slabs, columns and walls..... -5, +10 mm;
  - (ii) in slabs-on-ground..... -10, +20 mm; and
  - (iii) in footings cast in the ground..... -10, +40 mm,

where a positive value indicates the amount the cover may increase and a negative value indicates the amount the cover may decrease.
- (b) For positions not controlled by cover, namely—
  - (i) the location of tendons on a profile..... 5 mm;
  - (ii) the position of the ends of reinforcement ..... 50 mm; and
  - (iii) the spacing of bars in walls and slabs and of fitments in beams and columns..... 10% of the specified spacing or 15 mm, whichever is greater.

## 17.6 FORMWORK

### 17.6.1 General

The materials, design and construction of formwork shall conform with AS 3610. Stripping of forms and removal of formwork supports from members cast in situ shall conform with the requirements of Clause 17.6.2 where these are more stringent than the relevant requirements of AS 3610.

## 17.6.2 Stripping of forms and removal of formwork supports

### 17.6.2.1 General

The stripping of forms and the removal of formwork supports shall conform with the following:

- (a) Forms shall not be stripped or any formwork supports removed until the part of the member that will be left unsupported has attained sufficient strength to support, with safety and without detriment to its intended use, its own weight and any superimposed loads due to concurrent or subsequent construction works.
- (b) Removal of formwork supports shall be carried out in a planned sequence so the concrete structure will not be subject to any unnecessary deformation, impact, or eccentric loading during the process.
- (c) Removal of formwork from vertical surfaces shall be carried out in accordance with Clause 17.6.2.2.
- (d) Stripping of forms, from the soffits of reinforced slabs and beams between formwork supports, shall be carried out in accordance with Clause 17.6.2.3 or Clause 17.6.2.4 as appropriate. Where backpropping is used, the procedure shall conform with the appropriate requirements of AS 3610.
- (e) Removal of formwork supports from the soffits of reinforced slabs or beams shall be carried out in accordance with—
  - (i) Clause 17.6.2.5 for members not supporting structures above; or
  - (ii) Clause 17.6.2.6 for multistorey structures.
- (f) Stripping of forms and removal of formwork supports from prestressed beams and slabs shall be carried out in accordance with Clause 17.6.2.7.
- (g) Where formwork is stripped before the end of the specified curing period for the concrete element, exposed surfaces shall be cured until at least the end of the specified curing period.

### 17.6.2.2 Removal of formwork from vertical surfaces

Formwork shall not be removed from vertical surfaces unless the concrete in the member has achieved sufficient strength to withstand potential damage to its surfaces.

When formwork is stripped at less than 18 hours after casting, extra care shall be exercised to avoid surface damage during stripping.

### 17.6.2.3 Stripping of soffit forms from reinforced beams and slabs where control samples are available

Where control samples have been taken, cured and tested in accordance with Clause 17.6.2.8, soffit forms may be stripped from between the formwork supports of reinforced beams and slabs if—

- (a) the elapsed time between casting of the concrete and the commencement of stripping is greater than three days; and
- (b) the spans between the remaining formwork supports are such that the member will remain uncracked under the action effects of bending and shear due to the maximum concurrent or subsequent construction loads.

In determining whether sufficient curing time has elapsed, the design resistance of the member shall be taken as  $\phi R_u$  (see Clause 2.2), and the appropriate characteristic strength of the concrete is determined from the average strength of the control samples.

#### 17.6.2.4 Stripping of soffit forms from reinforced slabs of normal-class concrete

For reinforced slabs of normal-class concrete, for which an early-age strength has been specified and which are continuous over formwork supports, the period of time between casting of the concrete and the commencement of stripping of the forms between formwork supports shall be not less than that given in Table 17.6.2.4 for the appropriate average ambient temperature over the period. The periods given in the table shall be increased if—

(a)  $L_s / D > 280 / \sqrt{(D + 100)}$

where

$L_s$  is the span between the formwork supports; and

$D$  is the overall depth of the concrete member.

- (b) the superimposed construction load is greater than 2.0 kPa; or
- (c) the average ambient temperature over the period is less than 5°C, in which case the periods shall be increased by half a day for each day the daily average temperature was between 2°C and 5°C, or by a whole day for each day the daily average temperature was below 2°C.

**TABLE 17.6.2.4**  
**STRIPPING OF FORMWORK FROM REINFORCED SLABS**  
**CONTINUOUS OVER FORMWORK SUPPORTS**

Average ambient temperature over the period ( $T$ ) °C	Period of time before stripping <i>normal-class</i> concrete with specified early-age strength Days
$T > 20$	4
$20 \geq T > 12$	6
$12 \geq T > 5$	8

#### 17.6.2.5 Removal of formwork supports from reinforced members not supporting structures above

For the purpose of determining the minimum period before any undisturbed supports or backprops can be removed from the soffits of reinforced members not supporting a structure above, it may be taken that the requirements of Clause 17.6.2.1(a) shall be deemed to be satisfied if either—

- (a) it can be demonstrated by calculations, based on known or specified early-age strengths that, at the time of removal, the concrete has gained sufficient strength so that the degree of cracking or deformation that will occur, then or subsequently, is not greater than that which would occur if the design serviceability load were applied to the member when the concrete has attained its required design strength; or
- (b) in the absence of any early-age strength data, the period of time is not less than that given in Table 17.6.2.5 for the appropriate average ambient temperature over the period.

The periods given in Table 17.6.2.5 shall be increased if—

- (i) the superimposed construction load is greater than 2.0 kPa; or
- (ii) the average ambient temperature is less than 5°C, in which case the periods shall be increased by half a day for each day the daily average temperature was between 2°C and 5°C, or by a whole day for each day the daily average temperature was below 2°C.

**TABLE 17.6.2.5**  
**REMOVAL OF FORMWORK SUPPORTS FROM SLABS**  
**AND BEAMS NOT SUPPORTING STRUCTURES ABOVE**

Average ambient temperature over the period ( $T$ ) °C	Period of time before removal of all formwork supports from reinforced members Days
$T > 20$	12
$20 \geq T > 12$	18
$12 \geq T > 5$	24

**17.6.2.6** *Removal of formwork supports from reinforced members in multistorey structures*

In multistorey structures, the number of storeys (including the lowest storey) that are to remain supported by formwork at any one time and the maximum spacing of the formwork supports in any storey, shall be calculated on the basis of the relevant properties of the concrete in each floor at that time and the interaction between the formwork supports and the concrete structure.

Where removal of formwork supports from a storey will result in the floors above being supported mainly by formwork and supported concrete construction, all supported and supporting floors and beams shall be checked by calculation for cracking and deflection under the resulting loads. Removal of formwork supports from that storey shall be permitted only if the magnitude of the cracks and deflections so calculated will not impair the strength or serviceability of the completed structure.

No undisturbed supports or backprops shall be removed within two days of the placing of any slab directly or indirectly supported by such supports.

**17.6.2.7** *Stripping of forms and removal of supports from soffits of prestressed concrete slabs and beams*

Formwork shall not be stripped and formwork supports not removed from the soffits of prestressed concrete slabs or beams until the strength of the concrete in the member and the number of tendons stressed are such as to provide the necessary strength to carry the permanent action (dead load) and imposed actions due to construction loads, and meet the associated serviceability and other limit state requirements.

**17.6.2.8** *Control tests*

If specified, control test-samples of the concrete shall be taken where it is intended that removal of formwork or the stressing of tendons will occur before the concrete has attained the strength assumed in the design of the member.

Control test-samples shall be taken at a minimum frequency of one sample for each 50 m<sup>3</sup>, or part thereof, of a concrete grade placed on any one day and the sample specimens stored and cured under conditions similar to those of the concrete in the work.

At least two specimens from each grade shall be tested for strength at the desired time of stripping or stressing and the strength of the concrete at that age assessed on the basis of the average strength of the specimens.

**17.7 PREFABRICATED CONCRETE STRUCTURES**

**17.7.1 General**

Except where the provisions of Clause 17.7.4 govern, the minimum provisions of Clause 17.7.1 through to Clause 17.7.3 for structural integrity shall apply to all prefabricated concrete structures.

### 17.7.2 Prefabricated elements as floor or roof diaphragms

Connections between diaphragms and those members being laterally supported shall have a nominal tensile strength capable of resisting not less than 4.5 kN per linear metre.

### 17.7.3 Vertical integrity ties

All vertical structural members except for non-load bearing elements shall have connections at horizontal joints in accordance with the following requirements:

- (a) Connections between prefabricated columns shall have a nominal strength in tension not less than  $f_{vt} \times A_g$ , where  $f_{vt}$  is taken as 1.4 MPa and  $A_g$  is in mm<sup>2</sup>. For columns with a larger cross sections than required for strength, a reduced effective area  $A_g$  may be used based on the cross section required but shall not be less than one-half the total area;
- (b) Prefabricated wall panels shall have a minimum of two ties per panel, with a nominal strength not less than 45 kN per tie; and
- (c) When design forces result in no tension at the base, the ties required by Clause 17.7.3(b) shall be anchored into a reinforced concrete floor slab-on-ground or footing.

Connection details that rely solely on friction caused by gravity loads shall not be permitted.

### 17.7.4 Prefabricated concrete load bearing walls

For prefabricated concrete load bearing wall structures ties between structural elements shall be provided to meet the requirements of the load bearing wall system and designed using principles of mechanics.

### 17.7.5 Grouted ducts to joints

Grouted duct used to join prefabricated elements shall be fully confined using Ductility Class N reinforcement anchored perpendicular to the duct in accordance with Section 11.

### 17.7.6 Seating of prefabricated floor elements

Seating of prefabricated floor elements shall have seating at least 1.5 times the Ultimate Limit State drifts.

## SECTION 18 DESIGN FOR FATIGUE

### 18.1 GENERAL

The fatigue design action  $Q_{\text{fat}}$  and the foreseen effective number of stress cycles  $n_{\text{sc}}$  shall be determined in accordance with Clause 2.1.5. The load combinations shall be determined in accordance with Clause 2.5.2.3.

NOTE: Fatigue is a strength limit state considered at the operational/serviceability load combination.

In the case of prestressed members, the relevant section shall be assessed for sensitive to cracking. If any load combinations cause tensile stresses at the concrete surface, the stress ranges for reinforcing steel and prestressing steel shall be calculated assuming a cracked section.

Nominal stresses shall be calculated at the site of potential fatigue initiation. The critical section for fatigue analysis shall be taken in accordance with Section 6, including Clause 6.2.1 and Clause 6.2.3. Additional checks shall be undertaken at changes in section or changes in quantity or direction of reinforcement.

NOTE: A support is any column, wall or supporting beam.

The concrete compression stress under the permanent design actions  $[G, \gamma_p P]$  shall not exceed  $0.45 f'_c$ .

### 18.2 MAXIMUM COMPRESSIVE STRESS IN CONCRETE

Detailed fatigue design for concrete in compression is not required if the maximum calculated stresses  $\sigma_{c, \text{max}}$  under the combination of loads specified in Clause 2.5.2.3 satisfy the following:

$$\eta_c \sigma_{c, \text{max}} \leq 0.45 \phi_{c, \text{fat}} f_{c, \text{fat}} \quad \dots 18.2(1)$$

where

$\sigma_{c, \text{max}}$  = maximum compressive stress at the extreme fibres under consideration, compression is taken as positive:  $[\{G, P, \psi_s Q\}, Q_{\text{fat}}]$

$$f_{c, \text{fat}} = 0.85 \beta_{cc}(t_o) f'_c \left(1 - \frac{f'_c}{400}\right)$$

$$\beta_{cc}(t_o) = e^{\left[s \left(1 - \sqrt{28/t_o}\right)\right]}$$

$s$  = coefficient of strength gain given in Table 18.3

$\eta_c$  = concrete stress gradient factor and may be taken as equal to 1.0 or determined as follows:

$$\eta_c = \frac{1}{1.5 - 0.5(\sigma_{c, 300} / \sigma_{c, \text{max}})} \quad \text{but not taken as less than 0.67}$$

$\sigma_{c, 300}$  = compressive stress at a depth of 300 mm from the extreme compressive fibre for same load case as  $\sigma_{c, \text{max}}$ , but not less than zero (compression is positive)

**TABLE 18.3**  
**COEFFICIENT OF STRENGTH GAIN**

Concrete	<i>s</i>	$\beta_{cc}(t_0)$			
		$\frac{f_{cm}(7d)}{f_{cm}(28d)}$	$\frac{f_{cm}(56d)}{f_{cm}(28d)}$	$\frac{f_{cm}(90d)}{f_{cm}(28d)}$	$\frac{f_{cm}(365d)}{f_{cm}(28d)}$
High early strength	0.20	0.82	1.06	1.09	1.16
Normal	0.25	0.78	1.08	1.12	1.20
Delayed strength	0.38	0.68	1.12	1.18	1.32

The fatigue requirements under the load combination specified in Clause 2.5.2.3 shall be deemed to be satisfied if the foreseen number of effective stress cycles ( $n_{sc}$ ) is less than or equal to the resisting number of cycles to failure ( $N$ )—

$$n_{sc} \leq N \quad \dots 18.2(2)$$

The number of resisting stress cycles, to be calculated from the fatigue strength equations are calculated from

For  $0 < S_{cd,min} \leq 0.80$

$$\log(N_1) \leq 8, \text{ then } \log(N) = \log(N_1)$$

For  $S_{cd,min} > 0.80$

$$\log(N_1) > 8, \text{ then } \log(N) = \log(N_2)$$

$$\log(N_1) = \frac{8}{Y-1} (S_{cd,max} - 1)$$

$$\log(N_2) = 8 + \frac{8 \ln(10)}{Y-1} (Y - S_{cd,min}) \log\left(\frac{S_{cd,max} - S_{cd,min}}{Y - S_{cd,min}}\right) \text{ and}$$

$$Y = \frac{0.45 + 1.8 S_{cd,min}}{1 + (1.8 - 0.3 S_{cd,min}) S_{cd,min}}$$

where

$\sigma_{c,max}$  = maximum compressive stress at the extreme fibre under consideration, compression measured positive,  $\gamma_F [G, P, Q_{fat}]$

$S_{cd,max}$  = maximum compressive stress level calculated as  $\frac{\eta_c \sigma_{c,max}}{\phi_{c,fat} f_{c,fat}}$

$\sigma_{c,min}$  = minimum compressive stress at the extreme fibre under consideration, taken as zero if tensile,  $\gamma_F [\{G, P, \psi_s Q\}, Q_{fat}]$

$S_{cd,min}$  = minimum compressive stress level calculated as  $\frac{\eta_c \sigma_{c,min}}{\phi_{c,fat} f_{c,fat}}$

For multiple cycles with variable amplitudes, the fatigue damage may be determined using—

$$D_{Ed} = \sum_{i=1}^m \frac{n_i}{N_i} \leq 1.0 \quad \dots 18.2(3)$$

where

$m$  = number of intervals with constant amplitude



$n_i$  = foreseen number of constant amplitude cycles in interval  $i$

$N_i$  = ultimate number of constant amplitude cycles in interval  $i$  that can be carried before failure

### 18.3 PLAIN CONCRETE WITH COMPRESSION-TENSION STRESS

For plain (unreinforced) concrete in compression with tension stress, the maximum tensile stress in the concrete shall be limited by—

$$\sigma_{c,\max} \leq \sigma_{c,\max}/38.5 \quad \dots 18.3(1)$$

$$\log(N) = 9 \left( 1 - \frac{\sigma_{c,\max}}{\phi_{c,\text{fat}} f_{c,\text{fat}}} \right) \quad \dots 18.3(2)$$

where

$\sigma_{ct,\max}$  = maximum tensile stress

$\sigma_{c,\max}$  = maximum compressive stress

$$f_{c,\text{fat}} = 0.85 \beta_{cc}(t_o) f'_c \left( 1 - \frac{f'_c}{400} \right)$$

$$\text{NOTE: } \sigma_{c,\max} \leq \phi_{c,\text{fat}} f_{c,\text{fat}} \left( 1 - \frac{\log(n_{sc})}{9} \right).$$

### 18.4 PLAIN CONCRETE WITH PURE TENSION OR COMBINED TENSION-COMPRESSION STRESS

For plain (unreinforced) concrete with pure tension or in combined tension-compression, the maximum tensile stress shall be limited by—

$$\sigma_{c,\max} > \sigma_{c,\max}/38.5 \quad \dots 18.4(1)$$

$$\log(N) = 12 \left( 1 - \frac{\sigma_{ct,\max}}{\phi_{c,\text{fat}} f'_{ct,f}} \right) \quad \dots 18.4(2)$$

where

$\sigma_{ct,\max}$  = maximum tension stress

$\sigma_{c,\max}$  = maximum compression stress

$$\text{NOTE: } \sigma_{ct,\max} \leq \phi_{c,\text{fat}} f'_{ct,f} \left( 1 - \frac{\log(n_{sc})}{12} \right)$$

### 18.5 SHEAR LIMITED BY WEB COMPRESSIVE STRESSES

The maximum shear force in flexural members under the combination of permanent effects and the fatigue design loading shall not exceed  $0.6\phi V_{u,\max}$ , where  $V_{u,\max}$  is calculated in accordance with Clause 8.2.3.3.

### 18.6 SHEAR IN SLABS

#### 18.6.1 General

The maximum shear force in concrete slabs, as determined in accordance with Clause 9.2 under the combination of permanent effects and the fatigue design loading, shall be limited to the values specified in this clause.

### 18.6.2 Allowable shear force under cyclic loading

The calculated shear force under fatigue design loads shall not exceed:

- (a) For slabs where the number of stress cycles is not greater than  $2 \times 10^6$  and where the slab can act as a wide beam and a shear failure could occur across a substantial width,  $0.6\phi V_u$ ; otherwise  $0.54\phi V_u$ .

If the longitudinal tensile reinforcement ratio  $(A_{st} + A_{pt})/(b_{do})$  is less than 0.01, the permissible shear shall be further reduced by multiplying the permissible value by the factor  $(100(A_{st} + A_{pt})/(b_{do}))^{1/3}$ .

- (b) Where the potential failure surface could form a truncated cone or pyramid around a support or loaded area, the maximum calculated shear shall not exceed  $0.5\phi V_u$  specified in Clause 9.2.4.

### 18.7 BOND STRENGTH IN REINFORCEMENT AND PRESTRESSED STEEL

The effect of differences in bond behaviour of prestressing and reinforcing steel shall be accounted for. Unless a more refined method is used, a linear elastic model for stress calculation that fulfils the compatibility in strains and multiplying the stress in the reinforcing steel by the following factor may be used:

$$\eta_s = \frac{1 + (A_p / A_s)}{1 + (A_p / A_s) \sqrt{\xi (d_b / d_p)}} \quad \dots 18.7$$

where

$\eta_s$  = differential bond factor (between reinforcing steel and prestressing steel)

$A_s$  = area of reinforcing steel

$A_p$  = area of prestressing steel

$d_b$  = smallest diameter of the reinforcing steel in the relevant cross-section

$d_p$  = diameter of the prestressing steel (for bundles an equivalent diameter  $1.6\sqrt{A_p}$  is chosen, where  $A_p$  is the cross-section area of the bundle)

For post-tensioned members:

$\xi = 0.2$  for smooth prestressing steel

$\xi = 0.4$  for strands

$\xi = 0.6$  for ribbed prestressing wires

$\xi = 1.0$  for ribbed prestressing bars

For pretensioned members:

$\xi = 0.6$  for strands

$\xi = 0.8$  for ribbed prestressing steels

### 18.8 TENSILE STRESS RANGE IN STEEL

The fatigue strength for nominal stress ranges given by a series of  $\log(\Delta\sigma_R) - \log(N)$  curves ( $S-N$  curves) that correspond to the reinforcement detail categories.

Detailed fatigue design for reinforcement is not required if the maximum calculated stress range  $\Delta\sigma_s$  under the applied design action from AS 1170.0 with load combination specified in Clause 2.5.2.3, satisfy the following criteria:

Unwelded straight reinforcement	$\Delta\sigma_s \leq 70 \text{ MPa}$
Welded or bent reinforcement	$\Delta\sigma_s \leq 35 \text{ MPa}$

The damage of a single stress amplitude  $\Delta\sigma_s$  shall be determined by using the corresponding  $S-N$  curves given in Figure 18.8 for reinforcing and prestressing steel. The applied design action from AS 1170.0 shall be factored in accordance with Clause 2.5.2.3. The resisting stress range  $\Delta\sigma_{Rsk}$  at  $N_{Rsk}$  cycles obtained shall be multiplied by the strength capacity reduction factor  $\phi_{s,fat}$ .

The fatigue requirement will be met if the calculated maximum acting stress range,  $\Delta\sigma_s$ , satisfies the condition—

$$\Delta\sigma_s \leq \phi_{s,fat} \Delta\sigma_{Rsk}(n_{sc}) \quad \dots 18.8(1)$$

where

$\Delta\sigma_s$  = maximum nominal steel stress range under the acting load combination

$\phi_{s,fat}$  = fatigue strength reduction factor for steel, taken as 0.85

$\Delta\sigma_{Rsk}(n_{sc})$  = resisting stress range relevant to  $n_{sc}$  cycles obtained from a characteristic fatigue strength function

$$\Delta\sigma_{Rsk}(n_{sc}) = \Delta\sigma_{Rsk}(N_{Rsk}) \left( \frac{N_{Rsk}}{n_{sc}} \right)^{1/m}$$

$N_{Rsk}$  = number of cycles for resistance of constant amplitude stress  $\Delta\sigma_{Rsk}$

$m = m_1$  for  $n_{sc} \leq N_{Rsk}$ , or

$m = m_2$  for  $n_{sc} > N_{Rsk}$

Values of parameters  $N_{Rsk}$ ,  $m_1$  and  $m_2$  are given in Table 18.8.

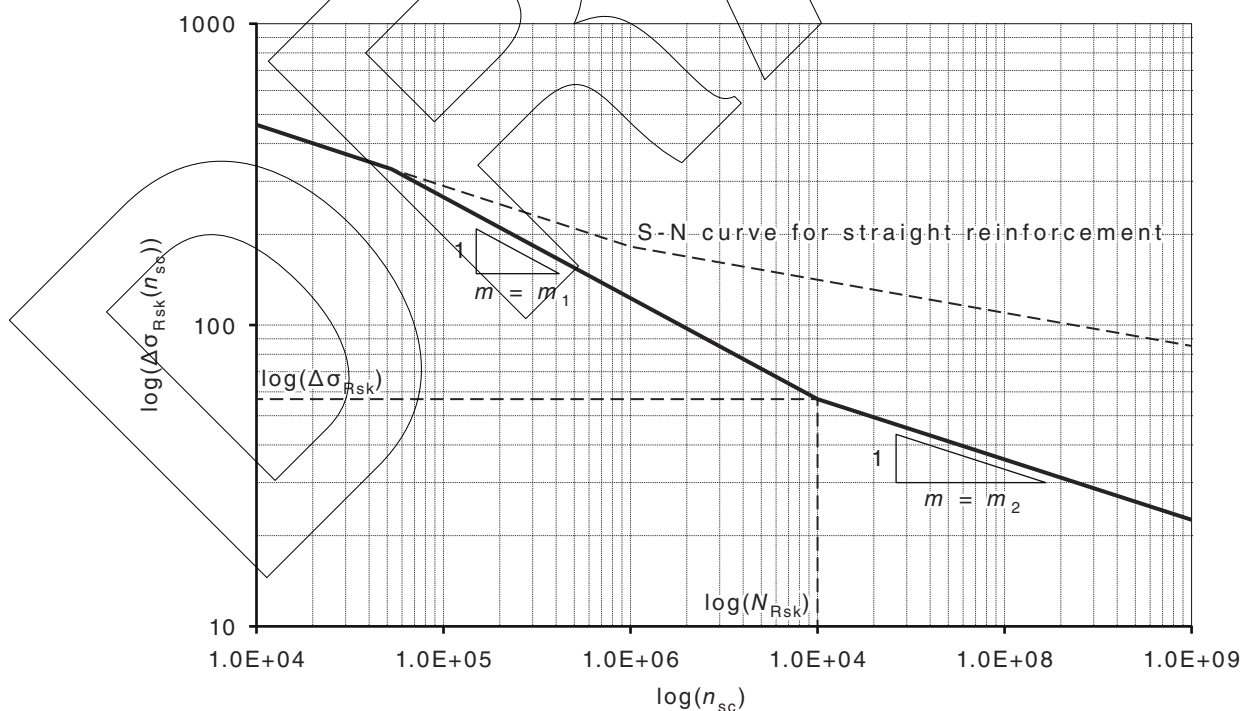


FIGURE 18.8 S-N CURVE FOR REINFORCING STEEL AND TENDONS

**TABLE 18.8**  
**PARAMETERS FOR CHARACTERISTIC S-N CURVES FOR REINFORCING**  
**STEEL AND TENDONS**

Category	Reinforcement or tendon detail	$N_{Rsk}$	$m_1$	$m_2$	$\Delta\sigma_{Rsk}$ at $N_{Rsk}$
<b>Reinforcement</b>					
A	Straight reinforcement $d_b \leq 16$	$10^6$	5	9	210
B	Straight reinforcement $d_b > 16$	$10^6$	5	9	$(240 - 2d_b)$
C	Bent reinforcement $d_b \leq 16$ (see Note 2)	$10^6$	5	9	$210 k_d$
D	Bent reinforcement $d_b > 16$ (see Note 2)	$10^6$	5	9	$(240 - 2d_b)k_d$
E	Welded bar and wire mesh (see Note 1)	$10^7$	3	5	50
F	Mechanical connectors	$10^7$	3	5	50
G	Marine environment (see Note 3)	$10^7$	3	5	65
<b>Tendons</b>					
H	Pre-tensioning	$10^6$	5	9	185
I	Post-tensioning—Single strands in plastic ducts	$10^6$	5	9	185
J	Post-tensioning—Straight tendons or curved tendons in plastic ducts	$10^6$	5	10	150
K	Post-tensioning—Curved tendons in steel ducts	$10^6$	5	7	120
L	Post-tensioning—Splicing devices	$10^6$	5	5	80

Definitions:

$$k_d = 0.35 + 0.026(d_i/d_b) \leq 1.0$$

$d_i$  = Mandrel pin diameter for bending reinforcement

$d_b$  = Reinforcement bar diameter

$N_{Rsk}$  = Reference number of fatigue stress cycles for  $\Delta\sigma_{Rsk}$

NOTES:

- 1 The welding of reinforcement, including non-structural welds (e.g. tack-welds) shall be in accordance with AS/NZS 1554.3.
- 2 The stress range of welded bars  $\Delta\sigma_{Rsk}(n_{sc})$  shall not exceed the stress range for straight bar, wire or tendon at the same number of cycles ( $n_{sc}$ ).
- 3 Valid for all ratios ( $d_i/d_b$ ) and all diameters  $d_b$ .
- 4 The values given in the table are characteristic and do not include the strength reduction factor  $\phi_{s,fat}$ . These values or higher values shall be validated by appropriate testing.
- 5 The values given in the table do not cover coiled and re-straightened bars.

For multiple cycles with variable amplitudes the damage may be added by using—

$$D_{Ed} = \sum_i \frac{n_{sc}(\Delta\sigma_{si})}{N_{Rsk}(\Delta\sigma_{si} / \phi_{s,fat})} \quad \dots 18.8(2)$$

where

$n_{sc}(\Delta\sigma_{si})$  = applied number of cycles for a stress range  $\Delta\sigma_{si}$

$N_{Rsk}(\Delta\sigma_{si})/\phi_{s,fat}$  = resisting number of cycles for a stress range  $\Delta\sigma_{si}$

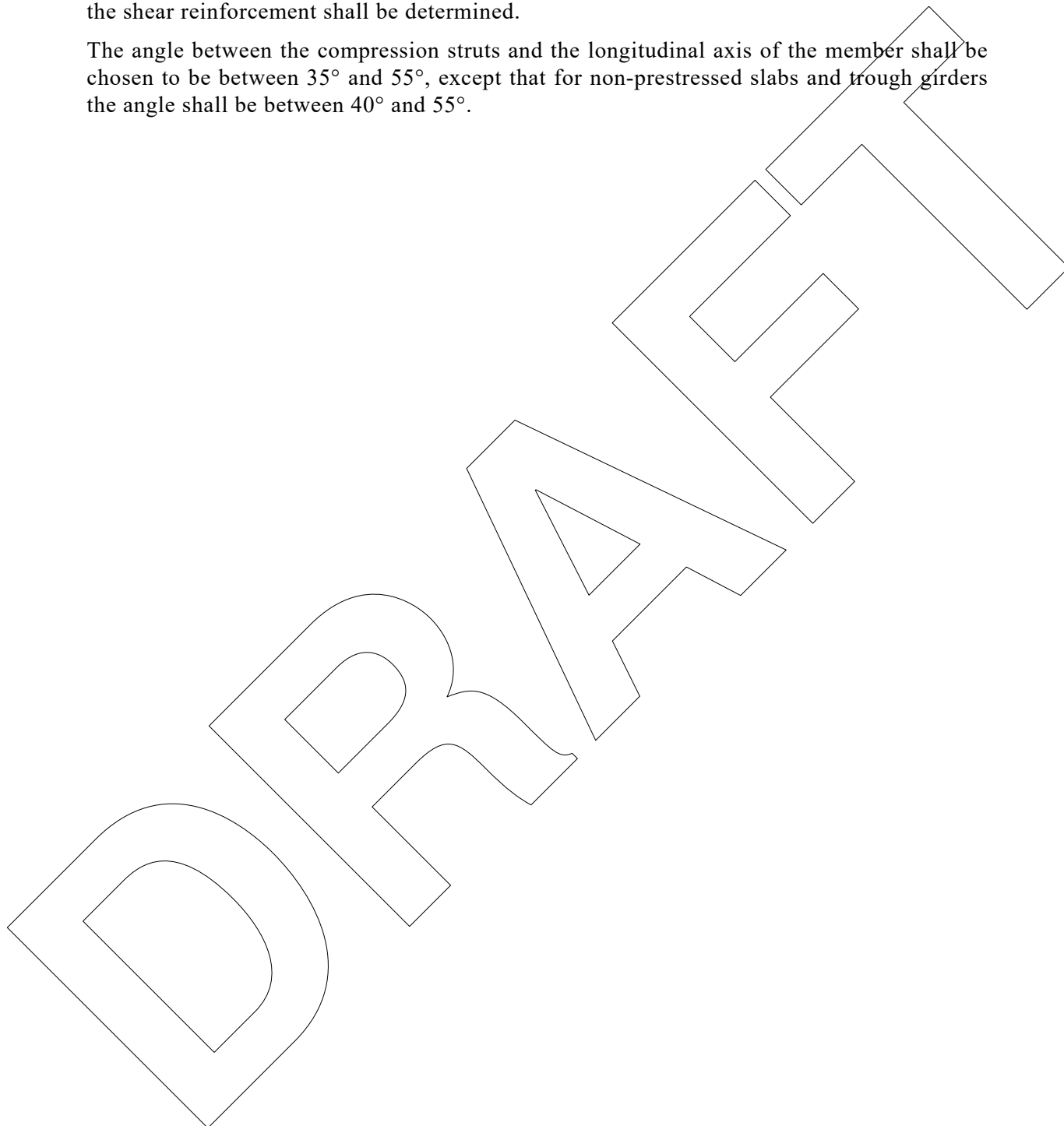
For prestressing or reinforcing steel exposed to fatigue loads, the calculated stresses shall not exceed the design yield strength of the steel. The yield strength should be verified by tensile tests for the steel used.

In areas of high fluctuating stresses, welded lap splices shall not be used.

### **18.9 CALCULATION OF STRESSES IN REINFORCEMENT AND TENDONS OF FLEXURAL MEMBERS**

In flexural members fatigue resistance of the longitudinal reinforcement and tendons and the shear reinforcement shall be determined.

The angle between the compression struts and the longitudinal axis of the member shall be chosen to be between  $35^\circ$  and  $55^\circ$ , except that for non-prestressed slabs and trough girders the angle shall be between  $40^\circ$  and  $55^\circ$ .



## SECTION 19 JOINTS, EMBEDDED ITEMS AND FIXINGS

### 19.1 JOINTS

#### 19.1.1 General

A joint in a structure or member shall be designed and constructed so the load-carrying capacity and serviceability of the structure or member is maintained while serving its intended function. Joints shall be for construction purposes ('construction joint') or to control movement ('movement joint'), as appropriate.

#### 19.1.2 Construction joints

##### 19.1.2.1 General

Construction joints shall be designed and installed to satisfy intended construction practice for the specific application. Construction joints shall be designed to produce a well-bonded interface between hardened concrete and freshly placed concrete.

##### 19.1.2.2 Joint spacing

Construction joints shall be located to facilitate the placement of concrete in accordance with Clause 17.1.3 and to meet concrete placement restrictions and finishing requirements. They shall be located in regions of minimal shear force and, where possible, in unobtrusive locations. The spacing shall be determined by the rate of concrete placement and finishing on site or as a result of any unplanned interruption to placement operations.

Where an interruption to the placing of concrete occurs such that the requirements of Clause 17.1.3(c) or Clause 17.1.3(d) or Clause 17.1.3(e) cannot be fulfilled, a construction joint shall be made at an appropriate location.

#### 19.1.3 Movement joints

##### 19.1.3.1 General

Movement joints shall be designed and constructed to—

- (a) control movement at a joint;
- (b) control cracking at design locations; or
- (c) provide articulation and separation between concrete members in a structure and meet their intended function without impairing the load-carrying capacity or serviceability of either the structure or member.

Movement joints shall be formed between two concrete members to allow movement to occur, typically as a result of shrinkage, creep, diurnal effects and differential settlement. Generally, they will extend throughout the member depth imparting complete discontinuity between adjoining concrete members. However, joints for shrinkage are not necessarily formed through the depth of the member.

##### 19.1.3.2 Joint spacing

In reinforced concrete members, the spacing of movement joints shall take into account effects such as shrinkage, temperature movement, moisture change, creep and other relevant factors. The level of prestress and subsequent member shortening shall also be considered in prestressed concrete members.

### 19.1.4 Joint fillers and sealants

Material infill in a joint shall remain in place and deform in response to loading and movement without undergoing any change that will adversely affect the functionality of the joint. Such infills shall consist of either a joint filler or sealant (or both). Fillers may be inserted into a joint in a compressed condition either when the concrete is plastic or is in its hardened state.

A sealant shall be employed to provide weatherproofing, fire resistance, acoustic insulation, chemical resistance, prevention of deleterious material ingress or other function that cannot be imparted by a filler.

Joints shall be sealed or otherwise designed and detailed to prevent the entry of dirt or incompressible material into the joint which would detrimentally affect the joint movement or operation. Where loading across the joint is anticipated, the designer shall ensure the infill materials are of sufficient hardness and the concrete edges adequately protected, as appropriate, to prevent joint edge spalling.

## 19.2 EMBEDDED ITEMS

### 19.2.1 General

For the purpose of this clause, embedded items include pipes and conduits with their associated fittings, sleeves, permanent inserts for fixings and other purposes, holding-down bolts and other supports.

Items may be embedded in concrete members provided the required strength and serviceability of the member is satisfied, and the durability requirements of Clause 4.10.3.7 are met.

### 19.2.2 Pipes

Embedded conduits and pipes shall conform with the relevant Standards.

NOTE: For example,

- (a) for electrical purposes..... AS/NZS 3000; and
- (b) for plumbing purposes ..... AS/NZS 3500.

Pipes intended to contain liquid, gas or vapour under pressure or extremes of temperature may be embedded in structural concrete, provided the maximum pressure to which any piping or fitting is intended to be subjected will not exceed 2000 kPa, and the effect that inclusion of the pipe has on the strength and serviceability behaviour of the member is taken into account.

### 19.2.3 Spacing

The minimum clear distance between embedded items, and between embedded items and bars (including bundled bars), tendons or ducts, shall be sufficient to ensure the concrete can be placed and compacted to conform with Clause 17.1.3.

## 19.3 FIXINGS

Fixings, including holding-down bolts, inserts and ferrules, shall conform with the following:

- (a) A fixing shall be designed to transmit all forces, acting or likely to act on it.
- (b) Forces on fixings used for lifting purposes shall include an impact factor in assessing the load.
- (c) Fixings shall be designed to yield before ultimate failure in the event of overload.

- (d) The anchorage of any fixings shall be designed in accordance with Section 13, as appropriate. The design strength of this anchorage shall be taken as  $\phi$  times the ultimate strength, where  $\phi = 0.6$ . In the case of shallow anchorages, cone-type failure in the concrete surrounding the fixing shall be investigated taking into account edge distance, spacing, the effect of reinforcement, if any, and concrete strength at time of loading.
- (e) In the absence of calculations, the strength of a fixing shall be determined by load testing of a prototype to failure in accordance with Paragraph B4, Appendix B. The design strength of the fixing shall be taken as  $\phi$  times the ultimate strength where the ultimate strength is taken as the average failure load divided by the appropriate factor given in Table B4.3, Appendix B and  $\phi = 0.6$ .
- (f) The spacing between, and cover to, fixings shall be in accordance with Clause 14.2.3. The cover for fixings shall be in accordance with Section 4. The cover for fire resistance shall be in accordance with Section 5.

Fixings that are intended for lifting purposes shall have a ratio of ultimate strain to either yield strain or proportional limit strain of not less than three.

NOTE: SA TS 101:2015 *Design of post-installed and cast-in fastenings for use in concrete* has further information on the minimum requirements for the design of fastenings used to transmit loads to concrete for safety-critical applications.



## SECTION 20 PLAIN CONCRETE PEDESTALS AND FOOTINGS

### 20.1 GENERAL

The provisions of this section apply to—

- (a) plain concrete pedestals, provided the unsupported height of the member is not greater than three times the least lateral dimension; and
- (b) plain concrete pad footings supported by the ground.

The value of  $\phi$  throughout the section shall be determined from Table 2.2.2.

### 20.2 DURABILITY

Plain concrete members shall conform with the appropriate provisions of Section 4. The cover to any reinforcement shall conform with that determined in accordance with the appropriate provisions of Section 4.

### 20.3 PEDESTALS

Pedestals subject to combined flexural and axial load shall be proportioned so the maximum compressive stress under the design actions does not exceed  $\phi 0.4f'_c$  and the maximum tensile stress does not exceed  $\phi 0.45\sqrt{f'_c}$ .

The minimum eccentricity shall be taken as  $0.1a$ , where  $a$  is the cross-section dimension in the direction being considered.

### 20.4 FOOTINGS

#### 20.4.1 Dimensions

The minimum nominal depth of a footing shall be 200 mm.

When calculating the strength of a footing, the entire cross-section shall be considered and the depth of the footing shall be assumed to be 50 mm less than the nominal depth.

#### 20.4.2 Strength in bending

The design strength under bending shall be based on a linear stress-strain relationship in both tension and compression.

The design strength in bending shall be taken as  $\phi M_{uo}$ , where  $M_{uo}$  is calculated using the characteristic flexural tensile strength ( $f'_{ct,f}$ ).

The critical section for bending shall be taken at—

- (a) the face of the column, pedestal or wall for concrete members;
- (b) halfway between the centre and face of the wall for a masonry wall; or
- (c) halfway between the face of the column and the edge of the base plate for a steel column and base plate.

### 20.4.3 Strength in shear

The design strength of a member in shear shall be determined in accordance with either or both of the following, as appropriate:

- (a) Where the member acts essentially as a one-way member, and a shear failure can occur across the width of the rectangular cross-section ( $b$ ) of the member, the design strength in shear shall be taken as  $\phi V_u$  where—

$$V_u = 0.15bD(f'_c)^{1/3} \quad \dots 20.4.3(1)$$

The critical section for one-way shear shall be taken at  $0.5D$  from the face of the support.

- (b) Where a shear failure can occur locally around a support or loaded area, the design strength in shear shall be taken as—

$$\phi V_u / [1 + (uM^*/8V^*aD)] \quad \dots 20.4.3(2)$$

where

$$V_u = 0.1uD(1 + 2/\beta_h)\sqrt{f'_c} \leq 0.2uD\sqrt{f'_c}$$

$u$  = effective length of the shear perimeter [see Figure 9.3(A)]

$a$  = dimension of the critical shear perimeter, which is parallel to the direction of bending being considered [see Figure 9.3(B)]

$\beta_h$  = ratio given in Clause 9.3.1.5

## SECTION 21 SLAB-ON-GROUND FLOORS, PAVEMENTS AND FOOTINGS

### 21.1 GENERAL

This section sets out additional design considerations for slab-on-ground floors and pavements and footings including plain concrete pavements, but excluding residential floors and pavements.

### 21.2 DESIGN CONSIDERATIONS

The design of pavements and slabs supported by the ground and any joints therein shall take into account, but not be limited to, the following considerations:

- (a) The determination of appropriate design loading.
- (b) Soil-structure interaction.
- (c) The influence of the pavement or slab on the behaviour of the other parts of the structure.
- (d) Effects of traffic on joints.
- (e) Differential movement at joints.
- (f) The limitation of moisture passing through the slab or pavement.
- (g) The effect of water pressure, if any.
- (h) Techniques to control and minimize cracking.
- (i) Techniques to minimize shrinkage warping.
- (j) Techniques to minimize differential temperature effects.

### 21.3 FOOTINGS

#### 21.3.1 Reinforced footings

Two-way footings shall be designed in accordance with Section 9 and the minimum reinforcement shall be given by  $0.19(D/d)^2 f'_{ct.f} / f_{sy}$ .

#### 21.3.2 Plain concrete footings

Plain concrete footings shall be designed in accordance with Clause 15.4.

## APPENDIX A

### REFERENCED DOCUMENTS

(Normative)

#### AS

- 1012 Methods of testing concrete
- 1012.1 Method 1: Sampling of fresh concrete
- 1012.2 Method 2: Preparation of concrete mixes in the laboratory
- 1012.3.1 Method 3.1: Determination of properties related to the consistency of concrete—Slump test
- 1012.4 Method 4: Determination of air content of freshly mixed concrete (series)
- 1012.9 Method 9: Determination of the compressive strength of concrete specimens
- 1012.10 Method 10: Determination of indirect tensile strength of concrete cylinders ('Brazil' or splitting test)
- 1012.11 Method 11: Determination of the modulus of rupture
- 1012.12 Method 12: Determination of mass per unit volume of hardened concrete (series)
- 1012.13 Method 13: Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
- 1012.14 Method 14: Method for securing and testing cores from hardened concrete for compressive strength
- 1012.16 Method 16: Determination of creep of concrete cylinders in compression
- 1012.17 Method 17: Determination of the static chord modulus of elasticity and Poisson's ratio of concrete specimens
- 1170 Structural design actions
- 1170.4 Part 4: Earthquake actions in Australia
- 1199 Sampling procedures for inspection by attributes (series)
- 1379 Specification and supply of concrete
- 1478 Chemical admixtures for concrete, mortar and grout
- 1478.1 Part 1: Admixtures for concrete
- 1530 Methods for fire tests on building materials, components and structures
- 1530.4 Part 4: Fire-resistance test of elements of construction
- 2758 Aggregates and rock for engineering purposes
- 2758.1 Part 1: Concrete aggregates
- 3610 Formwork for concrete (series)
- 3799 Liquid membrane-forming curing compounds for concrete
- AS/NZS
- 1170 Structural design actions
- 1170.0 Part 0: General principles
- 1170.1 Part 1: Permanent, imposed and other actions
- 1170.4 Part 4: Earthquake actions in Australia
- 1554 Structural steel welding
- 1554.3 Part 3: Welding of reinforcing steel

AS/NZS	
1768	Lightning protection
2425	Bar chairs in reinforced concrete—Product requirements and test methods
3101	Concrete structures standard—The design of concrete structures
4671	Steel reinforcing materials
4672	Steel prestressing materials
4672.1	Part 1: General requirements
4672.2	Part 2: Testing requirements
NZS	
1170	Structural design actions
1170.5	Part 5: Earth actions
ISO	
12491	Statistical methods for quality control of building materials and components
13270	Steel fibres for concrete—Definitions and specifications
EN	
14651	Test Method for Metallic Fibre Concrete—Measuring the Flexural Tensile Strength (Limit of Proportionality (LOP), Residual)
14721	Test Method for Metallic Fibre Concrete—Measuring the Fibre Content in Fresh and Hardened Concrete
BCA	Building Code of Australia
ASTM	
C42	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

## APPENDIX B

### TESTING OF MEMBERS AND STRUCTURES

(Normative)

#### B1 GENERAL

This appendix applies to the testing of a structure or prototype to check that the strength and serviceability requirements of this Standard are met. Methods for testing hardened concrete in place are also detailed. Testing shall be undertaken by persons competent in, and with appropriate expertise for, performing such tests.

#### B2 TESTING OF MEMBERS

##### B2.1 Purpose of testing

Structures designed by calculation in accordance with other parts of this Standard are not required to be tested. Tests can be accepted as an alternative to calculation (prototype testing), or may become necessary in special circumstances (proof testing), in order to satisfy the requirements of Clause 2.2 with respect to strength and Clause 2.3 with respect to serviceability.

Where testing is necessary, elements of structures or whole structures shall be either—

- (a) proof-tested in accordance with Paragraph B3, to ascertain the structural characteristics of an existing member or structure; or
- (b) prototype-tested in accordance with Paragraph B4, to ascertain the structural characteristics of a particular class of member, which are nominally identical to the elements tested.

##### B2.2 Test set-up

All measuring equipment shall be chosen and calibrated to suit the range of measurements anticipated, in order to obtain measurements of the required precision. Care shall be exercised to ensure that no artificial restraints are applied to the test specimen. All necessary precautions shall be taken to ensure that in the event of collapse of any part of a structure being tested, the risk to life is minimized and the collapse will not endanger the safety of the structure being tested (for tests on members) and/or adjacent structures.

##### B2.3 Test load

The test load shall simulate 100% of the design loads for the limit states for strength and serviceability, as appropriate. The test load shall be applied gradually at a rate as uniform as practicable and without impact. The distribution and duration of forces applied in the test shall be representative of those forces to which the structure is deemed to be subject under the requirements of this Standard.

##### B2.4 Test deflections

The deflections of each test specimen shall be measured with respect to an appropriate datum. Deflections shall, as a minimum requirement, be recorded at the following times:

- (a) Immediately prior to the application of the test load.
- (b) Incrementally during the application of the test load.
- (c) Immediately the full test load has been applied.
- (d) Immediately prior to removing the test load.
- (e) Immediately after the removal of the test load.

## **B3 PROOF TESTING**

### **B3.1 Test procedures**

A proof test shall be conducted as follows:

- (a) Before applying any load, record the original position of the members involved.
- (b) Apply the test load as determined from Paragraph B2.3, for the relevant limit state.
- (c) Maintain the test load for the necessary period as stated in Paragraph B3.2.
- (d) Remove the test load.

### **B3.2 Criteria for acceptance**

Criteria for acceptance shall be as follows:

- (a) *Acceptance for strength* The test structure or member shall be deemed to conform with the requirements for strength if it is able to sustain the strength limit state test load for at least 24 h without incurring any significant damage such as spalling or excessive cracking.
- (b) *Acceptance for deflection* The test structure or member shall be deemed to conform with the requirements for serviceability if it is able to sustain the serviceability test load for a minimum of 24 h without exceeding the appropriate serviceability limits.

Appropriate deflection limits for beams and slabs shall be determined using Clause 2.3.2 and the deflections calculated taking into account long-term and short-term effects, allowing for the age and loading history of the structure.

### **B3.3 Damage incurred during test**

The test specimen shall be regularly inspected, to determine the nature and extent of any damage incurred during the test. The effects of the damage shall be considered and the test disbanded if collapse seems likely. At the completion of the test, appropriate repairs to damaged parts shall be carried out.

### **B3.4 Test reports**

A report shall be prepared, which shall contain, in addition to the test load-deflection history and serviceability criteria records, a clear description of the test set-up, including the methods of supporting and loading the members, the method of measuring deflections, crack widths, and so on, and any other relevant data. The report shall also contain a statement as to whether or not the structure, substructure or members tested satisfy the relevant acceptance criteria in Paragraph B3.2, as appropriate.

## **B4 PROTOTYPE TESTING**

### **B4.1 Construction of prototypes**

Prototypes shall be constructed from materials that conform with this Standard, and manufactured in accordance with the specification for the member.

### **B4.2 Number of prototypes**

The number of prototypes to be tested shall be selected so that statistically reliable estimates of the behaviour of the member, at relevant limit state values, can be determined from the results of the testing. No fewer than two prototypes shall be tested. More than one loading combination and more than one limit state condition may be applied to a prototype.

### B4.3 Test load

The test load for strength shall be applied gradually until the total load on the prototype is equal to the design load for the strength limit state as determined by Section 2, and multiplied by the relevant factor given in Table B4.3. This factor shall be selected with respect to the expected coefficient of variation in the parameters that affect the strength and the sample size selected for the testing program, unless a reliability analysis shows that a different value is appropriate.

The total load for each prototype used to assess serviceability shall be the design load for the serviceability limit state as determined from Section 2 multiplied by a factor of 1.2.

**TABLE B4.3**  
**FACTOR TO ALLOW FOR VARIABILITY**  
**IN PRODUCTION OF UNITS**

Number of similar units to be tested	Expected coefficient of variation		
	10%	20%	30%
2	1.3	1.7	2.3
3	1.3	1.6	2.1
5	1.2	1.5	1.8
10	1.1	1.3	1.5

NOTE: Intermediate values may be obtained by linear interpolation. The above values are based on a target safety index of 3.0 for a confidence level of 90%.

### B4.4 Test procedure

The method of applying the test load to the prototype shall reflect the most adverse conditions expected to occur during construction and the in-service condition.

A prototype test shall be conducted as follows:

- (a) Before applying any load, record the original position of the members in the test specimen.
- (b) Apply the test load for the relevant limit state, as determined from Paragraph B4.3.
- (c) Maintain the test load for the necessary period, as stated in Paragraph B4.5.
- (d) Remove the test load.
- (e) Inspect and record the prototype for damage, spalling, cracking and any other relevant observations.

### B4.5 Criteria for acceptance

The units represented by the prototypes shall be deemed to conform with this Standard for serviceability and strength where Item (a) is satisfied and Item (b) or Item (c) is satisfied, as follows:

- (a) *Variability* Production units shall be similar in all respects to the prototypes tested, and variability of production shall be equal to or less than the expected variability determined at prototype testing for serviceability or for strength, as appropriate.
- (b) *Acceptance for strength* The test prototype shall be deemed to conform with the requirements for strength if it is able to sustain the strength limit state test load for at least 5 min without incurring any significant damage, such as spalling or excessive cracking.



- (c) *Acceptance for serviceability* The test prototype shall be deemed to conform with the requirement for serviceability if it is able to sustain the serviceability test load for a minimum period of 1 h without exceeding the serviceability limits appropriate to the member. Deflection limits shall be determined using Clause 2.3.2, taking into account only short-term effects.

Qualitative indicators for the parameters affecting strength and serviceability shall be determined for the expected variability during production. These indicators shall be routinely monitored and measured in manufactured units and used to ensure the actual coefficient of variation in production does not exceed the expected coefficient of variation. Alternatively, manufactured units shall be routinely tested to failure, to determine the coefficient of variation.

#### **B4.6 Test reports**

A report shall be prepared in accordance with Paragraph B3.4, except that instead of the requirement in the final sentence of Paragraph B3.4, the report shall contain a statement as to whether or not the prototypes tested satisfied the relevant acceptance criteria in Paragraph B4.5 as appropriate.

### **B5 QUALITY CONTROL**

#### **B5.1 General**

This Paragraph applies to the assessment of a group of units that are part of a production run of similar units. Paragraphs B5.2, B5.3 and B5.4 identify three methods to routinely assess production. One of these methods shall be nominated by the manufacturer as the means of demonstrating that the manufactured group is similar to the tested prototypes. The routine examination nominated shall include the determination of the variability in a production run by relating key indicators in the sample to the previously performed prototype testing and the application of a test load to each sample, as appropriate.

#### **B5.2 Statistical sampling**

A sampling plan, in accordance with AS 1199, shall be established for the routine inspection and testing of a produced batch. Sampling shall be undertaken in accordance with this plan and the selected specimens shall be routinely tested to ensure conformance with this appendix is maintained.

For concrete specified by strength, the methods of production and assessment, taken together, shall provide a reliable operating characteristic curve so that—

- (a) concrete with a proportion defective of 0.05 has a probability of acceptance of not less than 50%; and
- (b) concrete with proportion defective of 0.30 has a probability of rejection of not less than 98%.

#### **B5.3 Product certification**

Independent assurance of the claim by a manufacturer or contractor of batch consistency shall be permitted, to ascertain whether a production run or application routinely complies with the requirements of this appendix.

NOTE: The certification should meet the criteria described in HB 18.28 in order that effective quality planning to control production is achieved.

## **B5.4 Quality system**

Confidence in routine assessment of production shall be achieved where the manufacturer or contractor can demonstrate that an audited and registered quality management system conforming with the requirements of the appropriate or stipulated Australian or international Standard for a quality system is in place.

Such a system shall include a quality or inspection plan and test plan, to ensure product conformity.

## **B6 TESTING OF HARDENED CONCRETE IN PLACE**

### **B6.1 Application**

This Paragraph applies to the assessment of the strength and other properties of hardened concrete in place by non-destructive testing, by testing of samples cut from representative test panels, or samples cut from members.

### **B6.2 Preparation of samples**

The samples to be tested shall be representative of the concrete under investigation. Prior to testing, surfaces shall be cleaned to remove oil, laitance, curing compounds and surface treatments.

Where required, test panels shall be made of concrete that is identical in composition and which is placed, compacted and cured in a manner similar to concrete used in the member. Dimensions of test panels shall be such that at least three representative samples can be cut from each panel. Test samples of standard dimensions shall be obtained from the test panels by coring or sawing.

### **B6.3 Non-destructive testing**

Non-destructive testing (including impact or rebound hammer, ultrasonic pulse velocity, pullout and abrasion testing, or a combination of techniques) may be used to compare the properties of concrete under investigation with that of a representative sample of known quality. In particular, comparable concrete should be of similar maturity, curing history and mix composition. Alternatively, where specified, values obtained by non-destructive tests may be used directly to assess some properties of concrete.

The method of testing and assessment shall be specified and carried out in accordance with internationally recognized procedures.

NOTE: Combined non-destructive techniques have been found to substantially improve the order of accuracy of the estimated values compared with those obtained from testing by a single method.

### **B6.4 Tests on samples taken from the structure**

#### **B6.4.1 Test requirements**

Taking and testing of cores and beams from members and sample panels shall conform with the following:

- (a) Core and beam locations shall be selected so as to minimize any consequent reduction of strength of the structure.
- (b) The cores and beams shall be representative of the whole of the concrete concerned and in no case shall less than three samples be tested.
- (c) Cores and beams shall be examined visually before and after testing, to assess the proportion and nature of any voids, cracks and inclusions present. These factors shall be considered in the interpretation of the test results.

- (d) Cores shall be taken and tested for compressive strength in accordance with AS 1012.14 and beams shall be taken in accordance with ASTM C42. The beams shall be tested for flexural strength in accordance with AS 1012.11, and shall be tested dry unless the concrete concerned will be more than superficially wet in service. The density of cores and beams shall be determined in accordance with AS 1012.12, in the same condition as applicable to testing for compressive strength using AS 1012.1 or AS 1012.2 by sealing or wrapping samples where appropriate.

**B6.4.2** *Interpretation of results*

The strength of the concrete in the member may be estimated—

- (a) as 1.15 times the average strength of the cores and beams; or
- (b) by using test data from cores or beams taken from another member for which the strength of the concrete is known.

## APPENDIX C

### RESIDUAL TENSILE STRENGTH TEST FOR SFRC

(Normative)

#### C1 GENERAL

This section shall apply where direct tension testing is used to determine the residual tensile strength of strain softening steel fibre reinforced concrete.

#### C2 TESTING

The residual tensile strength shall be obtained using the testing arrangement shown in Figure C1 and shall conform with the following criteria:

- (a) The critical section shall be where the cross-sectional area is a minimum.
- (b) The SFRC mix shall be batched to ensure a uniform distribution of fibres, and shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres. The SFRC shall be compacted using lightly applied external vibration.
- (c) The specimen shall be connected to the testing machine in such a manner that the machine does not apply a load to the specimen during the process of tightening of the grips and prior to testing.
- (d) One end of the specimen shall be connected to the testing machine through a universal joint such that no moment is applied to the end of the specimen.
- (e) Displacement measurements shall be taken on each of the four sides with the COD taken as the average of these measurements.
- (f) A minimum of six specimens shall be tested.
- (g) Tests where the failure of the specimen is outside of the testing region, or where the results are influenced by the test specimen boundaries, shall be retested.
- (h) The characteristic values of the tensile strength  $f_{0.5}$  and  $f_{1.5}$ , corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 95 percentile confidence value assuming the population is normally distributed.
- (i) The mean values of  $f_{0.5m}$  and  $f_{1.5m}$ , corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 50th percentile confidence value assuming the population is normally distributed.

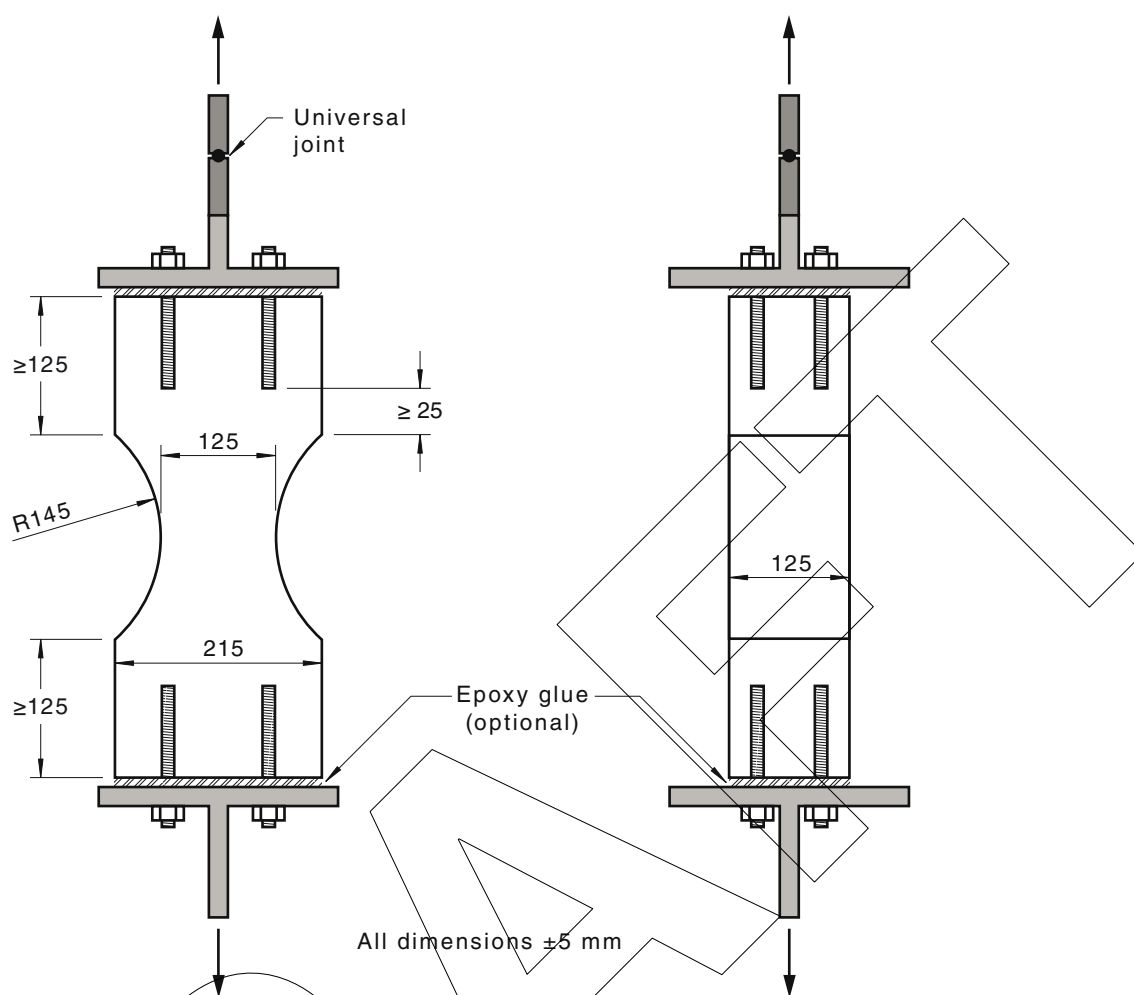
The stress results obtained from the test shall be multiplied by the three-dimensional orientation factor  $k_{3Dt}$ , where—

$$k_{3Dt} = \frac{1}{0.94 + 0.6l_f / b} \leq 1 \quad \dots C1$$

and  $l_f$  is the length of the steel fibre and  $b$  is taken as the average of the width and depth of the specimen taken at the critical section.

#### NOTES:

- 1 The factor  $k_{3Dt}$  removes the influence of the boundaries on the fibre distribution and converts the results of the test to a state where the fibres can be considered to be randomly orientated in three-dimensional space.
- 2 Testing should be undertaken in a laboratory accredited by the National Association of Testing Laboratories (NATA).



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## **PREPARATION OF AUSTRALIAN STANDARDS**

Australian Standards are prepared by a consensus process involving representatives nominated by organizations drawn from all major interests associated with the subject. Australian Standards may be derived from existing industry Standards, from established international Standards and practices or may be developed within a Standards Australia technical committee.

During the development process, Australian Standards are made available in draft form at all sales offices and through affiliated overseas bodies in order that all interests concerned with the application of a proposed Standard are given the opportunity to submit views on the requirements to be included.

The following interests are represented on the committee responsible for this draft Australian Standard:

Australian Building Codes Board  
Bureau of Steel Manufacturers of Australia  
Cement Concrete and Aggregates Australia—Cement  
Cement Concrete and Aggregates Australia—Concrete  
Concrete Institute of Australia  
Consult Australia  
Engineers Australia  
La Trobe University  
Master Builders Australia  
National Precast Concrete Association Australia  
Steel Reinforcement Institute of Australia  
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