

Australian Standard[®]

Concrete structures



This Australian Standard® was prepared by Committee BD-002, Concrete Structures. It was approved on behalf of the Council of Standards Australia on 8 October 2009. This Standard was published on 23 December 2009.

The following are represented on Committee BD-002:

- AUSTROADS
 - Association of Consulting Engineers Australia
 - Australian Building Codes Board
 - Bureau of Steel Manufacturers of Australia
 - Cement Concrete & Aggregates Australia—Cement
 - Cement Concrete & Aggregates Australia—Concrete
 - Concrete Institute of Australia
 - Engineers Australia
 - La Trobe University
 - Master Builders Australia
 - National Precast Concrete Association Australia
 - Steel Reinforcement Institute of Australia
 - University of Adelaide
 - University of Melbourne
 - University of New South Wales
 - University of Western Sydney
-

This Standard was issued in draft form for comment as DR 05252.

Standards Australia wishes to acknowledge the participation of the expert individuals that contributed to the development of this Standard through their representation on the Committee and through the public comment period.

Keeping Standards up-to-date

Australian Standards® are living documents that reflect progress in science, technology and systems. To maintain their currency, all Standards are periodically reviewed, and new editions are published. Between editions, amendments may be issued.

Standards may also be withdrawn. It is important that readers assure themselves they are using a current Standard, which should include any amendments that may have been published since the Standard was published.

Detailed information about Australian Standards, drafts, amendments and new projects can be found by visiting **www.standards.org.au**

Standards Australia welcomes suggestions for improvements, and encourages readers to notify us immediately of any apparent inaccuracies or ambiguities. Contact us via email at **mail@standards.org.au**, or write to Standards Australia, GPO Box 476, Sydney, NSW 2001.

Australian Standard[®]

Concrete structures

First published in part as AS CA2—1934.
AS A26 first published 1934.
AS CA2 redated 1937.
MP 13 first published 1957.
AS CA2—1937 and AS A26—1934 revised, amalgamated and redesignated AS CA2—1958.
Third edition 1963.
MP 13—1957 revised and redesignated AS CA35—1963.
Second edition 1973.
Fourth edition AS CA2—1973.
AS CA2—1973 revised and redesignated AS 1480—1974.
AS CA35—1973 revised and redesignated AS 1481—1974.
Second edition AS 1481—1978.
Second edition AS 1480—1982.
AS 1480—1982 and AS 1481—1978 revised, amalgamated and redesignated AS 3600—1988.
Fourth edition 2009.
Reissued incorporating Amendment No. 1 (November 2010).

COPYRIGHT

© Standards Australia Limited

All rights are reserved. No part of this work may be reproduced or copied in any form or by any means, electronic or mechanical, including photocopying, without the written permission of the publisher, unless otherwise permitted under the Copyright Act 1968.

Published by SAI Global Limited under licence from Standards Australia Limited, GPO Box 476, Sydney, NSW 2001, Australia

ISBN 0 7337 9347 9

PREFACE

This Standard was prepared by Standards Australia Committee BD-002, *Concrete Structures*, to supersede AS 3600—2001.

This Standard incorporates Amendment No. 1 (November 2010). The changes required by the Amendment are indicated in the text by a marginal bar and amendment number against the clause, note, table, figure or part thereof affected.

Objective of the Standard

The principal objective of the 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 compliance with the relevant design requirements.

Background to the fourth edition

Amendment No. 1 to the 2001 edition of the Standard was issued in May 2002 to address various editorial errors in the Standard. At the time the committee embarked on a full revision of the Standard to include design rules for advances in concrete design, including the use of high strength concrete as well as a restructure of the design procedures section to align the Standard to the new editions of the AS/NZS 1170 series, *Structural design actions*.

Amendment No. 2 was published in October 2004 to address two matters the committee believed required immediate attention. These matters included the use of low Ductility Class L reinforcement and its limited ability to distribute moments as implied by the simplified analysis. The minimum reinforcement requirements for crack control introduced in the 2001 edition were also amended as they increased the amount of reinforcement required sometimes by up to 50% of that which was required for the minimum strength provisions.

These two Amendments have been incorporated into this revised edition of AS 3600 as well as a number of other changes.

Areas of major change in the Standard are as follows:

- (a) Increase in concrete strength specified in design rules from 65 MPa to 100 MPa. This has resulted in the review of all equations in AS 3600 for strength and has meant, in some instances, modification of equations such as the rectangular stress block model and inclusion of requirements for confinement to the core of columns.
- (b) Section 2, Design procedures, actions and loads, has been revised to align with the editions of AS/NZS 1170 series, *Structural design actions*, and contains additional design check methods for designers to consider.
- (c) Section 3, Design properties of materials, (previously Section 6) has been reviewed to—
 - (i) include new shrinkage equations, which will address autogenous and drying shrinkage; and
 - (ii) revisions to creep calculations, which modify the creep factor by revising the k_2 and k_3 factors and include the addition of environmental and humidity factors.
- (d) Specification of additional severe exposure classifications and requirements for sulfate soils introduced in Section 4 on durability.

- (e) The fire resistance criteria in Section 5, Design for fire resistance, have been reviewed to take into account the latest developments in EN 1992-1-2:2004, Eurocode 2. *Design of concrete structures Part 1-2: General rules—Structural fire design*.
- (f) Section 6, Methods of structural analysis, (previously Section 7) has been completely revised.
- (g) A new Section 7, Strut-and-tie modelling, which provides rules on strut-and-tie modelling, has been included.
- (h) Clause 10.7.3 regarding confinement to the core of columns in Section 10 has been significantly changed due the importance of this issue for high strength concrete.
- (i) Section 11, Design of walls, has been revised to be more consistent with Section 10, Design of columns for strength and serviceability.
- (j) Section 13, Stress development, splicing of reinforcement and coupling of tendons, has been completely revised.
- (k) Section 17, Liquid retaining structures—Design requirements, and Section 18, Marine structures, of the 2001 edition of the Standard have been deleted as they did not provide specific design advice.
- (l) This Standard traditionally used the terms ‘tie’ and ‘fitment’ interchangeably. The word ‘tie’ is now used only in the strut-and-tie analysis section while the term ‘fitment’ is used for units such as stirrups and ligatures that perform various functions, such as restraining the longitudinal reinforcement and resisting shear.

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

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

CONTENTS

	<i>Page</i>
SECTION 1 SCOPE AND GENERAL	
1.1 SCOPE AND APPLICATION.....	8
1.2 NORMATIVE REFERENCES.....	9
1.3 EXISTING STRUCTURES.....	9
1.4 DOCUMENTATION.....	9
1.5 CONSTRUCTION.....	9
1.6 DEFINITIONS	10
1.7 NOTATION	16
SECTION 2 DESIGN PROCEDURES, ACTIONS AND LOADS	
2.1 DESIGN PROCEDURES.....	28
2.2 DESIGN FOR STRENGTH.....	28
2.3 DESIGN FOR SERVICEABILITY	33
2.4 ACTIONS AND COMBINATIONS OF ACTIONS.....	35
SECTION 3 DESIGN PROPERTIES OF MATERIALS	
3.1 PROPERTIES OF CONCRETE	37
3.2 PROPERTIES OF REINFORCEMENT	43
3.3 PROPERTIES OF TENDONS.....	44
3.4 LOSS OF PRESTRESS IN TENDONS.....	46
3.5 MATERIAL PROPERTIES FOR NON-LINEAR STRUCTURAL ANALYSIS.....	49
SECTION 4 DESIGN FOR DURABILITY	
4.1 GENERAL	50
4.2 METHOD OF DESIGN FOR DURABILITY.....	50
4.3 EXPOSURE CLASSIFICATION	50
4.4 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATIONS A1, A2, B1, B2, C1 AND C2	53
4.5 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATION U.....	54
4.6 ABRASION.....	54
4.7 FREEZING AND THAWING.....	54
4.8 AGGRESSIVE SOILS.....	55
4.9 RESTRICTIONS ON CHEMICAL CONTENT IN CONCRETE.....	57
4.10 REQUIREMENTS FOR COVER TO REINFORCING STEEL AND TENDONS....	57
SECTION 5 DESIGN FOR FIRE RESISTANCE	
5.1 SCOPE	60
5.2 DEFINITIONS	60
5.3 DESIGN PERFORMANCE CRITERIA.....	62
5.4 FIRE RESISTANCE PERIODS (FRPs) FOR BEAMS.....	63
5.5 FIRE RESISTANCE PERIODS (FRPs) FOR SLABS.....	66
5.6 FIRE RESISTANCE PERIODS (FRPs) FOR COLUMNS.....	69
5.7 FIRE RESISTANCE PERIODS (FRPs) FOR WALLS.....	72
5.8 INCREASE OF FIRE RESISTANCE PERIODS (FRPs) BY USE OF INSULATING MATERIALS.....	74

Page

SECTION 6 METHODS OF STRUCTURAL ANALYSIS

6.1	GENERAL	76
6.2	LINEAR ELASTIC ANALYSIS	79
6.3	ELASTIC ANALYSIS OF FRAMES INCORPORATING SECONDARY BENDING MOMENTS.....	80
6.4	LINEAR ELASTIC STRESS ANALYSIS	81
6.5	NON-LINEAR FRAME ANALYSIS	81
6.6	NON-LINEAR STRESS ANALYSIS.....	82
6.7	PLASTIC METHODS OF ANALYSIS	82
6.8	ANALYSIS USING STRUT-AND-TIE MODELS	83
6.9	IDEALIZED FRAME METHOD OF ANALYSIS	83
6.10	SIMPLIFIED METHODS OF FLEXURAL ANALYSIS	85

SECTION 7 STRUT-AND-TIE MODELLING

7.1	GENERAL	93
7.2	CONCRETE STRUTS.....	93
7.3	TIES	98
7.4	NODES.....	98
7.5	ANALYSIS OF STRUT-AND-TIE MODELS	99
7.6	DESIGN BASED ON STRUT-AND-TIE MODELLING.....	99

SECTION 8 DESIGN OF BEAMS FOR STRENGTH AND SERVICEABILITY

8.1	STRENGTH OF BEAMS IN BENDING	100
8.2	STRENGTH OF BEAMS IN SHEAR	105
8.3	STRENGTH OF BEAMS IN TORSION	109
8.4	LONGITUDINAL SHEAR IN COMPOSITE AND MONOLITHIC BEAMS	111
8.5	DEFLECTION OF BEAMS	113
8.6	CRACK CONTROL OF BEAMS.....	116
8.7	VIBRATION OF BEAMS.....	118
8.8	T-BEAMS AND L-BEAMS	118
8.9	SLENDERNESS LIMITS FOR BEAMS.....	119

SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY

9.1	STRENGTH OF SLABS IN BENDING	120
9.2	STRENGTH OF SLABS IN SHEAR.....	123
9.3	DEFLECTION OF SLABS.....	127
9.4	CRACK CONTROL OF SLABS	130
9.5	VIBRATION OF SLABS	133
9.6	MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS	133
9.7	LONGITUDINAL SHEAR IN COMPOSITE SLABS	133

SECTION 10 DESIGN OF COLUMNS FOR STRENGTH AND SERVICEABILITY

10.1	GENERAL	134
10.2	DESIGN PROCEDURES	134
10.3	DESIGN OF SHORT COLUMNS.....	135
10.4	DESIGN OF SLENDER COLUMNS	136
10.5	SLENDERNESS.....	137
10.6	STRENGTH OF COLUMNS IN COMBINED BENDING AND COMPRESSION	141
10.7	REINFORCEMENT REQUIREMENTS FOR COLUMNS.....	144
10.8	TRANSMISSION OF AXIAL FORCE THROUGH FLOOR SYSTEMS	152

SECTION 11 DESIGN OF WALLS

11.1 GENERAL 153

11.2 DESIGN PROCEDURES 153

11.3 BRACED WALLS 154

11.4 EFFECTIVE HEIGHT..... 154

11.5 SIMPLIFIED DESIGN METHOD FOR WALLS SUBJECT TO VERTICAL
COMPRESSION FORCES..... 155

11.6 DESIGN OF WALLS FOR IN-PLANE SHEAR FORCES 155

11.7 REINFORCEMENT REQUIREMENTS FOR WALLS 156

SECTION 12 DESIGN OF NON-FLEXURAL MEMBERS, END ZONES AND
BEARING SURFACES

12.1 GENERAL 158

12.2 STRUT-AND-TIE MODELS FOR THE DESIGN OF NON-FLEXURAL
MEMBERS..... 158

12.3 ADDITIONAL REQUIREMENTS FOR CONTINUOUS CONCRETE NIBS
AND CORBELS..... 160

12.4 ADDITIONAL REQUIREMENTS FOR STEPPED JOINTS IN BEAMS AND
SLABS 160

12.5 ANCHORAGE ZONES FOR PRESTRESSING ANCHORAGES 160

12.6 BEARING SURFACES..... 162

12.7 CRACK CONTROL..... 162

SECTION 13 STRESS DEVELOPMENT OF REINFORCEMENT AND TENDONS

13.1 STRESS DEVELOPMENT IN REINFORCEMENT 163

13.2 SPLICING OF REINFORCEMENT..... 169

13.3 STRESS DEVELOPMENT IN TENDONS..... 171

13.4 COUPLING OF TENDONS..... 172

SECTION 14 JOINTS, EMBEDDED ITEMS AND FIXINGS

14.1 JOINTS..... 173

14.2 EMBEDDED ITEMS 174

14.3 FIXINGS 174

SECTION 15 PLAIN CONCRETE PEDESTALS AND FOOTINGS

15.1 GENERAL 176

15.2 DURABILITY 176

15.3 PEDESTALS..... 176

15.4 FOOTINGS 176

SECTION 16 SLAB-ON-GROUND FLOORS, PAVEMENTS AND FOOTINGS

16.1 GENERAL 178

16.2 DESIGN CONSIDERATIONS..... 178

16.3 FOOTINGS 178

SECTION 17 MATERIAL AND CONSTRUCTION REQUIREMENTS

17.1 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR CONCRETE
AND GROUT..... 179

17.2 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR REINFORCING
STEEL..... 181

17.3 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR PRESTRESSING
DUCTS, ANCHORAGES AND TENDONS..... 184

	<i>Page</i>
17.4 CONSTRUCTION REQUIREMENTS FOR JOINTS AND EMBEDDED ITEMS.....	186
17.5 TOLERANCES FOR STRUCTURES AND MEMBERS.....	186
17.6 FORMWORK.....	187
APPENDICES	
A REFERENCED DOCUMENTS	191
B TESTING OF MEMBERS AND STRUCTURES	193
C REQUIREMENTS FOR STRUCTURES SUBJECT TO EARTHQUAKE ACTIONS.....	199
BIBLIOGRAPHY	205

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/m³ to 2800 kg/m³.

- (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) Prestressing tendons complying 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, such as concrete residential slabs and footings, and swimming pools.

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 comply 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 drawings forming part of the documents setting out the work to be executed.

1.6.2.3 *Specification*

The specification 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 Column strip

See Clause 6.1.4.1.

1.6.3.12 Characteristic strength

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

1.6.3.13 Composite concrete member

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

1.6.3.14 Concrete

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

1.6.3.15 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.

1.6.3.16 Cover

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

1.6.3.17 Creep coefficient

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

1.6.3.18 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 9.2.1(A)].

1.6.3.19 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 9.2.1(A)(b)].

1.6.3.20 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.

1.6.3.21 Design strip

See Clause 6.1.4.2.

1.6.3.22 Discontinuity

Abrupt change in geometry or loading, including prestress.

1.6.3.23 Direct loading

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

1.6.3.24 *D-region*

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

1.6.3.25 *Duct*

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

1.6.3.26 *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.

1.6.3.27 *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.

1.6.3.28 *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 9.2.1(A)].

1.6.3.29 *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.

1.6.3.30 *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.

1.6.3.31 *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).

1.6.3.32 *Fan-shaped compression field*

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

1.6.3.33 *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.

1.6.3.34 *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)].

1.6.3.35 *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 must comply with the BCA requirements, the appropriate Standard is AS 1530.4.

1.6.3.36 *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.

1.6.3.37 *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.

1.6.3.38 *Fixing*

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

1.6.3.39 *Flat plate*

Flat slab without drop panels.

1.6.3.40 *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.

1.6.3.41 *Footing*

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

1.6.3.42 *Foundation*

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

1.6.3.43 *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.

1.6.3.44 *Headed reinforcement*

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

1.6.3.45 *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.

1.6.3.46 *Hollow-core slab or wall*

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

1.6.3.47 *Initial force*

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

1.6.3.48 *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.

1.6.3.49 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.

1.6.3.50 Jacking force

Force in a tendon measured at the jack.

1.6.3.51 Ligature (reinforcement)

See fitment.

1.6.3.52 Lightweight concrete

Concrete having a saturated surface-dry density in the range of 1800 kg/m³ to 2100 kg/m³.

1.6.3.53 Limit state

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

1.6.3.54 Loadbearing function

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

1.6.3.55 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 $0.03 f'_c A_g$.

1.6.3.56 Mean strength

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

1.6.3.57 Middle strip

See Clause 6.1.4.3.

1.6.3.58 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.

1.6.3.59 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.

1.6.3.60 Nodal zone

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

1.6.3.61 One-way slab

Slab characterized by flexural action mainly in one direction.

1.6.3.62 Plain concrete member

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

1.6.3.63 Post-tensioning

Tensioning of tendons after the concrete has hardened.

1.6.3.64 Prestressed concrete

Concrete into which internal stresses are induced deliberately by tendons.

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

1.6.3.65 *Prestressing steel*

See tendon.

1.6.3.66 *Pretensioning*

Tensioning of tendons before the concrete is placed.

1.6.3.67 *Prismatic compression field*

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

1.6.3.68 *Reinforcement*

Steel bar, wire or mesh but not tendons.

NOTE: Commonly referred to as reinforcing steel.

1.6.3.69 *Ribbed slab*

Slab incorporating parallel ribs in one or two directions.

1.6.3.70 *Shear wall*

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

1.6.3.71 *Short column*

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

1.6.3.72 *Slender column*

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

1.6.3.73 *Span support*

See Clause 6.1.4.4.

1.6.3.74 *Strength grade*

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

1.6.3.75 *Structural adequacy (fire)*

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

1.6.3.76 *Strut-and-tie model*

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

1.6.3.77 *Tendon*

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

1.6.3.78 *Tie*

Tension member in a strut-and-tie model.

1.6.3.79 *Torsion strip*

Strip of slab of width a , whose longitudinal axis is perpendicular to the direction of M_v^* [see Figure 9.2.1(B)].

1.6.3.80 *Transfer*

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

1.6.3.81 *Transmission length*

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

1.6.3.82 *Transverse width*

See Clause 6.1.4.5.

1.6.3.83 *Two-way slab*

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

1.6.3.84 *Uniform strain*

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

1.6.3.85 *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 (see Clause 8.3.5)
A_{tr}	= cross-sectional area of a transverse bar along the development 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 9.2.1(B)]
a_m	= average axis distance (see Clause 5.2.1)
a_s	= axis distance (see Clause 5.5.2)
a_{sup}	= length of a support in the direction of the span (see Clause 6.1.4.4)
a_v	= distance from the section at which shear is being considered to the face of the nearest support (see Clause 8.2.7.1)
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 Paragraph C4.3.2, Appendix C)
b_o	= dimension of an opening (see Clause 9.2.1.2 and 9.2.1.5)
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 Paragraph C4.3.2, Appendix C)
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	= the smaller of the concrete covers to the deformed bar or half the clear distance to the next parallel (see Clause 13.1.2.3)
D	= overall depth of a cross-section in the plane of bending; <i>or</i> = 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 8.1.10.1)
d	= effective depth of a cross-section in the plane of bending; <i>or</i> = 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); <i>or</i> = 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_f	= 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; <i>or</i> = 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_c^*	= absolute value of the design force in the compressive zone due to flexure (see Clause 8.3.6)
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 8.2.7.1 and 9.2.3)
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 position of the bars being anchored with respect to the transverse reinforcement (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
A1 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
$L_{\text{sy.c}}$	= 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)
$L_{\text{sy.cb}}$	= basic development length of a deformed bar in compression (see Clause 13.1.5.2)
$L_{\text{sy.t}}$	= 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(A)]
$L_{\text{sy.t.lap}}$	= the tensile lap length for either contact or non-contact splices (see Clause 13.2.2)
$L_{\text{sy.tb}}$	= 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)]
$l_{0,\text{fi}}$	= 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; <i>or</i> = 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}	= bending moment causing cracking of the section with due consideration to prestress, restrained shrinkage and temperature stresses
M_0	= total static moment in a span (see Clause 6.10.4.2); <i>or</i> = decompression moment (see Clause 8.2.7.2)
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); <i>or</i>

	= number of laterally restrained longitudinal bars (see Clauses 10.7.3.3 and 10.7.3.4)
P	= force in the tendons; <i>or</i> = maximum force occurring at the anchorage during jacking (see Clause 12.5.4); <i>or</i> = 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; <i>or</i> = 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 ϕR_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 Paragraph C2.9, Appendix C)
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_{uc}	= ultimate torsional strength of a beam without torsional reinforcement and in the presence of shear (see Clause 8.3.5)
T_{us}	= ultimate torsional strength of a beam with torsional reinforcement (see Clause 8.3.5)
$T_{u,max}$	= 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 9.2.1.5)
u_e	= exposed perimeter of a member cross-section plus half the perimeter of any closed voids contained therein, used to calculate t_h
u_t	= perimeter of the polygon defined for A_t (see Clauses 8.3.5 and 8.3.6)
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_t	= shear force, which, in combination with the prestressing force and other action effects at the section, would produce a principal tensile stress of f'_{ct} at either the centroidal axis or the intersection of flange and web, whichever is the more critical (see Clause 8.2.7.2)
V_u	= ultimate shear strength
$V_{u,max}$	= ultimate shear strength limited by web crushing failure
$V_{u,min}$	= ultimate shear strength of a beam or slab provided with minimum shear reinforcement (see Clauses 8.2.9 and 9.2.4 respectively)
V_{uc}	= ultimate shear strength excluding shear reinforcement (see Clause 8.2.7)
V_{uo}	= ultimate shear strength of a slab with no moment transfer (see Clause 9.2.3)
V_{us}	= contribution by shear reinforcement to the ultimate shear strength of a beam or wall (see Clauses 8.2.10 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 9.2.1(A)]
x	= shorter overall dimension of a rectangular part of a cross-section

Y	= a dimension [see Figure 9.2.1(A)]
y	= larger overall dimension of a rectangular part of a cross-section
y_1	= larger overall dimension of a closed fitment (see Clause 9.2.1.5)
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)
α	= coefficient; <i>or</i> = angle of divergence between bottled shape compression fields and idealized parallel sided strut (see Clause 7.2.4)
α_b	= coefficient for beams (see Clause 8.1.6.1)
α_c	= coefficient (see Clause 10.3.1)
α_n	= coefficient (see Clause 10.6.4)
α_s	= correlation factor (see Clause 10.4.3)
α_{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)
α_v	= angle between the inclined shear reinforcement and the longitudinal tensile reinforcement (see Clause 8.2.10)
α_x, α_y	= short and long span bending moment coefficients respectively, for slabs supported on four sides (see Clause 6.10.3.2)
β	= 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)
β_d	= a factor (see Clause 10.4.3)
β_h	= a ratio (see Clause 9.2.1.5)
β_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)
β_p	= an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects (see Clause 3.4.2.4)
β_s	= strut efficiency factor (see Clause 7.2.2)
β_x, β_y	= short and long span bending moment coefficients respectively, for slabs supported on four sides (see Clause 6.10.3.2)
γ	= the ratio, under design bending or design combined bending and compression, of the depth of the assumed rectangular compressive stress block to $k_u d$
γ_1, γ_2	= column end restraint coefficients, determined in accordance with Clause 10.5.3
γ_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)
Δ	= 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)
ε	= a strain
ε_{cc}	= strain due to concrete creep (see Clauses 3.1.8.1 and 3.4.3.3)
ε_{cs}	= design shrinkage strain, determined in accordance with Clause 3.1.7.1
ε_{cs}^*	= final design shrinkage strain of concrete
ε_{csd}	= drying shrinkage strain, determined in accordance with Clause 3.1.7.2
ε_{cse}	= autogenous shrinkage strain, determined in accordance with Clause 3.1.7.2
ε_{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
ε_{pu}	= strain at maximum stress of a prestressing tendon
ε_{su}	= uniform strain at maximum stress, corresponding to the onset of necking
θ	= 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); <i>or</i> = angle between tie leg and confinement plane (see Clause 10.7.3.3)
θ_v	= angle between the axis of the concrete compression strut and the longitudinal axis of the member (see Clause 8.2.10)
λ	= a factor (see Clause 13.1.2.3)
λ_{uc}	= a ratio of loads (see Clause 10.4.3)
μ	= friction curvature coefficient (see Clause 3.4.2.4); <i>or</i> = coefficient of friction (see Clause 8.4.3); <i>or</i> = structural ductility factor (see Appendix C)
ν	= Poisson's ratio for concrete, determined in accordance with Clause 3.1.5
ρ	= density of concrete, in kilograms per cubic metre (kg/m^3), determined in accordance with Clause 3.1.3
ρ_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)
ρ_s	= volumetric ratio of the fitments relative to the volume of the core (see Clause 10.7.3.3)
σ_{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)
σ_{cp}	= average intensity of effective prestress in concrete
$\sigma_{cp.f}$	= compressive stress due to prestress, at the extreme fibre where cracking occurs (see Clause 8.2.7.2)

σ_{cs}	= maximum shrinkage-induced tensile stress on the uncracked section at the extreme fibre at which cracking occurs (see Clause 8.5.3.1)
σ_o	= a constant sustained stress (see Clause 3.1.8.1)
σ_{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)
σ_{pi}	= stress in the tendon immediately after transfer
σ_{pj}	= stress in the tendon at the jacking end (see Clause 3.4.2.4)
σ_{pu}	= maximum stress that would be reached in a tendon at ultimate strength of a flexural member
σ_{sc}	= a compressive stress being developed in a bar in compression (see Clause 13.1.5.4)
σ_{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)
$\sigma_{scr,1}$	= 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)
σ_{st}	= tensile stress in reinforcement (see Clause 13.1.2.4)
τ^*	= design shear stress acting on the interface (see Clause 8.4.2)
τ_u	= unit shear strength (see Clause 8.4.3)
ϕ	= capacity reduction factor for design using linear elastic analysis (see Clause 2.2.2)
ϕ_s	= stress reduction factor for design using linear stress analysis (see Clause 2.2.3)
ϕ_{st}	= strength reduction factor for design using strut-and-tie analysis (see Clause 2.2.4)
ϕ_{sys}	= system strength reduction factor for design using non-linear methods of analysis (see Clauses 2.2.5 and 2.2.6)
ϕ_{cc}	= design creep coefficient at any time t (see Clause 3.1.8.3)
ϕ_{cc}^*	= final design creep coefficient (see Clause 3.1.8.3)
$\phi_{cc,b}$	= basic creep coefficient of concrete, determined in accordance with Clause 3.1.8.2
ψ_a	= factor for reduction of imposed (live) floor loads due to area (refer to AS/NZS 1170.1)
ψ_c	= combination imposed action (live load) factor used in assessing the design load for strength (refer to AS/NZS 1170.0)
ψ_s	= short-term imposed action (live load) factor used in assessing the design load for serviceability (refer to AS/NZS 1170.0)
ψ_l	= long-term imposed action (live load) factor used in assessing the design load for serviceability (refer to AS/NZS 1170.0)
Ω	= dimension of node [see Figure 7.2.4(A)]

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 comply with that Standard, this Standard and the provisions of Appendix C of this Standard.

Values for the structural ductility factor (μ) and the structural performance factor (S_p) for concrete members and structures complying with this Standard shall be as given in Paragraph C3 of Appendix C.

Structures designed assuming a structural ductility factor (μ) greater than 3 fall outside the scope of AS 1170.4.

NOTE: AS 1170.4 suggests in this situation that structures be designed in accordance with NZS 1170.5. In this situation, concrete structures should be designed and detailed in accordance with NZS 3101 and this will involve the use of Ductility Class E reinforcement.

2.1.3 Design for robustness

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

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 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 ϕR_u)

E_d = design action effect

- (ii) The design capacity, $R_d = \phi R_u$, shall be obtained using the appropriate capacity reduction factor (ϕ), 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 (ϕ)

Type of action effect	Capacity reduction factor (ϕ)
(a) Axial force without bending:	
(i) Tension	0.8
(ii) Compression	0.6
(b) Bending without axial tension or compression—	
(i) for members with Class N reinforcement only	$0.6 \leq (1.19 - 13k_{uo}/12) \leq 0.8$
(ii) for members with Class L reinforcement	$0.6 \leq (1.19 - 13k_{uo}/12) \leq 0.64$
(c) Bending with axial tension	$\phi + [(0.8 - \phi)(N_u/N_{uot})]$ and ϕ is obtained from Item (b)
(d) Bending with axial compression, where—	
(i) $N_u \geq N_{ub}$	0.6
(ii) $N_u < N_{ub}$	$0.6 + [(\phi - 0.6)(1 - N_u/N_{ub})]$ and ϕ is obtained from Item (b)
(e) Shear	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

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:

$$\phi_s \beta 0.9f'_c$$

... 2.2.3

where

ϕ_s = stress reduction factor with values taken from Table 2.2.3
 β = 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, β shall be evaluated by rational calculation taking account of the amount of confining steel and the details used, but shall not exceed 2.
- (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 (ϕ_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 (ϕ_s)

Material	Stress reduction factor (ϕ_s)
Concrete in compression	0.6
Steel in tension	
Class N	0.8
Class L	0.64
Tendons	0.8

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 (ϕ_{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 (ϕ_{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 (ϕ_{st}) shall be in accordance with Table 2.2.4.

TABLE 2.2.4
STRENGTH REDUCTION FACTORS (ϕ_{st}) FOR
DESIGN USING STRUT-AND-TIE ANALYSIS

Material	Strength reduction factor (ϕ_{st})
Concrete in compression	0.6
Steel in tension	0.8

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 (ϕ_{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 (ϕ_{sys})
(For application with Clauses 2.2.5 and 2.2.6)

Type of failure	System strength reduction factor (ϕ_{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 (ϕ_{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 (Δ/L_{ef}) for spans (Notes 1 and 2)	Deflection limitation (Δ/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 Δ/L_{ef} given in this Table apply only if the rotation at the support is included in the calculation of Δ .
- 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, 9.4.1, 9.4.2, 9.4.4 or 9.4.5, as appropriate.
- (b) Cracking caused by shrinkage and temperature in concrete slabs shall be controlled in accordance with Clause 9.4.3.

- (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 ACTIONS AND COMBINATIONS OF ACTIONS

2.4.1 Actions and loads

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

2.4.2 Combinations of actions and loads

The combinations of actions, loads and forces used in the design shall be in accordance with AS/NZS 1170.0.

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 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.

2.4.3 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.

2.4.4 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.

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 (ρ) 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/m³.

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 (ν) 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 (ϵ_{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 (ϵ_{cs}) is to be calculated, it shall be determined as the sum of the chemical (autogenous) shrinkage strain (ϵ_{cse}) and the drying shrinkage strain (ϵ_{csd})—

$$\epsilon_{cs} = \epsilon_{cse} + \epsilon_{csd}$$

... 3.1.7.2(1)

The autogenous shrinkage strain shall be taken as—

$$\varepsilon_{\text{cse}} = \varepsilon_{\text{cse}}^* \times (1.0 - e^{-0.1t}) \quad \dots 3.1.7.2(2)$$

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

$$\varepsilon_{\text{cse}}^* = (0.06f'_c - 1.0) \times 50 \times 10^{-6} \quad \dots 3.1.7.2(3)$$

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

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

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 ($\varepsilon_{\text{csd.b}}$) is given by—

$$\varepsilon_{\text{csd.b}} = (1.0 - 0.008f'_c) \times \varepsilon_{\text{csd.b}}^* \quad \dots 3.1.7.2(5)$$

where the final drying basic shrinkage strain ($\varepsilon_{\text{csd.b}}^*$) depends on the quality of the local aggregates and shall be taken as 800×10^{-6} for Sydney and Brisbane, 900×10^{-6} for Melbourne and 1000×10^{-6} elsewhere.

Based on a value of $\varepsilon_{\text{csd.b}}^* = 1000 \times 10^{-6}$, this method gives the typical design shrinkage strains shown in Table 3.1.7.2.

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 ε_{cs} has a range of $\pm 30\%$.

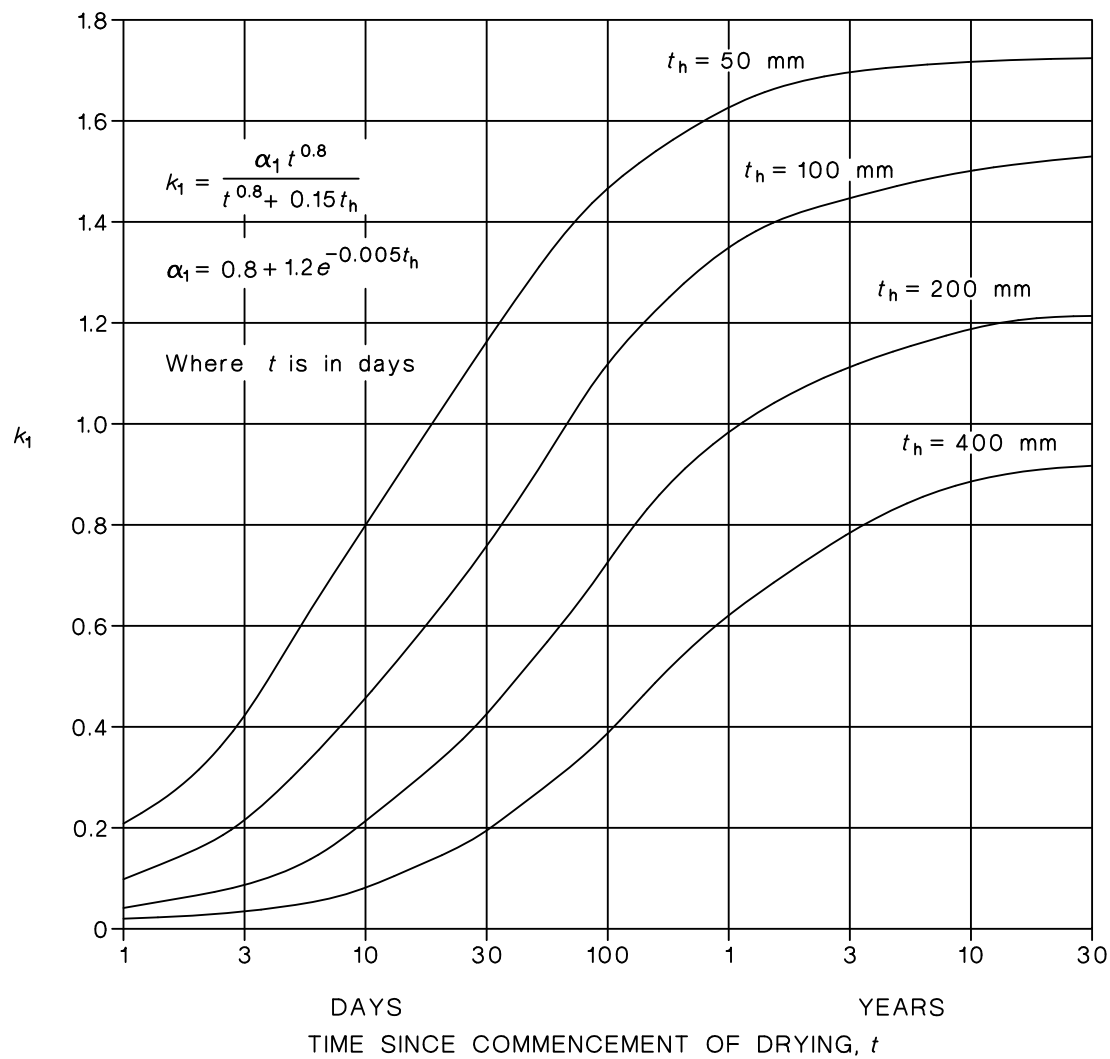


FIGURE 3.1.7.2 SHRINKAGE STRAIN COEFFICIENT (k_1) FOR VARIOUS VALUES OF t_h

TABLE 3.1.7.2
TYPICAL FINAL DESIGN SHRINKAGE STRAINS AFTER 30 YEARS

f'_c (MPa)	Final design shrinkage strain ε_{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	990	870	710	550	920	810	660	510	850	750	610	470	720	630	510	400
32	950	840	680	530	880	780	640	500	820	720	590	460	690	610	500	390
40	890	790	650	510	830	740	610	480	780	690	570	450	660	590	490	390
50	830	740	610	490	770	690	580	460	720	650	540	440	620	550	470	380
65	730	650	560	460	680	620	530	440	640	580	500	410	560	510	440	370
80	630	570	500	420	590	540	480	410	560	520	450	390	500	460	410	360
100	490	460	420	380	480	450	410	370	460	430	400	360	420	400	370	340

3.1.8 Creep

3.1.8.1 General

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

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

where

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

φ_{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—

- (a) determined from measurements on similar local concrete; or
- (b) determined by tests in accordance with AS 1012.16; or
- (c) 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 , (φ_{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, φ_{cc} at any time shall be taken as—

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

where k_2 and k_3 are obtained from Figure 3.1.8.3(A) and Figure 3.1.8.3(B) respectively; $k_4 = 0.70$ for an arid environment, 0.65 for an interior environment, 0.60 for a temperate inland environment and 0.50 for a tropical or near-coastal environment; k_5 is a modification factor for high strength concrete and shall be taken as—

$$k_5 = 1.0 \quad \text{when } f'_c \leq 50 \text{ MPa; or}$$

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

the factor $\alpha_3 = 0.7/(k_4\alpha_2)$; and α_2 is defined in Figure 3.1.8.3(A).

Consideration shall be given to the fact that φ_{cc} has a range of approximately $\pm 30\%$. This range is likely to be exceeded if—

- (a) the concrete member is subjected to prolonged periods of temperature in excess of 25°C ; or
- (b) the member is subject to sustained stress levels in excess of $0.5f'_c$.

The final design creep coefficients (ϕ_{cc}^*) (after 30 years) predicted by this method for concrete first loaded at 28 days are given in Table 3.1.8.3.

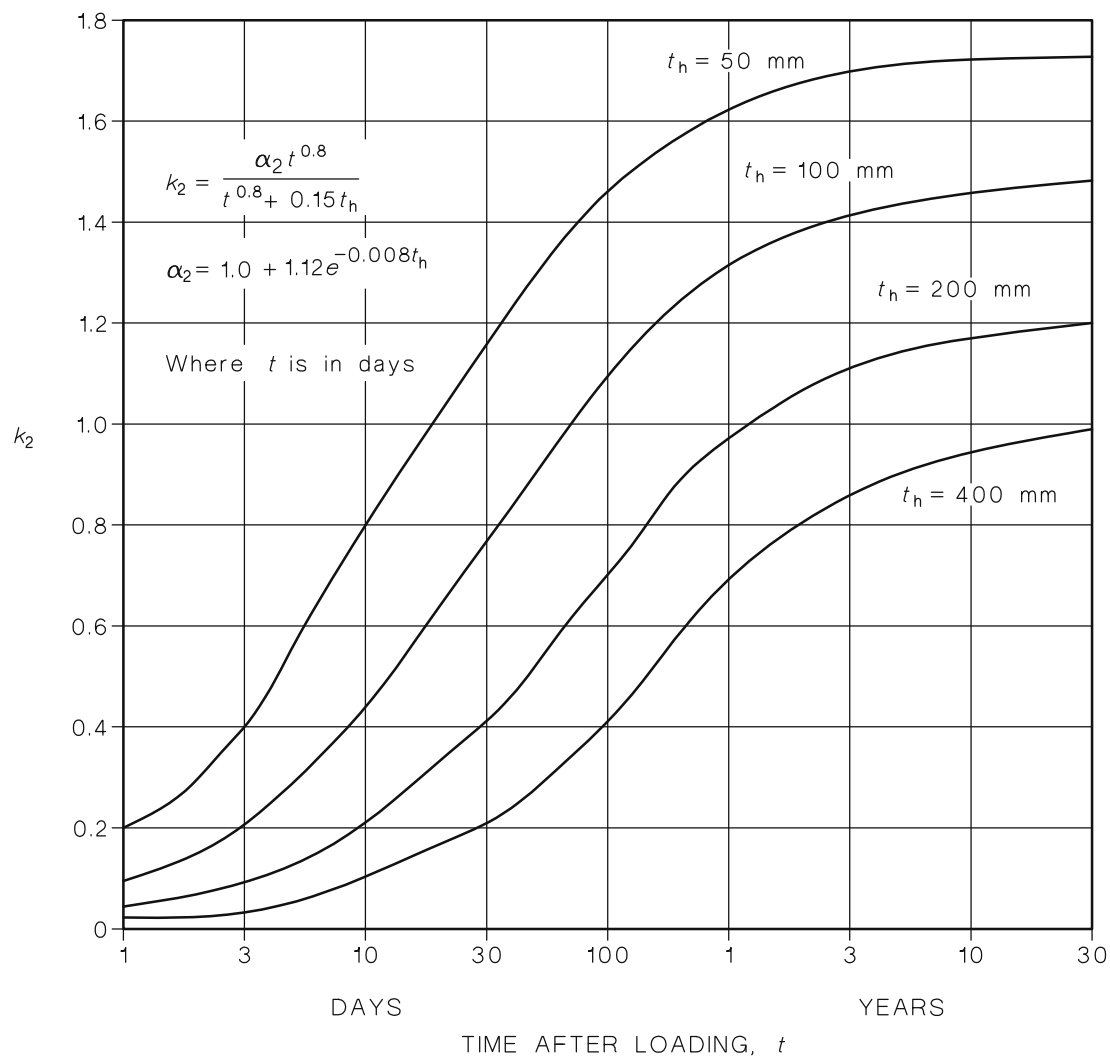


FIGURE 3.1.8.3(A) COEFFICIENT (k_2)

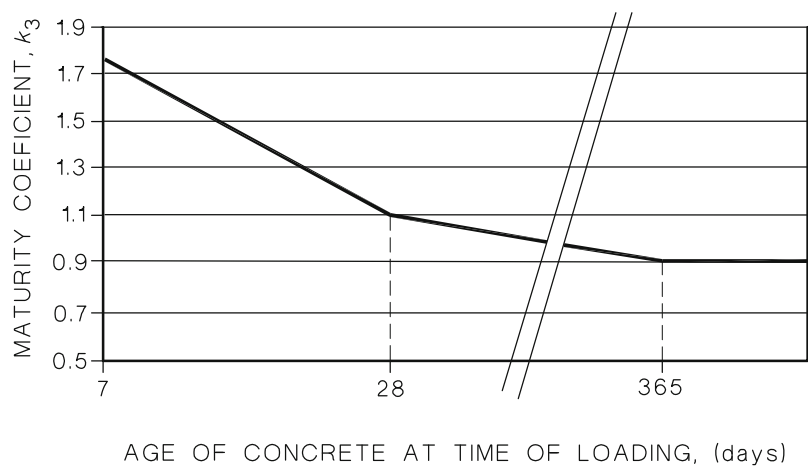


FIGURE 3.1.8.3(B) MATURITY COEFFICIENT (k_3)

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

f'_c (MPa)	Final creep coefficient (ϕ_{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 (ε_{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 comply with AS/NZS 4671.

NOTE: In AS/NZS 4671, ε_{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	Uniform strain (ε_{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 wire mesh, plain, deformed or indented to AS/NZS 4671	D500L	500	0.015	L
	D500N	500	0.05	N

NOTE: Reference should be made to AS/NZS 4671 for explanation to designations applying to 500 MPa steels.

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 200×10^3 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 ²	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	140	250	1790
	15.2	143	261	1830
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 ±10 GPa;
- (ii) for stress-relieved steel wire, to AS/NZS 4672.1 205 ±10 GPa;
- (iii) for stress-relieved steel strand, to AS/NZS 4672.1 200 ±5 GPa; or
- (iv) for hot-rolled high tensile alloy steel bars, to AS/NZS 4672.1 ... 205 ±10 GPa; 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 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—

$$R = k_4 k_5 k_6 R_b$$

... 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
- sk_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.

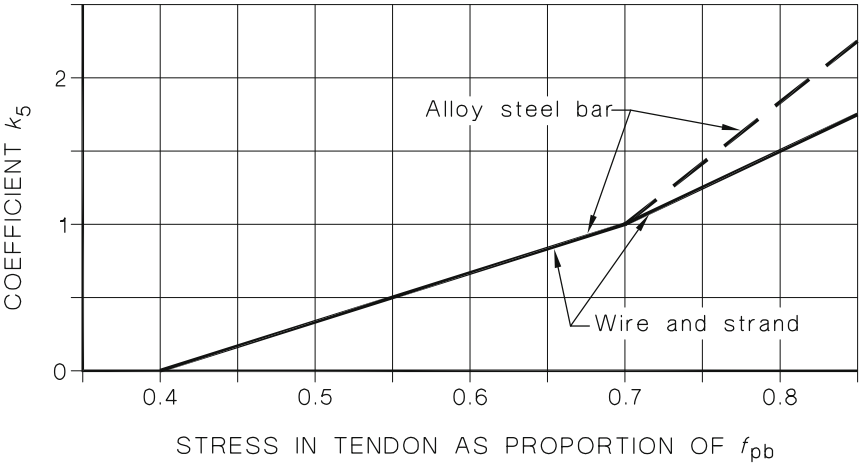


FIGURE 3.3.4.3 COEFFICIENT k_5

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 (σ_{pa}) shall be taken as—

$$\sigma_{pa} = \sigma_{pj} e^{-\mu(\alpha_{tot} + \beta_p L_{pa})} \quad \dots 3.4.2.4$$

where

σ_{pj} = stress in the tendon at the jacking end

e = base of Napierian logarithms

μ = 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

α_{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})

β_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—

- (i) for sheathing containing tendons other than bars and having an internal diameter—
 - ≤ 50 mm: 0.024 to 0.016 rad/m
 - > 50 mm but ≤ 90 mm: 0.016 to 0.012 rad/m
 - > 90 mm but ≤ 140 mm: 0.012 to 0.008 rad/m;
- (ii) for flat metal ducts containing tendons other than bars: 0.024 rad/m to 0.016 rad/m;
- (iii) for sheathing containing bars and having an internal diameter of 50 mm or less: 0.016 rad/m to 0.008 rad/m; and
- (iv) 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 $E_p \varepsilon_{cs}$, 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 ε_{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:

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

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 $E_p \varepsilon_{cc}$, in which ε_{cc} is given by—

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

where

φ_{cc} = design creep coefficient calculated in accordance with Clause 3.1.8.3

σ_{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

σ_{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 Clause 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 less than 50 years (e.g., temporary structures).
- 2 Durability is a complex topic and compliance 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, complying 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: <ul style="list-style-type: none"> (a) Members protected by a damp-proof membrane (b) Residential footings in non-aggressive soils (c) Other members in non-aggressive soils (d) Members in aggressive soils: <ul style="list-style-type: none"> (i) Sulfate bearing (magnesium content <1g/L) (ii) Sulfate bearing (magnesium content ≥1g/L) (see Note 2) (iii) Other (e) Salt rich soils and soils in areas affected by salinity 	A1 A1 A2 See Table 4.8.1 U U See Table 4.8.2
2 Surfaces of members in interior environments: <ul style="list-style-type: none"> (a) Fully enclosed within a building except for a brief period of weather exposure during construction: <ul style="list-style-type: none"> (i) Residential (ii) Non-residential (b) In industrial buildings, the member being subject to repeated wetting and drying 	A1 A2 B1
3 Surfaces of members in above-ground exterior environments in areas that are: <ul style="list-style-type: none"> (a) Inland (>50 km from coastline) environment being: <ul style="list-style-type: none"> (i) Non-industrial and arid climatic zone (see Note 3) (ii) Non-industrial and temperate climatic zone (iii) Non-industrial and tropical climatic zone (iv) Industrial (see Note 4) and any climatic zone (b) Near-coastal (1 km to 50 km from coastline), any climatic zone (c) Coastal (see Note 5) and any climatic zone 	A1 A2 B1 B1 B1 B2
4 Surfaces of members in water: <ul style="list-style-type: none"> (a) In freshwater (b) In soft or running water 	B1 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 (b) In spray zone (see Note 6) (c) In tidal/splash zone (see Note 7)	B2 C1 C2
6 Surfaces of members in other environments, i.e., any exposure environment not specified in Items 1 to 5 above (see Note 8)	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.
- 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 The spray zone is the zone from 1 m above wave crest level.
- 7 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.
- 8 Further guidance on measures appropriate in exposure classification U may be obtained from AS 3735.
- 9 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.

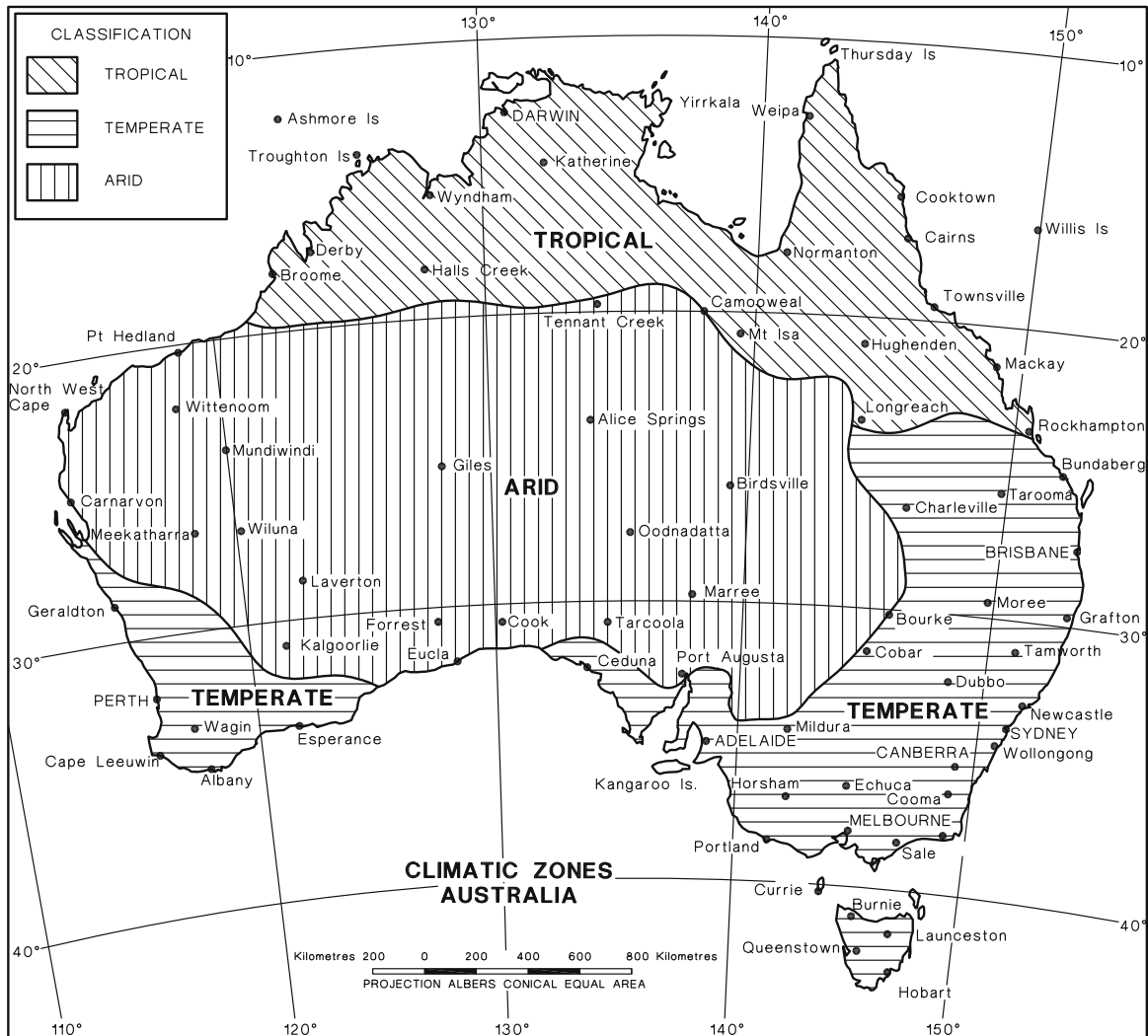


FIGURE 4.3 CLIMATIC ZONES REFERRED TO IN TABLE 4.3

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 ₄)*		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 SO₄ = 80 ppm SO₃
† 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 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 under ground 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 _e)	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_e 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 comply with AS 1478.1 and chemical content in concrete shall comply 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 complying 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 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 or rolling concrete, the cover for the corrosion protection shall be as specified in the appropriate Standard.

NOTE: For example, refer to AS 4058 and AS/NZS 4065.

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—

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

... 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.

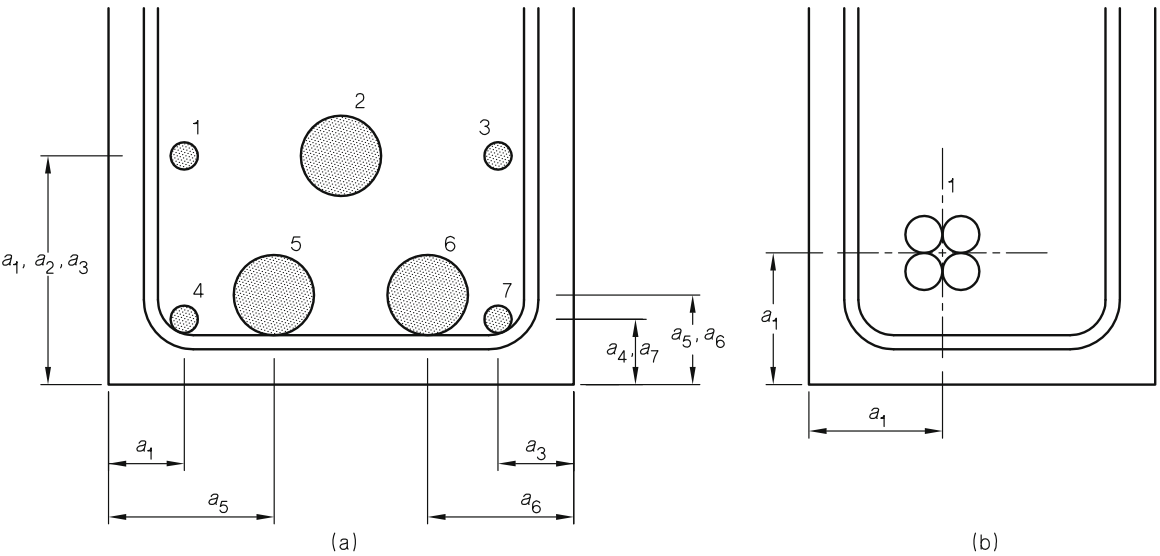
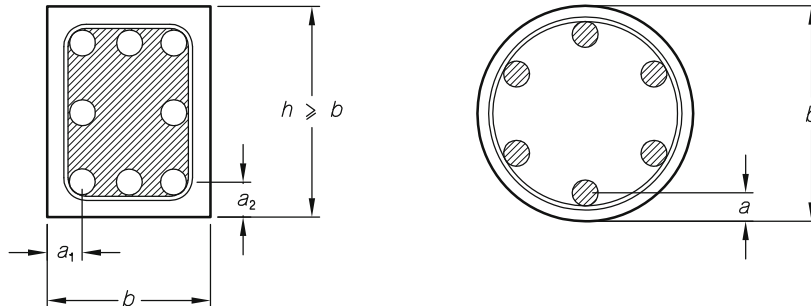


FIGURE 5.2.1 DIMENSIONS USED TO CALCULATE AVERAGE AXIS DISTANCE

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 distances are nominal values and no allowance for tolerance need be added.

FIGURE 5.2.2 SECTIONS THROUGH STRUCTURAL MEMBERS
SHOWING AXIS DISTANCE, a

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, e.g., 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 criterion (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 reinforcing bars shown in the Tables and Figures in this Section shall be increased by 10 mm where prestressing tendons are used.

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 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 complying 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.

A1

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.

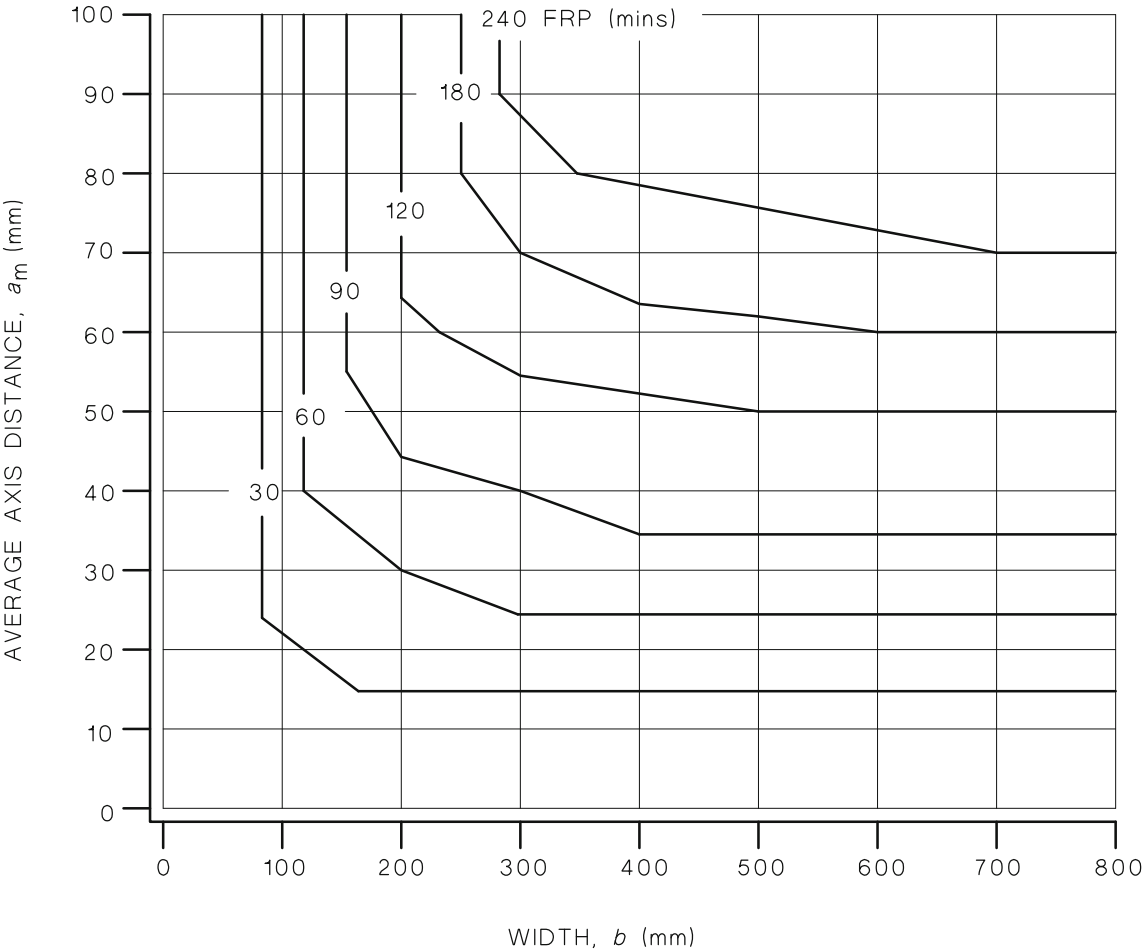


FIGURE 5.4.1(A) FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
FOR SIMPLY SUPPORTED BEAMS

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.

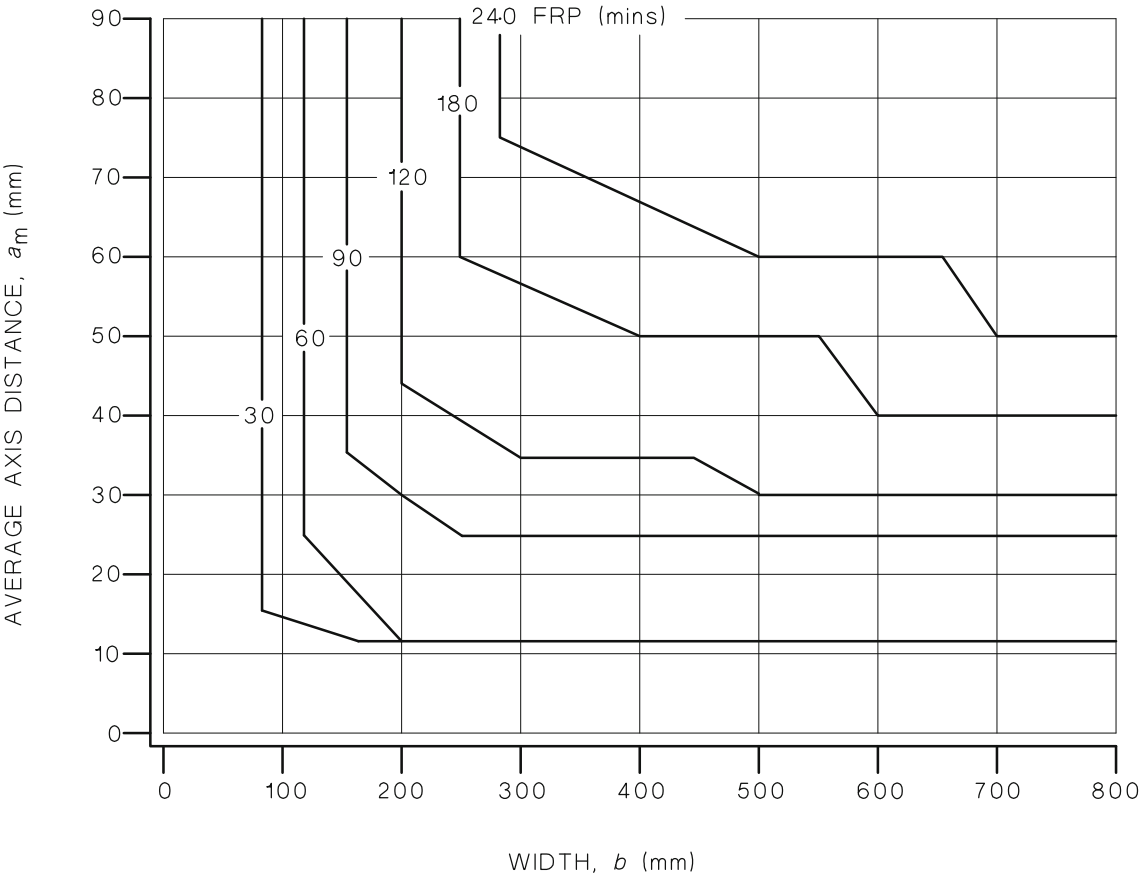


FIGURE 5.4.1(B) FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY FOR CONTINUOUS BEAMS

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 given by—

- (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)], if, for the appropriate support conditions, the average axis distance to the bottom layer of reinforcement and tendons from the soffit of the rib is not less than the value given in the Table—and
 - (i) ‘Text deleted’

A1

- (ii) at least 20% of the total top reinforcement in each direction over intermediate supports is continuous over the full span and placed in the column strip,
- (c) 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 comply 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:

- 1 l_y = longer span of a two-way slab
 l_x = shorter span of a two-way slab
- 2 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.
- 3 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:
- 1 a_s = axis distance to the reinforcement in the lowest layer.
 - 2 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	≥200	10	80
90	45	120	40	160	30	≥250	15	100
120	60	160	55	190	40	≥300	20	120
180	75	220	70	260	60	≥410	30	150
240	90	280	75	350	70	≥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)							
	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	h_s
	a_s	b	a_s	b	a_s	b		
30	10	80	—	—	—	—	10	80
60	25	100	15	120	10	≥ 200	10	80
90	35	120	25	160	15	≥ 250	15	100
120	45	160	40	190	30	≥ 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 comply with the appropriate criteria for walls given in Clause 5.7.1.

5.6.2 Structural adequacy for columns

The fire resistance period (FRP) for structural adequacy for a column shall be determined using Clause 5.6.3, except—

- (a) for columns in a braced structure where Clause 5.6.4 may be used; and
- (b) where the ratio of the longer cross-section dimension of the column is equal to or greater than 4 times the shorter cross-section dimension, Clause 5.7.2 may be used.

In the situation of Item (b), 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 Calculation method to determine structural adequacy for columns

The FRP for structural adequacy for a column shall be as given by Table 5.6.3 provided the column is proportioned so that the value for the smaller cross-sectional dimension and the average axis distance to the longitudinal reinforcement are not less than the values for that period.

The value of the load level shall be taken as 0.7 or calculated as follows:

$$\frac{N_f^*}{N_u} \quad \dots 5.6.3$$

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

Where $A_s \geq 0.02A_c$ and the required FRP is greater than 90 min, the bars shall be distributed along all the faces of the column.

Dimension b in Table 5.6.3 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 be not nearer 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.

When using Table 5.6.3 the effective length of the column under fire conditions shall be less than 3 m and the maximum eccentricity shall be limited to $0.15b$. For columns where the effective length falls outside these limits, reference shall be made to alternative design approaches noted 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 min, 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 (min)	Minimum dimensions (mm)							
	Combinations for column exposed on more than one side						Column exposed on one side	
	$\frac{N_f^*}{N_u} = 0.2$		$\frac{N_f^*}{N_u} = 0.5$		$\frac{N_f^*}{N_u} = 0.7$		$\frac{N_f^*}{N_u} = 0.7$	
	a_s	b	a_s	b	a_s	b	a_s	b
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	350 450 ³	25	155
120	40 35	250 350	45 ³ 40 ³	350 ³ 450 ³	57 ³ 51 ³	350 ³ 450 ³	35	175
180	45 ³	350 ³	63 ³	350 ³	70 ³	450 ³	55	230
240	61 ³	350 ³	75 ³	450 ³			70	295

NOTES:

- 1 a_s = axis distance
- b = smaller cross-sectional dimension of a rectangular column or the diameter of a circular column
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.
- 3 These combinations for columns with a minimum of 8 bars.

5.6.4 Alternative method to determine structural adequacy for braced columns

The FRP for structural adequacy for a column given in Table 5.6.4 shall be used, provided—

- (a) the column is in a braced structure; and

- (b) 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.

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

FRP for structural adequacy (min)	Reinforcement ratio (p)	Minimum dimensions (mm)							
		Combinations of (a_s) and (b)							
		$\eta = 0.2$		$\eta = 0.3$		$\eta = 0.5$		$\eta = 0.7$	
		a_s	b	a_s	b	a_s	b	a_s	b
30	0.01	25	150	25	150	30	200	30	300
						25	250	25	350
	0.05	25	150	25	150	25	150	30	200
								25	250
	0.10	25	150	25	150	25	150	30	200
								25	300
60	0.01	30	150	40	200	40	300	25	500
		25	200	25	300	25	500		
	0.05	25	150	35	150	35	250	40	350
				25	200	25	350	25	550
	0.10	25	150	30	150	40	200	50	300
				25	200	25	400	30	600
90	0.01	40	200	40	300	50	500	40	550
		25	250	25	400	25	550	25	600
	0.05	35	150	45	200	45	300	50	500
		25	200	25	300	25	550	40	600
	0.10	25	200	40	200	40	250	50	500
				25	300	25	550	45	600
120	0.01	50	250	50	400	25	550	60	550
		25	350	25	550			45	600
	0.05	45	200	45	300	50	450	60	500
		25	300	25	550	25	600	50	600
	0.10	40	200	50	250	45	450	60	600
		25	250	25	400	30	600		
180	0.01	50	400	60	500	60	550	Note 3	
		25	500	25	550	30	600		
	0.05	45	300	50	450	60	500	75	600
		25	450	25	600	50	600		
	0.10	35	300	50	450	60	500	Note 3	
		25	400	25	550	45	600		

(continued)

5.7.2 Structural adequacy for walls

The FRP for structural adequacy for a wall given in Table 5.7.2 shall be used, provided the effective thickness of the wall is not less than the corresponding value 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 then structural adequacy will be considered to be achieved by satisfying 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							
	$\frac{N_f^*}{N_u} = 0.35$				$\frac{N_f^*}{N_u} = 0.7$			
	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

NOTE: For prestressing tendons, the axis distance shall be increased as given 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 m² of wall face, is not more than 10 000 mm² 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 mm²; and
 - (iii) the total face area of chases within any 5 m² of wall face is not greater than 100 000 mm² 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 1 part cement to 4 parts vermiculite (by volume) concrete or of 1 part cement to 4 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 m³ 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_{nom} = 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 complying 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 complying 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° 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)].

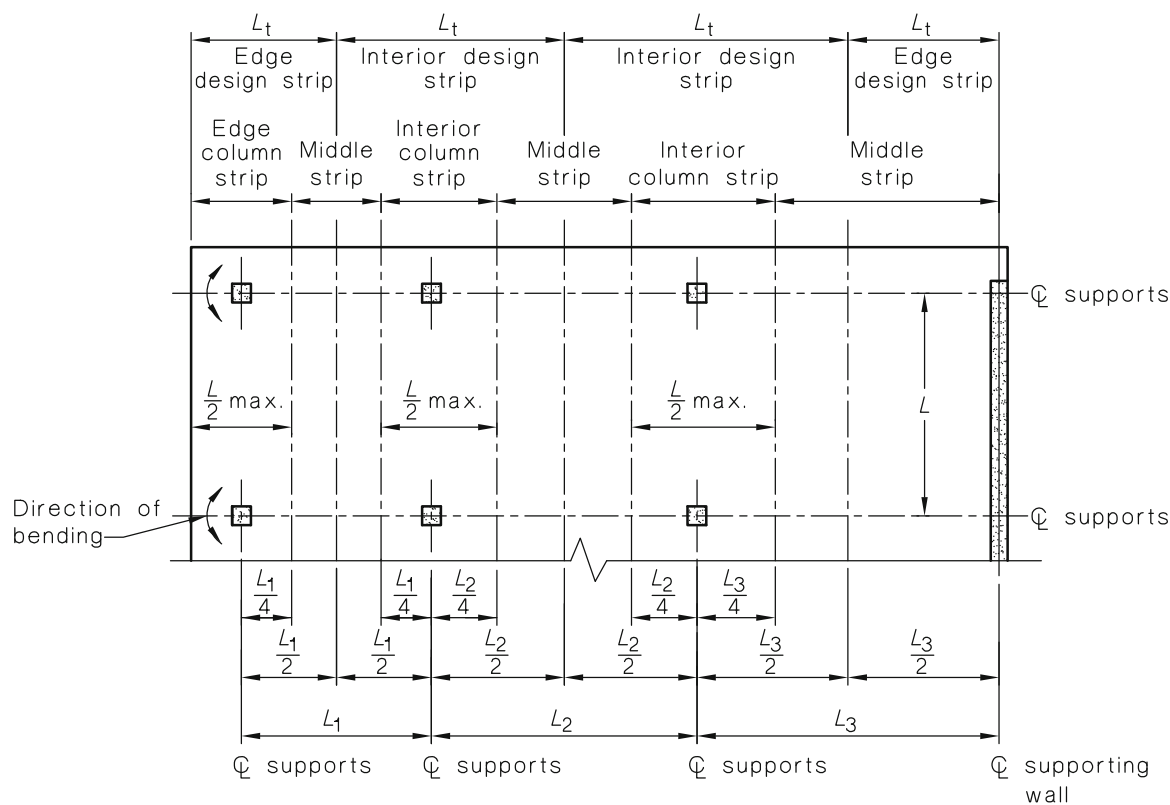


FIGURE 6.1.4(A) WIDTHS OF STRIPS FOR TWO-WAY SLAB SYSTEMS

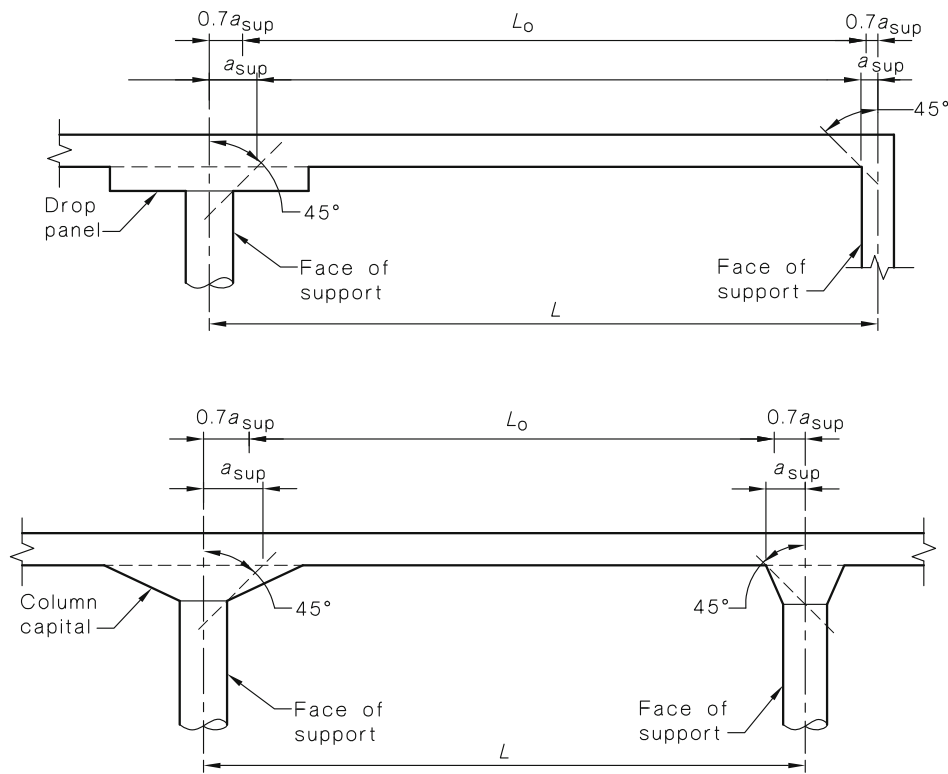


FIGURE 6.1.4(B) SPAN SUPPORT AND SPAN LENGTHS FOR FLAT SLABS

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

The stiffnesses of members shall be chosen to represent the conditions at the limit state being analysed.

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.

6.2.5 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 ε_{su} and ε_{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-comply 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 comply 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 may be taken as $0.4E_cI_f$ and $0.8E_cI_c$ respectively.
- (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 comply 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 comply 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 2.4.4. 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.

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 complying 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_t); 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 8.3.7.

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 comply 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 β_x and β_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 comply 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) α_x or α_y times the midspan values in the direction considered when they are taken from Table 6.10.3.2(B), where the value of α_x or α_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 (β_x)								Long span coefficients (β_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	
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 (β_x and α_x)									Long span coefficients (β_y and α_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	β	0.021	0.025	0.029	0.032	0.034	0.036	0.039	0.041	0.042	0.020
	α	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	β	0.027	0.030	0.033	0.035	0.037	0.039	0.041	0.042	0.042	0.024
	α	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	β	0.024	0.028	0.034	0.038	0.043	0.047	0.056	0.061	0.070	0.028
	α	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	β	0.032	0.035	0.037	0.038	0.039	0.040	0.042	0.042	0.042	0.024
	α	2.09	2.05	2.03	2.01	2.00	2.00	2.00	2.00	2.00	—
5 Two long edges discontinuous	β	0.024	0.028	0.035	0.042	0.049	0.056	0.071	0.085	0.125	0.039
	α	—	—	—	—	—	—	—	—	—	2.31
6 Two adjacent edges discontinuous	β	0.031	0.036	0.041	0.046	0.050	0.053	0.060	0.064	0.070	0.034
	α	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)	β	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.066	0.070	0.035
	α	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)	β	0.033	0.039	0.047	0.054	0.061	0.067	0.082	0.093	0.125	0.046
	α	—	—	—	—	—	—	—	—	—	2.12
9 Four edges discontinuous	β	0.044	0.052	0.059	0.066	0.073	0.079	0.091	0.100	0.125	0.049
	α	—	—	—	—	—	—	—	—	—	—

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 complying 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—

- (a) the reactions apply directly when all edges are continuous;
- (b) 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
- (c) when adjacent edges are discontinuous, the reactions are adjusted for elastic shear considering each span separately.

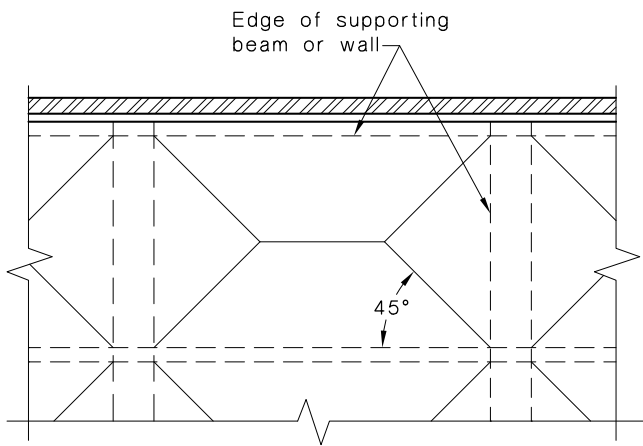


FIGURE 6.10.3.4 ALLOCATION OF LOAD

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 8.1.10.6, 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—

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

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—

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

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 comply with the requirements of Clauses 6.9.5.5(a) and 6.9.5.5(b) shall be permitted in slabs.

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:

- (a) Loads shall be applied at nodes, and the struts and ties shall be subjected only to axial force.
- (b) The model shall provide load paths to carry the loads and other actions to the supports or into adjacent regions.
- (c) The model shall be in equilibrium with the applied loads and the reactions.
- (d) In determining the geometry of the model, the dimensions of the struts, ties, and nodal zones shall be taken into account.
- (e) Ties shall be permitted to cross struts.
- (f) Struts shall cross or intersect only at nodes.
- (g) 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°.
- (h) 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 (β_s) that is used to determine the design strength shall be taken as 1.0.

For fan- 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 (θ) 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 (θ) is different for nodes at each end of a strut, the smallest value of θ shall be used in determining β_s .

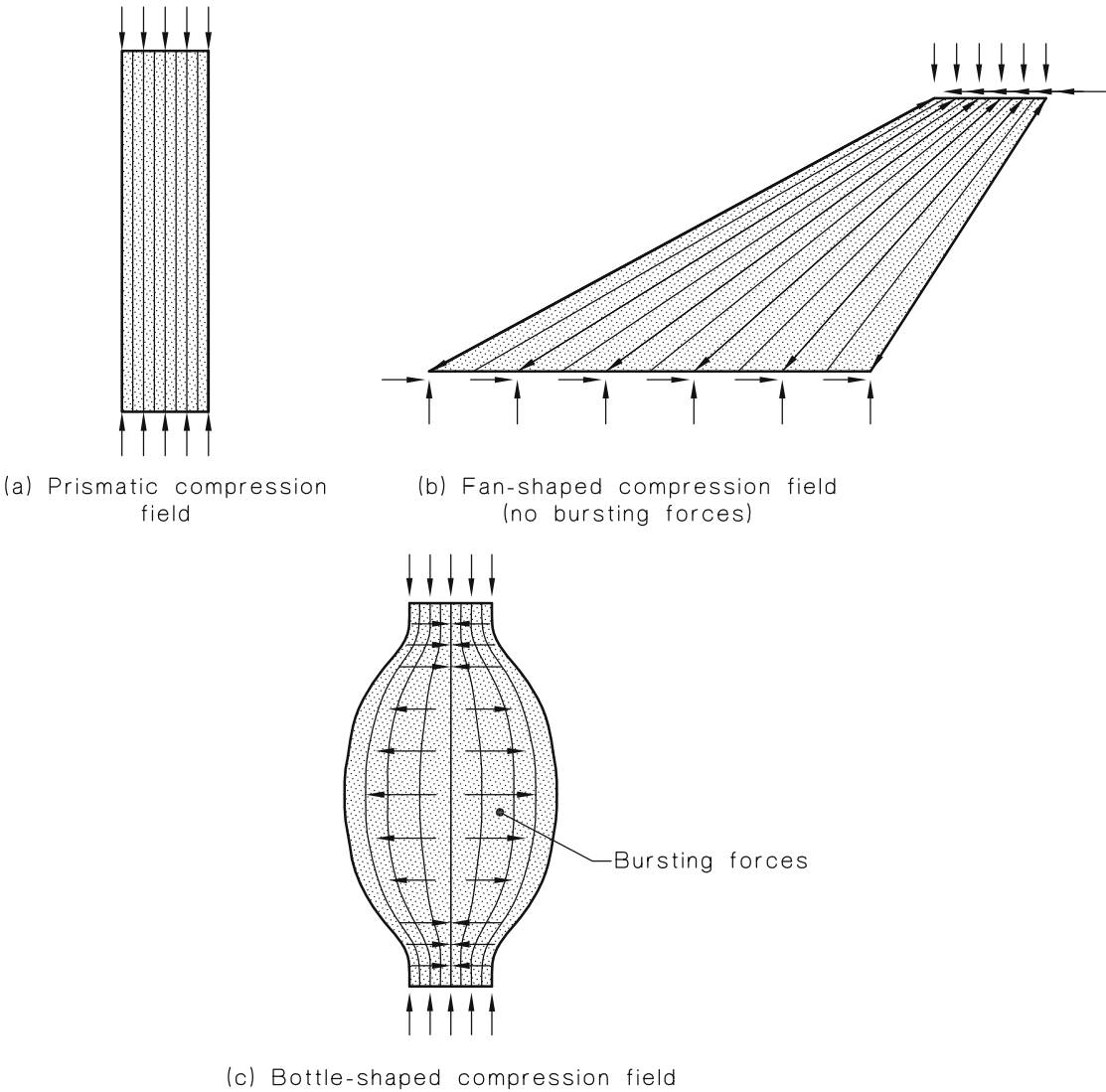
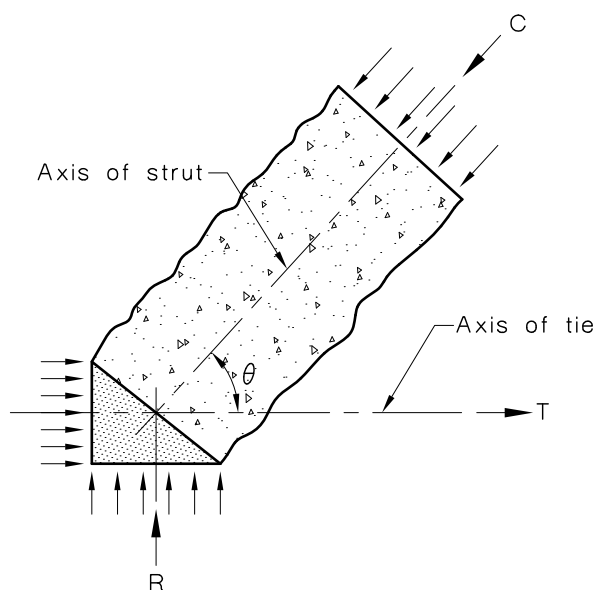


FIGURE 7.2.1 TYPES OF STRUTS

FIGURE 7.2.2 DEFINITION OF θ

7.2.3 Design strength of struts

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

$$\phi_{st} \beta_s 0.9 f'_c A_c \quad \dots 7.2.3$$

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

β_s = an efficiency factor given in Clause 7.2.2

The value of the strength reduction factor (ϕ_{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 (α) 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 $0.5T_{b,cr}$, then transverse reinforcement shall be provided in either—

- (i) two orthogonal directions at angles γ_1 and γ_2 to the axis of the strut [see Figure 7.2.4(B)]; or
- (ii) one direction at an angle γ_1 to the axis of the strut, where γ_1 shall be not less than 40° and shall satisfy the following—

(A) for serviceability

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

(B) for strength

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

In the above expressions, A_{si} is the area of reinforcement in directions 1 and 2 crossing a strut at an angle γ_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)].

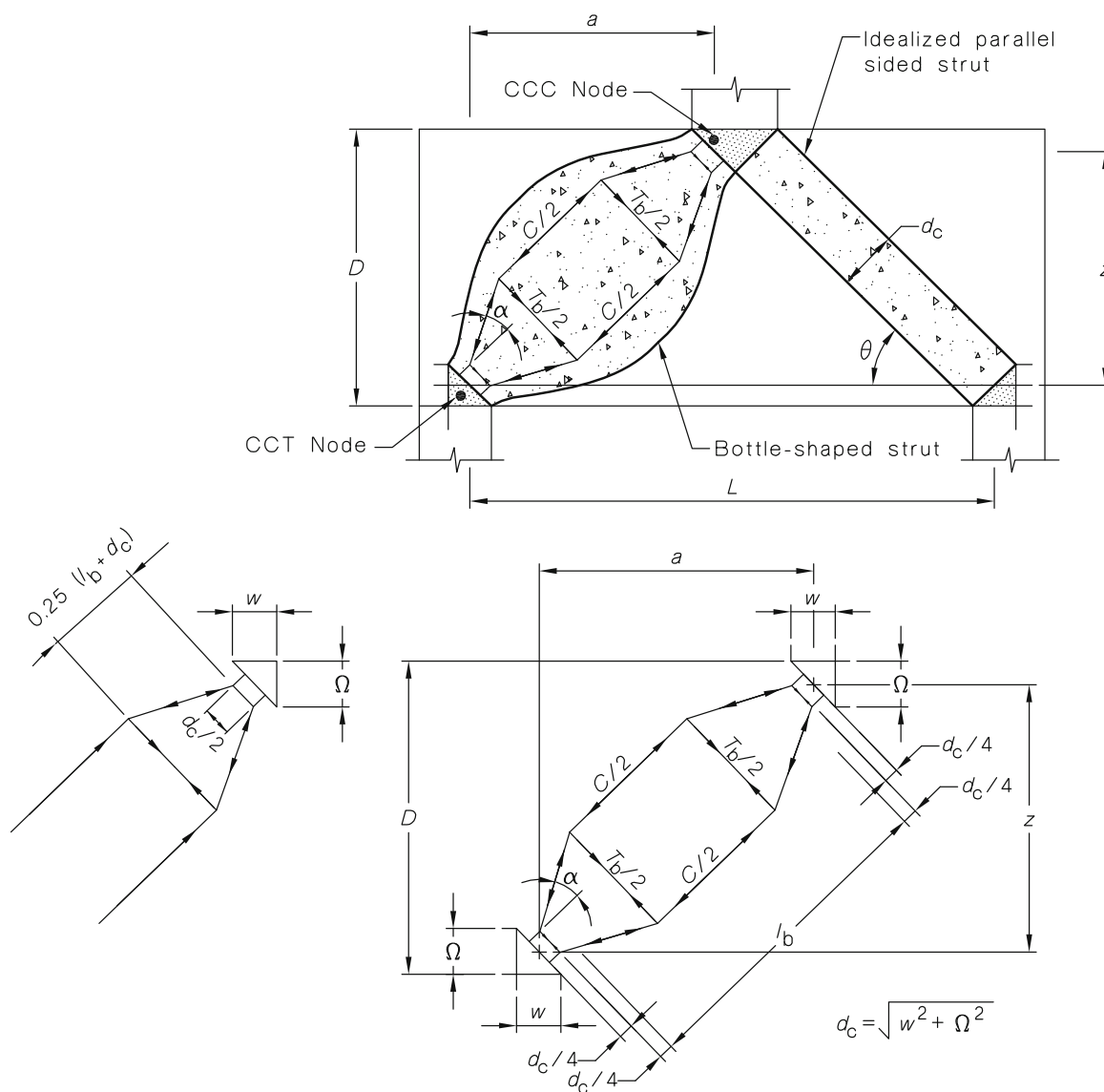


FIGURE 7.2.4(A) MODEL OF BURSTING FORCES IN BOTTLE-SHAPED STRUTS

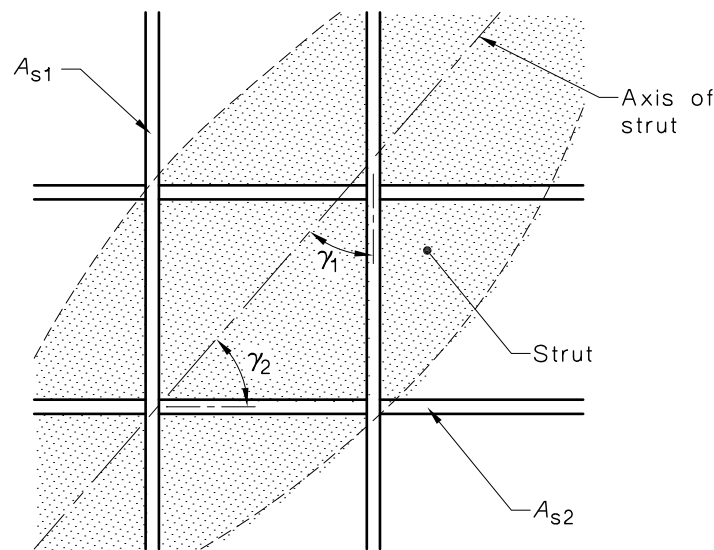


FIGURE 7.2.4(B) BURSTING REINFORCEMENT

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 ϕ_{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 (ϕ_{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 $\phi_{st} 1.8 f'_c$.

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 $\gamma k_u d$ from the extreme compressive fibre, where—

$$\alpha_2 = 1.0 - 0.003 f'_c \quad (\text{within the limits of } 0.67 \leq \alpha_2 \leq 0.85) \quad \dots 8.1.3(1)$$

$$\gamma = 1.05 - 0.007 f'_c \quad (\text{within the limits } 0.67 \leq \gamma \leq 0.85) \quad \dots 8.1.3(2)$$

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° (i.e., 30° 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 ϕM_{uo} , where ϕ is determined from Item (b) of Table 2.2.2.

A1

A1 | Sections with k_{uo} greater than 0.36 and $M^* > 0.6M_u$ shall be used only when—

- (a) the structural analysis is carried out in accordance with Clauses 6.2 to 6.6; and
- (b) compressive reinforcement of at least 0.01 times the area of concrete in compression is used and restrained as specified in Clauses 8.1.10.7 and 8.1.10.8, 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 (ϕ) 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 $0.5f_{cp}$ for a rectangular distribution of stress or $0.6f_{cp}$ for a triangular distribution of stress.

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 (σ_{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 (σ_{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 σ_{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(b).

8.1.10 Detailing of flexural reinforcement and tendons

8.1.10.1 General procedure for detailing reinforcement and tendons

The design for flexural strength and detailing of flexural reinforcement and pretensioned tendons at termination shall be extended from the theoretical cut-off point, or debonding point, by a length of $1.0D + L_{sy,t}$, or $1.0D + L_{pt}$, respectively, where D is the member depth at the theoretical cut-off point or debonding point.

Prestressing effects and the contribution to strength of post-tensioned tendons shall be disregarded over a length of the tendon from the point of termination equal to the development length (L_p).

Clauses 8.1.10.2 to 8.1.10.8 do not apply to tendons. Clause 8.1.10.9 applies to tendons only.

NOTE: When detailing flexural reinforcement, designers should be cautious in D-regions and design accordingly.

8.1.10.2 *Distribution of 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.

8.1.10.3 *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.

8.1.10.4 *Anchorage of positive moment reinforcement*

Anchorage of positive moment reinforcement shall comply 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 8.3.6, 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 (θ_v) is as defined in Clause 8.2.10, 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 $12d_b$ 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.

8.1.10.5 *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, $\phi V_u \geq 1.5 V^*$.
- (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.

8.1.10.6 *Deemed to comply arrangement of flexural reinforcement*

For continuous reinforced beams analysed using simplified methods of analysis, as detailed in Clause 6.10, the following deemed to comply arrangement shall be used and shall be deemed to satisfy the requirements of Clauses 8.1.10.3 to 8.1.10.5:

- (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 $12d_b$;
 - (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 comply with shear requirements, not more than a quarter of the maximum tensile reinforcement shall be terminated within any distance $2D$.

8.1.10.7 *Restraint of compressive reinforcement*

Compressive reinforcement required for strength in beams shall be adequately restrained by fitments in accordance with Clause 10.7.4.

8.1.10.8 *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.

8.1.10.9 *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

This Clause applies to reinforced and prestressed beams subjected to any combination of shear force, bending moment and axial force. When torsion acts in conjunction with shear force, the requirements given in Clause 8.3 also shall apply.

This Clause does not apply to non-flexural members covered by Section 12.

8.2.2 Design shear strength of a beam

The design shear strength of a beam shall be taken as ϕV_u where—

- (a) $V_u = V_{uc} + V_{us}$, taking account of Clauses 8.2.3 to 8.2.6, where V_{uc} is determined from Clause 8.2.7 and V_{us} is determined from Clauses 8.2.9 and 8.2.10; or
- (b) V_u is calculated by means of a method based on strut-and-tie modelling in accordance with Section 7; or
- (c) where design for shear and torsion interaction is required in accordance with Clause 8.3, V_{uc} shall be calculated in accordance with Clause 8.2.7.4.

8.2.3 Tapered members

In members that are tapered along their length, the components of inclined tension or compressive forces shall be taken into account in the calculation of shear strength.

8.2.4 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) diagonal cracking can not take place at the support or extend into it;
 - (ii) there are no concentrated loads closer than $2d_o$ from the face of the support;
 - (iii) the value of β_3 in Clauses 8.2.7.1 and 8.2.7.2 is taken to be equal to one; and
 - (iv) the transverse shear reinforcement required at d_o from the support is continued unchanged to the face of the support.

In both Items (a) and (b) above, longitudinal tensile reinforcement required at d_o from the face of the support shall be continued into the support and shall be fully anchored past that face.

8.2.5 Requirements for shear reinforcement

The following requirements for shear reinforcement shall apply:

- (a) Where $V^* \leq 0.5\phi V_{uc}$, no shear reinforcement is required, except that where the overall depth of the beam exceeds 750 mm, minimum shear reinforcement ($A_{sv,min}$) shall be provided in accordance with Clause 8.2.8.
- (b) Where $0.5\phi V_{uc} < V^* \leq \phi V_{u,min}$, minimum shear reinforcement ($A_{sv,min}$) shall be provided in accordance with Clause 8.2.8.

The minimum shear reinforcement requirements of Items (a) and (b) may be waived—

- (i) for beams, if $V^* \leq \phi V_{uc}$ and D does not exceed the greater of 250 mm and half the width of the web; and

(ii) for slabs to which this Clause applies, if $V^* \leq \phi V_{uc}$.

Where $V^* > \phi V_{u,min}$ shear reinforcement shall be provided in accordance with Clause 8.2.10.

8.2.6 Shear strength limited by web crushing

In no case shall the ultimate shear strength (V_u) be taken as greater than—

$$V_{u,max} = 0.2 f'_c b_v d_o + P_v \quad \dots 8.2.6$$

where

$$b_v = (b_w - 0.5 \Sigma d_d)$$

Σd_d = sum of the diameters of the grouted ducts, if any, in a horizontal plane across the web

P_v = vertical component of the prestressing force at the section under consideration

8.2.7 Shear strength of a beam excluding shear reinforcement

8.2.7.1 Reinforced beams

The ultimate shear strength (V_{uc}) of a reinforced beam, excluding the contribution of shear reinforcement, shall be calculated from the following equation:

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o f_{cv} \left(\frac{A_{st}}{b_v d_o} \right)^{1/3} \quad \dots 8.2.7.1$$

where

for members where the cross-sectional area of shear reinforcement provided (A_{sv}) is equal to or greater than the minimum area specified in Clause 8.2.8—

$$\beta_1 = 1.1(1.6 - d_o/1000) \geq 1.1$$

otherwise—

$$\beta_1 = 1.1(1.6 - d_o/1000) \geq 0.8$$

$\beta_2 = 1$, for members subject to pure bending; or

$$= 1 - (N^*/3.5A_g) \geq 0 \quad \text{for members subject to axial tension; or}$$

$$= 1 + (N^*/14A_g) \quad \text{for members subject to axial compression}$$

$\beta_3 = 1$, or may be taken as—

$2d_o/a_v$ but not greater than 2, provided the applied loads and the support are orientated so as to create diagonal compression over the length (a_v)

a_v = distance from the section at which shear is being considered to the face of the nearest support

$$f_{cv} = f'_c{}^{1/3} \leq 4 \text{ MPa}$$

A_{st} = cross-sectional area of longitudinal reinforcement provided in the tensile zone and fully anchored at the cross-section under consideration

8.2.7.2 Prestressed beams

The ultimate shear strength (V_{uc}) of a prestressed beam, excluding the contribution of shear reinforcement, shall be taken as not greater than the lesser of the values obtained from the following, unless the cross-section under consideration is cracked in flexure, in which case only Item (a) shall apply:

(a) *Flexure-shear cracking:*

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o f_{cv} \left[\frac{(A_{st} + A_{pt})}{b_v d_o} \right]^{1/3} + V_o + P_v \quad \dots 8.2.7.2(1)$$

where

$\beta_1, \beta_2, \beta_3, f_{cv}$ and A_{st} are as given in Clause 8.2.7.1 except that in determining β_2, N^* is taken as the value of the axial force excluding prestress

V_o = shear force which would occur at the section when the bending moment at that section was equal to the decompression moment (M_o) given by—

$$M_o = Z \sigma_{cp,f}$$

$\sigma_{cp,f}$ = compressive stress due to prestress, at the extreme fibre where cracking occurs

For statically determinate structures:

$$V_o = \frac{M_o}{|M^* / V^*|} \quad \dots 8.2.7.2(2)$$

where M^* and V^* are the bending moment and shear force respectively, at the section under consideration, due to the design loading for strength.

Where the prestress and the applied moment both produce tension on the same extreme fibre of a member, V_o shall be taken as zero.

For statically indeterminate structures, secondary shear forces and bending moments, due to prestress, shall be taken into account when determining M_o and V_o .

(b) *Web-shear cracking:*

$$V_{uc} = V_t + P_v \quad \dots 8.2.7.2(3)$$

where

V_t = shear force, which, in combination with the prestressing force and other action effects at the section, would produce a principal tensile stress of f'_{ct} at either the centroidal axis or the intersection of flange and web, whichever is the more critical

8.2.7.3 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.

8.2.7.4 Reversal of loads and members in torsion

Where loading cases occur that result in cracking in a zone usually in compression, the value of V_{uc} obtained from Clause 8.2.7.1 or 8.2.7.2 may not apply and V_{uc} shall be assessed or be taken as zero.

8.2.8 Minimum shear reinforcement

The minimum area of shear reinforcement ($A_{sv,min}$) provided in a beam shall be given by—

$$A_{sv,min} = 0.06 \sqrt{f'_c} b_v s / f_{sy,f} \geq 0.35 b_v s / f_{sy,f} \quad \dots 8.2.8$$

where

s = centre-to-centre spacing of shear reinforcement, measured parallel to the longitudinal axis of the member

8.2.9 Shear strength of a beam with minimum reinforcement

The ultimate shear strength of a beam provided with minimum shear reinforcement ($A_{sv,min}$), ($V_{u,min}$) shall be taken as—

$$V_{u,min} = V_{uc} + 0.10\sqrt{f'_c} b_v d_o \geq V_{uc} + 0.6b_v d_o \quad \dots 8.2.9$$

8.2.10 Contribution to shear strength by the shear reinforcement

The contribution to the ultimate shear strength by shear reinforcement in a beam (V_{us}) shall be determined from the following equations:

(a) For perpendicular shear reinforcement—

$$V_{us} = (A_{sv} f_{sy,f} d_o / s) \cot \theta_v \quad \dots 8.2.10(1)$$

(b) For inclined shear reinforcement—

$$V_{us} = (A_{sv} f_{sy,f} d_o / s) (\sin \alpha_v \cot \theta_v + \cos \alpha_v) \quad \dots 8.2.10(2)$$

where, for both Items (a) and (b)—

θ_v = angle between the axis of the concrete compression strut and the longitudinal axis of the member and shall be taken as either—

(i) 45°; or

(ii) chosen in the range of 30° to 60° except that the minimum value of θ_v shall be taken as varying linearly from 30°, when $V^* = \phi V_{u,min}$ to 45°, when $V^* = \phi V_{u,max}$

α_v = angle between the inclined shear reinforcement and the longitudinal tensile reinforcement

8.2.11 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.

8.2.12 Detailing of shear reinforcement

8.2.12.1 Types

Shear reinforcement shall comprise one or more—

- (a) stirrups or fitments making an angle of between 45° and 90° with the longitudinal bars; and
- (b) welded wire mesh placed to have wires perpendicular to the axis of the beam.

8.2.12.2 Spacing

Shear reinforcement shall be spaced longitudinally not further apart than $0.5D$ or 300 mm, whichever is less. Where $V^* \leq \phi V_{u,min}$, the spacing may be increased to $0.75D$ or 500 mm, whichever is less.

The maximum transverse spacing across the width of the member shall not exceed the lesser of 600 mm and D .

8.2.12.3 Extent

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. 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.

8.2.12.4 Anchorage of shear reinforcement

The anchorage of shear reinforcement transverse to the longitudinal flexural reinforcement may be achieved by a hook or cog complying with Clause 13.1.2.7 or by welding of the fitment to a longitudinal bar or by a welded splice.

NOTE: The type of anchorage used should not induce splitting or spalling of the concrete cover.

Notwithstanding the above, fitment cogs are not to be used when the fitment cog is located within 50 mm of any concrete surface.

8.2.12.5 End anchorage of mesh

Where mesh is used as shear reinforcement, the ends shall be anchored—

- (a) in accordance with Clause 8.2.12.4, 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 STRENGTH OF BEAMS IN TORSION

8.3.1 General

This Clause applies to reinforced and prestressed beams subjected to any combination of torsion, flexure and shear. It does not apply to non-flexural members covered by Section 12.

8.3.2 Secondary 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, it shall be permissible to disregard the torsional stiffness in the analysis and torsion in the member, if the torsion reinforcement requirements of Clauses 8.3.7 and the detailing requirements of Clause 8.3.8 are satisfied.

8.3.3 Torsional strength limited by web crushing

To prevent web crushing under the combined action of torsion and flexural shear, beams shall be proportioned so that the following inequality is satisfied:

$$\frac{T^*}{\phi T_{u, \max}} + \frac{V^*}{\phi V_{u, \max}} \leq 1 \quad \dots 8.3.3$$

where

$V_{u, \max}$ is calculated from Clause 8.2.6 and

$$T_{u, \max} = 0.2 f'_c J_t$$

The torsional modulus (J_t) may be taken as—

- = $0.33x^2y$ for solid rectangular sections;
- = $0.33\Sigma x^2y$ for solid T-shaped, L-shaped, or I-shaped sections; and
- = $2A_m b_w$ for thin walled hollow sections, A_m being the area enclosed by the median lines of the walls of a single cell and b_w being a minimum thickness of the wall of a hollow section

8.3.4 Requirements for torsional reinforcement

In the calculation of T^* and V^* in Items (a) and (b), the elastic uncracked stiffness shall be used.

Requirements for torsional reinforcement shall be determined from the following:

(a) Torsional reinforcement is not required if—

$$(i) \quad T^* < 0.25 \phi T_{uc}; \text{ or} \quad \dots 8.3.4(1)$$

$$(ii) \quad \frac{T^*}{\phi T_{uc}} + \frac{V^*}{\phi V_{uc}} \leq 0.5; \text{ or} \quad \dots 8.3.4(2)$$

(iii) the overall depth does not exceed the greater of 250 mm and half the width of the web; and

$$\frac{T^*}{\phi T_{uc}} + \frac{V^*}{\phi V_{uc}} \leq 1 \quad \dots 8.3.4(3)$$

where T_{uc} and V_{uc} are calculated in accordance with Clauses 8.3.5 and 8.2.7 respectively.

(b) If Item (a) above is not satisfied, torsional reinforcement consisting of transverse closed fitments and longitudinal reinforcement shall be provided, in addition to any other reinforcement, such that—

$$\frac{T^*}{\phi T_{us}} \leq 1 \quad \dots 8.3.4(4)$$

where T_{us} is calculated in accordance with Clause 8.3.5.

At least the minimum torsional reinforcement required by Clause 8.3.7 shall be provided in addition to any other fitments.

Longitudinal torsional reinforcement shall comply with Clause 8.3.6 and both transverse and longitudinal torsional reinforcement shall comply with Clause 8.3.7.

Shear reinforcement shall be provided with V_{uc} assessed in accordance with Clause 8.2.7.4.

8.3.5 Torsional strength of a beam

For the purpose of Clause 8.3.4, the ultimate strength of a beam in pure torsion (T_{uc} or T_{us}) shall be determined from the following:

(a) For a beam without closed fitments, the ultimate strength in pure torsion (T_{uc}) shall be calculated from—

$$T_{uc} = J_t \left(0.3 \sqrt{f'_c} \right) \sqrt{(1 + 10 \sigma_{cp} / f'_c)} \quad \dots 8.3.5(1)$$

(b) For a beam with closed fitments, the ultimate strength in pure torsion (T_{us}) shall be calculated from—

$$T_{us} = f_{sy.f} (A_{sw} / s) 2 A_t \cot \theta_v \quad \dots 8.3.5(2)$$

where

A_t = area of a polygon with vertices at the centre of longitudinal bars at the corners of the cross-section

8.3.6 Longitudinal torsional reinforcement

Longitudinal torsional reinforcement shall be provided to resist the following design tensile forces, taken as additional to any other design tensile forces:

- (a) In the flexural tensile zone, a force of—

$$0.5f_{\text{sy.f}}\left(\frac{A_{\text{sw}}}{s}\right)u_{\text{t}}\cot^2\theta_{\text{v}}; \text{ and} \quad \dots 8.3.6(1)$$

- (b) In the flexural compressive zone, a force of—

$$0.5f_{\text{sy.f}}\left(\frac{A_{\text{sw}}}{s}\right)u_{\text{t}}\cot^2\theta_{\text{v}} - F_{\text{c}}^*; \text{ but not less than zero,} \quad \dots 8.3.6(2)$$

where

θ_{v} = angle between the axis of the concrete compression strut and the longitudinal axis of the member (see Clause 8.2.10)

u_{t} = perimeter of the polygon defined for A_{t}

F_{c}^* = absolute value of the design force in the compressive zone due to flexure

8.3.7 Minimum torsional reinforcement

Where torsional reinforcement is required as specified in Clause 8.3.4—

- (a) longitudinal torsional reinforcement shall be provided in accordance with Clause 8.3.6; and
- (b) minimum transverse reinforcement shall be provided to satisfy the greater of—
 - (i) the minimum shear reinforcement required by Clause 8.2.8 in the form of closed ties or fitments; and
 - (ii) a torsional capacity equal to $0.25T_{\text{uc}}$.

8.3.8 Detailing of torsional reinforcement

Torsional reinforcement shall be detailed in accordance with Clause 8.2.12.4 and the following:

- (a) Torsional reinforcement shall consist of both closed fitments and longitudinal reinforcement.
- (b) A closed fitment shall be capable of developing full yield stress in each leg and capable of transferring the yield force to an adjacent leg, unless a more refined analysis shows that over part of the fitment full yield stress is not required. The spacing (s) of the closed fitments shall be not greater than the lesser of $0.12u_{\text{t}}$ and 300 mm.
- (c) The 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 fitments.

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 (τ^*) acting on the interface shall be taken as follows:

$$\tau^* = \beta V^* / (z b_f) \quad \dots 8.4.2$$

where

z = internal moment lever arm of the section

For a shear plane that passes through a region in compression—

β = 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—

β = 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 $\phi \tau_u$ 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

τ_u = unit shear strength

g_p = permanent distributed load normal to the shear interface per unit length, newtons per millimetre (N/mm)

μ = 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^2)

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	
	μ	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 μ 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 comply 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—

$$I_{ef} = I_{cr} + (I - I_{cr}) \left(M_{cr} / M_s^* \right)^3 \leq I_{ef,max} \quad \dots 8.5.3.1(1)$$

where

$I_{ef,max}$ = maximum effective second moment of area and is taken as I , for reinforced sections when $p = A_{st}/bd \geq 0.005$ and 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

$$M_{cr} = Z \left(f'_{ct,f} - \sigma_{cs} + P / A_g \right) + Pe \geq 0$$

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

σ_{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, σ_{cs} 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 = $A_{sc}/b_w d$

ε_{cs}^* = final design shrinkage strain of concrete

Where appropriate, σ_{cs} shall be increased to account for axial tension induced by restraint to shrinkage by the support to the beams.

Alternatively, as a further simplification but only for reinforced members, I_{ef} may be taken as—

$$I_{ef} = [(5 - 0.04 f'_c) p + 0.002] b_{ef} d^3 \leq [0.1 / \beta^{2/3}] b_{ef} d^3 \text{ when } p \geq 0.001 (f'_c)^{1/3} / \beta^{2/3} \quad \dots 8.5.3.1(2)$$

$$I_{ef} = [(0.055 (f'_c)^{1/3}) / \beta^{2/3} - 50 p] b_{ef} d^3 \leq [0.06 / \beta^{2/3}] b_{ef} d^3 \text{ when } p < 0.001 (f'_c)^{1/3} / \beta^{2/3} \quad \dots 8.5.3.1(3)$$

where

$$\beta = b_{ef} / b_w \geq 1$$

$$p = A_{st} / b_{ef} d \text{ at midspan}$$

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 (ε_{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 (φ_{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 calculated by multiplying the short-term deflection caused by the sustained loads 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 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 comply 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 comply with the requirements of Clause 2.3.2 if the ratio of effective span to effective depth satisfies the following:

$$L_{ef} / d \leq \left[\frac{k_1 (\Delta / L_{ef}) b_{ef} E_c}{k_2 F_{d,ef}} \right]^{1/3} \quad \dots 8.5.4$$

where

Δ / 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—

- $(1.0 + k_{cs})g + (\psi_s + k_{cs} \psi_l)q$ for total deflection; or

- (b) $k_{cs} g + (\psi_s + k_{cs} \psi_1)q$ 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 ψ_s and ψ_1 are given in AS/NZS 1170.0

$$\begin{aligned}
 k_1 &= I_{ef}/b_{ef}d^3, \text{ which may be taken as—} \\
 &= (5 - 0.04f'_c)p + 0.002 \leq 0.1/\beta^{2/3} \quad \text{when } p \geq 0.001(f'_c)^{1/3}/\beta^{2/3} \\
 &= 0.055(f'_c)^{1/3}/\beta^{2/3} - 50p \leq 0.06/\beta^{2/3} \quad \text{when } p < 0.001(f'_c)^{1/3}/\beta^{2/3}
 \end{aligned}$$

where $\beta = b_{ef}/b_w \geq 1$ and $p = A_{st}/b_{ef}d$ at midspan

k_2 = deflection constant, taken as—

- (a) 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 Crack control for tension and flexure in reinforced beams

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.

Cracking in reinforced beams subjected to tension, flexure with tension or flexure shall be deemed to be controlled if the appropriate requirements in Items (a) and (b), and either Item (c) for beams primarily in tension or Item (d) for beams primarily in flexure are satisfied. 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 crack control is not required, only Items (a) and (b) need be satisfied.

- (a) The minimum area of reinforcement in a tensile zone of a beam shall comply 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.
- (c) For beams primarily subject to tension, the calculated steel stress (σ_{scr}) shall not exceed the maximum steel stress given in Table 8.6.1(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 $0.8f_{sy}$.
- (d) For beams primarily subject to flexure, the calculated tensile steel stress (σ_{scr}) shall not exceed the larger of the maximum steel stresses given in—
 - (i) Table 8.6.1(A) for the largest nominal diameter (d_b) of the bars in the tensile zone; and

- (ii) Table 8.6.1(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 M_s^* and $M_{s.1}^*$ at the serviceability limit state will normally be estimated using elastic analysis. Significant errors may result if they are determined from the design bending moments M^* at the strength limit state when the amount of moment redistribution is unknown; for example, if plastic methods of analysis are used for strength design.

TABLE 8.6.1(A)
MAXIMUM STEEL STRESS FOR TENSION
OR FLEXURE IN REINFORCED BEAMS

Nominal bar diameter (d_b) mm	Maximum steel stress MPa
10	360
12	330
16	280
20	240
24	210
28	185
32	160
36	140
40	120

NOTE: Values for other bar diameters may be calculated using the equation—
Maximum steel stress = $-173 \log_e (d_b) + 760$ MPa.

TABLE 8.6.1(B)
MAXIMUM STEEL STRESS FOR FLEXURE
IN REINFORCED BEAMS

Centre-to-centre spacing mm	Maximum steel stress MPa
50	360
100	320
150	280
200	240
250	200
300	160

NOTE: Intermediate values may be calculated using the equation—
Maximum steel stress = $-0.8 \times \text{centre-centre spacing} + 400$ MPa.

8.6.2 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 limiting either—

- (a) the calculated maximum flexural tensile stress under short-term service loads to $0.6\sqrt{f'_c}$; or
- (b) the increment in steel stress near the tension face to that given in Table 8.6.2, as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the short-term service load value.

TABLE 8.6.2
MAXIMUM INCREMENT OF STEEL STRESS
FOR FLEXURE IN PRESTRESSED BEAMS

Nominal reinforcement bar diameter (d_b) mm	Maximum increment of steel stress MPa
≤12	330
16	280
20	240
24	210
≥28	200
All bonded tendons	200

8.6.3 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.

8.6.4 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 comply 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 $180b_{ef}/D$ 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 $100b_{ef}/D$ 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—

$$A_{sc} \geq 0.35A_{pt}f_{pb}/f_{sy} \quad \dots 8.9.4$$

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-comply 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 comply 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 comply with Clause 9.1.3.1(a).

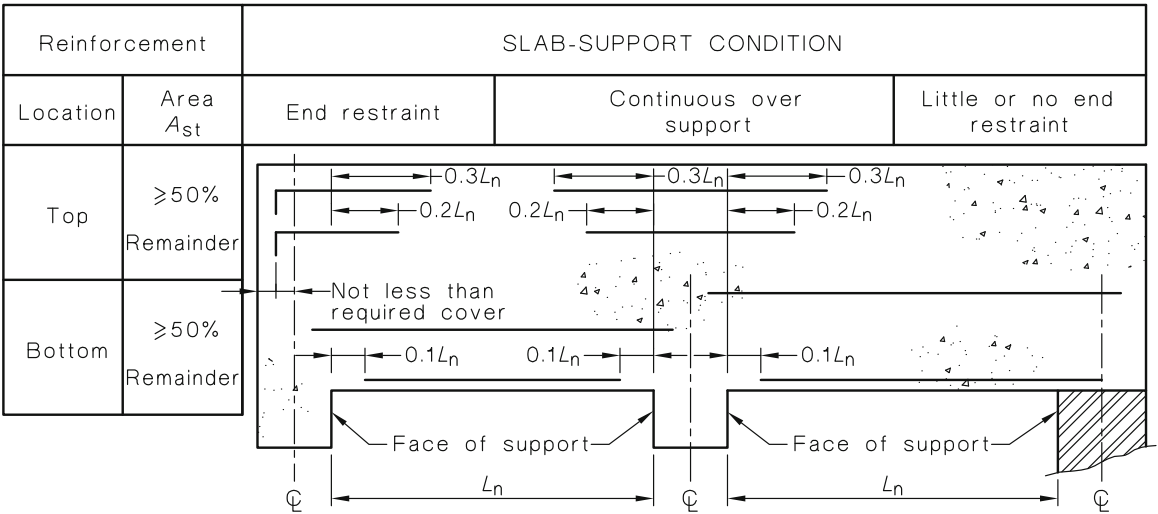


FIGURE 9.1.3.2 ARRANGEMENT OF REINFORCEMENT

9.1.3.3 Deemed-to-comply 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-comply 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 comply 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-comply arrangement for two-way flat slabs*

For multispan, reinforced, two-way flat slabs analysed using simplified elastic analysis, as detailed in Clause 6.10.4, the following deemed to comply 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-comply 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.

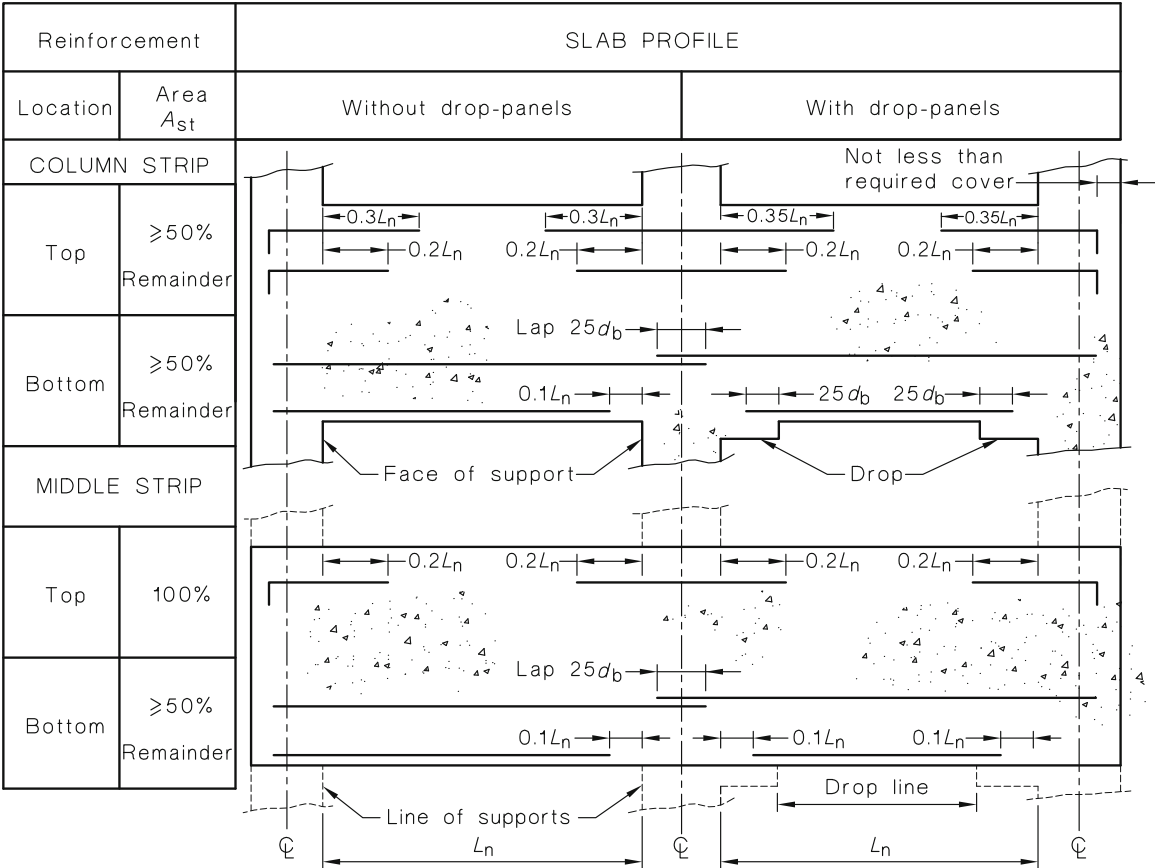


FIGURE 9.1.3.4 ARRANGEMENT OF REINFORCEMENT

9.1.4 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.

9.1.5 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 shall be determined in accordance with Clause 9.4.

9.2 STRENGTH OF SLABS IN SHEAR

9.2.1 Definitions and symbols

For the purpose of this Clause, the definitions and symbols below apply to flat slabs.

9.2.1.1 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 9.2.1(A)].

9.2.1.2 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 9.2.1(A)].

9.2.1.3 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 9.2.1(A)].

9.2.1.4 Torsion strip

A strip of slab of width a , whose longitudinal axis is perpendicular to the direction of M_v^* [see Figure 9.2.1(B)].

9.2.1.5 Symbols

The following symbols apply:

- a = dimension of the critical shear perimeter measured parallel to the direction of M_v^* [see Figure 9.2.1(B)]
- b_o = dimension of an opening [see Figure 9.2.1(A)]
- b_w = width of the web of a spandrel beam [see Figure 9.2.1(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 9.2.1(B)]
- u = length of the critical shear perimeter [see Figure 9.2.1(A)]
- v_1 = larger overall dimension of a closed fitment (see Figure 9.2.6)
- β_h = ratio of the longest overall dimension of the effective loaded area, Y , to the overall dimension, X , measured perpendicular to Y [see Figure 9.2.1(A)]

9.2.2 Strength

The strength of a slab in shear shall be determined in accordance with the following:

- (a) 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.
- (b) Where shear failure can occur locally around a support or concentrated load, the design shear strength of the slab shall be taken as ϕV_u , where V_u is calculated in accordance with one of the following, as appropriate:
 - (i) Where M_v^* is zero, V_u is taken as equal to V_{uo} calculated in accordance with Clause 9.2.3.
 - (ii) Where M_v^* is not zero, V_u is calculated in accordance with Clause 9.2.4.

NOTE: For types of shear reinforcement other than those covered in Clauses 9.2.3 and 9.2.4 strength may be determined by tests, in accordance with Appendix B.

9.2.3 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—

- (a) where there is no shear head—

$$V_{uo} = u d_{om} (f_{cv} + 0.3 \sigma_{cp}) \quad \dots 9.2.3(1)$$

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 σ_{cp} should be evaluated separately for the case of corner, edge and internal columns.

(b) where there is a shear head—

$$V_{uo} = u d_{om} (0.5 \sqrt{f'_c} + 0.3 \sigma_{cp}) \leq 0.2 u d_{om} f'_c \qquad \dots 9.2.3(2)$$

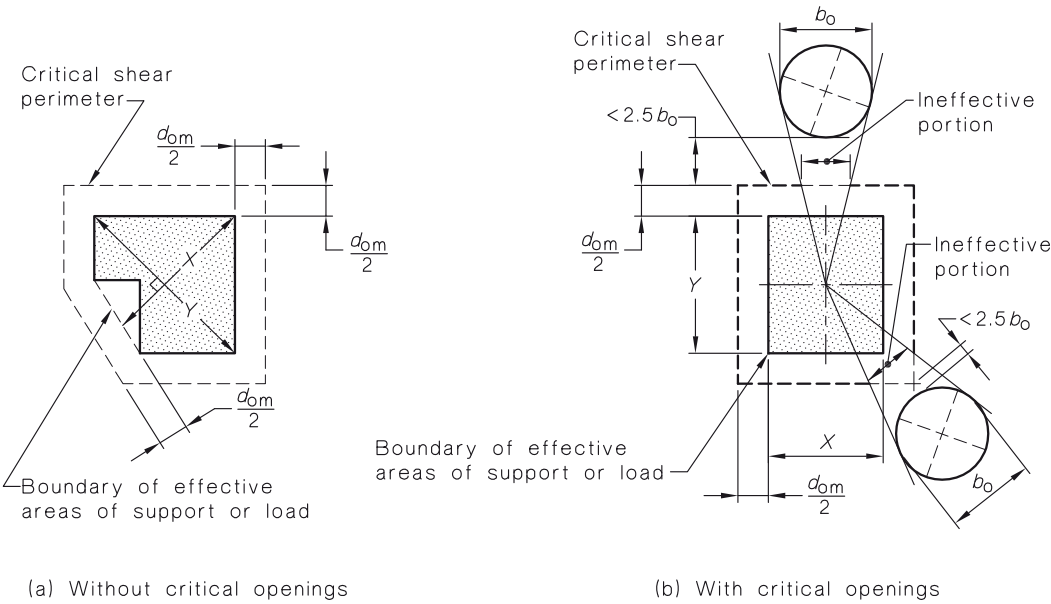


FIGURE 9.2.1(A) CRITICAL SHEAR PERIMETER

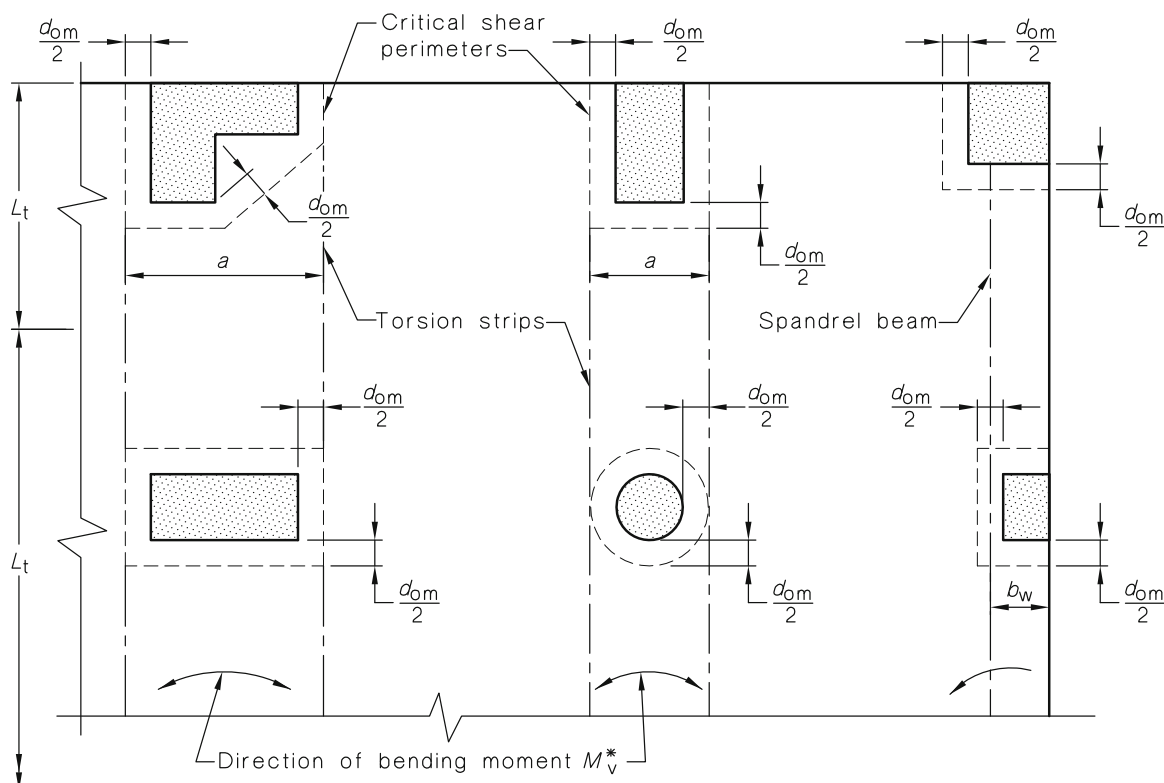


FIGURE 9.2.1(B) TORSION STRIPS AND SPANDREL BEAMS

9.2.4 Ultimate shear strength where M_v^* is not zero

Where M_v^* is not zero and shear reinforcement, if provided, complies with Clauses 9.2.5 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—

$$V_u = V_{uo} / [1.0 + uM_v^* / (8V^* a d_{om})] \quad \dots 9.2.4(1)$$

- (b) If the torsion strip contains the minimum quantity of closed fitments, V_u shall be taken as $V_{u,min}$ given by—

$$V_{u,min} = 1.2 V_{uo} / [1.0 + uM_v^* / (2V^* a^2)] \quad \dots 9.2.4(2)$$

- (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—

$$V_{u,min} = 1.2 V_{uo} (D_b / D_s) / [1.0 + uM_v^* / (2V^* a b_w)] \quad \dots 9.2.4(3)$$

- (d) If the torsion strip or spandrel beam contains more than the minimum quantity of closed fitments, V_u is given by—

$$V_u = V_{u,min} \sqrt{[(A_{sw} / s) / (0.2y_1 / f_{sy,f})]} \quad \dots 9.2.4(4)$$

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—

$$V_{u,max} = 3V_{u,min} \sqrt{(x/y)} \quad \dots 9.2.4(5)$$

where x and y are the shorter and longer dimensions respectively of the cross-section of the torsion strip or spandrel beam.

9.2.5 Minimum area of closed fitments

The minimum cross-sectional area of the reinforcement forming the closed fitments shall satisfy the following inequality:

$$A_{sw} / s \geq 0.2 y_1 / f_{sy.f} \quad \dots 9.2.5$$

9.2.6 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 $L_v/4$ 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 9.2.6.

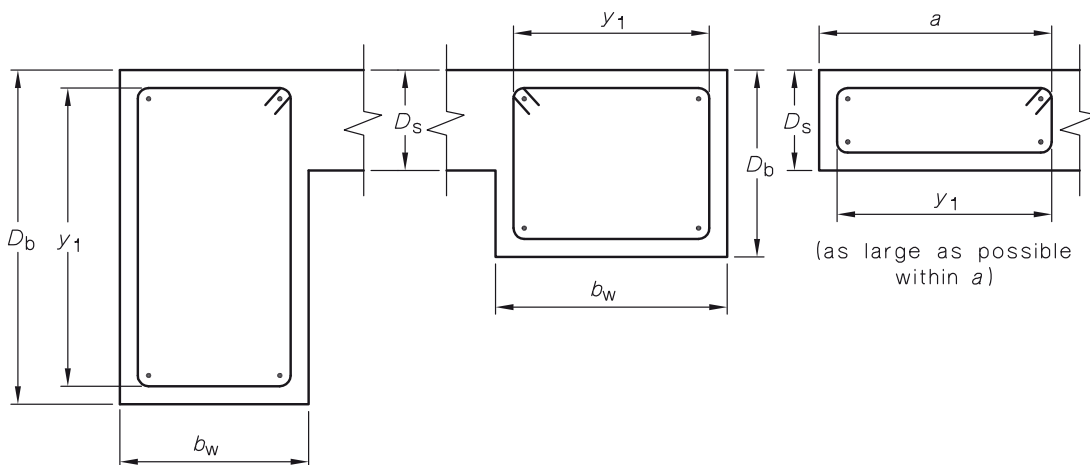


FIGURE 9.2.6 PARAMETERS AND DETAILS OF SHEAR REINFORCEMENT FOR SLABS

9.3 DEFLECTION OF SLABS

9.3.1 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 comply with Clause 9.3.4.

9.3.2 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.

- (c) Shrinkage and creep properties of the concrete.
- (d) Expected load history.
- (e) Expected construction procedure.
- (f) Deflection of formwork or settlement of props during construction, particularly when the slab formwork is supported off suspended floors below.

9.3.3 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:

- (a) For a one-way slab, a prismatic beam of unit width.
- (b) 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—

$$L_y^4 / (\alpha L_x^4 + L_y^4)$$

... 9.3.3

where α is given in Table 9.3.3 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 9.3.3
COEFFICIENT OF PROPORTIONALITY (α)

Edge condition		Coefficient (α)
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

9.3.4 Deemed to comply span-to-depth ratio for reinforced slabs

9.3.4.1 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 comply with the requirements of Clause 2.3.2 if the ratio of the effective span to the effective depth satisfies the following:

$$L_{ef} / d \leq k_3 k_4 \left[\frac{(\Delta / L_{ef}) 1000 E_c}{F_{d,ef}} \right]^{1/3}$$

... 9.3.4.1

NOTE: E_c is in megapascals.

where

Δ/L_{ef} = deflection limit selected in accordance with Clause 2.3.2 and the deflection (Δ) is taken on the centre-line between the supports used to calculate L_{ef}

L_{ef} = effective span

$F_{\text{d,ef}}$ = effective design service load, per unit area, in kilopascals, taken as—

- (a) $(1.0 + k_{\text{cs}})g + (\psi_{\text{s}} + k_{\text{cs}}\psi_{\text{l}})q$ for total deflection; or
- (b) $k_{\text{cs}}g + (\psi_{\text{s}} + k_{\text{cs}}\psi_{\text{l}})q$ for the deflection that occurs after the addition or attachment of the brittle partitions or finishes

k_{cs} is determined in accordance with Clause 8.5.3.2 and ψ_{s} and ψ_{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

9.3.4.2 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 comply 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 9.3.4.2.

TABLE 9.3.4.2
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

9.4 CRACK CONTROL OF SLABS

9.4.1 Crack control for flexure in reinforced slabs

Cracking in reinforced slabs subject to flexure shall be deemed to be controlled if the appropriate requirements in Items (a), (b), (c) and (d) are satisfied. For areas of slabs fully enclosed within a building except for a brief period of weather exposure during construction and, where it is assessed that crack control is not required, only Item (a) and Item (b) need be satisfied.

- (a) The minimum area of reinforcement in a tensile zone of a slab shall comply 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.
- (c) The calculated tensile steel stress (σ_{scr}) shall not exceed the larger of the maximum steel stresses given in—
 - (i) Table 9.4.1(A) for the largest nominal diameter (d_b) of the bars in the tensile zone; and
 - (ii) Table 9.4.1(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.
- (d) The calculated tensile steel stress $\sigma_{scr,1}$ shall not exceed $0.8f_{sy}$.

NOTE: Design bending moments M_s^* and $M_{s,1}^*$ at the serviceability limit state will normally be estimated using elastic analysis. Significant errors may result if they are determined from the design bending moments M^* at the strength limit state when the amount of moment redistribution is unknown; for example, if plastic methods of analysis are used for strength design.

TABLE 9.4.1(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)	
	≤ 300	> 300
6	375	450
8	345	400
10	320	360
12	300	330
16	265	280
20	240	
24	210	

NOTE: Values for other bar diameters may be calculated using the appropriate equation, as follows:

Maximum steel stress equals—

$$-173 \log_e (d_b) + 760 \text{ MPa for } d_b \geq 20 \text{ mm}$$

$$-173 \log_e (d_b) + 760 \text{ MPa for } d_b < 20 \text{ mm and } D_s > 300 \text{ mm}$$

$$-114 \log_e (d_b) + 580 \text{ MPa for } d_b < 20 \text{ mm and } D_s \leq 300 \text{ mm}$$

TABLE 9.4.1(B)
MAXIMUM STEEL STRESS FOR FLEXURE
IN REINFORCED SLABS

Centre-to-centre spacing (mm)	Maximum steel stress (MPa)
50	360
100	320
150	280
200	240
250	200
300	160

NOTE: Intermediate values may be calculated using the following equation:

$$\text{Maximum steel stress} = -0.8 \times \text{centre-to-centre spacing} + 400 \text{ MPa.}$$

9.4.2 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 limiting—

- the calculated maximum flexural tensile stress in the concrete under short-term service loads to $0.6\sqrt{f'_c}$; or
- the increment in steel stress near the tension face to that given in Table 9.4.2, as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the short-term service load value.

TABLE 9.4.2
MAXIMUM INCREMENT OF STEEL STRESS
FOR FLEXURE IN PRESTRESSED SLABS

Nominal reinforcement bar diameter (d_b) mm	Maximum increment of steel stress (MPa) for overall depth, D_s (mm)	
	≤ 300	> 300
≤ 10	320	360
12	300	330
16	265	280
20	240	
24	210	
≥ 28	200	
All bonded tendons	200	

9.4.3 Crack control for shrinkage and temperature effects

9.4.3.1 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 9.4.3.2 to 9.4.3.5.

For members greater than 500 mm thick, the reinforcement required near each surface may be calculated using 250 mm for D .

9.4.3.2 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 9.4.3.3 to 9.4.3.5, as appropriate.

9.4.3.3 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 $(1.75-2.5 \sigma_{cp}) bD \times 10^{-3}$.

9.4.3.4 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..... $(6.0-2.5 \sigma_{cp})bD \times 10^{-3}$.

9.4.3.5 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 9.4.3.3 and 9.4.3.4.

9.4.4 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.

9.4.5 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.

9.5 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.

9.6 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—

$$b_{ef} = \text{the load width} + 2.4a[1.0 - (a/L_n)] \quad \dots 9.6$$

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.

9.7 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 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 ϕ 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 (δ_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 ϕ 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 ϕ_{sys} shall be determined from Table 2.2.5, as appropriate.

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—

$$L_e/r \leq 25; \text{ or}$$

$$\leq \alpha_c (38 - f'_c/15) (1 + M_1^*/M_2^*) \quad \dots 10.3.1(1)$$

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—

$$L_e/r \leq 22 \quad \dots 10.3.1(2)$$

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 (ϕN_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 (δ).

The moment magnifier (δ) 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 δ , 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 (δ) for a braced column shall be taken to be equal to δ_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 (δ) for an unbraced column shall be taken as the larger value of δ_b or δ_s where—

- (a) δ_b for an individual column is calculated in accordance with Clause 10.4.2 assuming the column is braced; and
- (b) δ_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), δ_s may be calculated from a linear elastic critical buckling load analysis of the entire frame, where δ_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 $0.4E_cI_f$ and $0.8E_cI_c$ respectively

The frame shall be proportioned so that δ_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 (γ_1 and γ_2) shall be determined—

- (a) for regular rectangular framed structures where the axial forces in the beams are generally small, in accordance with Clause 10.5.4;
- (b) 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
- (c) 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.


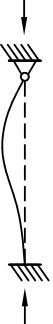
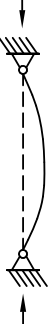
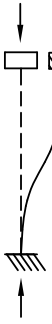
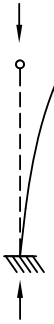
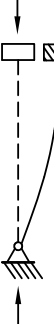




	Braced column			Unbraced column		
Buckled shape						
Effective length factor (k)	0.70	0.85	1.00	1.20	2.20	2.20
Symbols for end restraint conditions	 = Rotation fixed, translation fixed  = Rotation free, translation fixed			 = Rotation fixed, translation free  = Rotation free, translation free		

FIGURE 10.5.3(A) EFFECTIVE LENGTH FACTOR (k) FOR COLUMNS WITH SIMPLE END RESTRAINTS

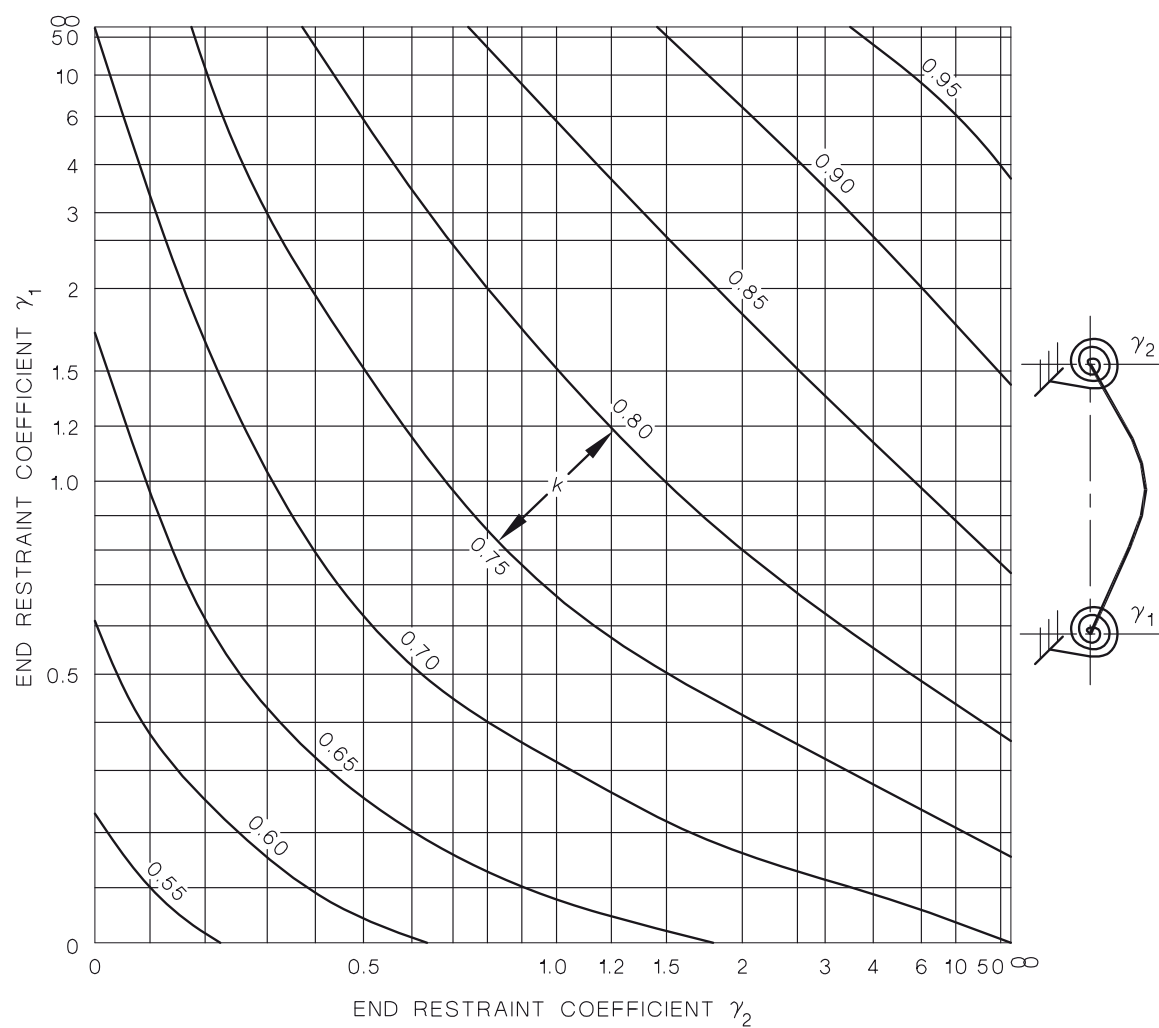


FIGURE 10.5.3(B) EFFECTIVE LENGTH FACTOR (k) FOR BRACED COLUMNS

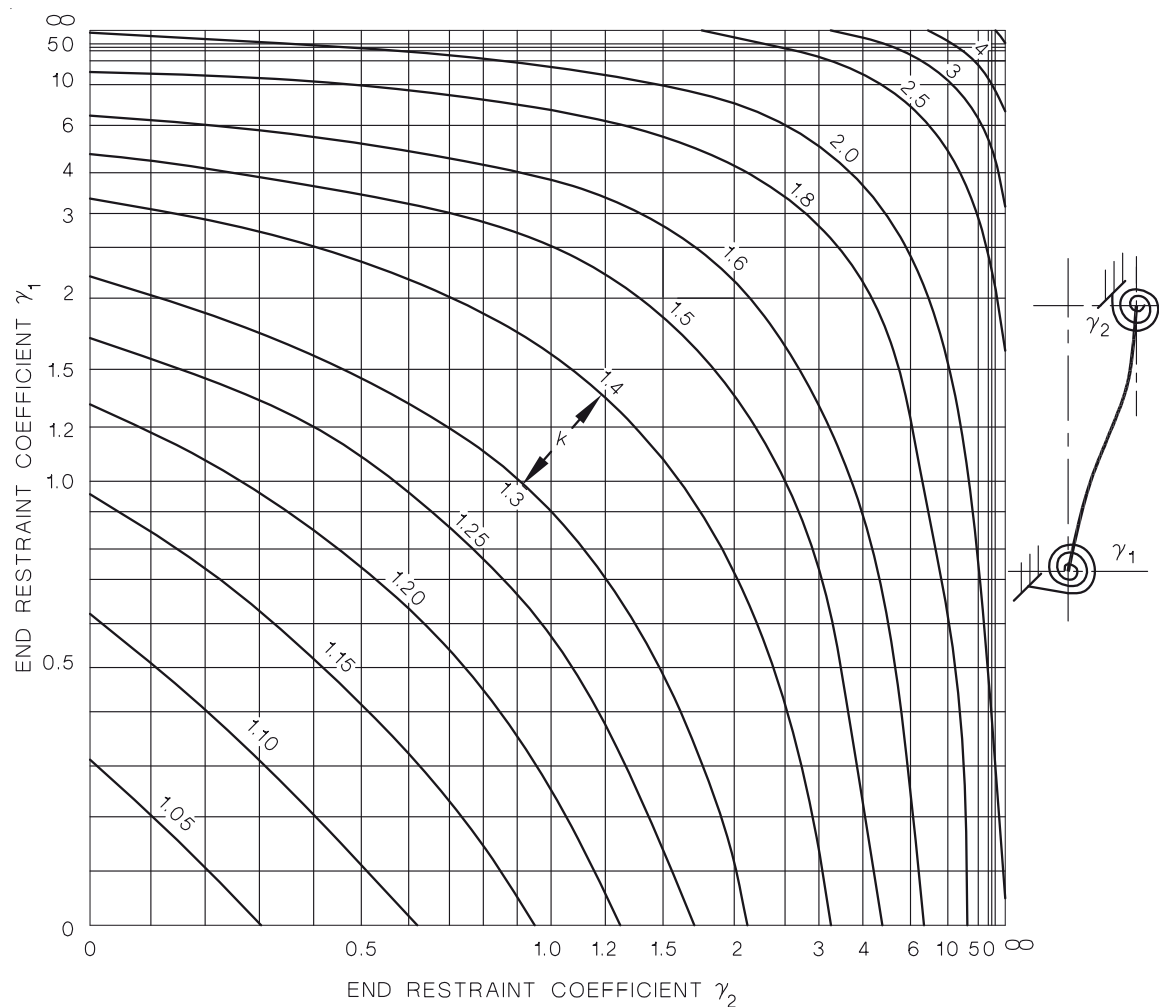


FIGURE 10.5.3(C) EFFECTIVE LENGTH FACTOR (k) FOR UNBRACED COLUMNS

10.5.4 End restraint coefficients for regular rectangular framed structures

For regular rectangular framed structures, the end restraint coefficient (γ_1) at one end of a column and the end restraint coefficient (γ_2) at the other end may each be calculated as—

$$\frac{\sum(I/L)_c}{\sum(\beta I/L)_b} \qquad \dots 10.5.4$$

where

- $\Sigma(I/L)_c$ = 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
- $\Sigma(\beta I/L)_b$ = 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
- β = 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 (β)

Fixity conditions at far end of a beam or slab, or both	Fixity factor (β)	
	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 (γ_1) at one end of a column and the end restraint coefficient (γ_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, γ is theoretically infinite but may be taken as 10.

Where a footing is specifically designed to prevent rotation of the end of a column, γ 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.

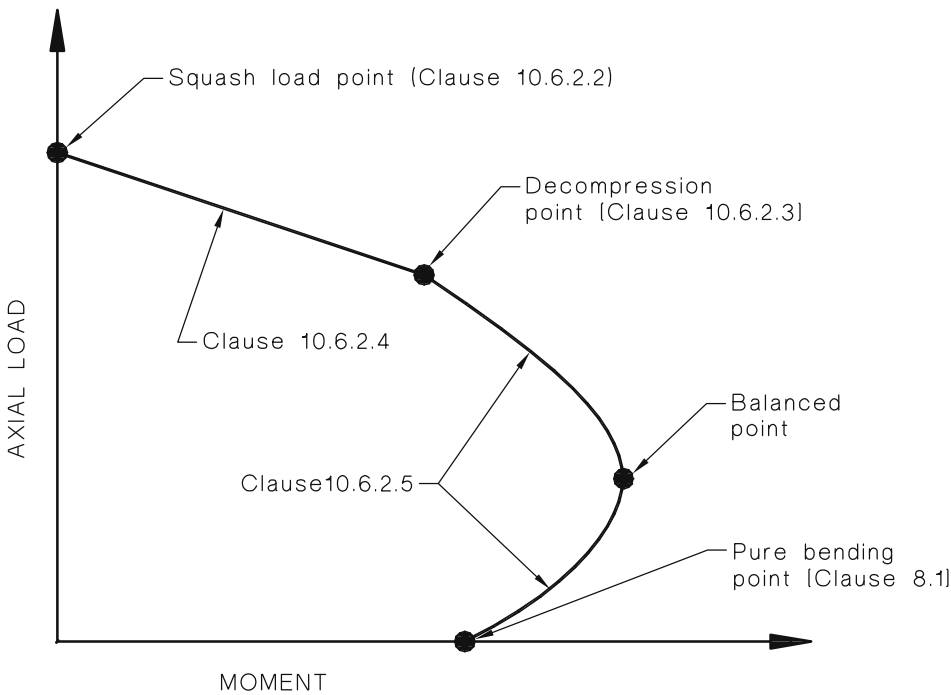


FIGURE 10.6.2.1 AXIAL LOAD—MOMENT DIAGRAM

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—

$$\alpha_1 = 1.0 - 0.003 f'_c \text{ with the limits } 0.72 \text{ to } 0.85; \text{ and} \qquad \dots 10.6.2.2$$

NOTE: The modification of $0.9 f'_c$ given in Clause 3.1.4 is included in the calculation of α_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—

$$\alpha_2 = 1.0 - 0.003 f'_c \quad (\text{within the limits } 0.67 \leq \alpha_2 \leq 0.85) \quad \dots 10.6.2.5(1)$$

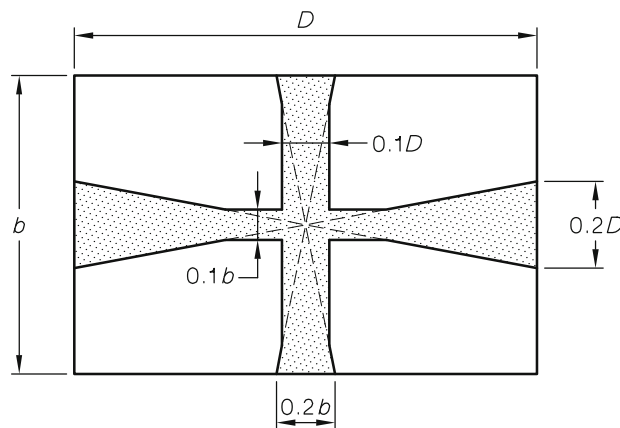
$$\gamma = 1.05 - 0.007 f'_c \quad (\text{within the limits } 0.67 \leq \gamma \leq 0.85) \quad \dots 10.6.2.5(2)$$

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—

$$\left(\frac{M_x^*}{\phi M_{ux}} \right)^{\alpha_n} + \left(\frac{M_y^*}{\phi M_{uy}} \right)^{\alpha_n} \leq 1.0 \quad \dots 10.6.4$$

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

α_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—

- (a) 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
- (b) 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 4 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—

- (a) 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
- (b) for columns where $f'_c > 50$ MPa confinement shall be provided—
 - (i) in special confinement regions using fitments detailed to provide a minimum effective confining pressure to the core of $0.01f'_c$, calculated in accordance with Clause 10.7.3.2, 10.7.3.3 or 10.7.3.4; and
 - (ii) 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 $0.6D_c$, 300 mm and that of Clause 10.7.4.

Special confinement regions are regions where the design action effects satisfy the following [see Figure 10.7.3.1(A)]:

$$(A) \quad N^* \geq 0.75\phi N_{uo}; \text{ or} \quad \dots 10.7.3.1(1)$$

$$(B) \quad N^* \geq \phi 0.3f'_c A_g \text{ and } M^* \geq 0.6\phi M_u \quad \dots 10.7.3.1(2)$$

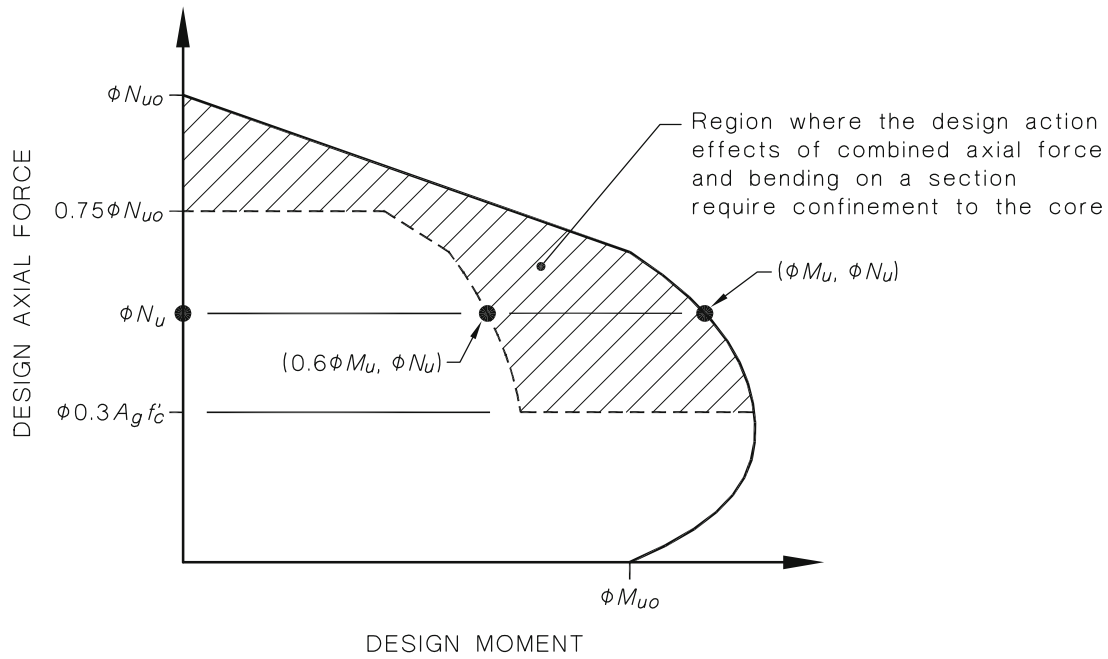
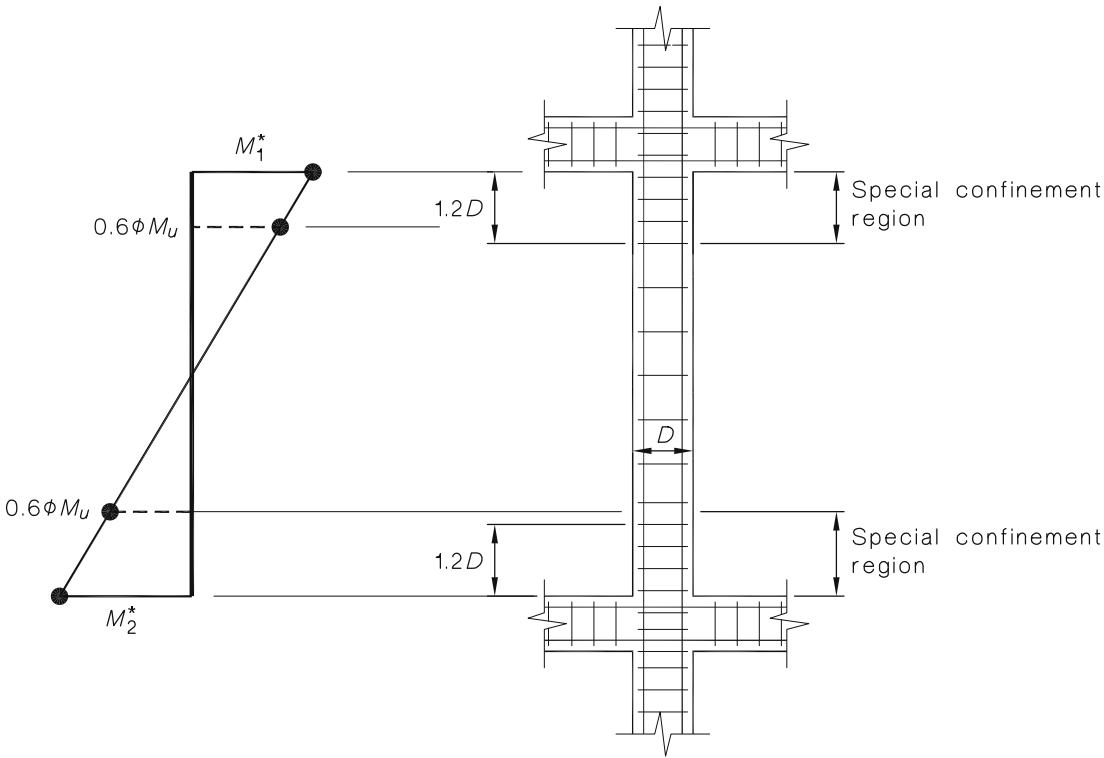


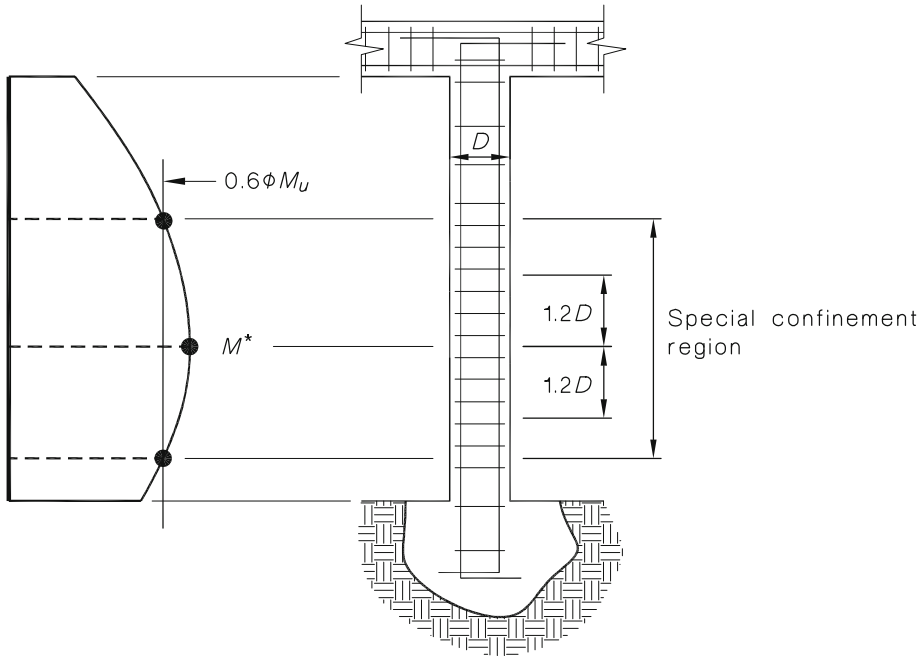
FIGURE 10.7.3.1(A) CONFINEMENT TO THE CORE

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) $0.4L \frac{|M_2^*|}{|M_1^*| + |M_2^*|}$; and

(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—

$$f_{r,eff} = k_e f_r \quad \dots 10.7.3.2(1)$$

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)

θ = angle between the tie 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 tied 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

$$k_e = \left(1 - \frac{s}{2d_s}\right)^2 \qquad \dots 10.7.3.3(4)$$

Alternatively, for rectangular or circular columns, the effective confining pressure may be calculated as follows:

$$f_{r,eff} = 0.5k_e\rho_s f_{sy,f} \qquad \dots 10.7.3.3(5)$$

where

ρ_s = the volumetric ratio of the fitments relative to the volume of the core
calculated as $\rho_s = \frac{A_{b,fit} \times \text{total perimeter of fitments crossing the section}}{A_c \times s}$

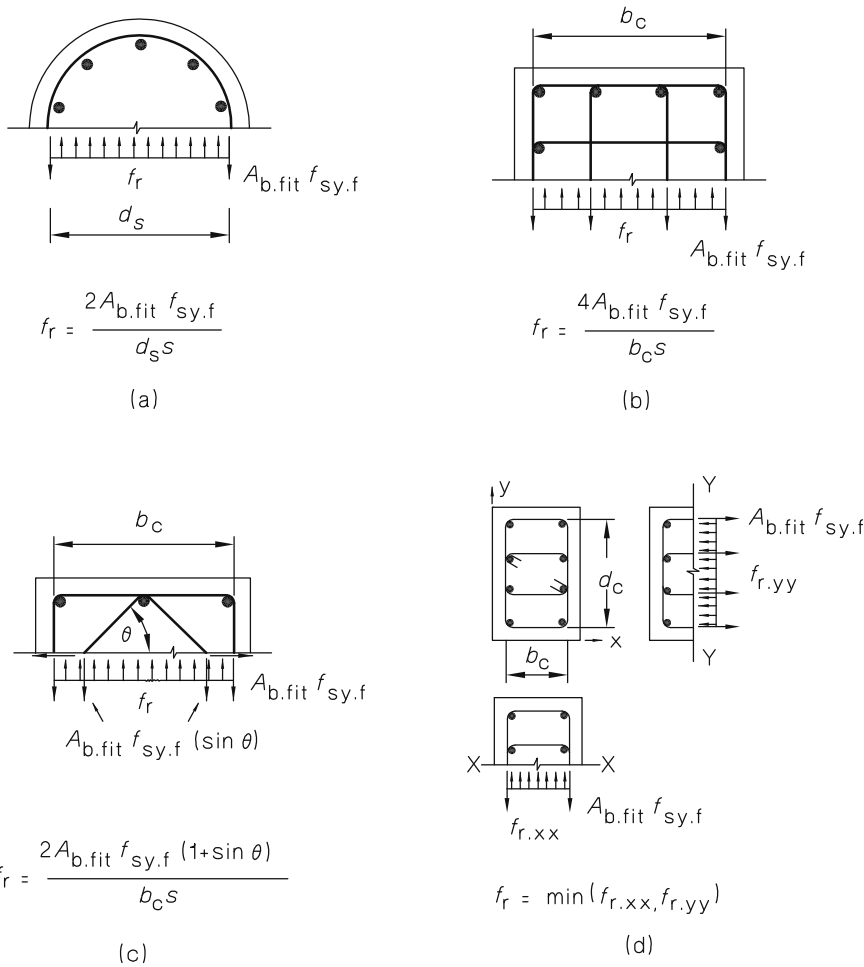


FIGURE 10.7.3.3 CALCULATION OF CONFINING PRESSURES

10.7.3.4 Deemed to comply 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; 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° or less; or
- (ii) between two 135° fitment hooks; or
- (iii) inside a single 135° fitment hook of a fitment that is approximately perpendicular to the column face; or
- (iv) for internal fitments with one leg, at a 90° fitment hook of a fitment that is approximately perpendicular to the column face, provided the other end of the fitment has a 135° angle hook around a longitudinal reinforcing bar and consecutive internal fitments are alternated end to end along the longitudinal reinforcement; or

(b) a circular fitment or helical reinforcement and the longitudinal reinforcing bars are equally spaced around the circumference.

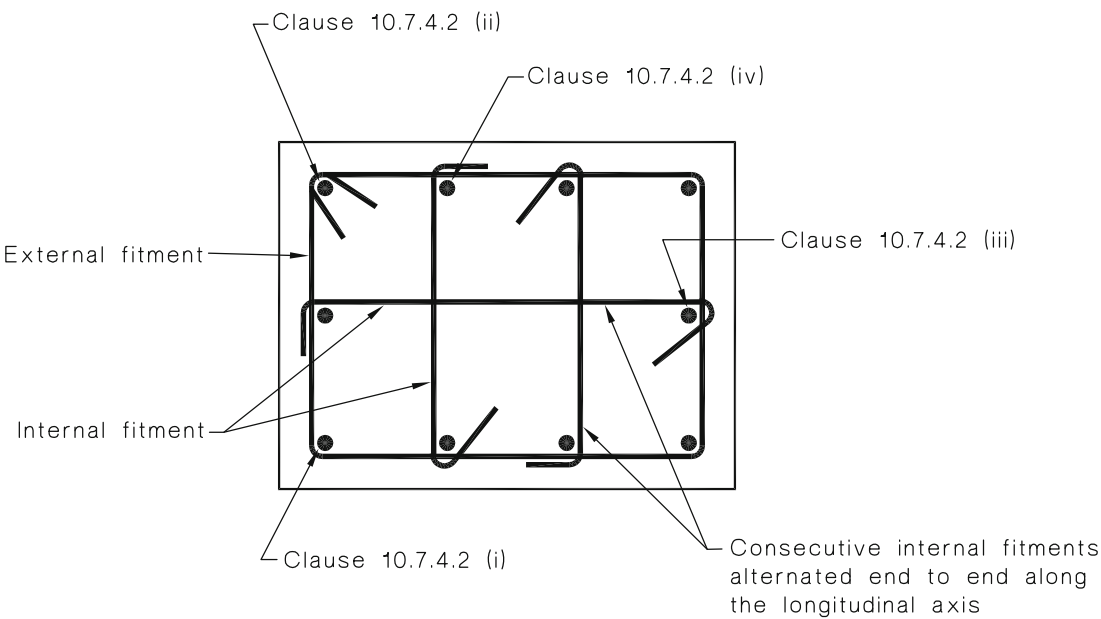


FIGURE 10.7.4.2 LATERAL RESTRAINT TO LONGITUDINAL BARS

10.7.4.3 Diameter and spacing of fitments and helices

The diameter and spacing of fitments and helices shall comply 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 28 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

- (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 unless restraint on all sides is provided by a floor system of approximately equal depth.

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 comply 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 1 in 6;

- (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 with 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).

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, in-plane bending may be neglected and 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.

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 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—

$$\phi N_u \quad \dots 11.5.1$$

where

$$\phi = 0.6$$

$$N_u = (t_w - 1.2e - 2e_a) 0.6 f'_c$$

N_u = ultimate strength per unit length of wall

t_w = thickness of the wall

e = eccentricity of the load measured at right angles to the plane of the wall, determined in accordance with Clause 11.5.2

e_a = an additional eccentricity taken as $(H_{we})^2/2500t_w$

11.5.2 Eccentricity of vertical load

The eccentricity of a vertical load applied to the top of a wall shall be determined as follows:

- (a) The eccentricity of the load applied by the floor (or roof) supported by the top of the wall being designed shall be taken as—
 - (i) for a discontinuous floor, one third of the depth of the bearing area measured from the span face of the wall,
 - (ii) for a cast in situ concrete floor continuous over the wall, zero.
- (b) 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—

$$\phi V_u \quad \dots 11.6.2$$

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—

$$V_{u,max.} = 0.2 f'_c (0.8 L_w t_w)$$

The value of ϕ shall be determined from Table 2.2.2.

11.6.3 Shear strength without shear reinforcement

The ultimate shear strength of a wall without shear 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 shear reinforcement

The contribution to the ultimate shear strength of a wall by shear 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 0.0015 and the value required for strength; 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 comply 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.
- (c) Walls designed for two-way buckling (based on Clauses 11.4(b) or 11.4(c)).

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 ≤ 50 MPa and either—
 - (i) the vertical reinforcement is not used as compressive reinforcement; or
 - (i) the vertical reinforcement ratio is not greater than 0.02, and a minimum horizontal reinforcement ratio of 0.0025 is provided.

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.

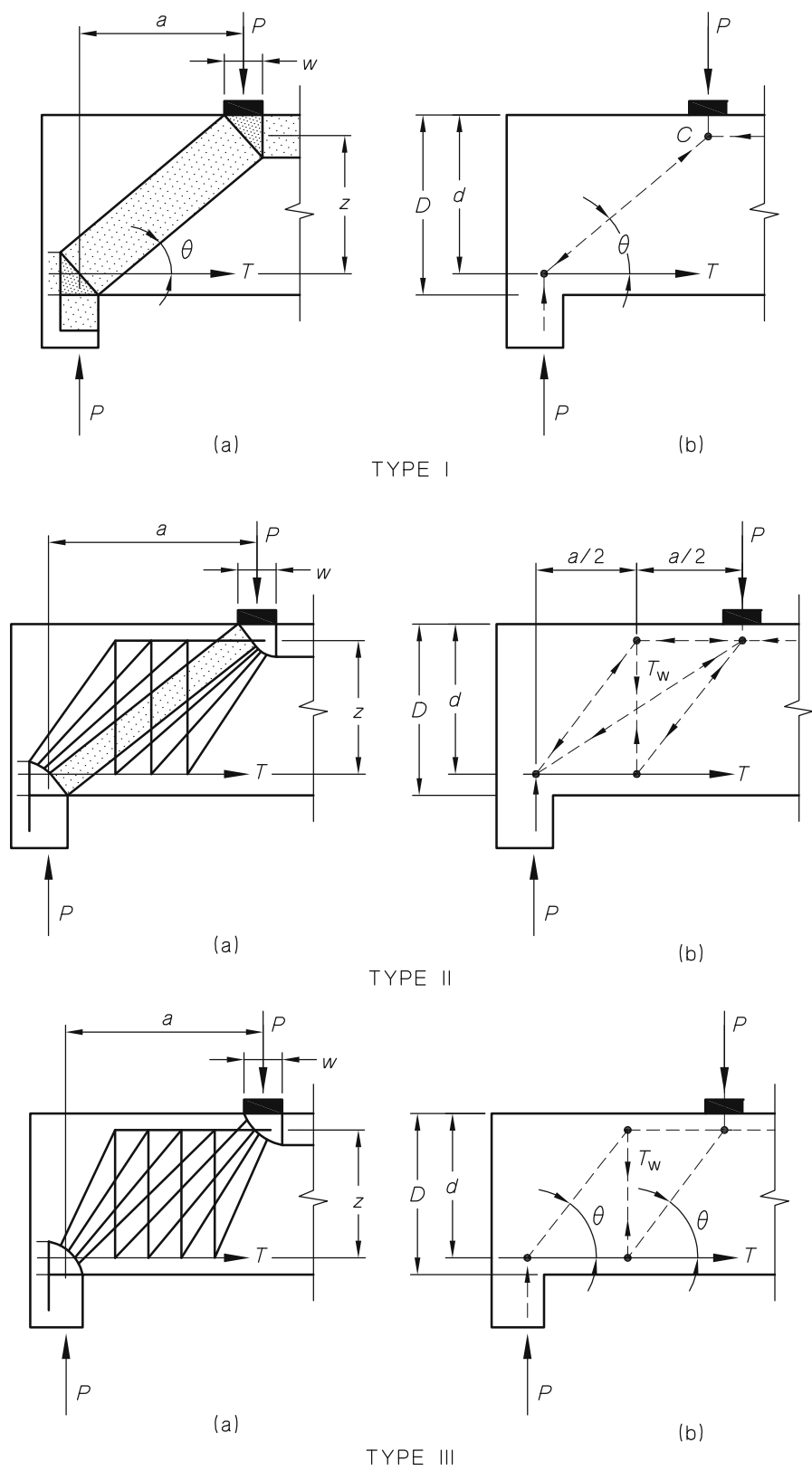


FIGURE 12.2.1 (a) STRUT-AND-TIE MODELS AND (b) SIMPLIFIED DESIGN MODELS

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 comply 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 comply 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 $\phi 0.9 f'_c \sqrt{(A_2 / A_1)}$ or $\phi 1.8 f'_c$, 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 1 longitudinally to 2 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 of crack control may be deemed to be satisfied if the stress in the reinforcement is not greater than the following:

- (a) Where a minor degree of control over cracking is required..... 250 MPa.
- (b) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view..... 200 MPa.
- (c) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes 150 MPa.

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 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—

$$L_{sy,tb} = \frac{0.5k_1k_3f_{sy}d_b}{k_2\sqrt{f'_c}} \geq 29k_1d_b \quad \dots 13.1.2.2$$

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 13.1.2.3(A).

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.

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

$$k_4 = 1.0 - K\lambda \quad (\text{within the limits } 0.7 \leq k_4 \leq 1.0)$$

$$k_5 = 1.0 - 0.04\rho_p \quad (\text{within the limits } 0.7 \leq k_5 \leq 1.0)$$

$$K = \text{a factor that accounts for the position of the bars being anchored with respect to the transverse reinforcement, with values as shown in Figure 13.1.2.3(B)}$$

$$\lambda = (\Sigma A_{tr} - \Sigma A_{tr,min})/A_s$$

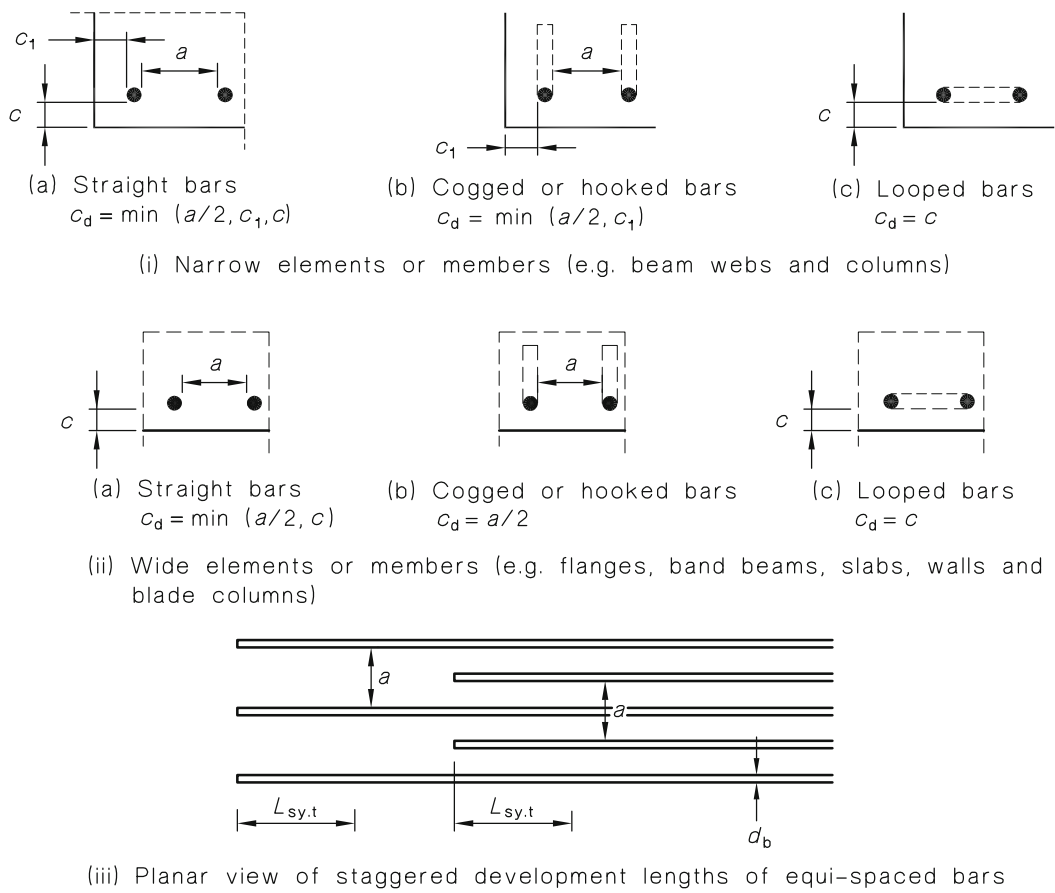
$$\Sigma A_{tr} = \text{sum of cross-sectional area of the transverse reinforcement along the development length } (L_{sy,t})$$

$$\Sigma A_{tr,min} = \text{sum of cross-sectional area of the minimum transverse reinforcement, which may be taken as } 0.25A_s \text{ for beams or columns and 0 for slabs or walls}$$

$$A_s = \text{cross-sectional area of a single bar of diameter } (d_b) \text{ being anchored}$$

$$\rho_p = \text{transverse compressive pressure (in MPa), at the ultimate limit state along the development length and perpendicular to the plane of splitting}$$

The product $k_3 k_4 k_5$ shall be not taken as less than 0.7.



NOTE: For wide elements or members (such as band beams, slabs, walls and blade columns), edge cover (c_1) should be ignored.

FIGURE 13.1.2.3(A) VALUES OF c_d

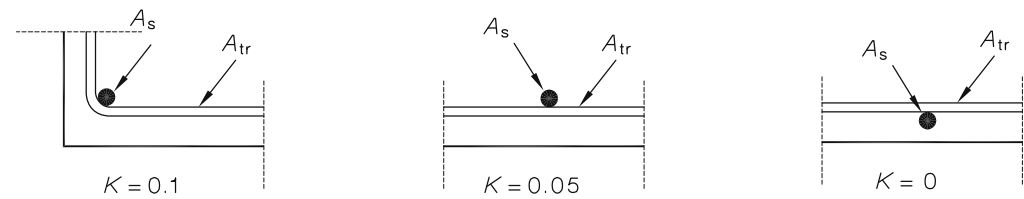


FIGURE 13.1.2.3(B) VALUES OF K FOR DIFFERENT BAR POSITIONS

13.1.2.4 Development length to develop less than the yield strength

The development length (L_{st}) to develop a tensile stress (σ_{st}), less than the yield strength (f_{sy}), shall be calculated from—

$$L_{st} = L_{sy,t} \frac{\sigma_{st}}{f_{sy}} \quad \dots 13.1.2.4$$

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 complying 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).

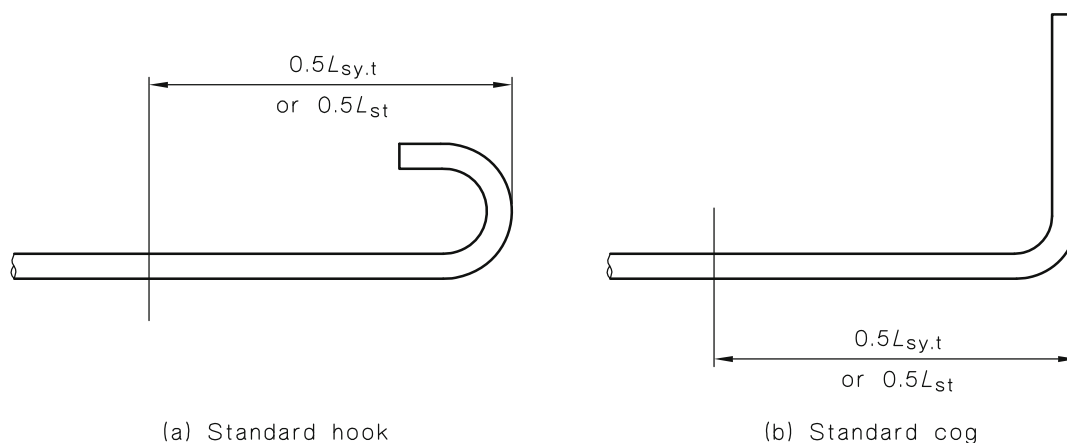


FIGURE 13.1.2.6 DEVELOPMENT LENGTH OF A DEFORMED BAR WITH A STANDARD HOOK OR COG

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 complying 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 complying 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 complying 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/m³;
- (c) the net bearing area of head shall be not less than 4 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 3 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 ΣA_{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 (σ_{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 bundle20%; and
- (b) for a 4-bar bundle33%.

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 2 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 (σ_{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:

- (a) Splices of reinforcement shall be made only as required or permitted on the design drawings or in specifications.
- (b) The splice shall be made by welding, by mechanical means, by end-bearing, or by lapping.
- (c) Splicing of reinforcement shall take into account the requirements of Clause 17.1.3 regarding the placement of concrete.
- (d) Splices required in bars in tension-tie members shall be made only by welding or mechanical means.
- (e) Lapped splices shall not be used for bars in compression or tension with diameter larger than 40 mm.
- (f) 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 $29k_1 d_b$ in Equation 13.1.2.2 does not apply); and

A1 | 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.

A1 | 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 $29k_1 d_b$, $k_7 L_{sy.t}$ 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 $L_{sy.t.lap}$.

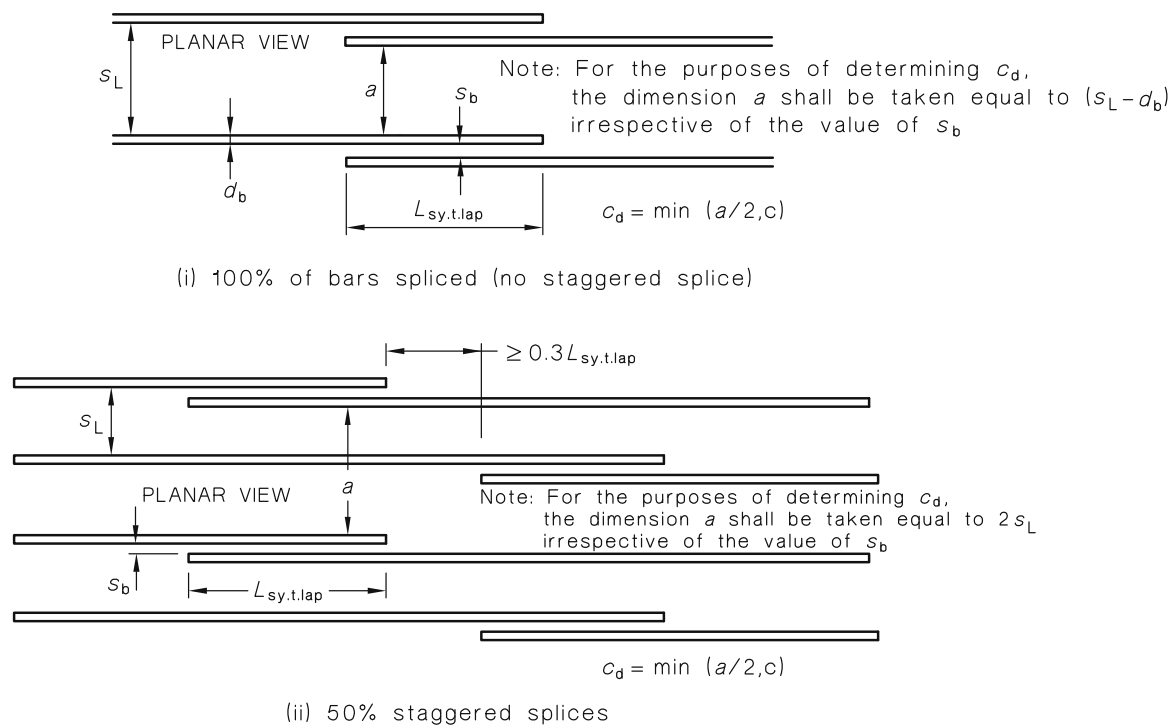


FIGURE 13.2.2 VALUE OF c_d FOR LAPPED SPLICES

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 3 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 3 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 bundle33%.

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	<i>L_{pt}</i> for gradual release	
	<i>f_{cp}</i> ≥ 32 MPa	<i>f_{cp}</i> < 32 MPa
Indented wire	100 <i>d_b</i>	175 <i>d_b</i>
Crimped wire	70 <i>d_b</i>	100 <i>d_b</i>
Ordinary and compact strand	60 <i>d_b</i>	60 <i>d_b</i>

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—

$$L_p = 0.145(\sigma_{pu} - 0.67 \sigma_{p,ef}) d_b \geq 60 d_b$$

... 13.3.2.2

where $\sigma_{p,ef}$ is the effective stress in the tendon after allowing for all losses.

Both σ_{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 comply 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.

SECTION 14 JOINTS, EMBEDDED ITEMS AND FIXINGS

14.1 JOINTS

14.1.1 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.

14.1.2 Construction joints

14.1.2.1 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.

14.1.2.2 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.

14.1.3 Movement joints

14.1.3.1 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.

14.1.3.2 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.

14.1.4 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.

14.2 EMBEDDED ITEMS

14.2.1 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.

14.2.2 Pipes

Embedded conduits and pipes shall comply 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.

14.2.3 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 comply with Clause 17.1.3.

14.3 FIXINGS

Fixings, including holding-down bolts, inserts and ferrules, shall comply 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 ϕ 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 ϕ 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 3.

SECTION 15 PLAIN CONCRETE PEDESTALS AND FOOTINGS

15.1 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 ϕ throughout the Section shall be determined from Table 2.2.2.

15.2 DURABILITY

Plain concrete members shall comply with the appropriate provisions of Section 4. The cover to any reinforcement shall comply with that determined in accordance with the appropriate provisions of Section 4.

15.3 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.

15.4 FOOTINGS

15.4.1 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.

15.4.2 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 ϕM_{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.

15.4.3 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 ϕV_u where—

$$V_u = 0.15bD(f'_c)^{1/3} \quad \dots 15.4.3(1)$$

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—

$$\phi V_u / [1 + (uM^*/8V^*aD)] \quad \dots 15.4.3(2)$$

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 9.2.1(A)]

a = dimension of the critical shear perimeter, which is parallel to the direction of bending being considered [see Figure 9.2.1(B)]

β_h = ratio given in Clause 9.2.1.5

SECTION 16 SLAB-ON-GROUND FLOORS, PAVEMENTS AND FOOTINGS

16.1 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.

16.2 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.

16.3 FOOTINGS

16.3.1 Reinforced footings

Two-way footings shall be designed in accordance with Section 9 and the minimum reinforcement shall be given by $0.19(D/d)^2 f'_{ct.f} / f_{sy}$.

16.3.2 Plain concrete footings

Plain concrete footings shall be designed in accordance with Clause 15.4.

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 comply 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, and 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 comply 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 compliance

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 compliance 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 compliance 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 comply 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 comply 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 comply with Clause 17.2.3.
- (c) Welding if required shall comply with AS/NZS 1554.3. Locational tack welding shall also comply 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 complying 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.

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.

NOTE: If site rebending is to occur, then the following procedures should be stipulated:

- (a) Rebending of bars should preferably be undertaken using an approved rebending tool. Bars should preferably 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 17.2.3.2. Bars should not be over-bent beyond the original bend, typically 90°.
- (b) A pipe with an internal diameter not greater than $2d_b$ inserted over the bar may be used, provided adequate care is taken and supervision provided; however, bending with pipes should be carried out with a single, smooth continuous action. The pipe should be not less than 1.2 m long.
- (c) If scabbling tools have to be used near bars because of concrete leakage or contamination, extreme care should be exercised to prevent any impact or damage to the bars.
- (d) The bar should be positioned with the initial bend of the bar clear of the concrete.

17.2.3.2 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) below, 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, spacers and tie wires used for this purpose shall be made of concrete, steel or plastics, as appropriate.

17.2.6 Lightning protection by reinforcement

Where lightning protection is to be provided by the reinforcement, the reinforcement shall comply 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 comply with AS/NZS 4672.1, as applicable.

Tendons shall not be galvanized.

In the absence of assurance, such as a manufacturer's certificate, the quality of tendons shall be established by testing in accordance with the applicable Standards.

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

Anchorage 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, complying 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 4 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 14.1.2 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 17.1.3(d) or 17.1.3(e) cannot be fulfilled, a construction joint complying with Clause 14.1.2 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 comply 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 comply with AS 3610. Stripping of forms and removal of formwork supports from members cast in situ shall comply 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 comply 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 comply 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 3 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 ϕR_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) $L_s / D > 280 / \sqrt{(D + 100)}$. . . 17.2.6.4

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 (<i>T</i>) °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 2 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 m³, 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.

APPENDIX A

REFERENCED DOCUMENTS

(Normative)

A1	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 (all methods)
	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 (all methods)
	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 and tables for inspection by attributes (all parts)
	1379	Specification and supply of concrete
	AS/NZS	
	1170	Structural design actions
	1170.0	Part 0: General principles
	1170.1	Part 1: Permanent, imposed and other actions
	1554	Structural steel welding
	1554.3	Part 3: Welding of reinforcing steel
	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
	3799	Liquid membrane-forming curing compounds for concrete

AS/NZS	
1768	Lightning protection
4671	Steel reinforcing materials
4672	Steel prestressing materials
4672.1	Part 1: General requirements
4672.2	Part 2: Testing requirements
BCA	Building Code of Australia
ASTM	
C42	Test methods 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 such 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 comply 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 comply 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 satisfied 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 comply 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 from Section 2, 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 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 comply 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 units shall not be greater than the expected variability selected in Table B4.3.
- (b) *Acceptance for strength* The test prototype shall be deemed to comply 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 comply 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 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 compliance 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 complying 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 comply 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

REQUIREMENTS FOR STRUCTURES SUBJECT TO EARTHQUAKE ACTIONS

(Normative)

C1 GENERAL

This Appendix 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 Appendix.

Concrete structures and members shall be designed and detailed depending on the value adopted for the structural ductility factor (μ) as follows:

- (a) For $\mu \leq 2$ designed and detailed in accordance with the main body of this Standard.
- (b) For $2 < \mu \leq 3$ designed and detailed in accordance with the main body of this Standard and this Appendix, as appropriate.

NOTE: For $\mu > 3$ the structure should be designed and detailed in accordance with NZS 1170.5 and NZS 3101.

C2 DEFINITIONS

For the purpose of this Appendix, 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 back to AS 1170.4. Those marked with an asterisk have been modified from those in AS 1170.4 to suit their application in this Appendix.

C2.1 Connection

Mechanical means that provide a load path for actions between structural elements, non-structural elements and structural and non-structural elements.

C2.2 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.

C2.3 Earthquake actions

Inertia-induced actions arising from the response to earthquake of the structure.

C2.4 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.

C2.5 Moment-resisting frame, intermediate*

Concrete moment-resisting frame designed and detailed in accordance with this Standard to achieve moderate structural ductility (see Table C3) and which complies with the specific earthquake detailing requirements of this Appendix.

C2.6 Moment-resisting frame, ordinary*

Moment-resisting frame, with no particular earthquake detailing, specified in this Standard (see Table C3).

C2.7 Shear wall

Wall (either loadbearing or non-loadbearing) designed to resist horizontal earthquake forces acting in the plane of the wall.

C2.8 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.

C2.9 Structural performance factor (S_p)

Numerical assessment of the additional ability of the total building (structure and other parts) to survive earthquake motion.

C2.10 Structural ductility factor (μ)

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.

C3 STRUCTURAL DUCTILITY FACTOR (μ) AND STRUCTURAL PERFORMANCE FACTOR (S_p)

The structural ductility factor (μ) and the structural performance factor (S_p) for concrete structures and members constructed in accordance with this Standard shall be as given in Table C3.

TABLE C3
STRUCTURAL DUCTILITY FACTOR (μ) AND STRUCTURAL PERFORMANCE FACTOR (S_p)

Structural system description	μ	S_p
Intermediate moment-resisting frames (moderately ductile) designed in accordance with this Standard and Paragraph C4 of this Appendix	3	0.67
Combined systems of intermediate moment-resisting frames and ductile shear-walls designed in accordance with this Standard and Paragraphs C4 and C5 of this Appendix	3	0.67
Ordinary moment-resisting frames designed in accordance with the main body of this Standard	2	0.77
Limited ductile shear-walls designed in accordance with the main body of this Standard	2	0.77
Other concrete structures not listed above	1.5	0.77

NOTE: The design of structures with $\mu > 3$ is outside the scope of this Standard (see Clause 2.1.2 and Paragraph C1 and the Commentary to AS 1170.4).

C4 INTERMEDIATE MOMENT-RESISTING FRAMES (IMRFs)

C4.1 General

Reinforced IMRFs and prestressed IMRFs shall be regarded as ductile if, in addition to the detailing requirements of this Standard, they satisfy the detailing requirements of Paragraphs C4.2 to C4.5 and Paragraph C4.6 respectively, 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.

C4.2 Beams

C4.2.1 Longitudinal reinforcement

Beams shall be provided with longitudinal reinforcement as follows:

- (a) The top and bottom face of the beam shall be continuously reinforced.
- (b) The area of reinforcement 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.

C4.2.2 Shear reinforcement

Beams shall be provided with shear reinforcement complying with the following requirements:

- (a) 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 $0.5D$.
- (b) The area of shear reinforcement (A_{sv}) shall be not less than $0.5b_v s / f_{sy.f}$.
- (c) 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

C4.3 Slabs

C4.3.1 General

Slabs shall comply with Paragraph C4.2.1, Items (a), (b) and (c). Two-way flat slabs forming part of a moment-resisting frame shall also comply with Paragraph C4.3.2.

C4.3.2 Reinforcement detailing in flat slabs

Reinforcement in flat slabs shall be located and anchored in accordance with the following:

- (a) 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.
- (b) 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—

$$1 / \left\{ 1 + (2/3) \left[\sqrt{(b_1 + d_o) / (b_t + d_o)} \right] \right\} \quad \dots \text{C4.3.2}$$

or 0.5, whichever is greater,

where

b_1 = 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

- (c) Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.
- (d) Continuous bottom reinforcement in the column strip shall be not less than one third of the area of the top reinforcement in the column strip at the support.
- (e) Not less than one-half of all bottom reinforcement at midspan shall be continuous through the support for the distance required to develop its yield strength at the face of the support.
- (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.

C4.4 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 ties shall be not greater than required by Clauses 10.7.3 and 10.7.4, with the first tie located at half this spacing from the face of the relevant support. The cross-sectional area of the ties shall be sufficient to satisfy the shear requirements for the column but not less than required by Clauses 10.7.3 and 10.7.4.

C4.5 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 Clauses 10.7.3 and 10.7.4 and the cross-sectional area of the ties not less than required by Clauses 10.7.3 and 10.7.4, 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.

C4.6 Prestressed IMRFs

C4.6.1 General

Beams containing tendons shall satisfy the detailing requirements of Clauses 10.7.3 and 10.7.4 when designed for the prestressing force in accordance with Paragraph C4.6.4.

Tendons shall be fully bonded and shall be detailed so that anchorages or transmission lengths are not placed—

- (a) within beam-column joint cores; or
- (b) to lie along critical shear planes.

C4.6.2 Connections

Connections between members shall—

- (a) have a strength greater than that of the members being joined;
- (b) have adequate ductility to withstand the calculated deformations; and
- (c) be designed to fail in a ductile manner under reversals of loading.

C4.6.3 Supports

Supports shall be designed so that horizontal or vertical displacements, or both, will not cause the failure or collapse of any part of the structure.

C4.6.4 Prestressed beams

Prestressed beams shall satisfy the following:

- (a) The quantity of tensile steel (tensioned plus untensioned) shall be such that the flexural strength of any beam section is greater than $1.1(M_{uo})_{min}$ at that section.
- (b) Longitudinal reinforcement shall comply with Paragraph C4.2.1.
- (c) Unless tensile steel is provided at various depths throughout the section and the depth of the neutral axis at the design moment is less than 0.22 times the overall depth of the section, the quantity of tensile steel (tensioned and non-tensioned) shall be such that—

$$(A_{pt} f_{py} + A_{st} f_{sy}) / b d f'_c \leq 0.2 \quad \dots \text{C4.6.4}$$

- (d) Shear reinforcement consisting of closed ties shall be provided to carry the total design shear force for a distance of $2D$ from the face of the support. The closed ties shall be of not less than 6 mm bar diameter, with a maximum spacing of 100 mm or $d_o/4$, whichever is the lesser.

C4.6.5 Prestressed columns

The flexural strength of any prestressed column section shall be greater than $1.1(M_{uo})_{min}$ at that section, after allowance for the effect of axial loads. The columns shall also comply with Paragraph C4.4 and Paragraph C4.6.4(d).

C4.6.6 Beam-column joints

Beam-column joints shall satisfy the following:

- (a) At least one prestressing tendon in the beam shall be located in the mid-depth of the beam at the joint.
- (b) All joints between columns and prestressed beams shall be confined by transverse column reinforcement throughout the joint. Such reinforcement shall be as required by Paragraph C4.4.

- (c) The interfaces at connections between precast members at beam column joints shall be roughened to ensure good shear transfer and the retention of any jointing material after cracking.

C5 DUCTILE SHEAR WALLS

C5.1 General

Shear walls shall be provided with boundary elements in accordance with Paragraph C5.3. 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.

C5.2 Reinforcement

The reinforcement ratio (p_w) in the vertical and horizontal direction shall be not less than 0.0025. The reinforcement shall be divided equally between the two wall faces if—

- (a) the wall thickness is greater than 200 mm; or
- (b) the design horizontal shear force on the cross-section is greater than $(A_g f'_c)/6$.

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.

C5.3 Boundary elements

In any storey, boundary elements shall be provided at discontinuous edges of shear 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 comply with Clause 10.7.4 of this Standard or, if the extreme fibre compressive stress calculated as above exceeds $0.2 f'_c$, with Paragraph C4.4 of this Appendix.

BIBLIOGRAPHY

A1 |

AS

3600 Supp1

Concrete structures—Commentary (Supplement to AS 3600)

3735

Concrete structures retaining liquids

4058

Precast concrete pipes (pressure and non-pressure)

4997

Guidelines to the design of maritime structures

AS/NZS

3000

Electrical installation (known as the Australia/New Zealand Wiring Rules)

3500

Plumbing and drainage (all parts)

4065

Concrete utility services poles

SA

HB 18

Conformity assessment

HB 18.28

Guide 28: Guidance on a third-party certification system for products

HB 64

Guide to concrete construction

HB 67

Concrete practice on building sites

HB 79

Alkali Aggregate reaction—Guidelines on Minimising the Risk of Damage to Concrete Structures in Australia

BCA

Building Code of Australia

CCAA

T56

Guide to residential slabs and footings in saline environments

NZS

1170

Structural design actions

1170.5

Part 5: Earthquake actions

3101

Concrete structures standard—The design of concrete structures

EN1992-1-2

Eurocode 2

Design of concrete structures

Part 1-2: General rules—Structural fire design

AMENDMENT CONTROL SHEET

AS 3600—2009

Amendment No. 1 (2010)

CORRECTION

SUMMARY: This Amendment applies to Clauses 1.7, 5.4.1, 5.5.2, 8.1.5, 11.4, 13.1.4 and 13.2.2, Tables 5.6.3 and 5.6.4, Equations 7.2.4(4), 8.1.3(1), 8.1.3(2) and 8.4.3, Appendix A, and Bibliography.

Published on 17 November 2010.

NOTES

NOTES

Standards Australia

Standards Australia develops Australian Standards® and other documents of public benefit and national interest. These Standards are developed through an open process of consultation and consensus, in which all interested parties are invited to participate. Through a Memorandum of Understanding with the Commonwealth Government, Standards Australia is recognized as Australia's peak non-government national standards body. Standards Australia also supports excellence in design and innovation through the Australian Design Awards.

For further information visit www.standards.org.au

Australian Standards®

Committees of experts from industry, governments, consumers and other relevant sectors prepare Australian Standards. The requirements or recommendations contained in published Standards are a consensus of the views of representative interests and also take account of comments received from other sources. They reflect the latest scientific and industry experience. Australian Standards are kept under continuous review after publication and are updated regularly to take account of changing technology.

International Involvement

Standards Australia is responsible for ensuring the Australian viewpoint is considered in the formulation of International Standards and that the latest international experience is incorporated in national Standards. This role is vital in assisting local industry to compete in international markets. Standards Australia represents Australia at both the International Organization for Standardization (ISO) and the International Electrotechnical Commission (IEC).

Sales and Distribution

Australian Standards®, Handbooks and other documents developed by Standards Australia are printed and distributed under license by SAI Global Limited.

For information regarding the development of Standards contact:

Standards Australia Limited
GPO Box 476
Sydney NSW 2001
Phone: 02 9237 6000
Fax: 02 9237 6010
Email: mail@standards.org.au
Internet: www.standards.org.au

For information regarding the sale and distribution of Standards contact:

SAI Global Limited
Phone: 13 12 42
Fax: 1300 65 49 49
Email: sales@sai-global.com



ISBN 0 7337 9347 9

This page has been left intentionally blank.