NZS 3101.1:2006
NZS 3101.2:2006
Incorporating Amendment No. 1, 2, and 3

STANDARDS NEW ZEALAND
PAEREWA AOTEAROA

New Zealand Standard

Concrete structures standard

Part 1: The design of concrete structures
Part 2: Commentary on the design of concrete structures

NZS 3101.1:2006 & NZS 3101.2:2006

Single User PDF Terms & Conditions

COMMITTEE REPRESENTATION

This standard was prepared by the P 3101 Concrete Design Committee for the Standards Council established under the Standards Act 1988.

The committee consisted of representatives of the following organisations:

Name	Nominating Organisation
Dene Cook	Cement and Concrete Association of New Zealand (Chair)
Peter Attwood	New Zealand Contractor's Federation
Derek Chisholm	BRANZ
Richard Fenwick	Co-opted
Don Kirkcaldie	IPENZ
Graeme Lawrance	Department of Building and Housing
Len McSaveney	New Zealand Concrete Society Inc
John Mander	University of Canterbury
Les Megget	The University of Auckland
Bob Park	Co-opted
Ashley Smith	NZ Structural Engineering Society
Keith Towl	Business New Zealand

ACKNOWLEDGEMENT

Standards New Zealand gratefully acknowledges:

(a) The significant contribution towards the development of this Standard made by (the late) Professor Bob Park;

(b) The assistance provided by Stefano Pampanin for work on Appendix B; and

(c) The American Concrete Institute for permission to use extracts from ACI 318-02, Building Code Requirements for Reinforced Concrete. Appendix CF contains specific information related to ACI 318 provisions.

Published by Standards New Zealand, PO Box 1473, Wellington 6140. Telephone: (03) 943 4259, Website: www.standards.govt.nz.

AMENDMENTS

No.	*Date of issue*	*Description*	*Entered by, and date*
1	July 2006	Corrects minor errors	SNZ July 2006
2	August 2008	Amends clauses, equations, figures, notations, referenced documents and tables	SNZ August 2008
3	August 2017	Amends all sections, focusing on the areas outlined in the foreword, page ix	SNZ August 2017

NZS 3101.1:2006

Incorporating Amendment No. 1, 2, and 3

New Zealand Standard

Concrete structures standard

Part 1: The design of concrete structures

ISBN (Print) 978-1-77673-089-8
ISBN (PDF) 978-1-77673-090-2

NOTES

CONTENTS

Appendix

Table

Figure

REFERENCED DOCUMENTS

NEW ZEALAND STANDARDS

A3 NZS 1170:- - - -	Structural design actions
Part 5:2004	Earthquake actions – New Zealand
NZS 3106:2009	Design of concrete structures for the storage of liquids
NZS 3109:1997	Concrete construction
NZS 3112:- - - -	Methods of test for concrete
Part 1:1986	Tests relating to fresh concrete
Part 2:1986	Tests relating to the determination of strength of concrete
A3 NZS 3122:2009	Specification for Portland and blended cements (General and special purpose)
NZS 3152:1974 (R) 1980	Specification for the manufacture and use of structural and insulating lightweight concrete
NZS 3404:- - - -	Steel structures standard
Part 1:1997	Steel structures standard
A3 Part 1:2009	Steel structures standard Materials, fabrication, and construction

JOINT AUSTRALIA/NEW ZEALAND STANDARDS

AS/NZS 1170:- - - -	Structural design actions
Part 0: 2002	General principles
Part 1: 2002	Permanent, imposed and other actions
A3 Part 2: 2011	Wind actions
Part 3: 2003	Snow and ice actions
AS/NZS 1554:- - - -	Structural steel welding
A3 Part 3: 2014	Welding of reinforcing steel
AS/NZS 2699:- - - -	Built-in components for masonry construction
Part 3:2002	Lintels and shelf angles (durability requirements)
A3 AS/NZS 3582:- - - -	Supplementary cementitious materials
Part 3:2016	Amorphous silica
AS/NZS 4671:2001	Steel reinforcing materials
A2 AS/NZS 4672:- - - -	Steel prestressing materials
Part 1:2007	General requirements
A3 AS/NZS 4680:2006	Hot-dip galvanised (zinc) coatings on fabricated ferrous articles

AMERICAN STANDARDS

American Concrete Institute

ACI 210R-93	Erosion of Concrete in Hydraulic Structures (reapproved 1998)
A3 ACI 318-14	Building code requirements for structural concrete

American Society for Testing and Materials

ASTM C1152-04	Standard test method for acid-soluble chloride in mortar and concrete

AUSTRALIAN STANDARDS

AS 1012:- - - -	Methods of testing concrete
Part 10:2000	Determination of indirect tensile strength of concrete cylinders (“Brazil” or splitting test)
Part 11:2000	Determination of the modulus of rupture
A3 Part 13:2015	Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
Part 16:1996	Determination of creep of concrete cylinders in compression

Part 20.1:2016	Determination of chloride and sulfate in hardened concrete and concrete aggregates	A3
AS 1214:2016	Hot-dip galvanised coatings on threaded fasteners (ISO metric coarse thread series)	
AS 1313:1989	Steel tendons for prestressed concrete – Cold-worked high-tensile alloy steel bars for prestressed concrete	
AS 1530:- - - -	Methods for fire tests on building materials, components and structures	A3
Part 4:2014	Fire-resistance tests for elements of building construction	
AS 3582:- - - -	Supplementary cementitious materials for use with portland and blended cement	
Part 1:2016	Fly ash	
Part 2:2016	Slag – Ground granulated iron blast-furnace	
AS 3600:2009	Concrete structures	
AS 4072:- - - -	Components for the protection of openings in fire-resistant separating elements	
Part 1:2005	Service penetrations and control joints	

BRITISH STANDARDS

A3

BS 1377:- - - -	Methods of test for soils for civil engineering purposes	
Part 3:1990	Chemical and electro-chemical tests	
BS 1881:----	Testing concrete	
Part 124:2015	Methods for analysis of hardened concrete	A3
BS 8204:- - - -	Screeds, bases and in-situ floorings	
Part 2:2003	Concrete wearing surfaces	
BS 8500:- - - -	Concrete. Complementary British Standard to BS EN 206-1	A3
Part 1:2015	Method of specifying and guidance for the specifier	

EUROCODES

EN 1992:- - - -	Eurocode 2: Design of concrete structures	
Part 1.2:2004	General rules. Structural fire design.	A3

GERMAN STANDARDS

DIN 4030:- - - -	Assessment of water, soil and gases for their aggressiveness to concrete	A3
Part 2:2008	Sampling and analysis of water and soil samples	

A2 DIN 4102:- - - - Fire behaviour of building materials and building components
Part 2:1977 Building components; definitions, requirements and tests

INTERNATIONAL STANDARDS

ISO 834:- - - - Fire-resistance tests – Elements of building construction
Part 1:1999 General requirements

A3 ISO 15835:- - - - Steels for the reinforcement of concrete -- Reinforcement couplers for mechanical splices of bars
Part 1:2009 Requirements
Part 2:2009 Test methods

OTHER PUBLICATIONS

Alkali aggregate reaction: Minimising the risk of damage to concrete: Guidance notes and model specification clauses (Technical Report 3), 2004, Cement & Concrete Association of New Zealand.

Approved Code of Practice for the Safe Handling, Transportation and Erection of Precast Concrete, Occupational Safety and Health Service, Department of Labour, 2002.

A3 Bridge Manual (SP/M/022) third edition, New Zealand Transport Agency.

New Zealand Building Code Compliance Documents and Handbook, Department of Building and Housing, (formerly the Building Industry Authority), 1992 (as amended up to March 2005).

Creep and Shrinkage in Concrete Bridges, RRU Bulletin 70, Transit New Zealand 1984.

CEB-FIP Model Code 1990.

A3 European Organisation for Technical Approvals (2013). EOTA TR045, Design of Metal Anchors for Use in Concrete under Seismic Actions. Brussels. Belgium.

European Organisation for Technical Approvals (2013). ETAG 001 Annex E, Assessment of Metal Anchors for Use in Concrete under Seismic Actions. Brussels. Belgium.

NEW ZEALAND LEGISLATION

Building Act 2004

FOREWORD

This revision of NZS 3101 has been written with the objective of producing a concrete design standard which is:

(a) Compatible with the loading standards AS/NZS 1170 and NZS 1170.5, and other referenced loading standards;

(b) Intended to provide, in due course (once cited) a verification method for compliance with the New Zealand Building Code;

(c) Organised in component focused sections, for ease of use.

During the revision process, the opportunity has been taken to incorporate various technical advancements and improvements that have been developed since 1995. The non-seismic sections of this Standard are largely based upon ACI 318-02.

The following is a summary of some of the key changes in NZS 3101:

(d) The sections of the standard are component focused rather than force focused;

(e) Summary tables suitable as quick reference guides are provided in the commentary to the sections on beams, columns, walls, and joints;

(f) The expected curvature ductility that can be achieved from the specified detailing has been summarised;

(g) The seismic design philosophy has been made compatible with NZS 1170.5;

(h) Two approaches to capacity design have been included in Appendix D;

(i) The Standard now includes information on Grade 500 reinforcement;

(j) The durability section includes new information for zone C exposure classifications. Information is provided for structures with a specified intended life of 100 years. The durability section has been extended to include guidance on chemical exposure, aggressive soils, abrasion resistance, and fastening protection;

(k) Fire has been amended to include the latest revisions from AS 3600, and guidance is provided on the fire design of thin panel walls that are typically found in warehouse type structures;

(l) An Appendix has been provided on the design of fibre reinforced members;

(m) New provisions have been provided for the structural design of thin panel walls. These provisions include the latest developments in ACI 318 and research results of testing conducted in New Zealand; A3

(n) A new section has been provided on precast concrete;

(o) The strut and tie method of analysis has been introduced into Part 1 of the Standard. The information is based upon ACI 318-02;

(p) An Appendix has been provided for the design of ductile jointed precast systems.

Amendment No. 3 has involved significant revision and the introduction of new material, focusing on five aspects: A3

(q) A section on elongation has been added;

(r) Creep and shrinkage values for concrete have been added;

(s) The difference between design deformations and peak deformations, and between design displacements and peak displacements, has been introduced for the serviceability limit state, ultimate limit state and maximum considered level earthquakes;

(t) Substantial changes relating to design of walls have been implemented;

(u) Precast seating details have been significantly revised.

The standards development committee for Amendment No. 3 are acknowledged for their efforts.

NOTES

NEW ZEALAND STANDARD

CONCRETE STRUCTURES STANDARD

Part 1 – The Design of Concrete Structures

1 GENERAL

1.1 Scope

1.1.1 *Relationship to NZ Building Code*

1.1.1.1 *Minimum requirements*

This Standard sets out the minimum requirements for the design of reinforced and prestressed concrete structures. In addition to these requirements, every load or force acting on a structure shall have one or more dependable load paths that can transfer the force to the foundation soils. Each load path shall satisfy the fundamental structural design requirements of equilibrium and displacement compatibility. A3

This Standard does not cover the design of brittle elements. A brittle element is defined as a structural member that does not satisfy the minimum requirements specified in this Standard. A2

1.1.1.2 *Non Specific Terms*

Where this Standard has provisions that are in non-specific or unquantified terms then these do not form part of the verification method for the New Zealand Building Code and the proposed details must be submitted to a building consent authority for approval as part of the building consent application. This includes but is not limited to where the standard calls for special studies, a rational analysis, for engineering judgement to be applied or where the Standard requires tests to be "suitable" or "appropriate". "Assessed" is used in code clauses where exact practical methods of calculation are not available but approximate methods are used to determine the likely order of an action. A3

1.1.2 *Application to bridges*

While this standard has been developed with the intent that it be generally applicable to the design of bridges, and is referenced by the New Zealand Transport Agency Bridge Manual, some aspects are recognised to not be adequately covered by this Standard and designers are advised to make reference to appropriate specialised bridge design technical literature. Aspects of bridge design for which reference to the technical literature should be made include the following: A3

(a) Design for the combination of shear, torsion and warping in box girders;
(b) Design for deflection control taking into account the effects of creep, shrinkage and differential shrinkage and differential creep;
(c) Design for stress redistribution due to creep and shrinkage;
(d) Design for the effects of temperature change and differential temperature. (Refer to the New Zealand Transport Agency Bridge Manual for these design actions);
(e) Design for the effects of heat of hydration. This is particularly an issue where thick concrete elements are cast as second stage construction and their thermal movements are restrained by previous construction;
(f) Design for shear and local flexural effects, which may arise where out of plane moments are transmitted to web or slab members, or where the horizontal curvature of post-tensioned cables induces such actions;
(g) Seismic design of piers, where curvature ductility demand (material strain) exceeds the maximum permissible values in 2.6.1.3; A2
(h) Design for shear flow between flanges and webs in members. A3

1.1.3 *Materials and workmanship requirements*

It is applicable to structures and parts of structures constructed in accordance with the materials and workmanship requirements of NZS 3109.

1.1.4 *Interpretation*

1.1.4.1 *"Shall" and "should"*

In this Standard the word "shall" indicates a requirement that is to be adopted in order to comply with the Standard. The word "should" indicates practices which are advised or recommended

1.1.4.2 *Clause cross-references*

Cross-references to other clauses or clause subdivisions within this Standard quote the number only, for example: "… is given by 8.6.2.3 (a)".

A3

1.1.4.3 *Commentary*

The Commentary to this Standard, NZS 3101:Part 2:2006, does not contain requirements essential for compliance with this Standard but explains, summarises technical background and suggests approaches which satisfy the intent of the Standard.

1.2 Referenced documents

The full titles of reference documents cited in this Standard are given in the "Referenced Documents" list immediately preceding the Foreword.

1.3 Design

1.3.1 *Design responsibility*

The design of a structure or the part of a structure to which this Standard is applied shall be the responsibility of the design engineer or his or her representative.

1.3.2 *Design information*

Consent documentation and the drawings or specification, or both, for concrete members and structures shall include, where relevant, the following:

(a) The reference number and date of issue of applicable design Standards used;
(b) The fire resistance ratings, if applicable;
(c) The concrete strengths;
(d) The reinforcing and prestressing steel Class and Grades used and the manufacturing method employed in the production of the reinforcing steel;
(e) The testing methods, reporting requirements and acceptance criteria for any tests of material properties, components or assemblages that are required by this Standard.
(f) The locations and details of planned construction joints;
(g) Any constraint on construction assumed in the design;
(h) The camber of any members.

1.4 Construction

1.4.1 *Construction reviewer*

All stages of construction of a structure or part of a structure to which this Standard applies shall be adequately reviewed by a person who, on the basis of experience or qualifications, is competent to undertake the review.

1.4.2 *Construction review*

The extent of review to be undertaken shall be nominated by the design engineer, taking into account those materials and workmanship factors which are likely to influence the ability of the finished construction to perform in the predicted manner.

1.5 Definitions

The following terms are defined for general use in this Standard, noting that specialised definitions appear in individual sections:

ADMIXTURE. A material other than Portland cement, aggregate, or water added to concrete to modify its properties.

AGGREGATE. Inert material which is mixed with Portland cement and water to produce concrete.

ANCHORAGE. The means by which prestress force is permanently transferred to the concrete. Also, the method of ensuring that reinforcing bars and fixings acting in tension or compression are tied into a concrete member.

AXIS DISTANCE. The weighted average distance of a group of longitudinal bars or tendons from the axes of the bars to the nearest exposed surface; the weighting being conducted on the basis of bar area. A2

BEAM. An member subjected primarily to loads and forces producing flexure.

BINDER. A constituent phase of concrete, comprising a blend of cementitious materials, which on reaction bind the aggregates together into a homogenous mass.

BONDED TENDON. Prestressing tendon that is bonded to concrete either directly or through grouting.

CAPACITY DESIGN. In the capacity design of structures subjected to earthquake forces, regions of members of the primary lateral force-resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural members are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

COLUMN. An element subjected primarily to compressive axial loads.

COMPOSITE CONCRETE FLEXURAL MEMBERS. Concrete flexural members of precast and/or cast-in-place concrete elements or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

CONCRETE. A mixture of Portland cement or any other hydraulic cement, sand, coarse aggregate and water.

CONCURRENCY. The simultaneous occurrence of seismic actions not necessarily aligned to any principal direction of the structure, which result in actions in more than one principal direction of the structure. A3

CONSTRUCTION JOINT. An intentional joint in concrete work detailed to ensure monolithic behaviour at both the serviceability and ultimate limit states.

CORBEL. A short cantilver beam springing from a column, wall or the side of a beam which is used to provide a support surface. A3

CURVATURE FRICTION. Friction resulting from bends or curves in the specified prestressing tendon profile.

DEFORMED REINFORCEMENT. Deformed reinforcing bars conforming to AS/NZS 4671.

DESIGN ENGINEER. A person who, on the basis of experience or qualifications, is competent to design structural elements of the structure under consideration to safely resist the design loads or effects likely to be imposed on the structure.

A3 DESIGN INTER-STOREY DRIFT, DISPLACEMENT AND DEFORMATION. Design values are identified in NZS 1170.5. These values are predicted to be sustained several times during a design serviceability limit state or an ultimate limit state earthquake. These values are appropriate for assessing structural damage in ductile structural elements.

DEVELOPMENT LENGTH. The embedded length of reinforcement required to develop the design strength of the reinforcement at a critical section (see 8.6).

DIAPHRAGM. Elements transmitting in-plane lateral forces to resisting elements.

A3 DOUBLY REINFORCED WALL. A reinforced concrete wall with at least two layers of vertical reinforcement.

DUAL STRUCTURE. Lateral force-resisting system which consists of moment resisting frames and structural walls.

DUCTILE FRAME. A structural frame possessing ductility.

DUCTILITY. The ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.

EFFECTIVE PRESTRESS. The stress remaining in the tendons after all calculated losses have been deducted, excluding the effects of superimposed loads and the weight of the member.

EFFECTIVE THICKNESS. The effective thickness of ribbed or hollow-core wall panels is the net cross-sectional area divided by the width.

A3 ELONGATION. The increase in axial length that occurs when flexural or shear cracks form in a concrete member resulting in tensile strains that are greater than the compression strains and consequently the axial length increases. When, due to seismic action, plastic hinges form, or where rocking of walls against foundations or beams against columns occurs, the magnitude of elongation can be greatly increased and this has important implications for the performance of structures.

EMBEDMENT LENGTH. The length of embedded reinforcement provided beyond a critical section.

END ANCHORAGE. Length of reinforcement, or a mechanical anchor, or a hook, or combination thereof, required to develop stress in the reinforcement; mechanical device to transmit prestressing force to concrete in a post-tensioned member.

FIRE RESISTANCE. The ability of a structure or part of it to fulfil its required functions (load-bearing and/or separating function) for a specified exposure to fire, for a specified time. Refer to EN 1992-1-2.

A3 FIRE RESISTANCE RATING (FRR). The term used to classify fire resistance of building elements as determined in the standard test for fire resistance, or in accordance with a specific calculation method verified by experimental data from standard fire resistance tests in accordance with AS 1530.4. It comprises three numbers giving the time in minutes for which each of the criteria for stability, integrity and insulation are satisfied.

FIRE-SEPARATING FUNCTION. The function served by the boundary elements of a fire compartment, which are required to have a fire resistance rating, in preventing a fire in that compartment from spreading to adjoining compartments.

FLAT SLAB. A two-way continuous slab supported on columns, with no beams between supporting columns.

GRAVITY LOAD DOMINATED FRAMES. A frame with full or limited ductility capacity in which the design strength of members at the ultimate limit state is governed by gravity loads rather than by the most adverse combination of gravity loads and earthquake forces.

HOLLOW-CORE SLAB OR WALL. A slab or wall having mainly a uniform thickness and containing essentially continuous voids, where the thickness of concrete between adjacent voids and the thickness of concrete between any part of a void and the nearest surface is the greater of either one-fifth the required effective thickness of the hollow-core or 25 mm. Hollow-core units have no shear reinforcement.

INSULATION. 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.

INTEGRITY. The 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.

JACKING FORCE. In prestressed concrete, the temporary force exerted by the device which introduces the tension into the tendons.

LEDGE. A seating for beams and/or precast floor and roof units or precast cladding panels on a wall or above the uppermost flexural reinforcement in a beam or a corbel built out from the side of a wall or beam. A3

LIMIT STATE

SERVICEABILITY LIMIT STATE (SLS). The state at which a structure becomes unfit for its intended use through deformation, vibratory response, degradation or other operational inadequacy.

ULTIMATE LIMIT STATE (ULS). The state at which the design strength or ductility capacity of the structure is exceeded, when it cannot maintain equilibrium and becomes unstable.

LOADING STANDARD, REFERENCED. One of the documents referenced in C1.1.1 of the Concrete Structures Commentary which gives the range of design actions for which a structure is to be designed in order to satisfy the performance requirements of the New Zealand Building Code Clauses B1 and B2.

LOADS AND FORCES

LOAD, DEAD. The weight of all permanent components of a structure, including partitions, finishes, and permanently fixed plant and fittings.

LOAD, DESIGN. Combinations of loads and forces used in design as set out in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard for the applicable limit state. In seismic design for the ultimate limit state, the design load may be either the ultimate limit state forces or the forces resulting from the capacity design procedure depending on the case being considered.

LOAD, LIVE. Loads assumed or known to result from the occupancy or use of a structure, with values as specified in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard.

FORCE, EARTHQUAKE. Forces assumed to simulate earthquake effects as defined by AS/NZS 1170 and NZS 1170.5 or other referenced loading standard.

LOAD-BEARING FUNCTION. The ability of a structure or member to sustain specified actions under all relevant circumstances (e.g. fire – EN 1992-1-2). A3

MAXIMUM CONSIDERED EARTHQUAKE (MCE). The actions associated with the maximum considered earthquake are taken as 1.5 times the corresponding actions associated with the ultimate limit state seismic actions. A3

MEMBER. A physically distinguishable part of a structure such as a wall, beam, column, slab or connection.

NORMAL DENSITY CONCRETE. Concrete, excluding reinforcement with a density of between 2250 and 2350 kg/m^3.

OVERSTRENGTH. The overstrength value takes into account factors that may contribute to strength such as higher than specified strengths of the steel and concrete, steel strain hardening, confinement of concrete, and additional reinforcement placed for construction and otherwise unaccounted for in calculations.

P-DELTA EFFECT. Refers to the structural actions induced as a consequence of the axial loads being displaced laterally away from the alignment of the action.

A3 PEAK INTER-STOREY DRIFT, DISPLACEMENT AND DEFORMATION. Peak values are predicted to occur only once during an earthquake (ULS, SLS and MCE). These values are required in the design for the supports for stairs, ramps, precast floor units and panels, particularly where the collapse of these or other structural elements could block an egress route or create a falling hazard in public areas. The peak value is to be taken as $1/S_p$ times the deformations associated with a particular limit state (ULS, SLS and MCE). A peak MCE deformation is $1.5/S_p$ times the design ultimate limit state deformation given in NZS 1170.5. In addition peak values need to be considered in the design of brittle elements.

PIER. A vertical member (usually associated with bridge structures) subjected primarily to both compressive axial loads and seismic forces.

PLAIN CONCRETE. Concrete that contains less than the minimum reinforcement required by this Standard.

PLAIN REINFORCEMENT. Reinforcing bars conforming to AS/NZS 4671 and having no significant projections other than bar identification marks.

A2 PLASTIC HINGE (REGION). (POTENTIAL PLASTIC HINGE REGION.) Regions in a member as defined in this Standard, where significant rotations due to inelastic strains can develop under flexural actions

PRIMARY PLASTIC REGION. A potential plastic region identified in the ductile collapse mechanism, which is used as the basis for capacity design.

REVERSING PLASTIC HINGE. A potential plastic region which may be subjected to both negative and positive inelastic deformation in an earthquake.

SECONDARY PLASTIC REGION. A potential plastic region which may develop due to member elongation or higher mode effects in a structure.

UNIDIRECTIONAL PLASTIC HINGE. A plastic region which may be subjected to either negative or positive inelastic deformation rotation (but not both) in an earthquake.

PLASTIC HINGE (REGION) LENGTH.

DUCTILE DETAILING LENGTH. ℓ_y the length in which ductile detailing is required. In this length, inelastic deformation may develop in the reinforcement or spalling may occur in the concrete.

EFFECTIVE PLASTIC HINGE (REGION) LENGTH. ℓ_p, the length used to calculate curvature (2.6.1.3.3).

EFFECTIVE PLASTIC REGION LENGTH. As for ℓ_p, but specifically identified as L_{cb}, where it is used to calculate the shear strain in a diagonally reinforced coupling beam, (2.6.1.3.3).

POST-TENSIONING. A method of prestressing in which the tendons are tensioned after the concrete has hardened.

A2 POTENTIAL PLASTIC HINGE REGION. See plastic region.

PRECAST CONCRETE. A concrete element cast-in other than its final position in the structure.

PRESTRESSED CONCRETE. Concrete in which internal stresses of such magnitude and distribution have been introduced that the stresses resulting from loads are counteracted to some extent to ensure the required strength and serviceability are maintained.

PRETENSIONING. A method of prestressing in which the tendons are tensioned before the concrete is placed.

PRISMATIC MEMBER. A member of constant cross section along its length.

REINFORCED CONCRETE. Concrete containing steel reinforcement, and designed and detailed so that the two materials act together in resisting loads and forces.

RIBBED SLAB. A slab incorporating parallel ribs spaced at not greater than 1500 mm centre-to-centre in one or two directions.

A3

SELF-COMPACTING CONCRETE. Concrete that flows and consolidates under its own weight without the need of vibration. SCC is characterised by high flowability, filling ability and passing ability through congested reinforcement and shall exhibit adequate static and dynamic stability.

SELF-STRAIN ACTION. Structural actions that are induced as a result of one or more of the following: A3

(a) Temperature change, including differential temperature;
(b) Creep and shrinkage of concrete;
(c) Actions induced by heat of hydration of cement;
(d) Elongation in a plastic hinge.

SEPARATING FUNCTION. The ability of a separating member to prevent fire spread by passage of flames or hot gases (integrity) or ignition beyond the exposed surface (thermal insulation during the relevant fire). (Refer to EN 1992-1-2:. A3

SINGLY REINFORCED WALL. A reinforced concrete wall with one layer of vertical reinforcement, usually in the centre of the wall. A3

SPECIAL STUDY. A procedure for justifying departure from this Standard, or for determining information not covered by this Standard, which is consistent with AS/NZS 1170.0 and its Appendices A and B.

SPECIFIED INTENDED LIFE. For a building or structure, the period of time for which the building is proposed to be used for its intended use as stated in an application for a building consent.

SPIRAL. Continuously wound reinforcement in the form of a cylindrical helix.

STABILITY. The ability of a member to maintain its structural function when deformed.

STIRRUP OR TIES. Reinforcement used to resist shear and torsion in a structural member; typically bars or wires (smooth or deformed) bent around the longitudinal reinforcement and located perpendicular to, or at an angle to longitudinal reinforcement (the term “stirrups” is usually applied to lateral reinforcement in beams and the term “ties” to those in columns). Stirrup ties or hoops may also provide confinement to compressed concrete, stability to reinforcing bars subject to compression and clamping in shear-friction mechanisms in addition to acting as shear and torsional reinforcement.

STRENGTH

STRENGTH, COMPRESSIVE OF CONCRETE. The crushing resistance of cylindrical specimens of concrete, prepared, cured and tested in accordance with the standard procedures prescribed in Sections 3, 4 and 6 of NZS 3112:Part 2. This is normally denoted by the general symbol f'_c.

STRENGTH, DESIGN. The nominal strength multiplied by the appropriate strength reduction factor.

STRENGTH, LOWER CHARACTERISTIC YIELD OF NON-PRESTRESSED REINFORCEMENT. That yield stress below which fewer than 5 % of results fall when obtained in a properly conducted test programme. Refer to AS/NZS 4671.

STRENGTH, NOMINAL. The theoretical strength of a member section, calculated using the section dimensions as detailed and the lower characteristic reinforcement strengths as defined in this Standard and the specified compressive strength of concrete.

STRENGTH, OVER. See Overstrength.

STRENGTH, PROBABLE. The theoretical strength of a member section calculated using the expected mean material strengths as defined in this Standard.

STRENGTH REDUCTION FACTOR. A factor used to multiply the nominal strength to obtain the design strength.

STRENGTH, SPECIFIED COMPRESSIVE OF CONCRETE. A singular value of strength, normally at age 28 days unless stated otherwise, denoted by the symbol f'_c, which classifies a concrete as to its strength class for purposes of design and construction. It is that level of compressive strength which meets the production standards required by Section 6 of NZS 3109.

STRENGTH, UPPER CHARACTERISTIC BREAKING STRENGTH OF NON-PRESTRESSED REINFORCEMENT. That maximum tensile strength below which greater than 95 % of the results fall when obtained in a property conducted test programme.

STRUCTURAL. A term used to denote an element or elements which provide resistance to loads and forces acting on a structure.

STRUCTURAL ADEQUACY. The ability of a member to maintain its structural function when exposed to fire.

STRUCTURAL DUCTILITY FACTOR. A numerical assessment of the ability of a structure to sustain cyclic inelastic displacements.

STRUCTURAL LIGHTWEIGHT CONCRETE. A concrete containing lightweight aggregate and having a unit weight not exceeding 1850 kg/m^3. In this Standard, a lightweight concrete without natural sand is termed “all-lightweight concrete”, and lightweight concrete in which all of the fine aggregate consists of normal density sand is termed “sand-lightweight concrete” .

A3 STRUCTURAL PERFORMANCE FACTOR, S_p. This factor is given in 2.6.2.2. It is applied to the “elastic site hazard spectrum” to convert peak seismic actions (forces and displacements and deformations) into design values and makes allowance for structural members and the total structural capacity typically being stronger than predicted, and energy dissipation of the structure typically being higher than assumed (refer to NZS 1170.5).

SUPPLEMENTARY CROSS TIES. Additional ties placed around longitudinal bars supplementing the functions of stirrups or ties.

TENDON. A steel element such as wire, cable, bar, rod, or strand which when tensioned imparts a prestress to a concrete member.

TIES. See Stirrups.

TRANSFER. Act of transferring stress in prestressing tendons from jacks or pretensioning bed to a concrete member.

UNBONDED TENDONS. Tendons which are not bonded to the concrete either directly or through grouting. They are usually wrapped in a protective and lubricating coating to ensure that this condition is obtained.

A3 WALL. A vertical structural element with a thickness that is small compared with its in-plane dimensions, such that under ultimate limit state load combinations the out-of-plane flexural actions do not significantly reduce the in-plane flexural strength.

WALL PANEL. A structural or non-structural panel that is either part of a wall or a cladding panel fixed to the exterior face of a building.

WOBBLE FRICTION. In prestressed concrete, the friction caused by the unintended deviation of the prestressing sheath or duct from its specified profile.

NOTES

2 DESIGN PROCEDURES, LOADS AND ACTIONS

2.1 Notation A3

A_g Gross area of a cross section of a member, mm^2

A_r aspect ratio of wall = h_w/L_w

A_{sk} area of a bar used as skin reinforcement on one side of a beam, or slab, mm^2 A3

c_c clear cover between the reinforcement and the surface of the concrete, mm

c_m cover distance measured from the centre of the reinforcing bar, mm

d effective depth, distance from extreme compression fibre to centroid of tension reinforcement, mm

d_b diameter of reinforcing bar, mm A3

E_s modulus of elasticity of reinforcing steel, MPa

F_{ph} inertia force used in design of a part, N

f'_c specified compressive strength of concrete, MPa

f_s stress in reinforcement, MPa A3

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

G dead load, N, kPa or N/mm

g_s distance from centre of reinforcing bar to a point on surface of concrete where crack width is being assessed, mm

h overall depth of the member measured at right angles to the axis of bending, mm A2

h_b overall beam depth, mm

h_c overall depth of column in the direction of the horizontal shear force, mm

h_w height of wall, mm

k ratio of depth of neutral axis to effective depth, *d,* of member based on elastic theory for members cracked in flexure

k_1 factor for determining minimum slab thickness, see 2.4.3

K_d a factor used to define limiting inelastic material strains, see 2.6.1.3.4 A3

K_p a factor used in 2.6.1.3.3

ℓ_p effective plastic hinge length used to calculate equivalent uniform curvature in a plastic hinge (region), mm

L effective span length of beam, girder or one-way slab, as defined in 6.3.2; for a cantilever it is the clear projection

L_n length of clear span in long direction of two-way construction, measured face-to-face of columns in slabs without beams, and clear span of coupling beams between coupled walls, mm

L_s shortest span length of bridge deck slab, mm A2

L_w horizontal in-plane length of a wall, mm A3

M_e/V_e moment to shear force ratio for seismic actions found from an equivalent static analysis, or first mode values from a modal response spectrum analysis

M^* design moment action for ultimate limit state, N mm

M_n nominal flexural strength, N mm

M_s maximum bending moment calculated for serviceability limit state load combination with long-term live load, N mm

M_o^* overstrength bending moment, N mm

A3

N_o^* axial load that acts simultaneously with overstrength bending moment, N

A3

Q live load, N, kPa, or N/mm
S_n nominal strength at the ultimate limit state for the relevant action of moment, axial load, shear or torsion, N or N mm
S_p structural performance factor
S^* design action at the ultimate limit state, N or N mm
s centre-to-centre spacing of reinforcing bars, mm
A3 t thickness of wall, mm

V^* design shear action in ultimate limit state, N
w design crack width due to flexure, mm
y distance from the extreme compression fibre to the fibre being considered, mm
A3 Z_t section modulus related to extreme tension fibre calculated from gross section properties at the section sustaining the maximum bending moment, mm^3
α ratio of the flexural stiffness of beam to the flexural stiffness of a width of slab bounded laterally by the centrelines of adjacent panels, if any, on each side of the beam, see Table 2.2

A3

α_m average value of α for all beams on the edges of a panel
β ratio of clear spans in long to short direction of two-way slabs
β' ratio used to find strain in section in 2.4.4.6

A3

μ structural ductility factor
ϕ strength reduction factor as defined in 2.3.2.2 and 2.6.3.2
A2 ϕ_{max} limiting curvature, radians/mm
$\phi_{o,fy}$ overstrength factor depending on reinforcement grade, see 2.6.5.6
A2 ϕ_y curvature at first yield, radians/mm
A3 δ lateral deflection of a squat wall, mm
ρ density of concrete, kg/m^3
ω dynamic magnification factor
ψ_s short-term live load factor (see AS/NZS 1170)

2.2 Design requirements

2.2.1 *Design considerations*

The structure and its component members shall be designed to satisfy the requirements of this Standard for stiffness, strength, stability, ductility, robustness, durability and fire resistance.

2.2.2 *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:Part 0 or other referenced loading standard and the specific requirements of 2.3 to 2.6.

2.2.3 *Design for robustness, durability and fire resistance*

A3 Concrete structures shall be designed:

(a) To be robust in accordance with the procedures and criteria given in Part 0 of AS/NZS 1170 or other referenced loading standard;
(b) To be durable in accordance with the procedures and criteria given in Section 3;
(c) To be fire resistant in accordance with the procedures and criteria given in Section 4;
(d) Such that stairs and ramps that service different levels in buildings are able to be functional while sustaining the displacement corresponding to the peak maximum considered earthquake ($1.5/S_p$ × ultimate limit state displacement) (2.6.10.4) (notwithstanding the requirement to comply with the minimum dimensions for a sliding ledge given in NZS 1170.5);

(e) Such that precast floor units and cladding panels are able to sustain without collapse the peak displacements and peak deformations corresponding to the maximum considered earthquake (17.6.2 and 18.7.4). A3

2.2.4 *Material properties*

The properties of materials used in the design shall be in accordance with Section 5 unless material properties that are appropriate for specific calculations are given in the associated clauses. Strength reduction factors as given in 2.3.2.2 shall apply. A3

2.3 Design for strength and stability at the ultimate limit state

2.3.1 *General*

The structure and its component members shall be designed for the ultimate limit state by providing stiffness, strength and ductility and ensuring stability, as appropriate, in accordance with the relevant requirements of 2.3.2 to 2.3.3.

2.3.2 *Design for strength*

2.3.2.1 *General*

The design shall consider and take into account the construction sequence, the influence of the schedule for stripping of formwork and the method of back-propping on the loading of the structure during construction and their effect on the strength and deflection of the structure. The structural effects of differential settlement of foundation elements and lateral movement of the ground shall be considered where appropriate, and provided for in accordance with this Standard.

Structures and structural members shall be designed for strength as follows:

(a) The loads and forces giving rise to the ultimate limit state design action, S^*, shall be determined from the governing ultimate limit state combinations specified in AS/NZS 1170 or other referenced loading standard;

(b) The design strength of a member or cross section at the ultimate limit state shall be taken as the nominal strength, S_n, for the relevant action calculated in accordance with the requirements and assumptions of this Standard, multiplied by the applicable strength reduction factor, ϕ, specified in 2.3.2.2;

(c) Each member shall be proportioned so that the design strength is equal to or greater than the design action, in accordance with the following relationship:

$$S^* \leq \phi S_n \quad \text{(Eq. 2–1)}$$

where S is replaced in Equation 2–1 by the actions of moment, axial force, shear or torsion as appropriate.

2.3.2.2 *Strength reduction factors, ultimate limit state*

The strength reduction factor, ϕ, shall be as follows: A3

(a) For actions which have been derived from overstrengths of elements in accordance with the principles of capacity design (see 2.6.5) 1.00
(b) Anchorage and strength development of reinforcement 1.00
(c) Flexure with or without axial tension or compression 0.85
(d) Flexure and shear in singly reinforced walls for in-plane actions only……...0.70
(e) Shear and torsion.. 0.75
(f) Bearing on confined and unconfined concrete 0.75
(g) Tension in plain concrete ....not applicable[1]
(h) Strut and tie models 0.75

---

[1] See 2.3.2.3.

A3
(i) Corbels 0.75
(j) For design under fire exposure 1.00
(k) For beam-column joints not designed by capacity design……………………0.75
(l) For embedded items etc., values are given in Section 17.

A3
2.3.2.3 *Tensile strength of concrete*

Where it is required for the assessment of member stiffness values or deflections the design tensile strength of concrete shall be based on the appropriate value in Section 5.

Certain specific clauses of this Standard prescribe nominal strengths that rely, implicitly or explicitly, on the tensile strength of concrete. In these clauses the appropriate tensile strength is either given in the clause, or inherent in the prescribed nominal strength.

In all other cases, reinforcement with sufficient nominal strength to resist the total tension force required to maintain equilibrium of the structure shall be provided. This includes, but is not limited to, forces induced by:

(a) Flexure, or flexure and axial load in members and in the connections between structural members;
(b) Any tension force which provides support to a structural member.

2.3.3 *Design for stability*

For ultimate limit state load combinations, the structure as a whole and its component members shall be designed to prevent instability in accordance with AS/NZS 1170 or other referenced loading standard.

2.4 Design for serviceability

2.4.1 *General*

2.4.1.1 *Deflection, cracking and vibration limits*

The structure and its component members shall be designed for the serviceability limit state by limiting deflection, cracking and vibration, where appropriate, in accordance with the relevant requirements of 2.4.2 to 2.4.4.

2.4.1.2 *Vibration*

Appropriate measures shall be taken to evaluate and limit where necessary the effects of potential vibration from wind forces, machinery and vehicular, pedestrian or traffic movements on the structure, to prevent discomfort to occupants or damage to contents.

2.4.1.3 *Seismic actions*

Where seismic actions are included in a load combination the structure shall be proportioned to meet the requirements of 2.6.3.

2.4.1.4 *Strength reduction factor*

Where it is necessary to check or design for the strength associated with wind or seismic serviceability load combinations a strength reduction factor not exceeding 1.1 shall be used.

2.4.2 *Deflection of beams and slabs*

2.4.2.1 *Structures other than bridges*

Deflection in concrete structures and members shall either be determined in accordance with 6.8 or the minimum thickness provisions of 2.4.3 shall be applied.

The deflections computed in accordance with 6.8 shall, where required, meet the limits given by AS/NZS 1170, or for earthquake loading NZS 1170.5, or another referenced loading standard for the relevant serviceability limit state criteria.

2.4.2.2 *Bridges*

The design of bridge girders shall mitigate the deflection due to the combination of permanent loads, shrinkage, prestress and creep over the long-term to ensure appropriate ride quality and drainage of the bridge deck.

2.4.3 *Minimum thickness*

A3

2.4.3.1 *Slabs and beams for buildings*

The minimum thickness of non-prestressed beams and slabs subjected to gravity load combinations may be determined by calculation, as specified in 6.8 or by satisfying the minimum span to depth ratios and other requirements given in (a), (b), or (c) below, as appropriate.

(a) *One-way spans*

The limiting span to depth ratios shall only be used for determining the minimum thickness of non-prestressed beams or slabs where the condition in Equation 2–2 is satisfied. Where this condition is not satisfied deflection calculations shall be made as specified in 6.8. In Equation 2–2, M_s is the maximum bending moment in the serviceability limit state due to dead load and long-term live load calculated assuming uniform elastic properties, k_1 is a factor given in the Table 2.1 and Z_t is the section modulus for the extreme tension fibre calculated from the gross section.

$M_s < k_1 \sqrt{f_c'}\ Z_t$ .... (Eq. 2–2)

Table 2.1 – Minimum thickness of non-prestressed beams or one-way slabs

f_y (MPa)	Member	Minimum thickness, *h* and value of k_1 — Members not supporting or attached to partitions or other construction likely to be damaged by large deflections							
		Simply supported		One end continuous		Both ends continuous		Cantilever	
		h	k_1	h	k_1	h	k_1	h	k_1
300	Solid one-way slabs	$\frac{L}{25}$	1.0	$\frac{L}{30}$	1.1	$\frac{L}{35}$	1.2	$\frac{L}{13}$	1.0
	Beams or ribbed one-way slabs	$\frac{L}{20}$	1.0	$\frac{L}{23}$	1.0	$\frac{L}{26}$	1.0	$\frac{L}{10}$	1.0
500	Solid one-way slabs	$\frac{L}{18}$	1.0	$\frac{L}{20}$	1.1	$\frac{L}{25}$	1.2	$\frac{L}{9}$	1.0
	Beams or ribbed one-way slabs	$\frac{L}{14}$	1.0	$\frac{L}{16}$	1.0	$\frac{L}{19}$	1.0	$\frac{L}{7}$	1.0

NOTE –
The values given shall be used directly for members with normal density concrete ($\rho \approx 2400$ kg/m^3). For lightweight concrete having a density in the range of 1450-1850 kg/m³, the values shall be multiplied by (1.65 – 0.0003ρ) where ρ is the density in kg/m³.

(b) *Two-way construction (non-prestressed) for buildings*

For non-prestressed two-way slabs for buildings the minimum thickness of slabs without interior beams spanning between the supports shall be in accordance with the provisions of Table 2.2 and shall be equal to or greater than the following values dependant on the provision of drop panels that conform with 12.5.6.1:

(i) Slabs without drop panels ....120 mm

(ii) Slabs with drop panels ....100 mm

Table 2.2 – Minimum thickness of slabs without interior beams

f_y (MPa)	Without drop panels (1)			With drop panels (1)		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams (2)		Without edge beams	With edge beams (2)	
300	$\frac{L_n}{33}$	$\frac{L_n}{36}$	$\frac{L_n}{36}$	$\frac{L_n}{36}$	$\frac{L_n}{40}$	$\frac{L_n}{40}$
500	$\frac{L_n}{28}$	$\frac{L_n}{31}$	$\frac{L_n}{31}$	$\frac{L_n}{31}$	$\frac{L_n}{34}$	$\frac{L_n}{34}$

NOTE –
(1) Drop panel is defined in 12.5.6.1.
(2) Slabs with beams between columns along exterior edges. The value of α for the edge beam shall be equal to or greater than 0.8.

(c) *For slabs supported by beams on all four sides, the minimum thickness shall depend on the value of α_m, as given below*:

(i) For α_m equal to or less than 0.2 the limits given in Table 2–2 shall apply;

(ii) For α_m between the limits of 0.2 and 2.0 the thickness shall be equal to or greater than:

$$h = \frac{L_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 5\beta(\alpha_m - 0.2)} \geq 120 \text{ mm} \quad \text{(Eq. 2–3)}$$

where α_m is the average value of α for all the beams and α is defined in 2.1.

(iii) For α_m greater than 2.0 the thickness shall be equal to or greater than:

$$h = \frac{L_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 9\beta} \geq 120 \text{ mm} \quad \text{(Eq. 2–4)}$$

(iv) For slabs without beams, but with drop panels extending in each direction from the centreline of support a distance equal to or greater than one-sixth the span length in that direction measured centre-to-centre of the supports, and a projection below the slab of at least one quarter of the slab thickness beyond the drop, the thickness required by Equations 2–3, or 2–4 may be reduced by 10 %.

(v) At discontinuous edges one of the following conditions shall be satisfied:

(A) An edge beam with a stiffness ratio, α, equal to or greater than 0.8 shall be provided;

(B) The minimum thickness of the slab shall be equal to or greater than the value given by Equation 2–3;

(C) The minimum slab thickness given by Equation 2–4 shall be increased by at least 10 % for the panel with the discontinuous edge.

(d) *Composite precast and in situ concrete construction for buildings*

If the thickness of non-prestressed composite members meets the requirements of Table 2–1 deflection need not be calculated except as required by 6.8.5 for shored construction.

A3 **2.4.3.2** *Bridge structure members*

The minimum thickness stipulated in Table 2.3 shall apply to flexural members of bridge structures unless calculation of deflection and design for the effects of traffic-induced vibration calculated in accordance with engineering principles indicates that a lesser thickness may be used without adverse effect.

For deck slabs designed by the empirical method of 12.8, the minimum slab thickness requirements of that clause shall take precedence over the requirements of Table 2.3. A3

Table 2.3 – Minimum thickness of prismatic flexural members of bridge structures

Superstructure type	Minimum thickness	
	Simple spans	Continuous spans
Bridge deck slabs	$1.2\left(100+\frac{L_s}{30}\right)$	$100+\frac{L_s}{30}$
T-girders	$0.070L$	$0.065L$
Box-girders	$0.060L$	$0.055L$

NOTES –
(1) For non-prismatic members the values given may be adjusted to account for change in relative stiffness of positive and negative moment sections.
(2) Slab span length, L_s, shall be determined in accordance with 12.8.4. A2

2.4.4 *Crack control*

2.4.4.1 *Cracking due to flexure and axial load in reinforced concrete members in buildings* A3

Crack widths for serviceability load combinations, excluding seismic and wind forces, shall be controlled by satisfying one of the following sets of criteria:

(a) Crack control measures need not be considered where the maximum longitudinal tensile stress calculated from gross section properties is equal to or less than $0.4\sqrt{f_c'}$ (MPa);

(b) The reinforcement shall be distributed and the maximum stress levels limited so that the requirements of 2.4.4.3, 2.4.4.4, 2.4.4.5 and 2.4.4.7 are satisfied;

(c) The reinforcement shall be distributed in the tension zone so that the maximum crack width calculated from 2.4.4.6 does not exceed an acceptable limit.

Where crack control is identified as an important issue the criterion in (c) shall be used. A3

2.4.4.2 *Crack widths in bridge structures*

For crack width requirements in road bridge structures refer to the New Zealand Transport Agency's Bridge Manual clause 4.2.1.a.

2.4.4.3 *Cracking due to flexure and axial load in prestressed concrete members*

For crack control in prestressed members see 19.3.3.

2.4.4.4 *Spacing of reinforcement for crack control on the extreme tension face*

The spacing of deformed reinforcement, s, crossing a potential crack and located next to the tension face of a member, shall be smaller than the values given by either:

$$s=\frac{90000}{f_s}-2.5c_c \qquad \text{(Eq. 2–5)}$$

or

$$s=\frac{70000}{f_s} \qquad \text{(Eq. 2–6)}$$

where f_s is the stress in the reinforcement at the serviceability limit state and c_c is the clear cover between the reinforcement and the surface of the concrete.

A3 **2.4.4.5** *Crack control on sides of beams and slabs*

Beams and slabs with an overall depth, *h*, of 1.0 m or more, subjected to bending or bending and axial tension in the serviceability limit state due to actions other than transitory live load, seismic or wind, shall have longitudinal reinforcement uniformly distributed along each side of the member for a distance *h*/2 from the extreme tension fibre of the member. The diameter of this reinforcement shall be equal to or larger than 10 mm, and the spacing equal to or less than the smallest of:

(a) 300 mm

(b) *h*/6

(c) 3*t*, where *t* is the thickness of the web of the member

(d) $s = \dfrac{A_{sk} f_y}{0.25(2c_c + d_b)\sqrt{f_c'}}$

Where if $2c_c + d_b$ is greater than $t/2$, $2c_c + d_b$ may be taken as $t/2$. A_{sk} is the area of a bar used as part of the skin reinforcement, and c_c is the clear cover to the longitudinal bar and d_b is the diameter of the bar.

The skin reinforcement may be included in calculations to determine the flexural strength of the member.

2.4.4.6 *Assessment of surface crack widths*

Where limitations are placed upon the maximum allowable crack width, the design surface crack width, *w*, for members reinforced with deformed bars may be assessed from the equation:

$$w = 2.0\,\beta'\,\frac{f_{s,ch}}{E_s}\,g_s \qquad \text{(Eq. 2–7)}$$

A3 where

$f_{s,ch}$ is the change in stress in the reinforcement, equal to $f_s - 0.5\,f_{s,c}$ where $f_{s,c}$ is the stress in the reinforcement when the stress in the concrete alongside the reinforcement is zero prior to crack formation. The value of $f_{s,c}$ may be ignored when the long-term unrestrained shrinkage strain in the concrete is less than 400×10^{-6}. Appendix CE contains a method of assessing $f_{s,c}$,

g_s is the distance from the centre of the nearest reinforcing bar to the surface of the concrete at the point where the crack width is being calculated, and

β' is a coefficient, given by:

$$\beta' = \frac{y - kd}{d - kd} \qquad \text{(Eq. 2–8)}$$

where

kd is the depth of the neutral axis, and

y is the distance from the extreme compression fibre to the fibre being considered.

For the case where a crack width is being calculated between two bars, the critical value of g_s is given by:

$$g_s = \sqrt{(s/2)^2 + c_m{}^2} \qquad \text{(Eq. 2–9)}$$

where

c_m is the cover distance measured from the centre of the bar to the surface of the concrete, and

s is the centre-to-centre spacing of the bars.

2.4.4.7 *Crack control in flanges of beams*

Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over the effective flange overhang width defined in 9.3.1.3 to control crack widths. Consideration should also be given to adding reinforcement outside this width to control cracking. A2

2.4.4.8 *Control of thermal and shrinkage cracking*

Cracking of concrete due to differential temperature, heat of hydration or shrinkage of concrete shall be determined from first principles, where these actions may lead to a loss of serviceability of the structure.

Potential cracking due to plastic shrinkage shall be controlled by specification.

2.5 Other design requirements

2.5.1 *General*

Requirements such as those for fatigue, removal or loss of support, together with other performance requirements shall be considered in the design of the structure in accordance with established engineering principles.

2.5.2 *Fatigue (serviceability limit state)*

2.5.2.1 *General*

The effects of fatigue shall be considered where the imposed loads and forces on a structure are repetitive in nature.

2.5.2.2 *Permissible stress range*

At sections where frequent stress fluctuations occur, the stress range in reinforcing bars, excluding stirrups and ties, caused by the repetitive loading at the serviceability limit state, shall be equal to or less than the appropriate limit given in either (a) or (b) below: A2

(a) The stress range shall be equal to or less than the value given in the Table below, where D is the diameter of the bend measured to the inside of the bar and d_b is the diameter of the bar.

Stress range, MPa	**150**	**135**	**120**	**90**	**50**
D/d_b	>25	20	15	10	5

Interpolation may be used for intermediate values of D/d_b.

(b) Appropriate values are found from a special study in which the influence of the following factors is considered:

(i) The shape of deformations and bar marks;
(ii) The composition and diameter of the reinforcement;
(iii) The method of manufacture;
(iv) The diameter of bends in the reinforcement;
(v) The influence of embedment of the bar in cracked concrete;
(vi) The histogram of stress variation over the expected life of the structure.

2.5.2.3 *Highway bridge fatigue loads*

For highway bridges, the vehicle loading specified by the New Zealand Transport Agency's Bridge Manual shall be used as a basis for assessing the fatigue stress range.

A3

2.5.3 *Structural diaphragms*

In assessing the serviceability limit state requirements for floor and roof diaphragms the lateral forces transferred to or between lateral force resisting elements shall be considered. These forces may arise from gravity, seismic or wind actions and soil pressures. In addition forces induced by thermal and shrinkage effects shall be considered where appropriate.

2.6 Additional design requirements for earthquake effects

2.6.1 *General*

2.6.1.1 *Deformation capacity*

In addition to the requirements of 2.3.2 for strength, the structure and its component parts shall be designed to have adequate ductility at the ultimate limit state for load combinations including earthquake actions.

2.6.1.2 *Classification of structures*

Structures subjected to earthquake forces shall be classified for design purposes as brittle structures, nominally ductile structures, structures of limited ductility or ductile structures, as specified below:

(a) Brittle concrete structures shall be those structures that contain primary seismic resisting members, which do not satisfy the requirements for minimum longitudinal and shear reinforcement specified in this Standard, or rely on the tensile strength of concrete for stability. Brittle structures are not considered in this Standard.

(b) Nominally ductile structures are those that are designed using a structural ductility factor of 1.25 or less.

(c) Structures of limited ductility are a sub-set of ductile structures, which are designed for a limited overall level of ductility. The structural ductility factor shall not exceed 3.0.

(d) Ductile structures are those structures designed for a high level of ductility. The structural ductility factor shall not exceed 6.0.

2.6.1.3 *Classification of potential plastic regions*

2.6.1.3.1 *Classification nomenclature*

Potential plastic regions shall be classified for the purpose of defining the required detailing as:

(a) Nominally ductile plastic region, NDPR;

(b) Limited ductile plastic region, LDPR;

(c) Ductile plastic region, DPR.

2.6.1.3.2 *Material strain limits in plastic regions*

A3

The classification depends on the level of deformation that each potential plastic region can safely sustain at the ultimate limit state. For plastic hinge regions, the material strain is taken as curvature equal to the rotation divided by the effective plastic hinge length, ℓ_p. In diagonally reinforced coupling beams, the material strain is taken as the average shear strain over the clear span of the coupling beam, L_n. Coupling beams in which the diagonal reinforcement does not extend over the entire clear span of the beam shall not be used unless a special study is made to ensure that yielding is limited to the diagonal portion of the reinforcement, and strain hardening and strain ageing in the bend of the diagonal reinforcement does not significantly reduce the ductility of the coupling beam. The strain limits for different classifications of potential plastic regions are given in 2.6.1.3.4 and they shall not be exceeded except where a special study (see AS/NZS 1170.0 and NZS 1170.5) shows that higher strain levels can be sustained with a high level of confidence.

Where the analysis of the structure has been based on an equivalent static or modal analysis, the rotation in a plastic hinge region shall be found by adding the elastic deformation given in (a) to the inelastic deformation given in (b) below:

(a) The elastic rotation of a plastic hinge shall be taken as the elastic curvature times the effective plastic hinge length, ℓ_p. The elastic curvature, ϕ_y, is given by $\phi_y = \dfrac{2f_y}{E_s h}$ but the value of f_y in this expression is not taken as greater than 425 MPa, or a lower value defined in 2.6.1.3.4(c) for the group (ii) members where the shear strength determines the strength of the member; A3

(b) The corresponding inelastic deformation shall be calculated from (i), (ii) or (iii) below as appropriate:

(i) For the reversing plastic hinge regions, the inelastic component of deformation shall be calculated from the total (elastic plus inelastic) inter-storey drift given in NZS 1170.5 clause 7.3 (which includes the drift modification factor), or other referenced Loading Standard, minus the elastic component of inter-storey drift which is defined in NZS 1170.5 clause 7.2.1.1

(ii) For unidirectional plastic regions the inelastic rotation, either adjacent to the column faces or in the span of the beam, shall be calculated as for reversing plastic hinges adjacent to the column face but multiplied by:

$1 + 0.63(\mu - 1)$ for $1 \leq \mu \leq 2$

$1.63\sqrt{(\mu - 1)}$ for $2.0 < \mu < 6$

where μ is the structural ductility factor

(iii) The corresponding values from another appropriate referenced loading Standard.

The shear deformation in a diagonally reinforced coupling beam, corresponding to the deflected shape profiles defined in NZS 1170.5, shall be calculated by finding the deflection at one end of the beam from a line drawn normal to the deflected shape of the wall at the other end of the beam, and dividing this deflection by the clear span of the coupling beam.

Where time history analysis has been used, in which inelastic deformation characteristics of members are modelled and P-delta actions are included in the analysis, the required rotations in plastic regions shall be taken as the critical values sustained in each plastic region when subjected to a suite of analyses using earthquake ground motions which meet the requirements of NZS 1170.5, or other appropriate referenced loading Standard.

2.6.1.3.3 *Effective plastic region lengths*

The appropriate value of effective plastic hinge length, ℓ_p, used to calculate curvatures, shall be taken as appropriate from (a), (b), or (c) below. L_n, which is used to calculate an average shear strain in a diagonally reinforced coupling beam, shall be taken as given in (d):

(a) For reversing plastic hinges in beams, columns and out of plane actions in walls:

$$0.25 k_p h \leq \ell_p = 0.25 k_p \frac{M_e}{V_e} \leq 0.5 k_p h \quad \text{(Eq.2–9(a))}$$

where h is equal to h_b, h_c, or t for beams, columns, or walls respectively, M_e/V_e is defined below and k_p is given by:

$$k_p = \left(\frac{h}{d} - 0.25\right) \geq 1.0 \quad \text{(Eq. 2–9(b))}$$

(b) For in-plane actions in walls the effective plastic hinge length, ℓ_p, is given by:

$$\ell_p = 0.15 \frac{M_e}{V_e} \leq 0.5 L_w \quad \text{(Eq. 2–9(c))}$$

where M_e/V_e in (a) and (b) is the moment to shear force ratio for seismic actions found from an equivalent static analysis, or a first mode values from a modal response spectrum analysis;

(c) For unidirectional plastic hinges, where inelastic rotation can develop on both sides of the critical section, ℓ_p, may be taken as twice the corresponding value found in (a) or (b) above;

(d) For diagonally reinforced coupling beams complying with 11.4.9.4, the effective plastic region length for shear, L_n shall be taken to equal the clear span of the beam between the walls.

2.6.1.3.4 *Material strain limits*

A3 The maximum design ultimate limit state rotation in a plastic hinge (elastic plus plastic rotation) is equal to the effective plastic hinge length, $\ell_{p,}$ times the appropriate limiting material strain, which is defined as $K_d\,\phi_y$ where ϕ_y is a nominal plastic hinge rotation at first yield of reinforcement given by:

$$\phi_y = \frac{2 f_y}{E_s h} \qquad \text{(Eq.2–9(d))}$$

For the sole purpose of calculating the nominal plastic hinge rotation at first yield (ϕ_y) the value of f_y shall not be taken greater than 425 MPa.

The value of K_d, varies with the type of plastic region and the type of structural member, as defined here.

(a) For plastic regions where the ultimate strength is limited by flexure and that are subjected to reversing plastic actions, the value of K_d shall be determined from the values shown in Table 2.4, according to the type of member and the detailing provided;

(b) For ductile members where the ultimate strength is limited by flexure and subjected to unidirectional plastic actions, the value of K_d shall be equal to twice the value determined from the values shown in Table 2.4 according to the type of member and the detailing provided. For nominally ductile plastic regions no increase shall be made for unidirectional plastic regions;

(c) Nominally ductile plastic regions in beams and columns divide into group (i) and group (ii):

For those in group (i), where the strength in the ultimate limit state is limited by flexure, and the longitudinal reinforcement in the compression side of the member is restrained against buckling by ties that comply with the spacing limits in 9.3.8.2.2 and 9.3.9.6.2, regardless of whether the bars are required to act in compression or not, the limiting curvatures are given in Table 2.4 for group (i);

For those in group (ii), which do not satisfy the requirements for group (i), the deformation is limited to elastic response for the ultimate limit state. Where the shear strength of the member limits the ultimate limit state strength, the value of K_d shall be given by $K_d = 0.8\frac{f_s}{f_y} \le 1.0$ where the value of f_s is the stress in the longitudinal reinforcement at the critical section for flexure when the member sustains its design shear strength;

(d) For ductile diagonally reinforced coupling beams complying with 11.4.9.3, the average shear strain rotation shall be equal to or less than 0.035;

(e) For squat walls where $h_w/L_w \le 1.0$ the lateral deflection, δ, over the height of the wall, h_w, shall:

(i) For nominally ductile walls be equal to or less than 0.003;

(ii) For limited ductile and ductile walls be equal to or less than 0.008;

(iii) Include the deformation due to both flexure and shear, but need not include the lateral deflection associated with development of starter reinforcement in the foundation beam or element.

2.6.1.4 *Stiffness of members for seismic analysis*

A3 Assessment of structural deflections involving seismic forces shall make due allowance for the anticipated levels of concrete cracking associated with the strain levels sustained by the reinforcement, and with the quantity of longitudinal reinforcement. In members, or regions of major members such as structural walls, where analysis indicates that flexural cracking will not occur, section properties shall be based on either gross sections or uncracked transformed sections, see 6.9.1.

(a) For the serviceability limit state and for members, which are not expected to sustain inelastic deformation in the ultimate limit state, allowance for flexural cracking shall be consistent with the maximum expected strain levels in the members;

(b) For the ultimate limit state, where elastic-based methods of analysis are used (equivalent static, modal response spectrum or elastic time history), the stiffness of members that are expected to sustain plastic deformation in a design level earthquake shall correspond to the stiffness under cyclic loading conditions to first yield of the member. For other members the stiffness should be consistent with the expected maximum stress level induced in the member when adjacent potential plastic regions are sustaining their nominal strengths. Any potential increase in actions above this level due to overstrength of potential plastic regions or due to dynamic magnification effects, should be ignored for the purpose of assessing stiffness.

Assessment of structural deflections for the ultimate limit state involving seismic forces shall make due allowance for the anticipated levels of post-elastic effects and P-delta actions, as specified in NZS 1170.5.

Table 2.4 – K_d factor for limiting curvatures in flexural plastic regions in beams, columns and to walls where $h_w/L_w \geq 1.0$ A3

	K_d for detailing type		
Classification of plastic region	**Nominally ductile**	**Limited ductile**	**Ductile**
Beams and columns [(1)]	Group (i) 3 Group (ii), see 2.6.1.3.4 (c)(ii) n/a	11	19
Walls, doubly reinforced with confined boundary elements	n/a	9	16
Walls, doubly reinforced	4	6	14
Walls, singly reinforced	0.8	n/a	n/a

NOTES –
(1) See 2.6.1.3.4 (c) (i) and (ii).
(2) A wall may be assumed to have confined boundary elements where the dimensional requirements of 11.4.3 and 11.4.5 and 10.4.7.5.1 for the compression force acting on the each boundary element.
(3) Group (i) and (ii) are described in C2.6.1.3.4

2.6.2 *Seismic design actions*

2.6.2.1 *General*

In the derivation of seismic actions for the serviceability and ultimate limit states, the design actions specified by NZS 1170.5, or other referenced loading standard, shall be found using:

(a) A structural performance factor which is equal to or greater than the appropriate value for the limit state being considered, as given in 2.6.2.2;
(b) A structural ductility factor which is equal to or less than the maximum appropriate value given in 2.6.2.3; A3
(c) The dynamic characteristics of the structure;
(d) The design response spectrum, return period factor and seismic zone factor, given in NZS 1170.5 or other referenced loading standard. A2

2.6.2.2 *Structural performance factor*

2.6.2.2.1 *S_p values*

The structural performance factor, S_p, shall be taken as equal to or greater than:

(a) For the serviceability limit state .... $S_p = 0.7$ A3
(b) For the ultimate limit state:
   (i) For nominally ductile structures .... $S_p = 0.9$
   (ii) For structures with a structural ductility factor of 3 or more..... $S_p = 0.7$
   Interpolation may be used between these limits.

2.6.2.2.2 *Lower S_p may be used when detailing requirements met*

A3

For nominally ductile structures and structures with a structural ductility factor of less than 3, which are proportioned to ensure that, under seismic actions larger than anticipated, mechanisms could only develop in the same form as those permitted by 2.6.7 for ductile structures, or those of limited ductility, an S_p factor of 0.7 may be used for determining the seismic actions provided that:

(a) Unless a particular clause specifies a value of S_p to be used in determining the action associated with that clause;

A3 (b) In beams all the detailing required for beams containing limited ductile plastic regions are satisfied. The numerical values of the overstrength moments at the critical sections in the potential plastic regions may be replaced by 1.15 times the nominal flexural strength at these sections;

(c) In walls all the requirements in the ductile detailing length for limited ductile walls shall be satisfied. The reduction in S_p is not permitted for singly reinforced walls;

(d) The design horizontal and vertical shear forces in beam-column joint zones given in 15.3.5 shall be multiplied by 1.15.

2.6.2.3 *Structural ductility factor*

2.6.2.3.1 *SLS*

A2 The structural ductility factor, μ, shall be unity for the SLS 1 serviceability limit state and equal to or less than 2 for the SLS 2 serviceability limit state.

2.6.2.3.2 *Ultimate limit state*

For the ultimate limit state two factors need to be considered in determining the structural ductility factor:

(a) The selected value shall not exceed the appropriate value given in Table 2.5;

(b) The value of the structural ductility factor shall be such that the maximum permissible material strain limit is not exceeded in the critical plastic region.

Table 2.5 – Maximum available structural ductility factor, μ, to be assumed for the ultimate limit state

A3

Type of structure	Reinforced concrete	Prestressed concrete with bonded non-prestressed reinforcement
1. Nominally ductile structures In-plane action of singly reinforced walls	1.25	1.25
2. Structures of limited ductility		
(a) Moment resisting frame	3	3
(b) Walls	3	3
(c) Cantilever face loaded walls (single storey only)	2	2
3. Ductile structures		
(a) Moment resisting frame	6	5
(b) Walls:		
(i) Two or more cantilevered	5	5
(ii) Two or more coupled	6	6
(iii) Single cantilever	4	4

NOTE – The structural ductility of all members may be limited by the maximum permissible material strains.

2.6.3 *Serviceability limit state*

2.6.3.1 *General*

The structure shall be proportioned to meet the serviceability requirements of NZS 1170.5 or other referenced loading standard. An analysis to determine the seismic induced deformations and inter-storey drifts for the serviceability limit state shall be made by using either method (a), or method (b), as detailed below.

(a) An elastic analysis may be used to determine the deformations sustained provided one of the two following conditions is satisfied:

(i) The structure is designed as a nominally ductile structure or a structure of limited ductility;

(ii) The members are proportioned so that the design strength exceeds the design actions for the critical serviceability seismic load combinations.

(b) Allowance is made, using engineering principles, for the influence of inelastic deformation of members under the action of load combinations including seismic forces and gravity loads. The analysis shall include, where appropriate, calculation of increased deflection of members due to shake down effects and determination of residual inter-storey drifts and crack widths.

2.6.3.2 *Strength reduction factor*

Strength checks for seismic load combinations shall be made using standard ultimate strength theory with a strength reduction factor, ϕ, which is equal to or less than 1.1.

2.6.4 *Ultimate limit state*

(a) For all structural classifications (2.6.1.2) the structure shall be proportioned such that: A3
  (i) The design strengths shall be equal to or exceed the design actions;
  (ii) The inter-storey drift limits and P-Delta stability coefficients given in NZS 1170.5 or other referenced loading standard, shall not be exceeded;
  (iii) The maximum permissible lateral displacement of the structure at site boundaries, as specified in NZS 1170.5, or other referenced loading standard, shall not be exceeded.

(b) Structures of nominal ductility, limited ductility and ductile structures, shall be proportioned to ensure that when the maximum lateral displacements for the ultimate limit state act on the structure, the material strains sustained in critical potential plastic regions do not exceed the maximum permissible values for the level of detailing that is used. A3

2.6.5 *Capacity design*

2.6.5.1 *General*

All ductile structures and structures of limited ductility shall be proportioned to meet the requirements of capacity design following the procedure outlined in NZS 1170.5 and 2.6.5. A2

2.6.5.2 *Identification of ductile mechanism*

In capacity design it is assumed that the structure is displaced laterally so that primary plastic regions form to give a ductile failure mechanism. A permissible failure mechanism shall be selected and potential primary plastic regions identified, see 2.6.7. The required seismic lateral forces to develop the primary plastic regions shall be assumed to act simultaneously with dead, and where appropriate long-term live load (for example G & ψ_ℓQ in AS/NZS 1170).

2.6.5.3 *Detailing of potential plastic regions*

The material strains in the critical potential plastic regions shall be assessed from the deformed shape in the ultimate limit state, as defined in NZS 1170.5 or other referenced loading standard, and from the appropriate length of the plastic regions as identified in 2.6.1.3. The magnitude of the material strain shall be used to identify the appropriate level of detailing.

2.6.5.4 *Overstrength actions*

The overstrength actions shall be determined for each potential primary plastic region on the basis of:

(a) The detailing used in the region;
(b) The critical load combinations which may occur in each region;
(c) The likely maximum material strengths in each potential plastic region as detailed in 2.6.5.5.

2.6.5.5 *Likely maximum material strengths*

Overstrength actions in potential plastic regions shall be determined assuming the appropriate cross section of the member and material overstrengths as set out below: A2

(a) The stress resisted by longitudinal reinforcement shall be taken as $\phi_{o,fy}$ times the nominal yield strength of the reinforcement, f_y, provided it increases the flexural strength. For reinforcement which complies with AS/NZS 4671, $\phi_{o,fy}$ shall be taken as 1.35 for Grades 300E and 500E; A3
(b) The stress in flexural reinforcement given in (a) above shall be used unless other values supplied by the manufacturer can be shown to be appropriate after peer reviewed special studies; A2
(c) The compression strength of concrete shall be taken as $\left[f'_c + 15\right]$ MPa;
(d) The stress in post-tension cables shall be determined from a special study. A3

2.6.5.6 *Ends of columns*

For potential inelastic regions in a column, which can form against a base slab or other members that effectively confine the compression region, the overstrength bending moment, M^*_o shall be calculated

A2

taking into account axial compression load as given in Equation 2–10. In no case shall the overstrength moment be taken as less than the value defined in 2.6.5.5.

A2

$$M_o^* = \left(\phi_{\mathrm{o,fy}} + 2\left(\frac{N_o^*}{f_c' A_g} - 0.1 \right)^2 \right) M_n \quad \text{(Eq. 2–10)}$$

A3 where

$\phi_{\mathrm{o,fy}}$ = 1.35

N_o^* is the axial load that acts concurrently with M_o^*.

2.6.5.7 *Capacity design for regions outside potential plastic regions*

Where the design strengths for regions outside the potential plastic regions are determined on the basis of actions, which can be transmitted to them through potential plastic regions, a strength reduction factor that is equal to or less than 1.0 shall be used. In assessing these design actions allowance shall be made for:

(a) The most adverse combination of overstrength actions in the potential plastic regions which may be transmitted into the member for the action being considered;

(b) Gravity loads which may act on the member;

(c) The change in dynamic behaviour of the structure changing the distribution of moments and shear forces (see Appendix D for dynamic magnification factors or defined distributions of actions);

A2 (d) An analysis which includes vertical seismic actions shall be made for sensitive horizontal members such as cantilever beams or slabs containing pretensioned members.

2.6.5.8 *Concurrency and capacity design*

In capacity design the effects of seismic actions occurring simultaneously along two axes at right angles shall be considered in the detailing of members, which are part of two-way horizontal force-resisting systems.

A3 Columns, including their joints and foundations, which are part of a two-way horizontal force-resisting system, with structural elements aligned along two axes, shall be detailed to sustain the concurrent actions as defined in (a), (b) and (c) below:

(a) In columns, overstrength bending moments and shears, amplified by dynamic magnification for one axis together with the overstrength actions from the other axis (ω = 1.0) with both possible combinations being considered for the two axes;

(b) Critical axial force found assuming concurrent yielding of all beams framing into the column, modified where appropriate, as defined in Appendix D, to allow for the limited number of plastic hinges which develop simultaneously on different levels of a multi-storey structure;

A3 (c) Walls may be designed for in-plane actions alone, where:

(i) The out-of-plane thickness of a wall, or part of a wall, is small compared with the in-plane dimension, and

(ii) The out-of-plane actions are a result of compatibility, that is, not required for equilibrium.

Where these two conditions are not met, the wall should be designed as a column.

2.6.5.9 *Transfer diaphragms*

A3 Floor and roof systems in buildings shall be designed to act as horizontal structural elements, where required, to transfer seismic forces to frames or structural walls, making allowance for the internal forces arising from the different lateral force deflection characteristics of the individual walls and frames in accordance with Section 13.

2.6.5.10 *Elongation in plastic regions*

Allowance shall be made for the deformation arising from elongation, and where appropriate the actions induced by this elongation, in the following:

A3 (a) The support of stairs and ramps that connect different levels in a building, see 2.6.10.4 and 18.8;

(b) The support and fixings of cladding panels, see Section 17;

(c) The support of precast floor elements, see 18.8; A3

(d) Coupling beams in coupled structural walls, see 11.4.9.2;

(e) Structural walls, in which axial forces are induced, see 11.4;

(f) The interconnection between floors with precast units and lateral force resisting elements, through which forces are transferred, see Section 13.

Where required, the magnitude of elongation that shall be considered is given in 7.8.

2.6.6 *Additional requirements for nominally ductile structures*

2.6.6.1 *Limitations for nominally ductile structures*

Nominally ductile structures shall be proportioned to ensure that when they are subjected to the seismic load combinations specified in AS/NZS 1170:Part 0 for the ultimate limit state or other referenced loading standard, the following conditions are satisfied.

(a) When the structural system is such that under seismic actions larger than anticipated, mechanisms could only develop in the same form as those permitted by 2.6.7 for ductile structures, or those of limited ductility, the selected structure is exempt from the additional seismic requirements of all sections of this Standard.

(b) When a mechanism could develop in a form which is not permitted for a ductile or limited ductile structures, the relevant mechanism or mechanisms shall be identified. Potential plastic hinge regions shall be identified, and detailed for ductile or limited ductile plastic regions such that the material strain limits given in 2.6.1.3 are not exceeded, in accordance with the additional seismic design requirements of this Standard.

2.6.7 *Additional requirements for ductile frames and limited ductile moment resisting frames*

2.6.7.1 *Ductile and limited ductile moment resisting frames*

The requirements of 2.6.7 shall be satisfied for frames forming part of the primary lateral load resisting system. Frames that are secondary structural elements shall satisfy 2.6.10.

2.6.7.2 *Acceptable column sidesway mechanisms*

Column sidesway mechanisms may be used as a design solution for:

(a) The top storey of any moment resisting frame;

(b) Frames not exceeding two storeys where the columns are detailed using the ductile provisions of 10.4;

(c) Bridge piers.

In all other cases the design shall exclude the formation of a sidesway mechanism that forms in a single storey. A3

2.6.7.3 *Beam design for column sidesway structures*

Where the requirements of 2.6.7.2 are satisfied, and capacity design procedures mean that yielding of the beams is unlikely, the beams shall be detailed as specified in 9.3 and the beam-column joints designed to resist the overstrength actions from the potential plastic regions (see 15.4). A3

2.6.7.4 *Alternative design methods for columns in multi-storey frames*

Columns in ductile or limited ductile moment resisting frames shall be designed to have a high level of protection against the formation of a non-ductile failure mechanism in a major earthquake. Method A or Method B, as detailed in Appendix D shall be used to determine the critical design actions to achieve this objective.

2.6.7.5 *Design actions in columns*

When determining the design actions in columns:

(a) The axial load at critical sections shall be determined from the self weight of columns and attachments to the columns, gravity load shear forces and shear forces induced in the beams due to overstrength moments acting in the plastic hinge regions. In assessing the critical axial load level at a section, the axial load induced by all the beams framing into the column above the section being considered shall be included (see Appendix D).

(b) The nominal flexural strength of the column shall be equal to or greater than that required to sustain overstrength moments that act on the column from all beams intersecting the column amplified by appropriate dynamic magnification factors. Where a column acts in two moment resisting frames it shall be designed to sustain the moments applied simultaneously by the beams in the frames, amplified as required in 2.6.5.8.

(c) The columns shall be designed to sustain the critical shear forces transmitted to the columns by all the beams framing into the column above the section being considered (see appendix D).

A2 (d) For columns in two-way frames; design moments at primary plastic hinges, and immediately below the uppermost level in the frame, shall be determined from the critical ultimate limit state load combination, including seismic load combinations where 100 % of the seismic load on one axis is applied simultaneously, with 30 % of the seismic load on a second axis perpendicular to the first axis.

2.6.8 *Ductile walls and dual structures*

2.6.8.1 *Inelastic deformation of structural walls*

All structural walls, which are designed to provide lateral force resistance, shall be designed to dissipate energy by flexural yielding at the ultimate limit state.

2.6.8.2 *Shear strength of structural walls*

In providing the shear strength of a structural wall in the ultimate limit state, allowance shall be made in the shear force envelope for flexural overstrength and dynamic effects.

2.6.8.3 *Coupled walls*

A3 When two or more walls are interconnected by ductile beams, part of the seismic energy to be dissipated in the ultimate limit state shall be assigned to the coupling system. Capacity design procedures shall be used to ensure that the ductility of the coupling system can be maintained at its overstrength value by the strength of the walls. The overstrength of the coupling beams shall be calculated allowing for the increase in strength provided by the restraint to elongation of the coupling beams due to floors and other structural elements tying the coupled walls together, see 11.4.9.

2.6.8.4 *Ductile dual structures*

Where a combination of different lateral force-resisting structural elements is used in a structure, rational analysis shall be employed, taking into account the relative stiffness and location of elements, to allocate the seismic resistance to each element. Where diaphragms are required to transfer seismic forces between elements the design shall allow for the actions associated with overstrength of the elements.

2.6.9 *Structures incorporating mechanical energy dissipating devices*

The design of structures incorporating flexible mountings and mechanical energy dissipating devices is acceptable provided that the following criteria are satisfied at the ultimate limit state:

(a) The performance of the devices used is substantiated by tests;

(b) Proper studies are made towards the selection of suitable design earthquakes for the structure;

(c) The degree of protection against yielding of the structural members is at least as great as that implied in this Standard relating to the conventional seismic design approach without energy dissipating devices;

(d) The structure is detailed to deform in a controlled manner in the event of an earthquake greater than the design earthquake.

2.6.10 *Secondary structural elements*

2.6.10.1 *Definitions*

Secondary structural elements are those which at the ultimate limit state do not form part of the primary seismic action resisting system, but which are subjected to actions due to accelerations transmitted to them, or due to deformations of the structure as a whole. These are classified as follows:

(a) Elements of Group 1 are those which are subjected to inertia forces but which, by virtue of their detailed separations, are not subjected to forces induced by the deformation of the supporting primary elements or secondary elements of Group 2;

(b) Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to both inertia forces, as for Group 1, and to forces induced by the deformation of the primary elements.

2.6.10.2 *Group 1 secondary elements*

Group 1 elements shall be detailed for separation to accommodate deformations where the ultimate limit state lateral deflections of the primary seismic force-resisting system, calculated as specified in AS/NZS 1170 and NZS 1170.5, or other referenced loading standard are reached. Such separation shall allow adequate tolerances in the construction of the element and adjacent elements, and, where appropriate, allow for deformation due to other loading conditions such as gravity loading. For elements of Group 1:

(a) The inertia force, F_{ph}, used in the design shall be that specified in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard;

(b) Detailing shall be such as to allow ductile behaviour if necessary and in accordance with the assumptions made in the analysis. Fixings for non-structural elements shall be designed and detailed in accordance with 17.6.

2.6.10.3 *Group 2 secondary elements*

Group 2 elements shall be detailed to allow ductile behaviour if necessary where the ultimate limit state lateral deflections of the primary seismic force-resisting system, calculated as specified in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard, are reached for elements of Group 2:

(a) Additional seismic requirements of this Standard need not be satisfied when the design forces are derived from the imposed ultimate limit state lateral deflection, and analysis indicates that the element does not sustain inelastic deformation;

(b) Additional seismic requirements of this Standard shall be met when inelastic behaviour is assumed to occur at levels of deformation below the ultimate limit state lateral deflection;

(c) The inertia force, F_{ph}, shall be that specified by AS/NZS 1170 and NZS 1170.5;

(d) Forces induced by the deformation of the primary elements shall be those arising from the calculated ultimate limit state lateral deflection having due regard to the pattern and likely simultaneity of deformations;

(e) Analysis shall be by any rational method in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one-quarter of the above calculated ultimate limit state lateral deflection of the primary elements, as specified in AS/NZS 1170 and NZS 1170.5.

(f) Where inelastic deformation is required, potential plastic regions shall be identified and detailed to meet the requirements of this Standard. Plastic regions may be located in columns. A2

2.6.10.4 *Stairs, ramps and panels* A3

Notwithstanding the requirement to comply with the provisions of Section 8 of NZS 1170.5, stairs and ramps that service different levels, or span between different buildings, shall be designed to be functional after a maximum considered earthquake. The inter-storey displacements between the supports shall be considered in both the direction of the span of the stairs or ramp and in the direction normal to the span. The inter-storey displacement to be considered shall be equal to or greater than the ultimate limit state design displacement times $1.5/S_p$.

Cladding panels shall be designed to remain attached to the building after a maximum considered earthquake. The inter-storey displacement to be considered shall be equal to or greater than the peak inter-storey drift plus any additional deformation arising from elongation and rotation of supports. The peak inter-storey drift shall be taken as $1.5/S_p$ times the design ultimate limit state inter-storey drift.

Where the stairs or ramp are classified as a Group 1 element, the support shall comply with 18.7.6.

NOTES

3 DESIGN FOR DURABILITY

3.1 Notation

3.1.1 *Symbols*

f'_c specified compressive strength of concrete, MPa

3.1.2 *Abbreviations*

FA	Fly ash – AS 3582:Part 1 Supplementary cementitious materials for use with Portland and blended cement
GB	General purpose blended cement – NZS 3122
GP	General purpose Portland cement – NZS 3122
HE	High early strength cement – NZS 3122
MS	Amorphous silica – AS/NZS 3582:Part 3
SCM	Supplementary cementitious material
GBS	Ground granulated iron blast-furnace slag – AS 3582 Part 2

3.2 Scope

3.2.1 *Concrete*

The provisions of this section shall apply to the detailing and specifying for durability of plain, reinforced and prestressed concrete members with f'_c ranging from 20 MPa to 100 MPa and a design life of 50 years, or for a limited range of conditions, 100 years. A2

3.2.2 *Cementitious binders*

Durability design to this Standard shall be based on the use of concrete made with GP, GB or HE cement complying with NZS 3122 with or without supplementary cementitious materials complying with AS 3582.

3.2.3 *Design considerations*

Durability shall be allowed for in design by determining the exposure classification in accordance with 3.4 and, for that exposure classification, complying with the appropriate requirements for:

(a) Concrete quality and curing, in accordance with 3.5 to 3.12;
(b) Cover in accordance with 3.11 or 3.12;
(c) Chemical content restrictions, in accordance with 3.14;
(d) Alkali silica reaction precautions in accordance with 3.15;
(e) Protection of fixings to 3.13.

3.2.4 *Design for particular environmental conditions*

In addition to the requirements specified in 3.2.3:

(a) Members subject to aggressive soil and ground water shall satisfy the requirements of 3.5;
(b) Members subject to abrasion shall satisfy the requirements of 3.9;
(c) Members subject to cycles of freezing and thawing shall satisfy the requirements of 3.10.

3.3 Design life

3.3.1 *Specified intended life*

The provisions of this section shall apply to the detailing and specifying for durability of reinforced and prestressed concrete structures and members with a specified intended life of 50 or 100 years. Compliance with this section will ensure that the structure is sufficiently durable to satisfy the requirements of the NZ Building Code throughout the life of the structure, with only normal maintenance and without requiring reconstruction or major renovation. The 50 years corresponds to the minimum structural performance life of a member to comply with that code.

3.4 Exposure classification

3.4.1 *General*

A3 Where concrete will be in wet or saline conditions, aggressive soil or groundwater, or in contact with potentially aggressive chemicals, appropriate measures shall be highlighted in the drawings and specifications to ensure the durability of the structure.

3.4.2 *Environmental exposure classification*

3.4.2.1 *Exposure classification categories*

The exposure classification for a surface of a steel reinforced or prestressed member shall be determined from Table 3.1. Except for categories 4(b) and 5, this table need not apply to unreinforced members, members with non-metallic reinforcement, or steel fibre concrete provided that such concrete does not contain metals that rely on the concrete for protection against environmental degradation.

Table 3.1 – Exposure classifications

Surface and exposure environment	Exposure classification
1 Surfaces of members in contact with the ground:	
(a) Protected by a damp proof membrane	A1
(b) In non-aggressive soils	A2
2 Surfaces of members in interior environments:	
(a) Fully enclosed within a building except for a brief period of weather exposure during construction (1)	A1
(b) In buildings or parts thereof where the members may be subject to repeated wetting and drying (1)	B1
3 Surfaces of members in above-ground exterior environments in areas that are:	
(a) Inland (2)	A2
(b) Coastal perimeter (2)	B1
(c) Coastal frontage (see 3.4.2.4)	B2
4 Surfaces of members in water: (3)	
(a) (i) In fresh (not soft) water contact	B1
(ii) In fresh (not soft) water pressure	B2
(iii) In fresh (not soft) water running	B2
(b) (i) In fresh (soft) water contact	B2
(ii) In fresh (soft) water pressure	U
(iii) In fresh (soft) water running	U
(c) In sea water:	
(i) Permanently submerged	B2
(ii) Tidal/splash/spray (see 3.4.2.5)	C
5 Surfaces of members exposed to chemical attack (see 3.4.3) in:	
(a) Slightly aggressive chemical environment	XA 1
(b) Moderately aggressive chemical environment	XA 2
(c) Highly aggressive chemical environment	XA 3
6 Surfaces of members in other environments: Any exposure environment not otherwise described in items 1–5 above.	U

NOTE –

(1) Where concrete is used in industrial applications, consideration shall be given to the effects of any manufacturing process on the concrete which may require a reclassification to exposure classification U. (See 3.8)

(2) The boundary between the different exterior environments is dependent on many factors which include distance from sea, prevailing wind and its intensity.

(3) Water analysis is required to establish the characteristics of water softness.

3.4.2.2 *Mixed exposures*

For determining concrete quality requirements in accordance with 3.5 to 3.12, as appropriate, the exposure classification for the member shall be taken as the most severe exposure of any of its surfaces.

3.4.2.3 *Individual surfaces*

For determining cover requirements for corrosion protection in accordance with 3.11, the exposure classification shall be taken as the classification for the surface from which the cover is measured.

3.4.2.4 *Coastal frontage zone extent*

The extent of the coastal frontage zone shall be determined by reference to Table 3.2(b). General wind directions are indicated in Figure 3.1 (a) to (f). As an alternative solution, a site-specific evaluation can be made. The extent will depend on winds, wave action and topography. More specific wind frequency data can be obtained from the National Institute of Water and Atmospheric Research Ltd. A2

3.4.2.5 *Tidal/splash/spray zone*

The extent of the C tidal/splash/spray zone is given in Table 3.2(b). As an alternative a site - specific evaluation of spray drift can be made taking into account wind strength, wave action and local topography. Structures over the sea of body of saline water where breaking waves occur shall be classification C. A2

The boundary of the C zone and the B2 zone in the vertical direction shall be taken as mean low water level at depth and shall be determined from prevailing wind and sea conditions for height above sea level.

Table 3.2(a) – Prevailing or common winds A2

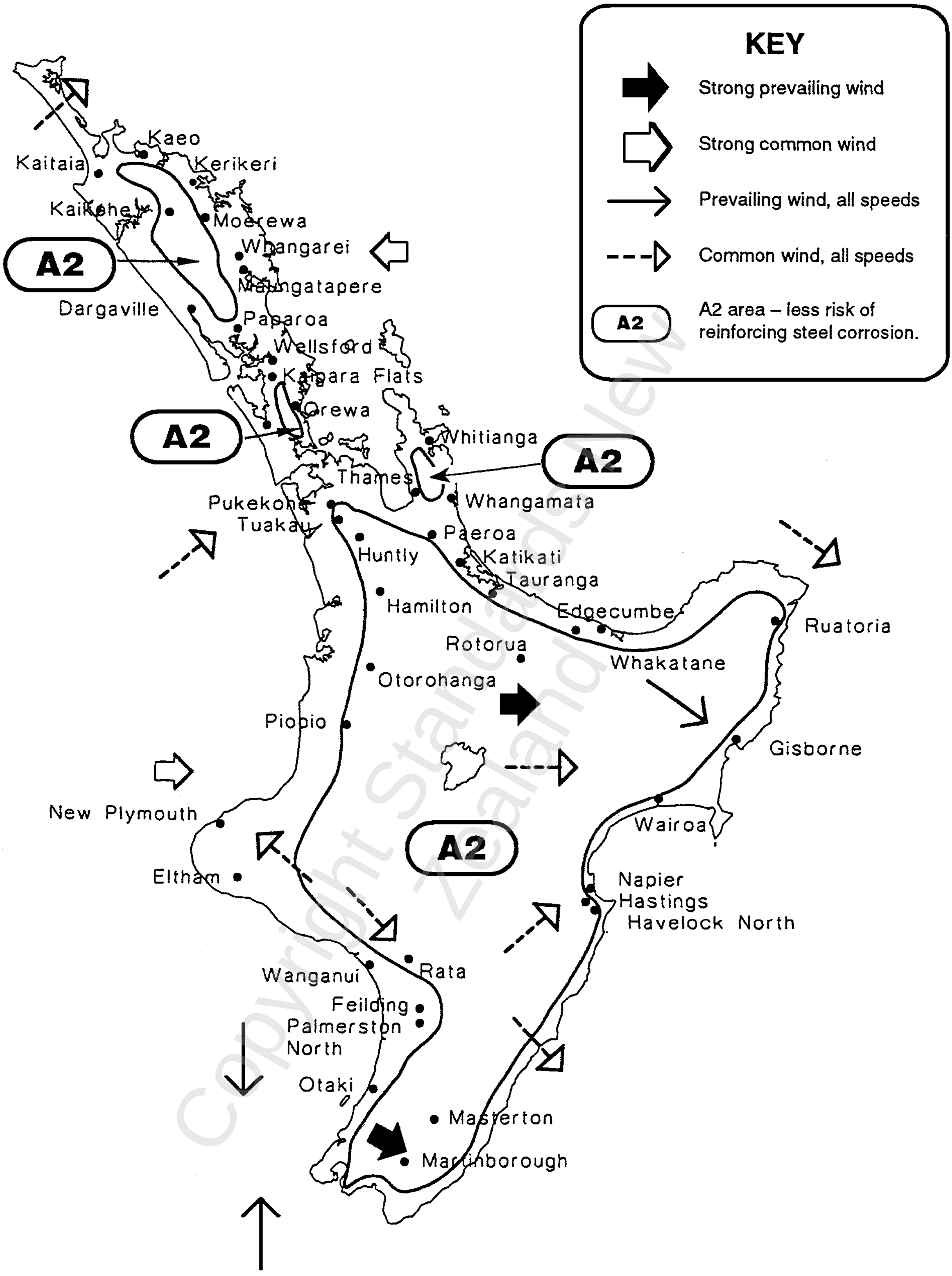
Locality	Direction prevailing or common wind comes from
Auckland	Southwest
Wellington	North, South or Northwest
Christchurch, Dunedin	Northwest, Northeast or Southwest
Other localities	Refer to Figure 3.1 (a) and (b)

Table 3.2(b) – Definition of B2 and C zones

Direction from coast to site exposed to common or prevailing wind	B2 (Coastal frontage)	C (Tidal / Splash / Spray)
Downwind	Between 30 m and 500 m inland of the high tide mark	Offshore and up to 30 m inland of the high tide mark
Directions other than downwind	From the high tide mark to 100 m inland	Offshore and up to the high tide mark

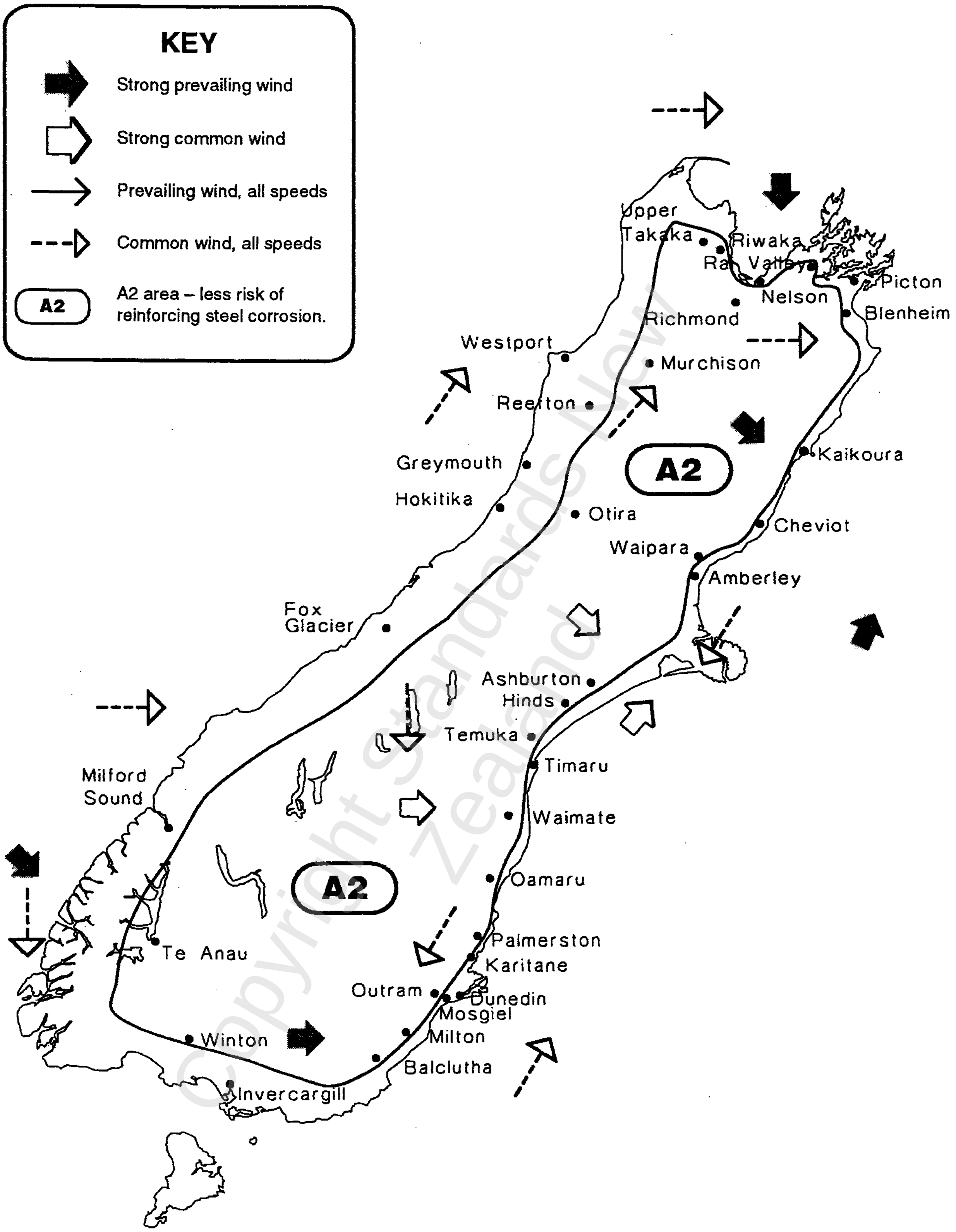
3.4.2.6 *Boundary between coastal perimeter and inland zones*

Figure 3.1(a) to (f) indicate the boundary between the inland (A2) and the coastal perimeter (B1) exposure classifications.



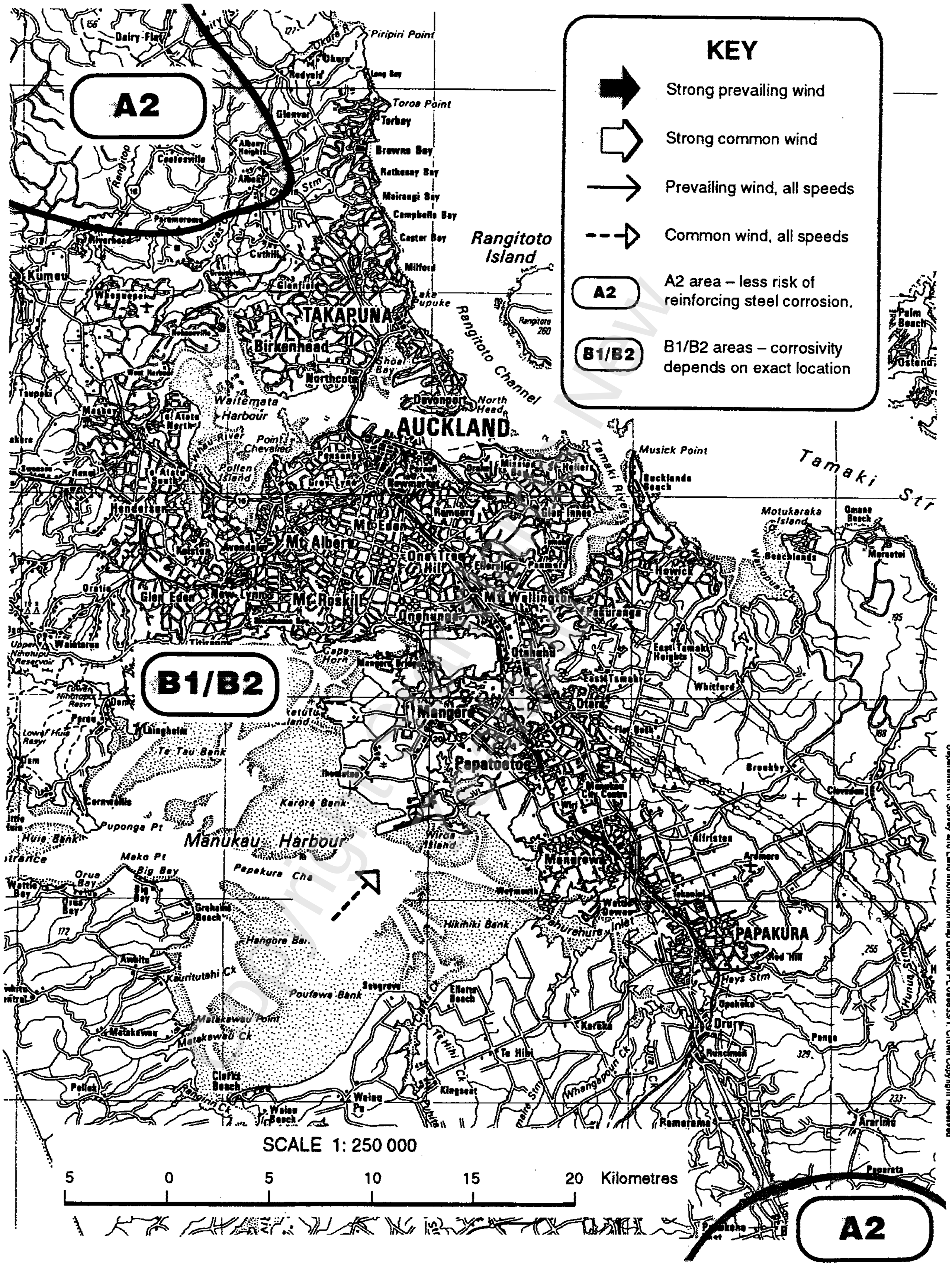
(a) North Island

Figure 3.1 – Exposure classification maps



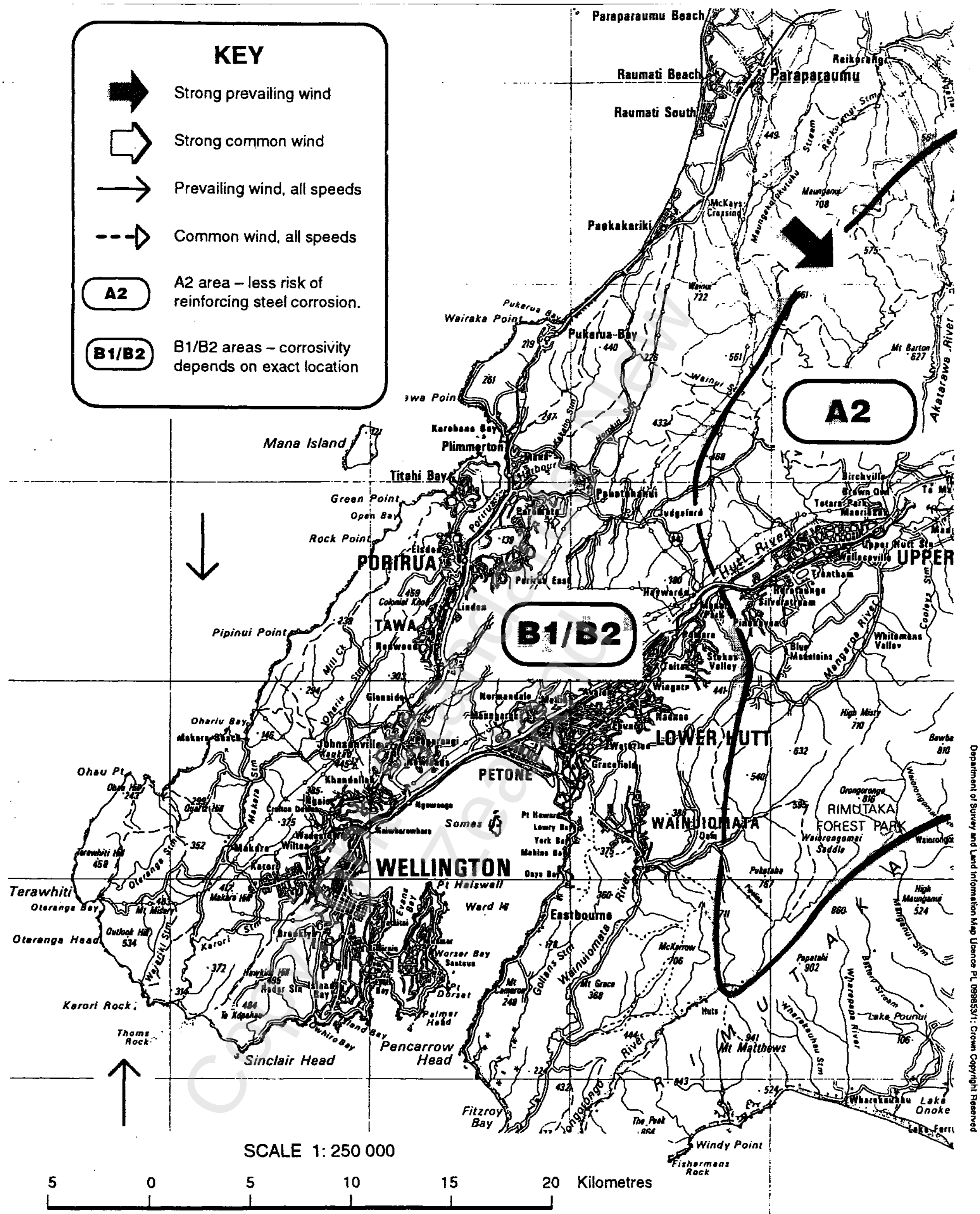
(b) South Island

Figure 3.1 – Exposure classification maps (continued)



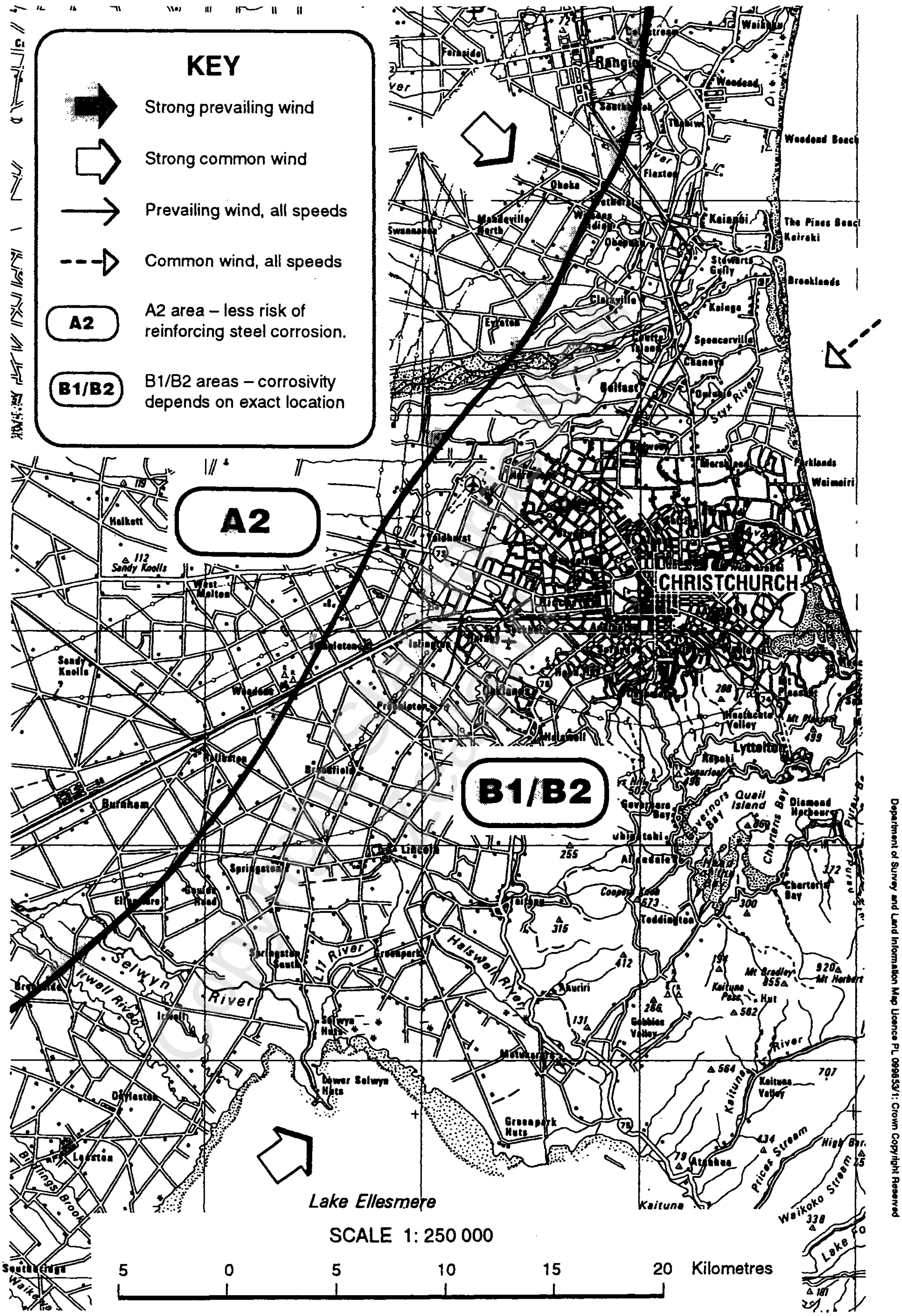
(c) Auckland

Figure 3.1 – Exposure classification maps (continued)



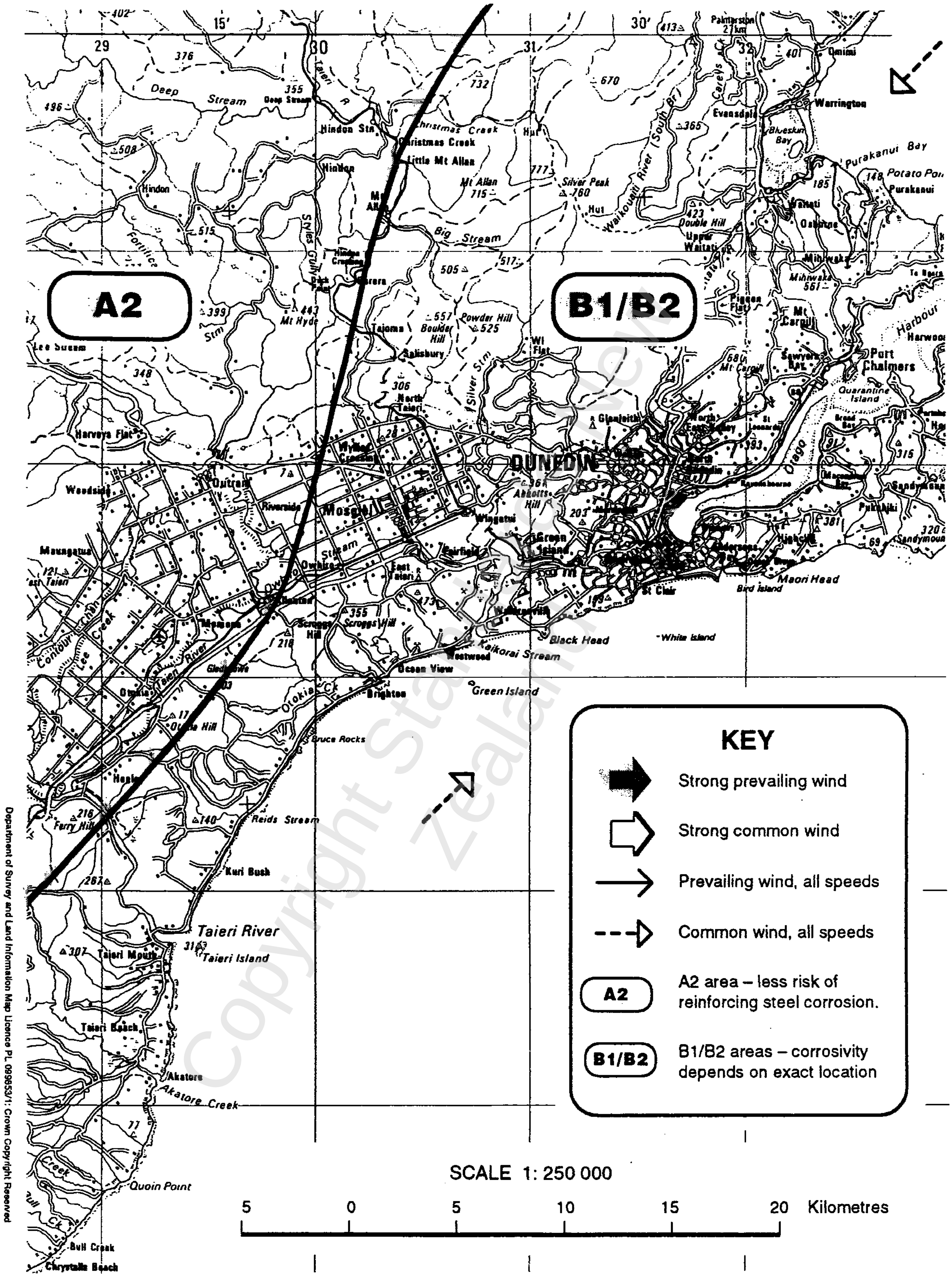
(d) Wellington

Figure 3.1 – Exposure classification maps (continued)



(e) Christchurch

Figure 3.1 – Exposure classification maps (continued)



(f) Dunedin

Figure 3.1 – Exposure classification maps (continued)

3.4.3 *Chemical exposure classification*

3.4.3.1 *Chemical attack from natural soil and groundwater*

The chemical exposure classification for a surface of a concrete member exposed to chemical attack from natural soil and ground water shall be determined from Table 3.3. The degree of aggressivity may be underestimated in cases where combined attacks plus high temperature and high relative humidity occur (e.g. geothermal areas). In this case the next higher degree of aggressivity determines the chemical exposure classification, unless a special study proves otherwise.

A2

Table 3.3 – Guide for exposure classification for chemical attack of concrete from natural soil and groundwater

Chemical exposure classification	Chemical constituents[1]				
	Ground water [2][3]		Soil ([3][4]		
	pH	Sulphate SO_4^{2-} (mg/ℓ)	Acidity[5] (ml/kg of air dry soil)	Acid soluble sulphate SO_4^{2-} [6] (% of air dry soil passing a 2 mm test sieve)	Water soluble sulphate $SO_4^{2-(6)}$ (g/ℓ in 2:1 water soil extract)
XA1	6.5 – 5.5	200 – 600	>200 (Baumann-Gully)	0.2 – 0.3	0.6 – 1.8
XA2	5.5 – 4.5	600 – 3000	–	0.3 – 1.2	1.8 – 3.7
XA3	4.5 – 4.0	3000 – 6000	–	1.2 – 2.4	3.7 – 6.7

NOTE –

(1) Magnesium content is considered to be less than 1000 mg/ℓ.
(2) Mobility of water is considered to be in an approximately static condition.
(3) Soil and groundwater temperature 5° C to 25° C.
(4) Nominally dry sites or soils with permeability less than 10^{-5} m/s (e.g. unfissured clay) may be moved into a lower class.
(5) The Baumann-Gully acidity is expressed as volume of 0.1 mol/litre sodium hydroxide required to neutralise acetic acid, in ml/kg of air dried soil (DIN 4030-2).
(6) Measure either acid soluble or water soluble sulphate in accordance with BS 1377-3, depending on which is more appropriate for the site being assessed. Sulphate results expressed in terms of SO_3 shall be multiplied by 1.2 to convert them to SO_4^{2-} values.
(7) A special study under exposure classification U is required where there is:
   (a) Limits outside of Table 3.3 or Table 3.3 Notes;
   (b) Direct contact with chemically aggressive environments; or
   (c) High water velocities and/or water under pressure in combination with the aggressive agents stated in Table 3.3. In these circumstances consult BS 8500-1 for guidance.

3.4.3.2 *Other chemical attack*

An acidity represented by a pH of 5.0 to 5.5 may be considered as a practical limit of tolerance of high quality concrete in contact with any acids. For pH lower than 5.0, the environment shall be assessed as exposure classification U.

3.5 Requirements for aggressive soil and groundwater exposure classification XA

Concrete in members subject to chemical attack shall be specified in accordance with Table 3.4. Such concrete shall be specified as 'Special Concrete' under NZS 3109 Clause 6.3.

Table 3.4 – Requirements for concrete subjected to natural aggressive soil and groundwater attack for a specified intended life of 50 years and 100 years A3

Chemical exposure classification	Max. water cementitious ratio	Min. cover 50 years (mm)	Min. cover 100 years (mm)	Min. binder content (kg)	Additional requirements
XA1	0.50	50	65	340	—
XA2	0.45	50	65	370	SCM
XA3	0.40	55	75	400	SCM

NOTE –

(1) Binders containing combinations of cement and supplementary cementitious materials (SCM) (30 % fly ash, 65 % slag or 8 % amorphous silica) provide significantly increased resistance to chemical attack mechanisms.

(2) Where low pH and high exchangeable soil acid conditions prevail, an additional protection (e.g. protective coating, or other form of physical protection) may be required. This may allow for reduction of originally specified concrete parameters.

3.6 Minimum concrete curing requirements

Minimum concrete curing requirements for the exposure classifications given in Table 3.1 are given in Table 3.5:

Table 3.5 – Minimum concrete curing requirements A2

Exposure classification	Curing period[(1) (3)] (under ambient conditions)
A1, A2, B1	3 days
B2	7 days
C	7 days[(2)]
XA1	3 days
XA2	7 days[(2)]
XA3	7 days[(2)]

NOTE –

(1) Curing shall comply with Clause 7.8 of NZS 3109.

(2) Concrete in C, XA2, and XA3 zones shall be cured continuously by direct water application such as ponding or continuous sprinkling, or by continuous application of a mist spray.

(3) Alternative curing methods may be used, provided a special study proves that the alternative method provides concrete durability performance equivalent to that provided by direct water application.

3.7 Additional requirements for concrete exposure classification C

3.7.1 *Supplementary cementitious materials*

The concrete shall contain a supplementary cementitious material. Table 3.6 and Table 3.7 give three compliant options using three different supplementary cementitious materials.

3.7.2 *Water/binder ratio and binder content*

Binder combination, minimum binder material content and maximum water/binder ratio shall be specified in addition to compressive strength to comply with Table 3.6 and Table 3.7.

3.7.3 *Special concrete*

Concrete to be used in exposure classification C shall be specified as ‘Special Concrete’ under NZS 3109 Amendment No.1, August 2003 Clause 6.3. The quality control requirements for the concrete supply, and any special durability related testing shall be ascertained between the specifier and the concrete producer.

A3

Table 3.6 – Minimum required cover for a specified intended life of 50 years

Exposure classification	Cement binder type	Specified compressive strength f'_c (MPa)							
		20	25	30	35	40	45	50	60 – 100
		Minimum required cover (mm)							
A1	GP, GB or HE	25	25	20	20	20	20	20	20
A2	GP, GB or HE	40	35	30	30	25	25	25	20
B1	GP, GB or HE	50	40	35	35	30	30	30	25
B2	GP, GB or HE	-	-	45	40	35	30	30	25
C (1) (2)	30 % FA	-	-	-	-	60	50	50	50
C (1) (2)	65 % GBS	-	-	-	-	-	50	50	50
C (1) (2)	8 % MS	-	-	-	-	-	60	50	50

NOTE –
(1) For zone C the total binder content shall be equal to or greater than 350 kg/m^3, and water-to-binder ratio shall not exceed 0.45.
(2) The minimum cover for the C zone shall be 50 mm.

A3

Table 3.7 – Minimum required cover for a specified intended life of 100 years

Exposure classification	Cement binder type	Specified compressive strength f'_c (MPa)						
		25	30	35	40	45	50	60 – 100
		Minimum required cover (mm)						
A1	GP, GB or HE	35	30	30	30	30	30	25
A2	GP, GB or HE	50	40	40	35	35	35	30
B1	GP, GB or HE	55	50	45	40	40	35	30
B2	GP, GB or HE	-	65	55	50	45	40	35
C (1) (2)	30 % FA	-	-	-	-	60	50	50
C (1) (2)	65 % GBS	-	-	-	-	60	50	50
C (1) (2)	8 % MS	-	-	-	-	-	60	60

NOTE –
(1) For zone C the total binder content shall be equal to or greater than 350 kg/m^3 and water to binder ratio shall not exceed 0.45.
(2) The minimum cover for the C zone shall be 50 mm.

3.8 Requirements for concrete for exposure classification U

Exposure Classification U represents an exposure environment not specified in Table 3.1 for which the degree of severity of exposure should be assessed by the designer. Concrete in members subject to exposure classification U shall be specified to ensure durability under the particular exposure environment and for the chosen design life. Protective coatings may be taken into account in the assessment of concrete requirements.

3.9 Finishing, strength and curing requirements for abrasion

3.9.1 *Abrasion from traffic*

Concrete for members subject to abrasion from traffic shall comply with the specified compressive strength and construction requirements given in Table 3.8.

Table 3.8 – Requirements for abrasion resistance for a specified intended life of 50 years

Class	Service conditions	Application	Finishing process	Curing	Minimum specified compressive strength f'_c (MPa)
Special	Severe abrasion and impact from steel or hard plastics wheeled traffic or scoring by dragged metal objects	Very heavy duty engineering workshops and very intensively used warehouses	Special flooring techniques may be used. The suitability of concrete flooring for this class should be established with the manufacturer or flooring contractor		
AR1	Very high abrasion: steel or hard plastics wheeled traffic and impact	Heavy duty industrial workshops and intensively used warehouses			
AR2	High abrasion: steel or hard plastics wheeled traffic	Medium duty industrial and commercial	Power floating and at least two passes with a power trowel	7 days water curing using ponding or covering; or the use of a curing membrane that meets NZS 3109	40 MPa
AR3	Moderate abrasion: Rubber tyred traffic	Light duty industrial and commercial			30 MPa
Commercial and industrial floors not subject to vehicular traffic			As nominated by the designer	3 days minimum	25 MPa

3.9.2 ***Abrasion by waterborne material***

Abrasion erosion damage caused by the abrasive effects of waterborne sediment (i.e. silt, sand, gravel, rock) and other debris impinging on a concrete surface may affect structures such as spillways, culverts and bridge piers. For guidance on materials and techniques to control abrasion erosion refer to erosion of Concrete in Hydraulic Structures", Report by ACI Committee 210, Report No. ACI 210R-93, American Concrete Institute, Michigan, USA, 1993.

3.10 Requirements for freezing and thawing

In addition to the other durability requirements of this section, where a surface may be exposed to cycles of freezing and thawing, concrete in the member shall:

(a) Contain a percentage of entrained air within the following ranges for:
  - (i) 10 mm to 20 mm nominal size aggregate .... 4 % to 8 %;
  - (ii) Greater than 20 mm nominal size aggregate .... 3 % to 6 %;

  where the percentage of entrained air is determined in accordance with NZS 3112:Part 1 and

(b) Have a specified compressive strength, f'_c, equal to or greater than:
  - (i) 30 MPa for frequent exposure (≥ 50 cycles per year);
  - (ii) 25 MPa for occasional exposure (25 – 49 cycles per year).

3.11 Requirements for concrete cover to reinforcing steel and tendons

3.11.1 *General*

3.11.1.1 *Cover*

The cover to reinforcing steel and tendons shall be the greater of the values determined from 3.11.2 and 3.11.3, as appropriate, unless greater covers are required by Section 4 for fire resistance. Cover shall be measured to the stirrups or reinforcement which is closest to the surface of the member.

3.11.1.2 *Effect of crack width control on cover*

Crack width control in accordance with 2.4.4 may limit maximum covers allowable for durability purposes.

3.11.2 *Cover of reinforcement for concrete placement*

3.11.2.1 *Reinforcing steel configuration*

The cover and arrangement of the steel shall be such that concrete can be properly placed and compacted in accordance with NZS 3109. The spacing requirements are given in Clause 8.3.

3.11.2.2 *Minimum cover*

The cover shall be equal to or greater than either:

(a) The maximum nominal aggregate size for Exposure Classifications A1 and A2 and 1.25 times the maximum nominal aggregates size for other exposure zones; or

(b) The nominal size of bar or tendon to which the cover is measured;

whichever is the greater.

3.11.3 *Cover for corrosion protection*

3.11.3.1 *General*

For corrosion protection, the cover shall be equal to or greater than the appropriate value given in 3.11.3.2 and 3.11.3.3.

3.11.3.2 *Formed or free surfaces*

Where concrete is compacted in accordance with NZS 3109; cover to formwork complying with NZS 3109, or to free surfaces shall be equal to or greater than the value given in Table 3.6Table 3.6 or Table 3.7 appropriate to the design life, the exposure classification and specified concrete strength. Table 3.6 and Table 3.7 give cover requirements for a specified intended life of 50 years and 100 years respectively.

3.11.3.3 *Casting against ground*

Where concrete is cast on or against ground and compacted in accordance with NZS 3109, the minimum cover for a surface in contact with the ground shall be 75 mm, or 50 mm if using a damp-proof membrane between the ground and the concrete to be cast.

3.12 Chloride based life prediction models and durability enhancement measures

3.12.1 *The use of life prediction models*

Life prediction models can be used as an alternative to Table 3.6 and Table 3.7 for the C zone and B2 zone, however they are outside the scope of this Standard. The tables will generally provide solutions which are more conservative than those derived from the use of a model. Guidance on the use of life prediction models to determine cover as an alternative to Table 3.6 and Table 3.7, is given in the commentary.

3.12.2 *Other durability enhancing measures*

There are a number of durability enhancing measures which can be taken to extend the life of concrete structures beyond those determined in accordance with 3.11.3.2. These include concrete coatings, corrosion inhibiting admixtures, galvanised or stainless steel reinforcement, controlled permeability formwork and GRC permanent formwork.

3.13 Protection of cast-in fixings and fastenings

3.13.1 *Fixing and fastening protection*

Where metallic fixings or fastenings are exposed in the finished structure, or where the cover to any part of the fixing is less than that required by 3.11 for the particular exposure classification, additional protection shall be provided in accordance with Table 3.9.

Table 3.9 – Protection required for steel fixings and fastenings for a specified intended life of 50 years

A2

Exposure classification	Material/protection
Dry, internal location, not subject to airborne salts or rain wetting	
A1	Mild steel (uncoated non-galvanised) Electroplated zinc anchors (1)
Open to airborne salts, but not rain washed	
A2, B1	Hot-dip galvanised steel (2) Hot-dip galvanised anchors(2)
B2(3), C	Stainless steel type 304(4) (5) Stainless steel anchors type 316
Open to airborne salts and rain washed	
A2	Hot-dip galvanised steel(2) (6) Hot-dip galvanised anchors(2) (6)
B1, B2(3), C	Stainless steel type 304 (4),(5) Stainless steel anchors type 316

NOTE –
(1) Anchors include cast-in, mechanical or chemical anchors.
(2) All galvanising weights to steel are to comply with Table 3.10
(3) Material/protection requirements apply only to the B2 zone where not permanently submerged in seawater. Material/protection requirements, where metallic fixings or fastenings are permanently submerged in seawater, should be the subject of a special study.
(4) Type 304 stainless steel may have surface rust. Type 316 should be used where appearance is a consideration.
(5) Where there is a final cover of at least 30 mm to any part of the fixing, galvanised fixings may be substituted.
(6) The minimum requirement is hot-dip galvanised steel plus additional protection, such as epoxy powder coating, under conditions defined in 4.4.4 and 4.4.5 in NZS 3604. Type 304 stainless steel is also a suitable option.

3.13.2 *Galvanised fixings*

Galvanised steel components shall have galvanised coating masses to meet a 50-year durability in accordance with Table 3.10.

Table 3.10 – Galvanising of steel components

Component	Standard	Material protection
Bolts in any location that require galvanising (refer Table 3.9)	AS 1214	375 g/m^2 average
Fixing plates, angles	AS/NZS 2699.3 or AS/NZS 4680	600 g/m^2

3.14 Restrictions on chemical content in concrete

3.14.1 *Restriction on chloride ion content for corrosion protection*

3.14.1.1 *Added chloride*

Chloride salts or chemical admixtures formulated with greater than 0.1 % by weight of chloride shall not be added to any steel reinforced concrete required for exposure classifications B1, B2 or C, or to any prestressed or steam cured concrete.

A3 **3.14.1.2** *Chloride*

At the time of placement, the maximum chloride ion content of all steel reinforced concrete shall not exceed the values given in Table 3.11. The concrete's chloride content may be calculated from chloride contents of individual components of cement, aggregate, mixing water (including recycled washwater) and admixtures, or may be measured directly on a sample of hardened concrete in accordance with 3.14.1.3.

Table 3.11 – Maximum values of chloride ion content in concrete as placed

Type of member	Maximum acid soluble chloride ion content (kg/m^3 of concrete)
Prestressed concrete	0.50
Reinforced concrete exposed to moisture or chloride in service	0.80
Reinforced concrete that will be dry or protected from moisture in service	1.6

A3 **3.14.1.3** *Testing for chloride content in concrete*

When testing is performed to determine the acid soluble chloride ion content, test procedures shall conform to ASTM C1152, AS 1012.20, or BS 1881-124. Alternatively XRF may be used if conducted at a suitably experienced IANZ-accredited chemical laboratory where the test has been calibrated against ASTM C1152, AS 1012.20 or BS 1881-124.

3.14.2 ***Restriction on sulphate content***

The sulphate content of concrete as placed, expressed as the percentage by mass of acid soluble SO_3 to cement shall not be greater than 5.0 %.

3.14.3 ***Restriction on other salts***

Other salts shall not be added to concrete unless it can be shown that they do not adversely affect durability.

3.15 Alkali silica reaction

A3 Where concrete aggregates proposed to be used are potentially reactive with alkalis, precautions to minimise the risk of damage shall be taken in accordance with CCANZ TR3.

4 DESIGN FOR FIRE RESISTANCE

4.1 Notation

A_c area of concrete, mm^2
A_s area of reinforcement, mm^2
a axis distance, mm A2
b column width, beam width, or wall thickness, mm
b_w web thickness, mm A3

L_y longer span of a two-way slab, mm
L_x shorter span of a two-way slab, mm
N_f^* factored design load on a column, N
N_u axial load capacity of a column at normal temperature, N A3

η_{fi} the ratio of factored design load in fire resistance to axial load capacity at normal temperature
ϕ strength reduction factor (as defined in 2.3.2.2) A3

4.2 Scope

The provisions of this section set out the requirements for the design of reinforced and prestressed concrete structures and members to resist the effects of fire, and gives methods for determining the fire resistance ratings required by the New Zealand Building Code.

4.3 Design performance criteria

4.3.1 *General performance criteria*

4.3.1.1 *Required fire resistance*

A member shall be designed to have a fire resistance rating (FRR) for each of structural adequacy, integrity and insulation equal to or greater than the required fire resistance.

4.3.1.2 *Integrity*

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, if applicable.

4.3.1.3 *Shear, torsion and anchorage*

Unless stated otherwise within this section when using the tabulated data or charts no further checks are required concerning shear and torsional capacity or anchorage details.

4.3.1.4 *Use of tabulated data or calculation*

The fire resistance rating (FRR) of concrete elements may be assessed using the tabulated data given in 4.4 to 4.7, or by calculation as specified in 4.10.

4.3.2 *General rules for the interpretation of tabular data and charts*

Linear interpolation between values given in the tables and charts 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 chart.

4.3.3 *Increase in axis distance for prestressing tendons*

The required axis distance for reinforcing bars shown in the tables shall be increased by the following distances where prestressing tendons are used:

(a) For prestressing bars .... 10 mm; and
(b) For prestressing strand and wires .... 15 mm.

4.3.4 *Joints*

Joints between members or between adjoining parts shall be constructed so that the fire resistance of the whole assembly is equal to or greater than that required for the member. The adequacy of methods used to protect service penetrations and control joints in walls or slabs shall be determined by testing in accordance with AS 1530: Part 4. Additional guidance can be found in AS 4072:Part 1.

4.3.5 *The effect of chases*

In concrete members subject to fire, chases shall be kept to a minimum. The effect of chases on the FRRs of walls shall be taken into account in accordance with the provisions of 4.7.3. The effects of chases in other members shall be taken into account using rational methods of analysis.

4.3.6 *Increasing FRRs by the addition of insulating materials*

4.3.6.1 *Use of insulation*

The FRRs 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 provisions of 4.9.

4.3.6.2 *Slabs*

For slabs, the FRRs may be increased by the addition of toppings and/or the application of insulating materials to the soffit.

4.3.6.3 *Other methods*

For walls, the FRRs may be increased, in accordance with 4.9, by the application of insulating materials to the face exposed to fire.

4.3.6.4 *Use of other methods*

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 4.9.

4.4 Fire resistance ratings for beams

4.4.1 *Structural adequacy for beams incorporated in roof or floor systems*

A2 The fire resistance period for structural adequacy for a beam incorporated in a roof or floor system, is given by:

(a) For simply supported beam Table 4.1; or

(b) For continuous beams Table 4.2; provided the beam:

   (i) Has the upper surface integral with, or protected by, a slab complying with clause 4.5;

   (ii) Has a web of uniform width or one which tapers uniformly over its depth;

   (iii) Is proportioned so that:

      (A) The beam width measured at the centroid of the lowest level of longitudinal bottom reinforcement; and

      (B) The axis distance to the longitudinal bottom reinforcement are not less than the values given in the appropriate tables;

      (C) The web width is not less than the values given in the appropriate table.

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

Table 4.1 – Fire resistance criteria for structural adequacy for simply-supported beams

Fire resistance rating (minutes)		**Minimum dimensions** (mm)				
		Possible combinations of *a and *b****				**Web thickness b_w*** (mm)
30	*b* *a*#	80 25	120 20	160 15	200 15	80
60	*b* *a*#	120 45	160 35	200 30	300 25	100
90	*b* *a*#	150 55	200 45	300 40	400 35	100
120	*b* *a*#	200 65	240 60	300 55	500 50	120
180	*b* *a*#	240 80	300 70	400 65	600 60	140
240	*b* *a*#	280 90	350 80	500 75	700 70	160
Column 1	2	3	4	5	6	7

LEGEND:

* Where *a* is the axis distance
b is the minimum width of the beam
b_w is the minimum width of the web (for a non-rectangular section) A2

In beams with only one layer of bottom reinforcement the distance from the centreline of the corner bar to the side of the beam (or tendons or wires) shall be increased by 10 mm except where the value of *b* is greater than that given in column 5 no increase is required. A2

NOTE – For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

Table 4.2 – Fire resistance criteria for structural adequacy for continuous beams

Fire resistance rating (minutes)		**Minimum dimensions** (mm)			
		Possible combinations of *a and *b****			**Web thickness b_w*** (mm)
30	*b* *a*#	80 15	160 12	200 12	80
60	*b* *a*#	120 25	200 12	300 12	100
90	*b* *a*#	150 35	250 25	400 25	100
120	*b* *a*#	200 45	300 35	500 30	120
180	*b* *a*#	240 60	400 50	600 40	140
240	*b* *a*#	280 75	500 60	700 50	160
Column 1	2	3	4	5	6

LEGEND:

* Where *a* is the axis distance
b is the minimum width of the beam
b_w is the minimum width of the web (for a non-rectangular section) A2

In beams with only one layer of bottom reinforcement the distance from the centreline of the corner bar to the side of the beam (or tendons or wires) shall be increased by 10 mm except where the value of *b* is greater than that given in column 5 no increase is required. A2

NOTE – For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

4.4.2 *Structural adequacy for beams exposed to fire on all sides*

A beam of approximately rectangular cross section, which can be exposed to fire on all four sides, has a particular fire resistance rating for structural adequacy if it is proportioned so that:

(a) The total depth of the beam is equal to or greater than the least value of *b*, obtained from Table 4.1 or Table 4.2 as appropriate;

(b) The cross-sectional area of the beam is equal to or greater than twice the area of a square with a side equal to *b* determined as for Item (a); and

(c) The axis distance is equal to or greater than the value determined using the minimum dimension of the beam for *b* in the relevant Table and applies to all longitudinal reinforcement or tendons.

4.5 Fire resistance ratings for slabs

4.5.1 *Insulation for slabs*

A slab has one of the FRRs for insulation given in Table 4.3 if the effective thickness of the slab is equal to or greater than the corresponding value given in the table.

The effective thickness of the slab to be used in Table 4.3 shall be taken as follows:

(a) For solid slabs, the actual thickness;

(b) For hollow-core slabs, the net cross-sectional area divided by the width of the cross section;

(c) For ribbed slabs, the thickness of the solid slab between the webs of adjacent ribs.

Table 4.3 – Fire resistance criteria for insulation for slabs

FRR for insulation (minutes)	**Effective thickness** (mm)
30	60
60	75
90	95
120	110
180	140
240	165

4.5.2 *Structural adequacy for slabs*

A slab has one of the FRRs for structural adequacy if it is proportioned so that:

(a) For solid or hollow-core slabs if, for the appropriate support conditions, the axis distance to the bottom layer of reinforcement and tendons is equal to or greater than the corresponding value given in Table 4.4.

(b) For flat slabs the axis distance to the bottom layer of reinforcement and tendons is equal to or greater than the corresponding value given in Table 4.5, provided that:

(i) The moment redistribution used in the analysis does not exceed 15 %; and

(ii) At least 20 % of the total top reinforcement in each direction over intermediate supports shall be continuous over the full span and placed in the column strip.

(c) For ribbed slabs if, for the appropriate support conditions it is proportioned so that:

A2 (i) The width of the ribs and the axis distance to the longitudinal bottom reinforcement in the ribs are equal to or greater than those given in Table 4.6; and

(ii) The axis distance to the bottom reinforcement in the slab between the ribs is equal to or greater than that determined in accordance with Item (a) above.

For the purpose of this clause, a slab shall be considered continuous if, under imposed load, it is flexurally continuous at least at one end.

Table 4.4 – Fire resistance ratings for solid and hollow-core slabs

Fire resistance rating (minutes)	Axis distance, *a*, to bottom layer of reinforcement [(1)] (mm)			
	Simply supported slabs			Continuous slabs (one-way and two-way)
	One-way	Two-way [(2)]		
		L_y/L_x[(3)] ≤ 1.5	1.5 < L_y/L_x[(3)] ≤ 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

NOTE –
(1) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.
(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) Where L_y is the longer span of a two-way slab
L_x is the shorter span of a two-way slab

Table 4.5 – Fire resistance ratings for flat slabs

Fire resistance rating (minutes)	Minimum dimensions (mm)	
	Slab thickness	Axis distance
30	150	10
60	180	15
90	200	25
120	200	35
180	200	45
240	200	50

NOTE –
(1) The axis distance relates to the reinforcement in the lower layer.
(2) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

Table 4.6 – Fire resistance criteria for structural adequacy for ribbed slabs

Fire resistance rating (minutes)	**Simply supported one-way and two-way ribbed slabs**		**Continuous one-way and two-way ribbed slabs**	
	Minimum width of rib (mm)	**Axis distance** (mm)	**Minimum width of rib** (mm)	**Axis distance** (mm)
30	80	15	80	10
60	100 120 ≥200	35 25 15	100 120 ≥200	25 15 10
90	120 160 ≥250	45 40 30	120 160 ≥250	35 25 15
120	160 190 ≥300	60 55 40	160 190 ≥300	45 40 30
180	220 260 ≥410	75 70 60	310 600	60 50
240	280 350 500	90 75 70	450 700	70 60

NOTE –

(1) The axis distance is measured to the longitudinal bottom reinforcement.

(2) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

(3) The minimum slab thickness in the flange should comply with Table 4.3 but be equal to or greater than 75 mm.

4.6 Fire resistance ratings for columns

4.6.1 *Insulation and integrity for columns*

FRRs 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 4.7.1.

4.6.2 *Structural adequacy for columns*

The FRR for square, rectangular or circular columns shall be determined by using the minimum dimensions shown in Table 4.7.

The value of the load level, η_{fi}, shall be taken as 0.7 or calculated as follows:

$$\eta_{fi} = \frac{N_f^*}{N_u} \quad \text{(Eq. 4–1)}$$

where

N_f^* is the factored design axial load on the column in fire conditions

N_u is the axial load capacity of the column at normal temperature

Where $A_s \geq 0.02A_c$ and the required FRR is greater than 90 minutes, the bars shall be distributed along the sides of the column.

The dimension *b* in Table 4.7 for columns exposed on one side only applies to columns that lie flush with a wall having the same FRR, or to columns protruding from the wall providing that the part within the wall is able to carry the whole load. Openings in the wall shall not be nearer to the column than the minimum dimension *b* for the column for the FRR. Otherwise the column shall be treated as a column exposed on more than one side.

Table 4.7 – Fire resistance criteria for structural adequacy for columns

A2

Fire resistance rating (minutes)		**Minimum dimensions** (mm)			
		Column exposed on more than one side			**Column exposed on one side**
		η_{fi} **= 0.2**	η_{fi} **= 0.5**	η_{fi} **= 0.7**	η_{fi} **= 0.7**
30	b a	200 25	200 25	200 30	155 25
60	b a	200 25	200 35	250 45	155 25
90	b a	200 30	300 45	350 50	155 25
120	b a	250 40	350 45	350 55	175 35
180	b a	350 45	350 60	450 70	230 55
240	b a	350 60	450 75	500 70	295 70
Column 1	2	3	4	5	6

NOTE –
(1) See 4.6.2.
(2) For prestressing tendons, the increase in axis distance given in 4.3.3 shall be noted.

4.7 Fire resistance ratings for walls

4.7.1 *Insulation for walls*

A wall has the fire resistance rating for insulation given in Table 4.8 if the effective thickness of the wall is equal to or greater than the corresponding value given in the table.

The effective thickness of the wall to be used in Table 4.8 shall be taken as follows:

(a) For solid walls, the actual thickness;

(b) For hollow-core walls the net cross-sectional area divided by the length of the cross section.

Table 4.8 – Minimum effective thickness for insulation

Fire resistance rating (minutes)	**Effective thickness** (mm)
30	60
60	75
90	95
120	110
180	140
240	165

4.7.2 *Structural adequacy for walls*

The FRR for structural adequacy for a wall shall be determined by using the values for minimum dimensions shown in Table 4.9.

Table 4.9 – Fire resistance criteria for structural adequacy for load-bearing walls

Fire resistance rating (minutes)		**Minimum dimensions** (mm)	
		Wall exposed to fire on one side	
		$\eta_{fi} = 0.35$	$\eta_{fi} = 0.7$
30	*b* *a*	100 10	120 10
60	*b* *a*	110 10	130 10
90	*b* *a*	120 20	140 25
120	*b* *a*	150 25	160 35
180	*b* *a*	180 40	210 50
240	*b* *a*	230 55	270 60
Column 1	2	3	4

NOTE –

(1) $\eta_{fi} = N^*_f / N_u$ see 4.6.2.

(2) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

4.7.3 *Chases and recesses for services in walls*

4.7.3.1 *When the effect of chases and recesses may be ignored*

The effect of chases and recesses for services, on the fire resistance ratings for structural adequacy, integrity and insulation of a wall, shall be ignored if the thickness of wall remaining under the bottom of the chase or recess is equal to or greater than half the wall thickness and the total recessed area, within any 5 m^2 of wall face, is not more than 10,000 mm^2 on one or both faces of the wall.

4.7.3.2 *Taking account of chases and recesses*

If the above limits are exceeded, the wall thickness, t, used to determine fire resistance ratings shall be taken as the overall thickness less the depth of the deepest chase or recess.

A3 4.8 External walls or wall panels that could collapse inward or outward due to fire

4.8.1 *General*

4.8.1.1 *Objective*

The purpose of this clause is to protect building occupants, prevent fire spread, protect fire fighters and protect other property.

4.8.1.2 *Application*

This clause applies to external walls and external panels that could collapse inward or outward from a building during or after a fire as a result of internal fire exposure.

4.8.1.3 *Portal frame buildings*

This clause also applies to portal frame buildings where external wall panels could collapse outward. In this application it is acceptable to allow controlled gradual internal collapse of the portal frame (with the panel(s) connected) to occur.

4.8.1.4 *Connection and wall requirements*

All walls and wall panels shall:

(a) Be attached to the building structure by steel connections;

(b) Be restrained by these connections, when subject to fire, from inwards or outward collapse of the wall relative to the building structure; and

(c) Comply with the appropriate provisions of this standard for walls.

4.8.2 *Forces on connections*

The connections between each wall and wall panel and the supporting structure shall be designed to resist all anticipated forces under both ambient and fire conditions. Under fire conditions, in the absence of a detailed analysis, the connections shall be designed to resist a face load in any direction of at least 0.5 kPa. A3

Any wall that may remain standing following a fire shall be designed to resist a face load in any direction of at least 0.5 kPa following the fire.

4.8.3 *Design of connections*

All connection components of the fire rated external walls and wall panels shall be fire rated.

Connections of non-fire rated external walls and external panels shall be designed to allow for reduced capacity in the fire conditions as follows:

(a) Components made from unprotected mild steel shall be designed using 30 % of the yield strength of the steel in ambient conditions;

(b) Components made from other types of steel shall be designed using the mechanical properties of the steel at 680 °C;

(c) Proprietary inserts shall be designed for a minimum fire resistance rating of at least 60 minutes for unsprinklered buildings and 30 minutes for sprinklered buildings.

4.8.4 *Fixing inserts*

The fixings of inserts shall comply with the following:

(a) Proprietary cast-in or drilled-in inserts with an approved fire resistance rating shall be installed in accordance with the manufacturer's specifications;

(b) Cast-in inserts without an approved fire resistance rating shall be anchored into the wall by steel reinforcement or fixed to the wall reinforcement;

(c) Adhesive anchors shall only be used if they have a fire resistance rating tested in accordance with ISO 834:Part 1, DIN 4102-2 or AS 1530:Part 4, and are used in accordance with the manufacturer's specifications. A2

4.8.5 *Walls spanning vertically*

Walls that span vertically shall have at least two upper connections per wall panel except where several narrow panels are connected to each other to act as a single unit in which case there shall be at least two upper connections per single unit. A2

4.8.6 *Walls spanning horizontally*

Walls that span horizontally between columns shall have at least two connections per column.

4.9 Increase of fire resistance periods by use of insulating materials

4.9.1 *General*

The fire resistance ratings 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.

4.9.2 *Acceptable forms of insulation*

Acceptable forms of insulation include the following:

(a) Thicknesses of 1:4 vermiculite concrete or of 1:4 perlite concrete, which are appropriately bonded to the concrete;

(b) Gypsum-vermiculite plaster or gypsum-perlite plaster, both mixed in the proportion of 0.16 m^3 of aggregate to 100 kg of gypsum, in the form of either thickness added and appropriately bonded to the concrete, or as a sprayed or trowelled application applied *in situ*;

(c) Any other fire protective building board or material, that has been demonstrated to be suitable for the purpose in a standard fire resistance test.

4.9.3 *Thickness of insulating material*

4.9.3.1 *Thickness determined by testing*

The minimum thickness of insulating material added to attain the required fire resistance rating shall be determined by testing in accordance with AS 1530:Part 4.

4.9.3.2 *Thickness determination in absence of testing*

A2 In the absence of such testing and only for the materials specified in 4.9.2, the minimum thickness of insulating material to be added may be taken as the difference between the required axis distance or effective thickness specified in this section and the actual axis distance or effective thickness, whichever governs, multiplied by:

(a) 0.75, for materials specified in 4.9.2 (a) and (b); or

(b) An appropriate factor for materials specified in 4.9.2(c), where the factor is derived from tests in which the difference calculated above lies within the range of insulation thicknesses tested.

4.9.4 *Reinforcement in sprayed or trowelled insulating materials*

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

4.10 Fire resistance rating by calculation

The fire resistance rating of a member may be assessed by a recognised method of calculation, such as given in Eurocode 2, using the load combinations given in AS/NZS 1170:Part 0 and the ϕ factor given by 2.3.2.2.

5 DESIGN PROPERTIES OF MATERIALS

5.1 Notation

E_c modulus of elasticity of concrete, MPa
E_s modulus of elasticity for non-prestressed reinforcing steel, MPa
E_{sp} modulus of elasticity of tendons, MPa
f'_c specified compressive strength of concrete, MPa
f_{ct} indirect tensile strength as defined in AS 1012:Part 10, MPa A2
f_{pu} ultimate tensile strength of prestressing steel, MPa
f_{py} yield strength of tendons, MPa
f_r average modulus of rupture used for calculation of deflections, MPa A3
f_t lower characteristic concrete direct tensile strength, MPa
f_y lower characteristic yield strength of longitudinal (main) reinforcement, MPa
f_{yt} lower characteristic yield strength of transverse (stirrup) reinforcement, MPa
λ modification factor reflecting the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength A3
ν Poisson's ratio for concrete
ρ density of concrete, kg/m^3

5.2 Properties of concrete

Unless otherwise specified, the values given in this section shall apply for self-compacting concrete and for conventionally placed concrete.

5.2.1 *Specified compressive strength*

The specified compressive strength of the concrete, f'_c shall be equal to or greater than 20 MPa, and shall not exceed 100 MPa, without special study. A3

For ductile elements and elements of limited ductility, the specified strength of the concrete, f'_c, shall not exceed 70 MPa, without special study.

5.2.2 *Applicable density range*

Concrete meeting the properties described in this section shall have a saturated surface-dry density, ρ, in the range 1800 kg/m^3 to 2800 kg/m^3.

5.2.3 *Modulus of elasticity*

The modulus of elasticity, E_c, for concrete shall be taken from the appropriate value given below: A2

(a) From a value established by testing of plain concrete in environmental conditions representative of those to which the proposed structure will be exposed;

(b) From the expression A3

$$\left[4700\sqrt{f'_c}\right]\left(\frac{\rho}{2300}\right)^{1.5} \text{ MPa} \quad \text{(Eq 5–1)}$$

(c) In analyses in which strain induced actions are critical, the modulus of elasticity shall be equal to or greater than the value calculated using Equation 5–1 but with a concrete strength of (f'_c + 10) MPa substituted for f'_c;

(d) In analyses of members for the serviceability limit state, the modulus of elasticity may be calculated using Equation 5–1 but with a concrete strength of (f'_c + 10) MPa substituted for f'_c.

5.2.4 *Direct tensile strength concrete*

The lower characteristic direct tensile strength of normal concrete is approximately:

$$f_t = 0.38\lambda\sqrt{f'_c} \quad \text{(Eq. 5–2)}$$

$$\text{where } \lambda = 0.4 + \frac{0.6\rho}{2200} \leq 1.0 \quad \text{(Eq. 5–3)}$$

A3

5.2.5 *Modulus of rupture for calculation of deflections*

For the purpose of calculating deflections using 6.8, the modulus of rupture shall be taken as:

$$f_r = 0.6\lambda\sqrt{f'_c} \quad \text{(Eq. 5–4)}$$

Where λ is as defined in 5.2.4.

5.2.6 *VOID*

5.2.7 *Poisson's ratio*

Poisson's ratio for normal density concrete, ν, shall be taken as 0.2 or as determined from suitable test data.

For lightweight concrete, ν, shall be determined from tests.

5.2.8 *Stress-strain curves*

A stress-strain curve for concrete may be either:

(a) Assumed to be of curvilinear form defined by recognised simplified equations; or

(b) Determined from suitable test data.

5.2.9 *Coefficient of thermal expansion*

A3

The coefficient of expansion of concrete for temperatures up to 40 °C shall be taken from one of the following options as:

(a) 12×10^{-6};

(b) From the appropriate value listed in Table 5.1;

(c) Determined from suitable test data for the proposed aggregate type.

Table 5.1 – Design values of coefficient of thermal expansion for concrete

Aggregate	Greywacke	Phonolite	Basalt	Andesite
Coefficient of thermal expansion $\times 10^{-6}$/°C	9.5 - 11.0	10.0 - 11.0	9.0 - 10.0	7.0 - 9.0

5.2.10 *Shrinkage*

A3

Where the effects of shrinkage may adversely affect the performance of a structure, such effects shall be assessed in the design process. Guidance on how this can be accomplished is provided in Appendix E for basic material values and Appendix CE for general comments on material properties and methods of analysis with worked examples.

5.2.11 *Creep*

A3

Where the effects of creep may adversely affect the performance of a structure, such effects shall be assessed in the design process. Guidance on how this can be accomplished is provided in Appendices E and CE, see 5.2.10.

5.3 Properties of reinforcement

5.3.1 *Use of plain and deformed reinforcement*

All reinforcement other than ties, stirrups, spirals, shear studs, seven-wire strand, welded wire fabric and wire, strands and high strength alloy steel bars for prestressing tendons shall be deformed unless there is special reason for using plain bars.

5.3.2 *Reinforcement grades*

5.3.2.1 *Reinforcement to comply with AS/NZS 4671*

Reinforcing bars shall conform to AS/NZS 4671. Grade 500 reinforcement shall be manufactured using either the microalloy process or the in-line quenched and tempered process. However, where the in-line quenched and tempered process, or equivalent, is used the restrictions of 5.3.2.2 shall apply.

5.3.2.2 *Restrictions on in-line quenched and tempered reinforcement*

Reinforcing bars manufactured by the in-line quenched and tempered process shall not be used where welding, hot bending, or threading of bars occurs. A3 A2

5.3.2.3 *Ductility class*

Reinforcing bars shall be ductility Class E. Ductility Classes L and N reinforcing bars shall not be used. A3

5.3.2.4 *VOID*

5.3.2.5 *Welded wire fabric*

Welded wire fabric shall be manufactured to AS/NZS 4671.

5.3.2.6 *Ductile welded wire fabric*

Welded wire fabric shall have a uniform elongation, as defined by AS/NZS 4671, of at least 10 % . A3

5.3.2.7 *VOID* A3

5.3.2.8 *Welding and bending of reinforcing bars*

The provisions of NZS 3109 shall apply to the welding, bending and re-bending of reinforcing bars. The method of manufacture, either microalloyed or quenched and tempered shall be taken into account.

5.3.3 *Strength*

The lower characteristic yield strength of reinforcement, f_y, used in design shall be equal to or less than 500 MPa. A3

Reinforcement with a lower characteristic yield strength other than 300 MPa shall carry permanent identification.

5.3.4 *Modulus of elasticity*

The modulus of elasticity, E_s, for non-prestressed reinforcing steel shall be taken as 200,000 MPa.

5.3.5 *Coefficient of thermal expansion*

The coefficient of thermal expansion for reinforcing steel shall either be taken as 12 × 10^{-6}/°C or determined from suitable test data.

5.4 Properties of tendons

5.4.1 *Strength*

A3 The characteristic tensile strength of tendons (f_{pu}) shall be selected according to AS/NZS 4672.

The yield strength of tendons (f_{py}) may either be taken as the 0.1 % or 0.2 % proof force as specified in AS/NZS 4672 or determined by test data. In the absence of test data it shall be taken as:

A2
(a) For wire used in the as-drawn condition ....$0.75f_{pu}$
(b) For stress-relieved wire.......................................$0.85f_{pu}$
(c) For all grades of strand and bar tendons ....$0.85f_{pu}$

A3

5.4.2 *Modulus of elasticity*

The modulus of elasticity, E_{sp}, of tendons shall be either:

(a) Taken as equal to:
  (i) For stress-relieved wire to AS/NZS 4672....................................200 × 10^3 MPa
  (ii) For stress-relieved steel strand to AS/NZS 4672 ....195 × 10^3 MPa
  (iii) For hot rolled steel bars to AS/NZS 4672...................................200 × 10^3 MPa; or
(b) Determined by test.

Consideration shall be given to the fact that the modulus of elasticity of tendons may vary by ±5 %.

5.4.3 *Stress-strain curves*

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

5.4.4 *Relaxation of tendons*

Relaxation of tendons is covered in 19.3.4.3.4.

5.5 Properties of steel fibre reinforced concrete

The design properties of steel fibre reinforced concrete shall be determined by means of deflection controlled bending tests with the specific fibre to be used, or with this information supplied by the fibre manufacturer. The methods of Appendix A to the Commentary on Section 5 may be used.

6 METHODS OF STRUCTURAL ANALYSIS

6.1 Notation

A3

A'_s area of longitudinal reinforcement in compression zone, mm^2 A2

b width of compression face of member, mm

B absolute value of reduction in bending moment to maximum bending moment in the member, see 6.3.7.2 A3

c neutral axis depth, mm

d distance from extreme compression fibre to centroid to tension reinforcement, mm

d_b diameter of reinforcing bar, mm A3

E_c modulus of elasticity for concrete, MPa

f'_c specified compressive strength of concrete, MPa

f_r modulus of rupture used for assessing deflections, MPa A3

f_y yield stress of reinforcement, MPa

G dead load, N or kPa

h_w height of wall, mm A3

I_{cr} effective second moment of area (moment of inertia) of cracked section about the centroidal axis, mm^4

I_e effective second moment of area (moment of inertia), mm^4

I_g moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement, mm^4

I_{se} effective second moment of area (moment of inertia) of a section, mm^4 A3

K_{cp} factor which allows for deflection due to creep and shrinkage

K_s $= \dfrac{\text{Dead load} + \text{Long term live load}}{\text{Dead load} + \text{Short term live load}}$

L length of member between centre lines of supports, mm A3

L_w length of wall, mm

M_a serviceability bending moment, either maximum bending moment in a member, or bending moment at a section, as appropriate, Nmm A3

M_{cr} cracking moment, N mm

p' proportion of longitudinal reinforcement in compression zone, A'_s/bd A2

Q live load, N or kPa

y_t distance from neutral axis to extreme tension fibre, mm A3

α_w coefficient in C6.9.1.3

ε_u tensile strain in longitudinal tensile reinforcement at ultimate limit state when the compression strain is 0.003

A3 ε_y yield strain of reinforcement

δ_h lateral deflection of squat wall due to development of reinforcement in a foundation beam (6.9.1.3), mm

6.2 General

6.2.1 *Basis for structural analysis*

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

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

6.2.2 *Interpretation of the results of analysis*

Irrespective of the method chosen for the structural analysis, the simplifications, idealisations 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.

6.2.3 *Methods of analysis*

6.2.3.1 *Permissible methods*

For the purpose of complying with the requirements for stability, strength and serviceability 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) Linear elastic analysis, in accordance with 6.3;
(b) Non-linear structural analysis, in accordance with 6.4;
(c) Plastic methods of analysis for slabs and frames, in accordance with 6.5;
(d) Strut and tie method of analysis, in accordance with 6.6;
(e) Structural model tests designed and evaluated in accordance with the principles of mechanics;
(f) The following simplified methods of analysis:
    (i) For reinforced continuous beams or one-way slabs, the simplified method given in 6.7.2;
    (ii) For reinforced two-way slabs supported by walls or beams on all four sides, the simplified method given in 6.7.3;
    (iii) For reinforced two-way slab systems having multiple spans, the simplified method given in 6.7.4;
(g) For non-flexural members, the methods given in 6.6 and Section 16.

6.2.3.2 *Frames or continuous construction*

All members of frames or continuous construction shall be designed for the maximum effects of loads in the serviceability and ultimate limit states. Moments obtained from elastic analyses of structures with factored loads for the ultimate limit state may be modified according to 6.3.7. However, the redistribution of moments permitted in 6.3.7.2 or 19.3.9 shall not be applied to the approximate moments of 6.7.

6.2.3.3 *Seismic loading*

For analysis involving seismic loading, refer to 6.9 which shall take precedence over other clauses in this section.

6.2.4 *Vertical loads on continuous beams, frames and floor systems*

In the analysis of continuous beams, two-dimensional frames and floor systems, or three-dimensional framed structures and floor systems, the arrangement of vertical loads to be considered shall consist of at least the following:

(a) The factored dead load, including considerations given in 2.3.2.1.;
(b) Where the live loading pattern is fixed – the factored live load;
(c) Where the live loading pattern (Q) can vary;
    (i) For continuous beams and two-dimensional frames or floor systems:
        (A) The factored live load on alternate spans; and

(B) The factored live load on two adjacent spans; and

(C) The factored live load on all spans.

(ii) For three-dimensional framed structures and floor systems, patterned variations of the factored live load on adjacent spans in a chequerboard arrangement on all spans to determine the peak design actions at each critical section.

6.2.5 *Slabs where critical actions arise from individual wheel loads*

In the design of slabs to carry highway vehicle loads as specified by the New Zealand Transport Agency's Bridge Manual, the empirical method based on membrane action defined in 12.8.2 may be used. A3

6.3 Linear elastic analysis

6.3.1 *Application*

In the design of structures for ultimate and serviceability limit states, linear elastic analysis may be used for the purpose of evaluating internal action effects.

6.3.2 *Span lengths*

For the purposes of calculating moments, shears, deflections or stiffness, the following shall be used:

(a) In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance centre-to-centre of supports;

(b) Solid or ribbed slabs built integrally with supports, with clear spans of not more than 3 m, may be analysed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected;

(c) Span length of members not built integrally with supports shall be considered to be the clear span plus the depth of member but need not exceed the distance between centres of seatings.

6.3.3 *Analysis requirements*

The analysis shall take into account:

(a) The stress-strain curves of the steel reinforcement and tendons;

(b) Static equilibrium of the structure after redistribution of the moments; and

(c) The properties of the concrete as defined in 5.2.

6.3.4 *Critical sections for negative moments*

Circular or regular polygon shaped supports may be treated as square supports with the same area for location of the critical section for negative moment.

6.3.5 *Stiffness*

6.3.5.1 *Stiffness to be appropriate to limit state*

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

6.3.5.2 *Variations in cross section*

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.

6.3.5.3 *Assumptions to be applied appropriately*

Appropriate assumptions regarding the stiffness of members shall be applied in a consistent and appropriate manner for the actions being considered. A3

6.3.5.4 *Effective stiffness*

Refer to 6.8 for effective stiffness to be used when calculating deflections and 6.9 for effective stiffness values appropriate for seismic analysis. A3

6.3.6 *Secondary bending moments and shears resulting from prestress*

Secondary actions due to prestress or other self-strain conditions in prestressed or partially prestressed members are considered in 19.3.8 and 19.3.9. A3

6.3.7 *Moment redistribution in reinforced concrete for ultimate limit state*

6.3.7.1 *General requirements*

6.3.7.1.1 *Redistribution permitted*

A3 In design calculations for strength of statically indeterminate members, the bending moments determined by elastic analysis at any support may be reduced or increased by redistribution, provided:

(a) Equilibrium is not violated;

(b) An analysis is undertaken to show that there is adequate inelastic rotational capacity in the critical regions, θ_{cap}, to meet the inelastic rotation demand, θ_{dem}, associated with the proposed moment redistribution.

The basis on which θ_{cap} and θ_{dem} shall be calculated are set out in (c) and (d) below:

(c) The inelastic rotation capacity is given by:

$$\theta_{cap} = \left(\frac{\varepsilon_u - k\varepsilon_y}{d - c} \right) \frac{d}{2} \quad \text{(Eq. 6–1)}$$

Where k = 1.733 for Grade 500 reinforcement and 2.0 for Grade 300, and ε_u is the strain in the longitudinal reinforcement closest to the tension side of the member when the compression strain in the concrete is 0.003. The term in brackets is the inelastic curvature and d/2 is the effective plastic hinge length for nominally ductile members.

(d) The rotational demand, θ_{dem}, is calculated by standard flexural theory from the increment of redistributed bending moments in the span (elastic bending moments minus the bending moments used to determine ultimate strength) divided by the strength reduction factor, 0.85. The inelastic rotation is found using the moment of inertia (second moment of area) for the member neglecting the contribution of concrete in tension, I_{cr}. Where I_{cr} varies for regions of the beam subjected to positive flexure from the corresponding values for negative flexure, when subjected to the design moments, either the smaller value of I_{cr} shall be used or the I_{cr} values may be used for their respective areas with the increment of redistributed bending moment.

6.3.7.1.2 *Redistribution due to creep and foundation movement*

Consideration shall be given to the significant redistribution of internal actions that may occur due to relative foundation movements and the considerable demands this can place on the rotational capacity of the critical sections. Where staged construction of members or structures is used, creep redistribution of structural actions shall also be considered.

A3 6.3.7.2 *Deemed-to-comply approach for reinforced concrete beams and one-way slabs for non-seismic cases and seismic load cases in nominally ductile plastic regions*

For non-seismic and seismic load cases in nominally ductile plastic regions, the requirements of 6.3.7.1.1 shall be deemed to be met provided all of the following requirements are satisfied:

(a) The member has a uniform depth and width along the span and the reinforcement has not been detailed to displace the location of potential plastic regions;

(b) The elastic bending moment distribution before redistribution is determined in accordance with 6.3.5;

(c) Equilibrium between the internal forces and the external loads must be maintained under each appropriate combination of factored vertical and horizontal loads and forces;

(d) The design strength after redistribution provided at any section of a member shall be equal to or greater than 70 % of the moment for that section obtained from a moment envelope covering all appropriate combinations of loads obtained from the analysis of the elastic structure;

(e) The moment at any section in a member obtained from the analysis of the elastic structure due to a particular combination of factored loads and forces shall not be reduced by more than 30 % of the numerically largest moment given anywhere by the elastic moment envelope for that particular member, covering all combinations of ultimate limit state loads and forces;

(f) Moment redistribution equal to the maximum value in (e) may be applied where the c/d ratio at the critical section of a plastic region associated with moment redistribution and its associated L/d ratio are less than the values in Table 6.1. Where redistribution less than 30 % at the critical section is applied the L/d ratio may be multiplied by $0.3/B$. B is the absolute value of the reduction in resistance to the largest ultimate limit state moment, determined by elastic analysis anywhere in the member; A3

(g) For any nominally ductile structure that contains limited ductile or ductile plastic regions the deformation limits in these regions shall comply with 2.6.

Interpolation may be used for intermediate values in Table 6.1.

The consequences of moment redistribution in the ultimate limit state shall be assessed for the serviceability limit state.

Table 6.1 – Ratios of neutral axis depth and beam or slab span to effective depth for 30 % moment redistribution

c/d at critical section	Span divided by critical depth, L/d	
	f_y = 500 MPa	f_y = 300 MPa
0.075	36	60
0.10	26	40
0.125	17	25
0.15	13	19
0.175	9	16
0.20	6	12
0.25	NA	9

6.3.8 *Idealised frame method of analysis*

The idealised frame method may be used to analyse structures of reinforced concrete and prestressed concrete that can be represented as a framework of line members with a regular layout. This method may also be applied to the analysis of framed structures with a regular layout incorporating two-way slab systems. Refer to the commentary for further details on this method.

6.4 Non-linear structural analysis

6.4.1 *General*

When used to evaluate the conditions in a structure in the serviceability limit state or at the ultimate limit state, non-linear analysis shall be carried out in accordance with the requirements of 6.2.1 to 6.2.3 and 6.4.2 to 6.4.4.

6.4.2 *Non-linear material effects*

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

(a) Non-linear relation between stress and strain for the reinforcement, the tendon 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 prestressing tendons.

6.4.3 *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.

6.4.4 *Values of material properties*

Non-linear analysis shall be undertaken using material stress-strain relationships based on either mean material strengths or design strengths of the material. However, in all cases the ultimate limit state section strengths shall be based on design strengths.

6.5 Plastic methods of analysis

6.5.1 *General*

Where plastic methods are used in determining the ultimate limit state of structures, the reinforcement shall be arranged with due regard to the serviceability requirements of the structure.

6.5.2 *Methods for beams and frames*

Plastic methods of analysis may be used for determining the ultimate limit state of continuous beams and frames provided it is shown that the high-moment regions possess sufficient moment-rotation capacity to achieve the plastic redistribution implied in the analysis.

6.5.3 *Methods for slabs*

6.5.3.1 *Use of plastic methods of analysis*

For the ultimate limit state of one-way and two-way slabs, plastic methods of analysis based on lower bound or upper bound theory may be used provided ductility Class E reinforcement is used throughout.

6.5.3.2 *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.

6.5.3.3 *Upper bound method for slabs*

Upper bound or yield line analysis for ultimate limit strength 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.

6.6 Analysis using strut-and-tie models

The analysis of squat elements and regions of discontinuity, where flexural theory relevant to members is not appropriate, may be based on strut-and-tie models. Requirements of equilibrium and strain compatibility shall be satisfied. Appendix A provides strut and tie methodologies.

6.7 Simplified methods of flexural analysis

6.7.1 *General*

In lieu of more detailed structural analysis, it is permissible to design ductile reinforced concrete beams and slabs for strength in accordance with the provisions of 6.7.2, 6.7.3, or 6.7.4 as appropriate. The simplified methods in this clause apply only to reinforced concrete beams or slabs containing ductility Class E steel as the principal longitudinal reinforcement.

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

6.7.2.1 *Application*

6.7.2.1.1 *Conditions for use of simplified methods*

Simplified methods 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 that:

(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 live load (Q) does not exceed twice the dead load (G);

(d) Members are of uniform cross section;

(e) The reinforcement is arranged in accordance with the requirements of Section 9;

(f) Bending moments at supports are caused only by the action of loads applied to the beam or slab; and

(g) There is no redistribution of bending moments.

6.7.2.1.2 *Design information*

Refer to the commentary for the simplified method to determine:

(a) Negative design moment;

(b) Positive design moment;

(c) Design shear force.

6.7.3 *Simplified method for reinforced two-way slabs supported on four sides*

6.7.3.1 *Application*

6.7.3.1.1 *Determination of bending moments*

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, may be determined by the simplified method provided that:

(a) The loads are essentially uniformly distributed;

(b) The reinforcement is arranged in accordance with the requirements of Section 12; and

(c) Bending moments at supports are caused only by the action of loads applied to the beam or slab.

6.7.3.1.2 *Referral to the commentary*

Refer to the commentary for the simplified method to determine:

(a) Design bending moments;

(b) Torsional moment at exterior corners;

(c) Load allocation onto supporting walls or beams.

6.7.4 *Simplified method for reinforced two-way slab systems having multiple spans*

6.7.4.1 *Conditions for the use of simplified method*

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 that the following requirements are met:

(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 centrelines 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 live load (Q) does not exceed twice the dead load (G);

(h) The reinforcement is arranged in accordance with Section 12.

6.7.4.2 *Referral to the commentary*

See commentary for the simplified method to determine:

(a) Total static moment for a span;

(b) Design moments;

(c) Transverse distribution of the design bending moment;

(d) Moment transfer for shear in flat slabs;

(e) Shear forces in beam and slab construction;

(f) Openings in slabs.

A3

6.8 Calculation of deflection of beams and slabs for serviceability limit state

6.8.1 *General*

Deflection calculations shall take into account the effects of cracking, tension stiffening, shrinkage and creep in concrete, and relaxation of prestressed reinforcement. Where appropriate, consideration shall be given to deformations that may result due to deflection of the formwork or settlement of the supporting props during construction.

Calculations shall be made to ensure that under the serviceability limit state conditions the deformations are such that they do not adversely affect the serviceability of the structure. Deflection shall be calculated as described in 6.8.2 or 6.8.3.

6.8.2 *Deflection calculation with a rational model*

A3 Rational methods of calculation may be used to determine deflections. Such methods shall make rational allowance for cracking in the concrete, the length of time the loading acts, the basic properties of concrete including its elastic, creep and shrinkage characteristics, the influence of the maturity of the concrete when the load is applied, the duration of the curing period, and the properties of the reinforcement.

6.8.3 *Calculation of deflection by empirical model*

A3 The short-term deflections of members that are not subjected to significant axial load are found as set out in (a), and these values are increased to allow for additional deformation due to creep and shrinkage in the concrete as set out in (b).

(a) *Short-term deflection*

Two alternative methods are given for calculating the short-term deflection of reinforced concrete beams and slabs. In the first of these, in (i) and (ii), an equivalent uniform second moment of area, I_e, is assessed for a simply supported member, or for continuous members the negative and positive moment regions separately, and these values are used to calculate the deflections by standard theory. In the second, in (iii), the second moment of areas are calculated at regular sections along the member and these values are used to find curvatures and hence the deflected shape.

(i) The effective second moment of area of a equivalent uniform beam that is subjected to only positive moments, or to only negative moments is given by:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \quad \text{(Eq. 6–2)}$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \quad \text{(Eq. 6–2a)}$$

where f_r is given by 5.2.5 and M_a is maximum moment in the beam or the maximum values in the positive and negative regions as appropriate.

(ii) For members subjected to both positive and negative moments along the length of the span I_e may be taken as the average of the I_e values found for the critical positive and negative moments, or alternatively, where there is a marked difference in the I_e values the positive I_e value may be used with positive moments and the I_e value for negative moments used with the region subjected to negative flexure.

(iii) For non-uniform members where, due to haunches, significant changes in reinforcement content, or the variation in stiffness along the member is important, deflection calculations shall be based on second moments of area calculated at regular intervals along the member from:

$$I_{se} = \left(\frac{M_{cr}}{M_a}\right)^4 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^4\right] I_{cr} \le I_g \quad \text{(Eq. 6–3)}$$

(b) *Calculation of long-term deflection*

Unless values are obtained by a more comprehensive analysis, the additional long-term deflection for flexural members (normal density and lightweight concrete) shall be obtained by multiplying the short-term deflection by K_s and by K_{cp}. K_s is the ratio of the maximum bending moment due to dead and long-term loading divided by the corresponding moment due to dead and short-term live loading. The value of K_{cp}, which allows for the additional deflection arising from creep and shrinkage in concrete, is given by:

$$K_{cp} = \frac{2}{1+50p'} \qquad \text{(Eq. 6–4)}$$

where p' is the proportion of longitudinal reinforcement in the compression zone. The rate at which this deflection may be expected to develop can be assessed from the creep-time curve for the appropriate concrete.

The resultant deflection is the sum of the short-term and long-term values.

6.8.4 *Calculation of deflection – prestressed concrete*

Where the deflection of prestressed members is determined:

(a) Where the member is designed not to form flexural cracks in the serviceability limit state, (tensile stress less than the limiting value for class T members in 19.3.3.5.1), the short-term deflection shall be determined by recognised elastic theory, using either section properties based on gross or transformed section properties;

(b) Where the section is designed to crack in flexure in the serviceability limit state rational allowance shall be made for the reduction in stiffness of the member, or zone where cracking is anticipated. In the zone where flexural cracking is predicted the section stiffness values may be based on either ignoring tension stiffening of concrete between the cracks or making a rational allowance for this stiffening in the bending moment range between decompression of the extreme tension fibre and the maximum bending moment sustained by the section; A3

(c) Additional long-term deflection for prestressed concrete members shall be calculated taking into account the stresses in the concrete and reinforcement under sustained load including the effects of creep and shrinkage of the concrete and relaxation of the prestressed reinforcement. (See Appendix CE for guidance.)

6.8.5 *Shored composite construction*

6.8.5.1 *Deflection after the removal of supports*

If composite flexural members are supported during construction so that, after removal of temporary supports, the dead load is resisted by the full composite section, the composite member may be considered equivalent to a monolithically cast member for calculation of deflection. Account shall be taken of the curvatures resulting from differential shrinkage of precast and cast-in-place components, and of the axial creep effects in a prestressed concrete member.

6.8.5.2 *Deflection of non-prestressed composite members*

The long-term deflection of the precast member shall be investigated including the magnitude and duration of load prior to the beginning of effective composite action.

6.9 Additional requirements for earthquake effects

6.9.1 *Linear elastic analysis*

6.9.1.1 *Analyses to be based on anticipated levels of cracking*

Elastic analyses of seismic response, which are used to assess inter-storey drifts, periods of vibration and internal actions, shall make allowances for the anticipated levels of concrete cracking and tension stiffening of concrete between cracks. Where analysis indicates tensile stresses due to flexure and axial A3

A3 load are less than $0.55\sqrt{1.2f'_c}$ in the ultimate limit state, flexural cracking is unlikely to occur and the section properties shall be based on gross or uncracked transformed sections properties.

6.9.1.2 *Ultimate limit state deflections to allow for post-elastic effects*

A3 Assessment of structural displacements and inter-storey drifts for the ultimate limit state involving seismic actions shall make due allowance for the elastic and inelastic deformation as specified in NZS 1170.5, or other appropriate referenced loading standard.

In calculation of deformation, allowance shall be made for elongation of potential plastic hinges where required by 2.6.5.10.

A3 **6.9.1.3** *Walls, coupling beams and other deep members*

In the estimation of stiffness or deformations of structural walls and other deep members, consideration shall be given to deformation associated with:

(a) Diagonal cracking due to shear, where the analysis indicates extensive diagonal cracking will occur;

(b) The deformation of foundations; and

(c) The development of reinforcement in foundations, or other structural elements.

A3 **6.9.1.4** *Dual structures*

Wherever a combination of different ductile structural systems is used, rational analysis, taking into account the relative stiffness and location of such elements, shall be employed to allocate the seismic resistance to each element at the serviceability and ultimate limit states. Diaphragms, shall be designed to ensure the design forces can be transmitted between lateral force-resisting elements in accordance with 13.4.

6.9.1.5 *Redistribution of moments and shear forces*

In ductile or limited ductile structures redistribution of moments or shear forces, derived from an elastic analysis for factored gravity loads and seismic forces at the ultimate limit state, may be made, provided:

A3 (a) The positive span moments for all design load combinations shall be modified in beams when terminal negative or positive moments are changed, to satisfy the requirements of equilibrium;

(b) Moment redistribution shall not be used where terminal beam negative moments for any load combination are based on nominal values;

(c) The requirements of 6.3.7 shall be satisfied when the strength of the structure at the ultimate limit state is governed by gravity loads and wind forces only;

(d) The lateral storey shear force or the torsional resistance in any storey shall not be reduced by the moment redistribution process;

(e) The material strain limits shall be equal to or less than the limits given in 2.6;

(f) The redistribution of design actions for the ultimate limit state shall not violate the requirements of the structure under the serviceability limit state.

6.9.1.6 *Capacity design for columns*

Refer to Appendix D for a recommended capacity design procedure for columns in ductile multi-storey frames subject to earthquake forces.

7 FLEXURE, SHEAR, TORSION AND ELONGATION OF MEMBERS A3

7.1 Notation

a depth of equivalent rectangular stress block as defined in 7.4.2.7, mm

A_g gross sectional area of member, mm^2 A3

A_{ℓ} total area of longitudinal reinforcement to resist torsion, mm^2

A_o area enclosed by line connecting the centres of longitudinal bars in the corners of closed stirrups (used to design torsional reinforcement), or for box girder type sections the area enclosed by the perimeter of the centre-line of transverse reinforcement that is within the width t_o resisting the torsional shear flow, mm^2 A3

A_{co} area enclosed by perimeter of section, mm^2

A_{cv} the effective shear area, mm^2

A2

A_t area of one leg of a closed stirrup resisting torsion within a distance s, mm^2

A_{vd} area of diagonal tension reinforcement crossing the shear plane, mm^2 A3

A_{vf} area of fully developed shear friction reinforcement normal to the shear plane, mm^2

c distance from extreme compression fibre to neutral axis, mm

d distance from extreme compression fibre to centroid of tension reinforcement, mm

d' distance from extreme compression fibre to centroid of compression reinforcement, mm A3

E_s modulus of elasticity of steel, MPa. see 5.3.4

f'_c specified compressive strength of concrete, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

f_{yt} design yield strength of transverse reinforcement provided for shear and/or torsion, MPa A2

h depth of structural member, equal to h_b for beam, h_c for column and L_w for wall as appropriate, mm A3

h_b overall beam depth, mm

h_w overall height of the wall

M^* design moment at section at the ultimate limit state, N mm

M_n nominal flexural strength of section, N mm

N^* design axial load at ultimate limit state, N

p_c perimeter of area A_{co}, mm A3

p_o perimeter of area A_o, (design of torsional reinforcement), mm

s centre-to-centre spacing of shear or torsional reinforcement measured in the direction parallel to the longitudinal reinforcement, mm

t_c 0.75 A_{co}/p_c – the equivalent tube thickness of a section prior to torsional cracking, but for a hollow section t_c shall be taken as the smaller of 0.75 A_{co}/p_c or the actual minimum wall thickness, mm A3

t_o 0.75 A_o/p_o – the equivalent tube thickness of a torsionally cracked section in mm but for a hollow section t_o shall be taken as the smaller of 0.75 A_o/p_o or the thinnest actual minimum wall thickness, mm

T_o maximum torsional design action for which torsional reinforcement is not required, N mm

T_n nominal torsional strength of section, N mm

$T_{n,min}$ minimum nominal torsional strength of section, N mm A3

T^* design torsional moment at section at the ultimate limit state, N mm

V_c nominal shear strength provided by concrete, N

V_{fd} increase in sliding shear resistance due to diagonal reinforcement crossing the shear plane, N A3

v_{max} maximum nominal shear stress, MPa

V_n total nominal shear strength of section, N

v_n nominal shear stress, MPa

V_s nominal shear strength provided by the shear reinforcement, N

v_{tn} nominal shear stress due to torsion, MPa

V^* design shear force at section at the ultimate limit state, N

A3 α_f angle of diagonal reinforcement to the shear plane

α_1 factor defined in 7.4.2.7

β_1 factor defined in 7.4.2.7

A3 $\delta_{e\ell}$ elongation at mid-depth of a member, mm

λ a factor for lightweight concrete (see 5.2.4)

μ_f coefficient of friction, see 7.7.4.3

ϕ strength reduction factor, see 2.3.2.2

A3 θ_m total rotation in the plastic region at ultimate limit state, radians

7.2 Scope

A3 The provisions of this section shall apply to the design of members for flexure, shear and torsion with or without axial loads. The section also includes provisions on the magnitudes of elongation that may be generated in plastic regions. Members subjected primarily to flexure and shear shall be designed as beams or slabs. Members subjected primarily to flexure, axial load and shear shall be designed as columns or walls.

7.3 General principles

Flexural and shear strengths may be determined independently without flexure-shear interaction effects. The influence of axial load shall be included in the determination of both the flexural and shear strength.

7.4 Flexural strength of members with shear and with or without axial load

7.4.1 *Flexural strength requirement*

Design of cross sections of members subjected to flexure, shear and with or without axial loads is based on:

$M^* \leq \phi M_n$ .... (Eq. 7–1)

Where M^* is the design bending moment at the section derived from the ultimate limit state loads and forces and M_n is the nominal flexural strength of the section.

7.4.2 *General design assumptions for flexural strength*

7.4.2.1 *Strength calculations at the ultimate limit state*

The design of members for flexure with or without axial loads at the ultimate limit state shall be based on strain compatibility and equilibrium using either:

(a) The assumptions of 7.4.2.2 to 7.4.2.9 when the full cross section is considered to contribute to the strength of the member; or

(b) Complete stress-strain relationships for reinforcing and concrete including the case when after spalling of the concrete only the core of the cross section is considered to contribute to the strength of the member. In the calculations the assumptions of 7.4.2.2 to 7.4.2.9 shall be satisfied except where spalling of the unconfined concrete is assumed to occur the limiting strain in the concrete consistent with the stress strain relationship for the concrete may be used in lieu of the of value 0.003 given in 7.4.2.3.

7.4.2.2 *Strain relationship to geometry*

Strain distribution in reinforcement and concrete shall be assumed to vary linearly through the depth of the member. For deep beams a strut-and-tie model shall be used.

7.4.2.3 *Maximum concrete strain*

The maximum strain at the extreme concrete compression fibre at the development of the nominal flexural strength shall be assumed equal to 0.003.

7.4.2.4 *Reinforcement stress-strain relationship* A3

The stress in reinforcement below the lower characteristic yield strength, f_y, for the grade of reinforcement used shall be taken as E_s times the reinforcement strain. For strains greater than that corresponding to f_y, the stress in reinforcement shall be considered independent of strain and equal to f_y. Alternatively, a stress-strain relationship including the effect of strain hardening supported by material tests may be used.

7.4.2.5 *Concrete tensile strength*

The tensile strength of concrete shall be neglected in flexural strength calculations of reinforced concrete.

7.4.2.6 *Concrete stress-strain relationship*

The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of the nominal flexural strength in substantial agreement with the results of comprehensive tests.

7.4.2.7 *Equivalent rectangular concrete stress distribution*

For the ultimate limit state, the requirements of 7.4.2.6 may be considered satisfied by an equivalent rectangular concrete compressive stress distribution defined by the following: A3

(a) Concrete stress of $\alpha_1 f'_c$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain;

(b) The distance, c, from the fibre of maximum compressive strain to the neutral axis shall be measured in a direction perpendicular to that axis;

(c) The factor α_1 shall be taken as 0.85 for concrete strengths, f'_c, up to and including 55 MPa. For strengths, f'_c, above 55 MPa, α_1 shall be taken as:

$$\alpha_1 = 0.85 - 0.004\,(f'_c - 55) \qquad \text{(Eq. 7–2)}$$

but with a minimum value of 0.75.

(d) Factor β_1 shall be taken as 0.85 for concrete strengths, f'_c, up to and including 30 MPa. For strengths above 30 MPa, β_1 shall be taken as:

$$\beta_1 = 0.85 - 0.008\,(f'_c - 30) \qquad \text{(Eq. 7–3)}$$

but with a minimum value of 0.65.

7.4.2.8 *Balanced conditions*

Balanced strain conditions exist at a cross section of a member when tension reinforcement near the extreme tension fibre of the cross section reaches the strain corresponding to its lower characteristic yield strength, f_y, just as the concrete in compression reaches its assumed ultimate strain of 0.003.

7.4.2.9 *Compression reinforcement*

Compression reinforcement in conjunction with additional tension reinforcement may be used to increase the flexural strength of beams and columns. Where compression reinforcement is required to satisfy strength transverse reinforcement shall be detailed as specified in 9.3.9.6, 10.3.10.5 and 10.3.10.6.

7.5 Shear strength of members

7.5.1 *General*

Design of cross sections of members shall be based on:

$$V^* \le \phi V_n \qquad \text{(Eq. 7–4)}$$

where V^* is the design shear action at the section derived from ultimate limit state and V_n is the nominal shear strength of the section.

The nominal shear strength of a section, V_n, is given by:

$$V_n = v_n A_{cv} \qquad \text{(Eq. 7–5)}$$

where v_n is the nominal shear stress and A_{cv} is the effective shear area, the values for which are defined in the appropriate sections for the type of member being considered.

7.5.2 *Maximum nominal shear stress, v_{max}*

A3 The shear stress obtained by adding the nominal shear stresses due to shear and torsion in beams shall not exceed the smaller of $0.20 f'_c$ or 10 MPa.

For structural walls the maximum nominal shear stress shall be the smaller of $0.16 f'_c$ or 6 MPa.

For columns the maximum shear stress shall be the smaller of $0.20 f'_c$ or 6 MPa.

For shear friction the maximum shear stress is given in 7.7.5 for different structural members.

7.5.3 *Nominal shear strength, V_n*

The total nominal shear strength of the section V_n for all cases except for shear-friction, shall be computed from:

$$V_n = V_c + V_s \qquad \text{(Eq. 7–6)}$$

A3 where V_c is the nominal shear strength provided by the concrete (7.5.4) and V_s is the nominal shear strength provided by shear reinforcement (7.5.5).

The nominal shear strength corresponding to shear-friction is defined in 7.7.

7.5.4 *Nominal shear strength provided by the concrete, V_c*

A3 When determining the nominal shear strength provided by the concrete, V_c:

(a) The effects of axial tension including those due to creep, shrinkage and temperature in restrained members, shall be considered, whenever applicable. The effect of inclined flexural compression in variable depth members shall be considered.

A3 (b) The methods to determine V_c for reinforced concrete beams and one-way slabs, columns and piers, walls, two-way slabs, beam-column joints and prestressed concrete are given in Sections 9, 10, 11, 12, 15 and 19 respectively.

7.5.5 *Nominal shear strength provided by the shear reinforcement*

When the design shear force V^* exceeds the design shear strength provided by the concrete ϕV_c shear reinforcement shall be provided to satisfy Equations 7–4 and 7–6. The design shear strength provided by the shear reinforcement, ϕV_s, shall be computed with V_s for reinforced concrete beams and one-way slabs, columns and piers, walls, two-way slabs, beam-column joint zones and prestressed concrete as given in Sections 9, 10, 11, 12, 15 and 19 respectively.

A3 With non-rectangular stirrups or ties, only the component of the tension force in a stirrup or tie where it crosses a diagonal tension crack that is parallel to the applied shear force shall be counted as resisting shear resistance. For circular hoops and spirals the shear resistance can be determined from Equation 10–18.

7.5.6 *Shear reinforcement details*

Shear reinforcement may consist of:

(a) Stirrups perpendicular to the longitudinal axis of member;
(b) Stirrups making an angle of 45° or more with the longitudinal tension bars;
(c) Vertical or inclined prestressing;
(d) Mechanically anchored bars with end bearing plates having an area at least 10 times the cross-sectional area of the bar;
(e) Longitudinal reinforcement with a bent portion making an angle of 30° or more with the longitudinal tension reinforcement;
(f) Combinations of stirrups and bent longitudinal reinforcement;
(g) Spirals;
(h) Diagonally reinforced members (as in diagonally reinforced coupling beams);
(i) Welded wire mesh, in members not located in potential plastic regions;
(j) Fibres, designed to the requirements of Appendix A of the Commentary to Section 5, in members not located in potential plastic regions.

7.5.7 *Location and anchorage of reinforcement*

7.5.7.1 *Anchoring of stirrups and ties*

Stirrups, ties or wires shall enclose the flexural tension reinforcement and be anchored as close as possible to the extreme compression fibre. Such stirrups and ties shall be anchored around longitudinal reinforcement by at least a 135° stirrup hook. Alternatively stirrups shall be spliced by welding to develop the breaking strength of the bar or anchored by mechanical anchors.

7.5.7.2 *Bent up bars*

Bent up bars, which are used as shear reinforcement, shall extend from the point where the bend starts in the bar in both the tension and compression zones for a distance equal to or greater than the development length.

7.5.7.3 *Lapped splices* A2

Lapped splices in stirrups and ties shall not be used unless the requirements of 8.7.2.8 are satisfied.

7.5.8 *Design yield strength of shear reinforcement*

Design yield strength of transverse reinforcement for torsion and or shear, f_{yt}, shall not exceed 500 MPa. A2

7.5.9 *Alternative methods for determining shear strength*

7.5.9.1 *Equilibrium and strain compatibility methods* A2

In lieu of the methods specified in 7.5.4 and 7.5.5 the resistance of a member in shear, or shear combined with torsion, may be determined by satisfying the applicable conditions of equilibrium and compatibility of strains and by using appropriate stress-strain relationships for reinforcement and for diagonally cracked concrete.

7.5.9.2 *Strut and tie*

Strut and tie models may be used to design for shear and/or torsion. Where this approach is used V_c shall be taken as zero. A2

7.5.10 *Minimum area of shear reinforcement*

A minimum area of shear reinforcement is required in most members. These minimum areas are specified in 9.3.9.4.13 for beams and one-way slabs, in 10.3.10.4.4 for columns, in 11.3.11.3.8(b) for walls, in 15.3.6 for beam-column joints and in 19.3.11.3.4 for prestressed beams and slabs. A3

7.6 Torsional strength of members with flexure and shear with and without axial loads

7.6.1 *Members loaded in torsion*

7.6.1.1 *Exceptions*

The provisions of 7.6 shall not apply to one-way slabs or slabs complying with Section 12. A2

7.6.1.2 *Threshold torsional reinforcement* A3

The maximum torsional design action, T^*, for which torsional reinforcement is not required is T_o, which is given by:

$$T_o = \phi 0.11 A_{co} t_c \lambda \sqrt{f_c'} \sqrt{\left(1 + \frac{N^*}{0.33 A_g \lambda \sqrt{f_c'}}\right)} \quad \text{(Eq. 7–6(a))}$$

7.6.1.3 *Requirement for torsional reinforcement*

If torsion is required in a member to maintain equilibrium in the structure and if the magnitude of the required torsional design action (T^*) exceeds T_o (calculated as per 7.6.1.2), torsion reinforcement designed in accordance with 7.6.4 shall be provided.

A3 **7.6.1.4** *Torsion due to deformation compatibility*

If torsion in a member arises because the member twists to maintain compatibility, the effect of torsion on the member may be neglected provided the requirements of either (a) or (b) given below are satisfied:

(a) The torsional reinforcement is not required if the torsional design action, T^*, calculated from an analysis based on gross section properties, is equal to or less than T_o;

(b) The effect of torsion on a member may be neglected provided the moments and shears in the structure are computed assuming the member has no torsional stiffness, and the following provisions are satisfied:

   (i) In regions of adjoining members where bending moments may be induced by torsional restraint in the member, adequately anchored flexural reinforcement shall be added to control potential flexural cracking in the serviceability limit state

   (ii) Torsional reinforcement shall be provided in the member in accordance with the provisions of 7.6.2.1 and detailed in accordance with 7.6.3.

7.6.1.5 *Sections within d of support*

Sections located less than a distance, *d*, from the face of the support shall be designed for the same torsion as that computed at a distance *d*.

A3 **7.6.1.6** *Torsional strength requirements*

Design of cross sections subject to torsion shall be based on the relationship:

$$T^* \leq \phi T_n \qquad \text{(Eq. 7–7)}$$

where T^* is the torsion at the section derived from the load on the structure at the ultimate limit state and T_n is the nominal torsional strength of the section.

A3 **7.6.1.7** *Torsional shear stress*

The torsional shear stress, v_{tn}, shall be computed from:

$$v_{tn} = \frac{T_n}{2A_o t_o} \qquad \text{(Eq. 7–8)}$$

A3 v_{tn} shall not exceed the limit given in 7.5.2.

7.6.1.8 *Torsion in flanged sections*

Where torsional shear stress and torsional reinforcement is determined for members with flanged sections, the value of A_o and A_{co} shall be based either on the stem of the section only, without flanges, or on the stem with flanges where the width of overhanging flange used shall not exceed three times the thickness of the flange.

A3 **7.6.1.9** *Torsional and flexural shear together*

Where torsional and flexural shear stresses occur together at a section the following condition shall be satisfied:

$$v_n + v_{tn} < v_{max} \qquad \text{(Eq. 7–9)}$$

where v_{max} is given by 7.5.2.

A2 **7.6.2** ***Reinforcement for compatibility torsion***

7.6.2.1 *Minimum reinforcement for compatibility torsion*

A3 Where required by 7.6.1.4(b), closed stirrup and longitudinal reinforcement meeting the requirements of 7.6.3 shall be provided for a minimum nominal torsional moment, T_n, equal to or greater than the smaller of:

A2

(a) T^*/ϕ calculated neglecting the reduction in torsional stiffness due to torsional cracking, or

(b) $T_{n,min,}$ given by:

$$T_{n,\min} = 0.30\, A_{co}\, t_c\, \lambda \sqrt{f_c'} \left(1 + \frac{N^*}{0.33\, A_g\, \lambda \sqrt{f_c'}}\right) \quad \text{(Eq. 7–10)}$$ A3

where, N^* is the design axial action, taken as positive for compression, and A_g is the gross section area.

The cross-sectional area of a leg or legs of closed stirrups on one side of the member, A_t, within a spacing of s, along the member, together with the corresponding area of longitudinal reinforcement, A_ℓ, located around the perimeter, p_o, shall satisfy Equation Eq. 7–11(a): A2

$$\left(\frac{A_t\, f_{yt}}{s}\frac{A_\ell\, f_y}{p_o}\right)^{0.5} \geq \frac{T_n}{2\, A_o} \quad \text{(Eq. 7–11(a))}$$

Satisfying the limits given in Equation 7–11(b):

$$\frac{0.6 A_\ell f_y}{p_o} \leq \frac{A_t\, f_{yt}}{s} \leq \frac{1.7\, A_\ell\, f_y}{p_o} \quad \text{(Eq. 7–11(b))}$$

7.6.2.2 *Contributions to A_t* A2

In calculating the term A_t/s in Equation Eq. 7–11(a), any closed stirrups provided for shear resistance or to satisfy minimum requirements may be included.

7.6.2.3 *Contributions to A_ℓ* A2

In calculating the term A_ℓ / p_o in Equation Eq. 7–11(a), longitudinal reinforcement used to resist flexure may be included provided that such reinforcement is anchored to provide full development.

7.6.3 *Torsional reinforcement details*

7.6.3.1 *Requirements*

Torsional reinforcement shall consist of closed stirrups perpendicular to the axis of the member combined with longitudinal bars.

7.6.3.2 *Maximum stirrup spacing*

Spacing of closed stirrups shall not exceed p_o /8, or 300 mm whichever is smaller.

7.6.3.3 *Maximum longitudinal bar spacing*

Spacing of longitudinal bars, distributed around the perimeter of the stirrups shall not exceed 300 mm centre-to-centre.

7.6.3.4 *Corner bar requirements*

At least one longitudinal bar or prestressed strand having a diameter equal to or greater than either s/16 or 10 mm shall be placed inside each corner of the closed stirrups. These corner bars or presstressing strands shall be anchored to provide full development A2

7.6.3.5 *Termination of torsional reinforcement*

Torsional reinforcement shall be provided at least a distance p_o /2 beyond the point of zero torsion.

7.6.3.6 *Anchoring of stirrups*

The closed stirrups shall be anchored by one of the appropriate methods detailed below: A2

(a) Welding, to give a continuous stirrup in accordance with 8.7.4.1(b);

(b) Anchored round a bar by a standard 135° hook, so that the tail is inside the concrete confined by the stirrup;

(c) Anchored by a corner bar by a standard 90° hook, where a flange protects the concrete on the outside of the hook from spalling.

7.6.3.7 *Torsional reinforcement in flanges*

A3 Where flanged sections are used, in accordance with 7.6.1.8, closed stirrups and longitudinal bars shall be provided also in the overhanging parts of the flanges which have been considered in determining A_o and A_{co}.

A2 **7.6.4** ***Design of reinforcement for torsion required for equilibrium***

7.6.4.1 *Design moment for torsion*

A3 Where the torsional design action, T^*, exceeds T_o, closed stirrups and longitudinal reinforcement shall be designed to resist a nominal torsional moment, T_n, equal to or greater than T^*/ϕ.

A2 **7.6.4.2** *Areas of closed stirrups and longitudinal reinforcement*

The minimum area, A_t, of a leg or legs of closed stirrups on one side of a member within a spacing of s along the member shall be equal to:

$$A_t = \frac{T_n\, s}{2\, A_o\, f_{yt}} \quad \text{(Eq. 7–12(a))}$$

The corresponding minimum area of longitudinal reinforcement, A_ℓ, around the perimeter p_o, shall be equal to:

$$A_\ell = \frac{T_n\, p_o}{2\, A_o\, f_y} \quad \text{(Eq.7–12(b))}$$

The reinforcement areas, A_t and A_ℓ, shall be added to the areas of reinforcement required to resist flexure, shear and axial load, and the detailing of the reinforcement shall comply with 7.6.3. The reinforcement area, A_ℓ, shall be distributed symmetrically round the perimeter, p_o, except where modified by 7.6.4.3.

7.6.4.3 *Longitudinal torsional reinforcement reduction in compression zone*

A3 In the flexural compression zone of a member, which is not subjected to prestress or axial load, the area of longitudinal torsional reinforcement required may be reduced by $\dfrac{M^*}{0.9df_y}$, where M^* is the design moment at the section acting in combination with T^*. Where a member is subjected to axial load, or prestress, any reduction in longitudinal torsional reinforcement in the compression zone may be calculated from first principles.

7.6.5 ***Interaction between flexure and torsion***

The requirements for longitudinal reinforcement to resist flexure and torsion are additive. However, it should be noted that when a plastic hinge forms the longitudinal reinforcement in tension is fully utilised by the bending moment and as a result the torsional resistance decreases to zero over the duration that the reinforcement yields in flexural tension.

7.7 Shear-friction

7.7.1 ***General***

The provisions of 7.7 shall apply when considering the shear transfer across a plane, such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times (a cold joint). The provisions may also be applied where there is an abrupt change in section, such as the junction between a corbel and other structural members. The strut and tie method of design can be used as an alternative to shear friction. A3

In ductile and limited ductile potential plastic regions sliding shear shall be considered at the critical section of the ductile region as detailed in 7.7.11. At other sections in uniform members, the provisions of 7.7 do not need to be satisfied where the design does not contain construction joints. Satisfying the shear friction provisions only ensures that sliding will not occur on the interface being considered. Failure may still occur due to diagonal tension or compression on another plane at a different angle in the same location unless the appropriate design requirements for shear in beams, columns or walls are satisfied.

7.7.2 *Shear-friction design*

Design of cross sections subject to shear transfer as described in 7.7.1 shall be based on Equation 7–4 where V_n is calculated in accordance with the provisions of 7.7.4.

7.7.3 *Design approach*

A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement, A_{vf}, across the shear plane shall be designed using 7.7.4 or any other shear transfer design methods that result in the prediction of strength in substantial agreement with results of comprehensive tests.

7.7.4 *Shear-friction design method*

7.7.4.1 *Shear-friction reinforcement perpendicular to shear plane*

When shear-friction reinforcement is perpendicular to the shear plane, nominal shear strength, V_n, shall be computed by:

$V_n = (A_{vf}f_y + N^*)\mu_f$ .... (Eq. 7–13) A3

where N^* is the design force at the ultimate limit state acting normal to the shear plane and is positive for compression, and μ_f is the coefficient of friction in accordance with 7.7.4.3. A3

7.7.4.2 *Shear-friction reinforcement inclined to shear plane*

When a portion of the diagonal reinforcement, with an area of A_{vd}, is inclined to the shear friction plane such that sliding occurs and tension is induced in the reinforcement, the additional shear resistance, V_{fd}, is given by: A3

$V_{fd} = A_{vd}\, f_y\, (\mu_f \sin \alpha_f + \cos \alpha_f)$ .... (Eq. 7–14)

where α_f is the angle between the diagonal tension reinforcement and the shear plane with the limits that $30° < \alpha_f < 60°$.

The total nominal sliding shear resistance is equal to the sum of the values given by Equations 7–13 and 7–14.

7.7.4.3 *Coefficient of friction*

The coefficient of friction μ_f in Equations 7–13 and 7–14 shall be: A3

(a) Concrete placed monolithically at the interface where a major change in section occurs.................. 1.4λ

(b) Concrete placed against hardened concrete with surface intentionally roughened as specified in 7.7.9.. 1.0λ

(c) Concrete placed against hardened concrete not intentionally roughened...................................... 0.6λ

(d) Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 7.7.10) 0.7λ

where

λ = 1.0 for normal density concrete,
= 0.85 for sand-lightweight concrete, and
= 0.75 for all-lightweight concrete.

Linear interpolation shall be permitted when partial sand replacement is used.

A3 Where the strut and tie method is used as an alternative to the shear friction method, the angle between the struts and the potential failure plane being considered shall be equal to or greater than $\tan^{-1}(1/\mu_f)$, where μ_f is the coefficient of friction given in (a) to (d) above.

A3 7.7.5 *Maximum shear stress for shear friction*

The maximum nominal permissible sliding shear strength shall be equal to or less than the shear area, A_{cv}, times the maximum permissible shear stress given in (a) where the reinforcement is normal to the shear plane (7.7.4.1), or in (b) where a component of the sliding shear force is resisted by diagonal tension reinforcement crossing the shear plane (7.7.4.2):

(a) The maximum shear stress is equal to the smaller of 0.20 f_c' or 6 MPa; or

(b) The maximum shear stress is equal to the smaller of 0.20 f_c' or 6 MPa plus the shear stress, v_{fd}, given by Eq. 7–14(a) where v_{fd} is the shear resistance of diagonal reinforcement given by 7.7.4.2:

$$v_{fd} = \frac{V_{fd}}{A_{cv}} \le 3\ MPa \quad \text{(Eq. 7–14(a))}$$

7.7.6 *Design yield strength of shear-friction reinforcement*

Design yield strength of shear-friction reinforcement shall not exceed 500 MPa.

7.7.7 *Reinforcement for net tension across shear plane*

Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vf}f_y$ when calculating required A_{vf}.

7.7.8 *Shear-friction reinforcement*

Shear-friction reinforcement shall be suitably distributed across the assumed crack and shall be adequately anchored to develop the yield strength on both sides by embedment, hooks, or welding to special devices. All reinforcement within the effective section, resisting flexure and axial load, normal to and crossing the potential sliding plane, may be included in determining A_{vf}.

7.7.9 *Concrete placed against previously hardened concrete*

For the purpose of 7.7, when concrete is placed against previously hardened concrete, the interface for
A3 shear transfer shall be clean and free of laitance. If μ_f is assumed equal to 1.0λ, the interface shall be roughened to a full amplitude of approximately 2 mm.

7.7.10 *Concrete placed against as-rolled structural steel*

When shear is transferred between as-rolled structural steel and concrete headed studs or welded reinforcing bars shall be used. The steel shall be clean and free of paint.

A3 7.7.11 *Shear friction in walls*

7.7.11.1 *Shear friction in potential plastic regions in walls*

The provision for sliding shear in potential plastic regions in walls given in 7.7.11.2 shall be satisfied unless at least one of the following conditions is satisfied:

(a) The aspect ratio h_w/L_w exceeds 1.0;

(b) The wall contains only nominally ductile plastic regions;

(c) The wall has been designed as nominally ductile and the maximum lateral displacement, over the height of the wall, neglecting deformation in the foundation beam and the soils, at the ultimate limit state is equal to or less than 0.003h_w.

A3

7.7.11.2 *Sliding shear in structural walls*

The potential plastic regions of structural walls which do not satisfy the requirements of 7.7.11.1 shall:

(a) Contain diagonal reinforcement across the potential sliding shear plane. This diagonal reinforcement shall be inclined at an angle of between 30° and 60° to the potential plane of sliding. The lateral components of the yielding strength of the diagonal reinforcement shall be equal to or greater than 30 % of the ultimate limit state shear force acting at the critical section. The diagonal reinforcement may be assumed to act in both tension and compression; and

(b) Have a lateral displacement over the height of the wall equal to or less than $0.008h_w$.

7.8 Elongation

7.8.1 *Elongation in reinforced concrete members and interaction of structural elements*

Allowance shall be made for displacements and structural actions induced by elongation from the deformation of plastic regions and the opening of gaps between structural elements. Deformations arising from elongation shall be considered where required by 2.6.5.10. The magnitude of elongation to be considered in design for seismic actions is given in 7.8.2.

7.8.2 *Magnitude of elongation in plastic regions for the ultimate limit state*

The magnitude of the design level of elongation in plastic regions depends upon the total rotation and average axial load level sustained in the design earthquake, and whether the plastic hinge is unidirectional or reversing. The values given are for elongation at the mid-depth of the member. In calculating elongation at any other level in a member, allowance shall be made for the rotation sustained by the member.

The total rotation in a plastic hinge, θ_m, shall be calculated in accordance with NZS 1170.5 or other referenced loading standard, where the total rotation is the yield rotation plus the plastic rotation.

(a) For unidirectional plastic hinges in beams, the elongation is given by:

$$\sigma_{e\ell} \approx \frac{\theta_m}{2}(d-d') \quad \text{(Eq. 7–15(a))}$$

(b) For reversing plastic hinges in beams, where there is no axial load, elongation is given by:

$$\sigma_{e\ell} = 2.6\frac{\theta_m}{2}(d-d') \le 0.036h_b \quad \text{(Eq. 7–15(b))}$$

(c) For columns and beams subjected to axial load, where the axial load ratio, $N^*/A_g f'_c$, is equal to or greater than 0.08, the design elongation shall be calculated from Equation 7–15(a). When the axial load ratio lies between 0 and 0.08 the design elongation shall be interpolated between Equations 7–15(a) and 7–15(b). For columns $(d-d')$ shall be replaced by the distance between the reinforcement closest to the tension and compression faces of the column;

(d) For structural walls, the design elongation is given by:

$$\sigma_{e\ell} = \theta_m(0.5L_w - c) \ \text{with}\ c \le 0.5L_w \quad \text{(Eq. 7–15(c))}$$

where c is equal to the neutral axis depth calculated in the ultimate limit state.

7.8.3 *Magnitude of elongation in plastic regions for the maximum considered earthquake*

The magnitude of elongation in plastic regions shall be calculated in accordance with the following:

(a) For the maximum considered earthquake the elongation shall be taken as 1.5 times the corresponding value determined for the ultimate limit state;

(b) The peak elongation associated with the maximum considered earthquake is the elongation associated with the design ultimate limit state elongation times $1.5/S_p$;

(c) For the elongations of (a) and (b) here, an upper limit of $0.036h$, where h is the overall depth of the member (h_b, h_c, or L_w as appropriate) shall apply.

7.8.4 *Magnitude of elongation in plastic regions for the serviceability limit state*

The effect of elongation need not be considered for the serviceability limit state.

NOTES

8 STRESS DEVELOPMENT, DETAILING AND SPLICING OF REINFORCEMENT AND TENDONS

8.1 Notation

A_b area of an individual bar, mm^2
A_{sp} area of flexural reinforcement provided, mm^2
A_{sr} area of flexural reinforcement required, mm^2
A_t area of a bar formed into a spiral or circular or rectangular hoop reinforcement, mm^2 A3
A_{tr} smaller of area of transverse reinforcement within a spacing s crossing plane of splitting normal to concrete surface containing extreme tension fibres, or total area of transverse reinforcement normal to the layer of bars within a spacing, s, divided by n, mm^2. If longitudinal bars are enclosed within spiral or circular hoop reinforcement, $A_{tr} = A_t$ when $n \le 6$.
A_v area of shear reinforcement within a distance s, mm^2
A_w area of an individual wire to be developed or spliced, mm^2
b_w web width, or diameter of circular section, mm
c_b neutral axis depth corresponding to balanced conditions, mm
c_m the smaller of the concrete cover or the clear distance between bars, mm
d distance from extreme compression fibre to centroid of tension reinforcement, mm
d_b nominal diameter of bar, wire or prestressing strand, or in a bundle, the diameter of a bar of equivalent area, mm
d_i diameter of bend measured to the inside of the bar, mm
f'_c specified compressive strength of concrete, MPa
f_{ps} calculated stress in prestressing steel at design load, MPa
f_s stress in reinforcing bar, MPa
f_{se} effective stress in prestressing steel after losses, MPa
f_y lower characteristic yield strength of non-prestressed reinforcement, MPa
f_{yt} lower characteristic yield strength of transverse reinforcement, MPa
L_b distance from critical section to start of bend, mm
L_d development length, mm
L_{db} basic development length of a straight bar, mm
L_{dh} development length of hooked bars, equal to straight embedment between critical section and point of tangency of hook, plus bend radius, plus one bar diameter, mm. (Refer to Figure 8.1)
L_{ds} splice length of bars in non-contact lap splices in flexural members, mm
M_n nominal flexural strength of section, N mm
n number of bars uniformly spaced around circular sections, or the number of longitudinal bars in the layer through which a potential plane of splitting would pass
s maximum spacing of transverse reinforcement within L_d, or spacing of stirrups or ties or spacing of successive turns of a spiral, all measured centre-to-centre, mm
s_b for a particular bar or group of bars in contact, the centre-to-centre distance or, measured perpendicular to the plane of the bend, to the adjacent bar or group of bars or, for a bar or group of bars adjacent to the face of the member, the cover plus one half of d_b, mm
s_L clear distance between bars of a non-contact lap splice, mm
s_w spacing of wires to be developed or spliced, mm
u_4, u_8 residual elongation after 4 and 8 cycles respectively A3
V^* design shear force at section at the ultimate limit state, N
V_s nominal shear strength provided by the shear reinforcement, N A3
α_1, α_2 parameters used in determining development lengths for standard hooks
α_a, α_b, α_c, α_d, α_e parameters used in determining development lengths for straight reinforcing bars
β_b ratio of area of reinforcement to be cut off to total area of tension reinforcement at the section, including those bars which are to be cut off

8.2 Scope

This section presents general provisions that shall apply to detailing of reinforcement and tendons, including spacing and design of anchorage, development and splices.

Provisions specific to particular elements are presented within the sections specific to those elements.

8.3 Spacing of reinforcement

8.3.1 *Clear distance between parallel bars*

The clear distance between parallel reinforcing bars in a layer shall be equal to or greater than the largest of the nominal diameter of the bars, or 25 mm, except that bars in slabs may be placed in two bar bundles.

8.3.2 *Nominal maximum size of aggregate*

The nominal maximum size of the aggregate shall be equal to or less than three-quarters of the minimum clear spacing between individual reinforcing bars or bundles or pretensioning tendons or post-tensioning ducts.

8.3.3 *Placement of parallel bars in layers*

Where parallel reinforcement is placed in two or more layers in beams, the bars in the upper layers shall be placed directly above those in the bottom layer with the clear distance between layers shall be the larger of the nominal diameter of the bars or 25 mm.

8.3.4 *Bundled bars*

Except in slabs, groups of parallel reinforcing bars bundled in contact and assumed to act as a unit shall only be used when the bundles are within the perimeter stirrups or ties. Bundles shall not contain any more than four bars. Bars larger than 32 mm shall not be bundled in beams or girders. Individual bars in a bundle cut off within the span of flexural members shall terminate at different points with at least 40 bar diameters stagger. Where spacing limitations and minimum clear cover are based on bar size, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

8.3.5 *Spacing of principal reinforcement in walls and slabs*

A3 The requirements for the spacing of reinforcement in walls is given by 11.3.11.3.8, 11.3.11.3.9, 11.3.12.2 and 11.4.5.5. The requirements for the spacing of reinforcement in slabs is given by 9.3.8.3, 12.5.6.3, and 12.8.2.3.

8.3.6 *Spacing of outer bars in bridge decks or abutment walls*

In bridge decks or abutment walls, the maximum spacing between adjacent bars in the outermost layer shall be 300 mm.

8.3.7 *Spacing between longitudinal bars in compression members*

In spirally reinforced and tied compression members, the clear distance between longitudinal bars shall be equal to or greater than $1.5d_b$, or 40 mm.

8.3.8 *Spacing between splices*

The limit on clear distance between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8.3.9 *Spacing between pretensioning reinforcement*

Except for hollow-core floor systems as provided for in 8.3.9, the clear distances between pretensioning reinforcement at each end of the member shall be equal to or greater than 4 d_b for individual wires or $3d_b$ for strands. Closer vertical spacing and bundling of strands is permitted in the middle portion of the spans, but the requirements of 8.3.3 shall be satisfied. In hollow-core floor systems the clear distance between prestressing strands shall be equal to or greater than $2d_b$.

8.3.10 *Bundles of ducts for post-tensioned steel*

Ducts for post-tensioning steel may be bundled if it can be shown that the concrete can be satisfactorily placed and provision is made to prevent the steel, when tensioned, from breaking through the duct.

8.4 Bending of reinforcement

8.4.1 *Compliance with NZS 3109*

Bending and re-bending of reinforcing bars shall comply with the provisions of NZS 3109 including its amendments.

8.4.2 *Bending of steel bar reinforcement*

8.4.2.1 *Minimum bend diameter for main bars*

The diameter of bend, measured to the inside of the bar, shall be equal to or greater than the greater of the appropriate value given in Table 8.1 for steel reinforcement manufactured to AS/NZS 4671 or the value given by Equation 8–1, except that Equation 8–1 need not apply in the case where two transverse bars of d_b greater than or equal to the bar being bent are placed in contact with the inside of the bend or where the stress at the start of the bend is less than $f_y/2$. Where transverse bars are required they shall extend for a minimum distance of $3d_b$ beyond the plane of the last bent bar.

Table 8.1 – Minimum diameters of bend

f_y (MPa)	Bar diameter, d_b (mm)	Minimum diameter of bend, d_i (mm)
300 or 500	6 – 20	5 d_b
	24 – 40	6 d_b

The diameter of bend measured to the inside of the bar shall be equal to or greater than:

$$d_i \geq 0.92\left(0.5+\frac{d_b}{s_b}\right)\frac{f_s d_b}{f_c'} \quad \text{(Eq. 8–1)}$$

where f_s is the stress in the bar at the start of the bend. This may be taken as f_y, or a lower value if this justified by a rational analysis which allows for the influence of diagonal cracking on stress.

8.4.2.2 *Minimum bend diameter in fatigue situations*

In members subjected to frequently repetitive loading situations, the minimum diameter of bends in flexural reinforcement shall comply with 2.5.2.2. A2

8.4.2.3 *Stirrup and tie bends*

The inside diameter of bends of stirrups shall be greater than or equal to the diameter of the largest enclosed bar, and greater than or equal to the values given in Table 8.2.

Table 8.2 – Minimum diameters of bends for stirrups and ties

f_y (MPa)	Stirrup or tie diameter d_b (mm)	Minimum diameter of bend, d_i (mm)	
		Plain bars	Deformed bars
300 or 500	6 – 20	2 d_b	4 d_b
	24 – 40	3 d_b	6 d_b

8.4.2.4 *Bends in galvanised deformed bars*

Where deformed bars are galvanised before bending, the minimum bend diameter shall be:

(a) $5d_b$ for bar diameters of 16 mm or less;

(b) $8d_b$ for bar diameters of 20 mm or greater.

8.4.3 *Bending of welded wire fabric*

The inside diameter of bends in welded wire fabric, plain or deformed, shall be equal to or greater than four wire diameters for deformed wire larger than 7 mm and two wire diameters for all other wires. Bends with an inside diameter of less than eight wire diameters shall be equal to or greater than four wire diameters from the nearest welded intersection.

8.5 Welding of reinforcement

8.5.1 *Compliance with AS/NZS 1554:Part 3*

Except as provided herein, all welding shall conform to AS/NZS 1554:Part 3. In the design and execution of welding of reinforcing bar, appropriate account shall be taken of the process of manufacture.

8.5.2 *In-line quenched and tempered steel bars*

Welding, including tack welding, and hot bending of bars that have been manufactured by the in-line quenched and tempered process shall not be permitted.

8.5.3 *Welds in proximity to bends*

Welds in reinforcing bars shall be at least $3d_b$ away from the commencement of bends or that part of a bar which has been bent and re-straightened in accordance with NZS 3109.

8.6 Development of reinforcement

8.6.1 *Development of reinforcement – General*

Calculated tension or compression in reinforcement at each section of a reinforced concrete member shall be developed on each side of that section by embedment length or end anchorage or a combination thereof. Hooks may be used in developing bars in tension.

8.6.2 *Development of shear and torsion reinforcement*

The development of shear and torsion reinforcement shall comply with the relevant requirements of 7.5.7 and 7.6.3 respectively.

8.6.3 *Development length of deformed bars and deformed wire in tension*

8.6.3.1 *Development length in tension*

The development length, L_d, of deformed bars and wire in tension shall be calculated from either 8.6.3.2 or 8.6.3.3, but L_d shall be equal to or greater than 300 mm.

8.6.3.2 *Basic development length in tension*

Unless a more detailed determination of L_d is made in accordance with 8.6.3.3, the development length, L_{db} shall be calculated from:

$$L_{db} = \frac{(0.5\alpha_a f_y)}{\sqrt{f_c'}} d_b \quad \text{(Eq. 8–2)}$$

where α_a = 1.3 for top reinforcement where more than 300 mm of fresh concrete is cast in the member below the bar, or 1.0 for all other cases.

The value of f_c' used in Equation 8–2 shall not exceed 70 MPa.

8.6.3.3 *Refined development length in tension*

The development length, L_d, in tension may be determined from:

$$L_d = \frac{\alpha_b}{\alpha_c \alpha_d} L_{db} \geq 300\,\text{mm} \quad \text{(Eq. 8–3)}$$

with α_b, α_c and α_d being defined as follows:

(a) Reinforcement provided in a flexural member (not subjected to seismic forces nor required for temperature or shrinkage in restrained members) in excess of that required:

$$\alpha_b = A_{sr}/A_{sp} \qquad \text{(Eq. 8–4)}$$

(b) When cover to bars in excess of $1.5d_b$ or clear distance between adjacent bars in excess of $1.5\ d_b$ is provided:

$$\alpha_c = 1 + 0.5\left(\frac{c_m}{d_b} - 1.5\right) \qquad \text{(Eq. 8–5)}$$

with the limitation of $1.0 \le \alpha_c \le 1.5$

where c_m = the lesser of the concrete cover or the clear distance between bars.

(c) When transverse reinforcement with at least 3 bars, spaced less than $8d_b$, transverse to the bar being developed, and outside it, are provided within L_d:

$$\alpha_d = 1 + \sqrt{\left(\frac{A_{tr}}{s}\right)\left(\frac{f_{yt}}{80\,nd_b}\right)} \qquad \text{(Eq. 8–6)}$$

with the limitation of $1.0 \le \alpha_d \le 1.5$. nd_b refers to the terminating reinforcement. A3

Transverse reinforcement used for shear, flexure or temperature may be included in A_{tr}.

8.6.4 *Development length of plain bars and plain wire in tension*

The development of plain bars and wire in tension shall rely on hooks. The development length shall be twice the value for L_{dh} calculated from Equation 8–12.

8.6.5 *Development length of deformed bars and deformed wire in compression*

8.6.5.1 *Development length in compression*

Development length L_d of deformed bars in compression shall be computed from either 8.6.5.2 or 8.6.5.3, but L_d must be greater than 200 mm.

8.6.5.2 *Basic development length in compression*

Unless a more detailed determination of L_d is made in accordance with 8.6.5.3 the development length in compression, L_{db}, shall be calculated from:

$$L_{db} = \frac{0.22 f_y}{\sqrt{f'_c}} d_b \qquad \text{(Eq. 8–7)}$$

with limitations of

$$L_{db} \ge 0.040 f_y d_b \ge 200 \text{ mm} \qquad \text{(Eq. 8–8)}$$

The value of f'_c used in Equation 8–7 shall not exceed 70 MPa.

8.6.5.3 *Refined development length in compression*

The development length in compression, L_d, may be determined from:

$$L_d = \alpha_b \alpha_e L_{db} \qquad \text{(Eq. 8–9)}$$

with α_b as defined in 8.6.3.3(a) and α_e as follows:

When transverse reinforcement with at least three bars, transverse to the bar being developed and outside it, are provided within L_{db}, and $\frac{A_{tr}}{s} \geq \frac{A_b}{600}$

α_e = 0.75, or

= 1.0 for all other cases.

8.6.6 *Development length of plain bars and plain wires in compression*

The development length for plain bars and wires in compression shall be twice the calculated value L_d or L_{db} for a deformed bar or wire.

8.6.7 *Development of bundled bars*

Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased by 20 % for a three-bar bundle, and 33 % for a four-bar bundle.

8.6.8 *Development of welded plain and deformed wire fabric in tension*

8.6.8.1 *Development length of wire fabric*

Development length, L_d, of welded plain and deformed wire fabric measured from the point of critical section to the end of the wire shall be computed from either 8.6.8.2 or 8.6.8.3.

8.6.8.2 *Development length of welded wire fabric – cross wires considered*

The yield strength of plain and deformed wires of welded wire fabric shall be considered developed by embedding at least two cross wires, with the first one equal to or greater than 50 mm from the critical section. However, development length L_d measured from the critical section to the outermost cross wire shall be equal to or greater than 100 mm:

$$L_d \geq \frac{3.25\alpha_b A_w f_y}{s_w \sqrt{f_c'}} \quad \text{(Eq. 8–10)}$$

where α_b is given by 8.6.3.3(a), but L_d shall be equal to or greater than 150 mm for plain wire fabric or greater than 100 mm for deformed wire fabric.

8.6.8.3 *Development length of welded wire fabric – cross wires not considered*

The development length of welded deformed and plain wire fabric, with no cross wires or when the cross wires within the development length as required by 8.6.8.2 are ignored, shall be determined by 8.6.3 or 8.6.4 as appropriate and shall be equal to or greater than 200 mm.

8.6.9 *Development of prestressing strand*

8.6.9.1 *Development length of pretensioning strand*

Three or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length given by:

$$L_d \geq \left(f_{ps} - \frac{2}{3} f_{se}\right)\frac{d_b}{7} \quad \text{(Eq. 8–11)}$$

8.6.9.2 *Development of pre-stressing strand*

Where bonding of a strand does not extend to the end of a member, the bonded development length specified in 8.6.9.1 shall be doubled.

A3 **8.6.9.3** *Prestressing strand transfer length*

The transfer length of the strand, i.e. the distance over which the strand must be bonded to the concrete to develop the prestress f_{se} in the strand, shall be taken as $f_{se}d_b/21$ provided the concrete strength at the time of transfer is equal to or greater than 21 MPa. For shear strength calculations in the development length see 19.3.11.2.3.

8.6.10 *Standard hooks*

8.6.10.1 *Standard hooks – definition*

The term "standard hook" as used herein shall mean either:

(a) A semi-circular turn plus an extension of at least four bar diameters but equal to or greater than 65 mm at the free end of the bar; or

(b) A 90° turn plus an extension of at least 12 bar diameters at the free end of the bar for a deformed bar and 16 bar diameters for a plain bar; or A2

(c) A stirrup hook, which is defined as a 135° turn around a longitudinal bar plus an extension of at least eight stirrup bar diameters for plain bars and six stirrup bar diameters for deformed bars at the free end of the bar embedded in the core concrete of the member.

The standard hooks defined in this clause are illustrated in Figure 8.1.

A2

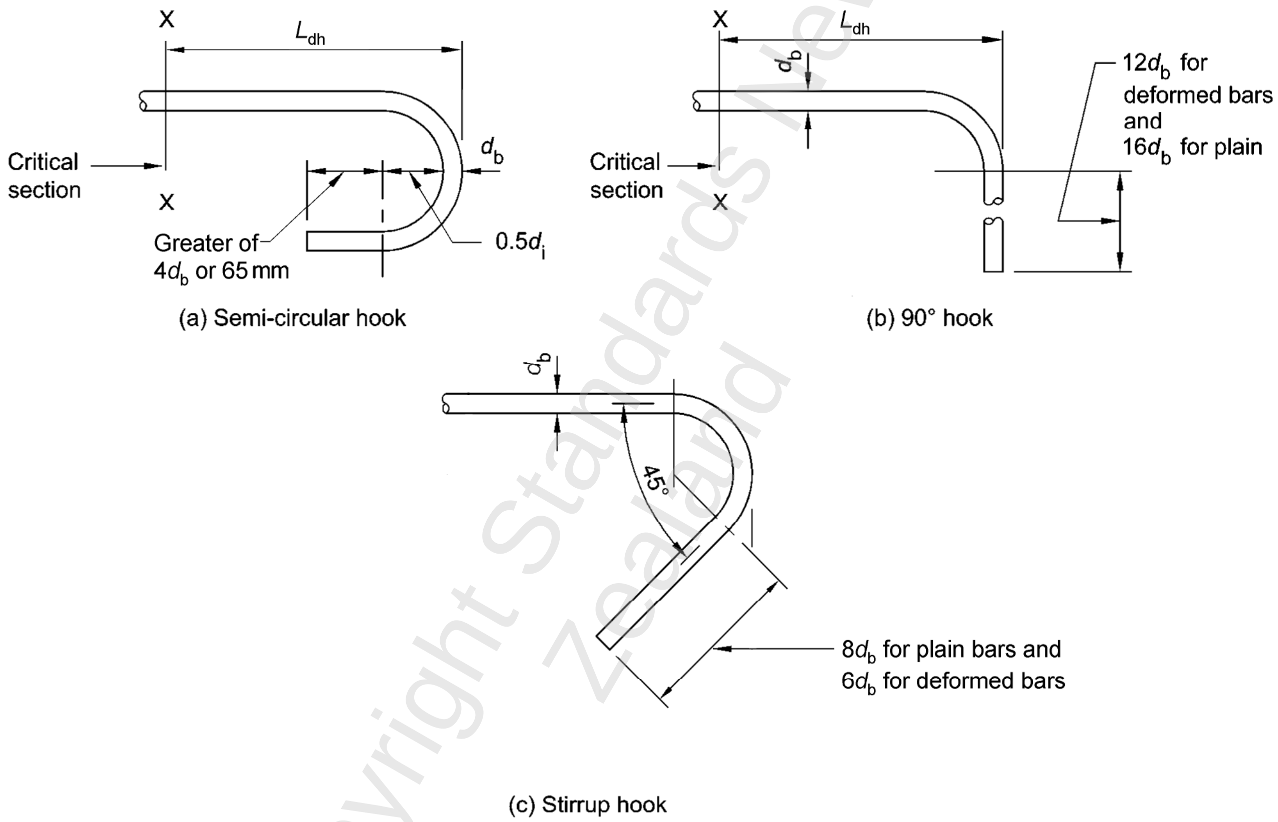


Figure 8.1 – Standard hooks

8.6.10.2 *Bars > 32 mm in diameter*

Bars with diameter greater than 32 mm shall not be developed in tension by the use of standard hooks.

8.6.10.3 *Development length of standard hooks in tension*

8.6.10.3.1 *Calculation of development length for hooked bars*

For the following two situations described in (a) and (b), the development length, L_{dh}, for hooks in tension shall be determined from Equation 8–12:

(a) Where the bar is anchored by a standard hook inside a volume of concrete confined by closed ties, spirals or stirrups perpendicular to the plane of the hook;

(b) Where the bar, is anchored by a standard hook inside a volume of concrete that is not confined by reinforcement perpendicular to the plane of the hook, but

   (i) The spacing between that bar and any adjacent bar or fixing loaded in a similar direction is greater than or equal to three times d_b over the development length (L_{dh}) for that bar; and A3

   (ii) The distance normal to the axis of the bar to the side or edge of the element is greater than or equal to two times d_b over the development length (L_{dh}) for that bar. A3

$$L_{dh} = 0.24\alpha_b\alpha_1\alpha_2 \frac{f_y d_b}{\sqrt{f_c'}} \geq 8d_b \qquad \text{(Eq. 8–12)}$$

where

f_c' shall not be taken greater than 70 MPa

α_b is given by 8.6.3.3 (a)

α_1 = 0.7 for 32 mm bars or smaller with side cover normal to the plane of the hook ≥ 60 mm, and cover on the tail extension of 90° hooks equal to or greater than 40 mm

= 1.0 for all other cases

α_2 = 0.8 where confined by closed stirrups or hoops spaced at $6d_b$ or less and which satisfy the relationship $\frac{A_{tr}}{s} \geq \frac{A_b}{1000}$

= 1.0 for all other cases

8.6.10.3.2 *Determination of development length where not covered by 8.6.10.3.1*

For situations other than as described by 8.6.10.3.1(a) and (b), the development length of a hook shall be determined from a rational analysis or suitable testing that takes into account the effects of the proximity of the anchored bar to edges of elements and to other loaded embedded items.

8.6.10.3.3 *Development length of standard hooks anchoring around transverse bars*

The development length L_{dh} of a deformed bar terminating in a standard hook as determined from 8.6.10.3 may be reduced by 20 %, provided that two transverse bars having a diameter equal to or larger than that of the bent bar are placed in contact with the inside of the bend and extend for a distance equal to or greater than $3d_b$ beyond the centreline of the bent bar.

8.6.10.4 *Hooks in compression*

Hooks shall not be considered effective in developing reinforcement in compression.

8.6.11 *Mechanical anchorage*

8.6.11.1 *General*

A3 For reinforcement complying with AS/NZS 4671, any mechanical device used alone as an anchorage, or used in combination with an embedment length beyond the point of maximum stress in the bar, shall be capable of developing the upper bound breaking strength of the reinforcing bar without damage to the concrete or overall deformation of the anchorage.

In addition, when tested with a bar complying with AS/NZS 4671, the mode of failure of the anchored bar shall be by ductile yielding of the bar, with the bar developing its ultimate tensile strength at a location outside the mechanical anchorage and away from any zone of the bar affected by working (e.g. by cold forging).

A3 **8.6.11.2** *Upper bound breaking strength of the reinforcing bar – definition*

The upper bound breaking strength of the reinforcing bar may be derived from 1.25 times the upper characteristic yield strength specified by AS/NZS 4671, or otherwise shall be determined from an appropriate testing programme.

8.6.11.3 *Adequacy of mechanical devices*

Mechanical anchorage systems relying on interconnecting threads or mechanical interlock with the bar deformations for attachment of the anchorage to the bar shall meet both the permanent extension and fatigue strength criteria of 8.7.5.2. A3 Where the mechanical anchor and ends of the bars are threaded as the means of achieving the connection between components, there shall be no thread stripping or evidence of significant distortion of the threads at the failure load of the bar.

8.6.11.4 *Brittle fracture resistance*

Mechanical anchors for the anchorage of reinforcing steel shall be proven by an appropriate test method to possess resistance to brittle fracture at the service temperatures at which they are intended for use. Where the mechanical anchors and ends of the bars are threaded as the means of achieving the connection between components, and/or the end of the bar is enlarged by cold forging prior to threading, appropriate testing of the processed bar end shall be applied to ensure that the potential for brittle fracture is avoided. Anchors manufactured from cast iron shall not be used. A3

8.6.12 *Development of flexural reinforcement*

8.6.12.1 *Bending across the web*

Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

8.6.12.2 *Critical sections*

Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent.

8.6.12.3 *Extension of tension reinforcement*

Except at supports of simply supported spans and at the free end of cantilevers, tension reinforcement shall extend beyond the point at which, according to the bending moment envelope and standard flexural theory, it is:

(a) Required at maximum stress for a distance equal to the development length, L_d, plus the effective depth of the member, and

(b) No longer required to resist flexure for a distance of 1.3 times the effective depth of the member.

8.6.12.4 *Termination in a tension zone*

Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

(a) Shear at the cut-off point is less than two-thirds of the shear strength provided by the concrete; or

(b) The shear strength provided by the web reinforcement , V_s, measured for a distance of 1.3d along the terminating bar from the cutoff point is equal to or greater than:

$$V_s = 1.2\frac{\sqrt{f_c'}}{16} b_w d \qquad \text{(Eq. 8–13)}$$

and the spacing, s, of stirrups or ties is equal to or less than the smaller of $d/2$ or $\dfrac{d}{8\beta_b}$.

8.6.12.5 *End anchorage in flexural members*

Adequate end anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets, deep flexural members; or members in which tension reinforcement is not parallel to the compression face.

8.6.12.6 *Anchorage of flexural reinforcement in external beam-column joints* A3

Longitudinal reinforcement in a beam terminating at an external beam-column joint shall be anchored by a 90° hook in which the leg of the hook is bent vertically into the joint zone. The distance from where the beam reinforcement terminates shall be:

(a) As close as possible to the vertical column reinforcement on the opposite face of the column from where the beam reinforcement enters the joint zone; and

(b) Equal to or greater than the larger of the development length, L_{dh}, or three quarters of the column depth, 0.75h_c.

Joint zone ties shall satisfy 15.3.6.2(b).

8.6.13 *Development of positive moment reinforcement in tension*

8.6.13.1 *Limitation in area of bars*

At least one-third the maximum positive moment reinforcement in simply supported members and one quarter the maximum positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm unless a lesser distance is demonstrated by test to be adequate and to provide the structural robustness required by AS/NZS 1170.0.

8.6.13.2 *Critical sections*

Where a flexural member is part of a primary horizontal force-resisting system, positive moment reinforcement required to be extended into the support by 8.6.13.1 shall be anchored to develop the lower characteristic yield strength, f_y, in tension at the face of support.

8.6.13.3 *Limitation in diameter of bars at simple supports*

A3 The positive tension reinforcement at simple supports shall be limited in diameter to enable the bars extending to the free end of the member to be fully developed from a point at a distance M_n/V^* from the centre of the support. M_n is the nominal bending moment capacity provided by the reinforcement at the centre of the support, calculated as the area of the positive tension reinforcement at the support multiplied by f_y and by the internal lever arm. V^* is the shear at the face of the support. Where the support induces compression in the anchorage zone of the reinforcement the development length may be reduced by 25 %.

8.6.13.4 *Limitation in diameter of bars at points of inflection*

The positive (and negative) tension reinforcement at points of inflection shall be limited in diameter to enable the bars, from a point at a distance M_n/V^* from the point of inflection, to be fully developed satisfying the requirements that:

$$L_d \le \frac{M_n}{V^*} + 12d_b \quad \text{(Eq. 8–14)}$$

and

$$L_d \le \frac{M_n}{V^*} + d \quad \text{(Eq. 8–15)}$$

A3 For both the positive and the negative tension reinforcement, the value of M_n/V^* shall be calculated at the point of inflection, where M_n equals the area of the positive or negative tension reinforcement at the point of inflection multiplied by f_y and by the internal lever arm.

8.6.14 *Development of negative moment reinforcement in tension*

8.6.14.1 *Anchorage of bars*

Negative moment reinforcement in a continuous, restrained or cantilever member, or in any member of a rigid jointed frame, shall be anchored in or through the supporting member by embedment length, hooks or mechanical anchorage.

8.6.14.2 *Embedment length adjacent to supports*

Negative moment reinforcement shall have an embedment length into the span as required by 8.6.1 and 8.6.12.3.

8.6.14.3 *Embedment length beyond the point of inflection*

At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection, according to the appropriate bending moment envelope, for a distance equal to or greater than 1.3 times the effective depth of the member.

8.6.14.4 *Limitation in diameter of bars*

The requirements of 8.6.13.4 at points of inflection for negative reinforcement shall be satisfied.

8.7 Splices in reinforcement

8.7.1 *General*

Splices in reinforcement shall be shown on the design drawings or specified in the specifications.

8.7.2 *Lap splices of bars and wire in tension*

8.7.2.1 *Bar sizes of lap splices*

Lap splices shall not be used for bars larger than 40 mm in diameter.

8.7.2.2 *Lap splices of bundled bars*

Lap splices of bundled bars shall be based on the lap splice length required for individual bars of the same size as the bars spliced, and such individual splices within the bundle shall not overlap each other. The length of lap, as prescribed in 8.7.2.3 or 8.7.3, shall be increased by 20 % for a three-bar bundle and 33 % for a four-bar bundle.

8.7.2.3 *Length of lap splices of deformed bars or wire*

The minimum length for lap splices of deformed bars and deformed wire in tension shall be equal to or greater than the development length, L_d, in 8.6.3. Plain straight bars or wires shall not be spliced by lapping unless using hooks or other anchorages.

8.7.2.4 *Length of lap splices of hooked plain bars or wire*

The length of lap splices for hooked plain bars or wire with a standard hook shall be equal to or greater than the development length required by 8.6.4. For bars with 50 mm of cover concrete or less, hooks shall be in a plane at a right angle to the adjacent concrete surface. Such splices shall not be used in potential plastic hinge regions of members. A3

8.7.2.5 *Length of non-contact lap splices*

Bars spliced by non-contact lap splices in flexural members spaced transversely farther apart than $3d_b$ shall have splice length, L_{ds}, given by $L_{ds} \geq L_d + 1.5\, s_L$.

8.7.2.6 *Strength developed at sections*

In computing the strength developed at each section, spliced bars shall be rated at the specified splice strength.

8.7.2.7 *Strength of bars where cut off*

Bars cut off near the section under consideration taking the requirements of 8.6.12.3 into account shall be rated only at a fraction of f_y, defined by the ratio of the embedded length past this section to the required development length.

8.7.2.8 *Lap splices of stirrups, ties and hoops*

Stirrups, ties and rectangular hoops in beams, columns, piers, beam-column joints or walls may be spliced A2
by lapping provided that the requirements in either (a) or (b) and (c) are satisfied: A3

(a) Lapping bars shall be terminated with standard hooks in accordance with 8.4.2.1 and 8.6.10.1, and the splice length shall be:
   (i) For plain bars, equal to or greater than the development length required by 8.6.4;
   (ii) For deformed bars, equal to or greater than L_{dh} in 8.6.10.3.

   When the lapped splice is located in cover concrete, the hooks shall be placed in a plane at right angles to the surface of the concrete. When located in a plastic region, the hooks shall be anchored around longitudinal reinforcement of at least equal or greater diameter.

(b) Straight lapped splices with deformed bars may be used where not specifically excluded in (i) to (iii) below:

A3 (i) Straight lapped splices shall not be used in nominally ductile, limited ductile or ductile plastic regions, or in beam-column joints;

A2 (ii) Straight lapped splices shall not be used in cover concrete where the reinforcement is required to provide confinement to the concrete;

(iii) Straight lapped splices shall not be used in cover concrete where the shear stress due to shear and torsion exceeds 0.5 $\sqrt{f_c'}$;

A3 (c) 90° hooked lap splices may be used in beam-column joints. The distance from where each bar enters the beam-column joint to the vertical leg of the hook is equal to or greater than three-quarters of the column depth provided the vertical legs of the hooks are located as close as possible to the vertical reinforcement on the far side of the joint from which each bar entered the joint. The lap distance is equal to or greater than that required for development (8.6.10) and the distance from where each bar enters the beam-column joint is equal to or greater than three-quarters of the column depth;

Where stirrup or tie legs are formed by overlapping deformed reinforcement bars with straight laps, as permitted by (b), the proportion of lapped splices in the cover concrete at any cross section shall be equal to or less than 50 % of the stirrup or tie reinforcement. In this context, cover concrete is defined as the concrete lying on the outside of a line connecting the centres of longitudinal bars located closest to the perimeter of the member.

8.7.3 *Lap splices of bars and wires in compression*

8.7.3.1 *General*

The minimum length of a lap splice in compression shall be the development length in compression L_d, in accordance with 8.6.5 and 8.6.6, but equal to or greater than $0.069 f_y d_b$ for f_y of 430 MPa or less, or $(0.12\ f_y - 22)\ d_b$ for f_y greater than 430 MPa, or 300 mm.

8.7.3.2 *Lap splices in compression with stirrups and ties*

In compression members with stirrups and ties where at least three sets of ties are present over the length of the lap, and

$$\frac{A_{tr}}{s} \geq \frac{A_b}{1000} \qquad \text{(Eq. 8–16)}$$

or where transverse reinforcement as required by either 10.4.7.4.3 or 10.4.7.4.5 has been provided, a lap length of 0.8 times that specified in 8.7.3.1 may be used but the lap length shall be equal to or greater than 300 mm.

8.7.3.3 *Lap splices in compression with spiral reinforcement*

In spirally reinforced compression members, if at least three turns of spiral are present over the length of the lap, and

$$\frac{A_{tr}}{s} \geq \frac{nA_b}{6000} \qquad \text{(Eq. 8–17)}$$

a lap length of 0.8 times that specified in 8.7.3.1 may be used, but the lap length shall be equal to or greater than 300 mm.

8.7.4 *Welded splices*

8.7.4.1 *Classification of welded splices*

Welded splices shall be classified as follows:

(a) A "full strength" welded splice is one in which the bars are butt welded to develop in tension the breaking strength of the bar;

(b) A "high strength" welded splice is one in which the bars are lap welded or butt welded to develop the lower characteristic yield strength of the bar or better.

8.7.4.2 *Limitations on the classification of welded splices for grade > 450 MPa reinforcement*

Butt welded splices in reinforcement with a lower characteristic yield stress of more than 450 MPa shall not be classified as “full strength” unless either:

(a) Yielding of the reinforcement will not occur; or

(b) Proof testing using a portion of the actual bar to be welded and the selected welding procedure, demonstrates that failure of the bar occurs away from the weld.

8.7.4.3 *Exceptions for welded splices*

The requirements 8.7.4.1(b) may be waived when the conditions of 8.7.5.4 are satisfied.

8.7.5 *Mechanical connections*

8.7.5.1 *Definition of mechanical connection*

A mechanical connection is defined as a connection that relies on interlocking threads or mechanical interlock with the bar deformations to develop the connection capacity.

8.7.5.2 *Performance requirements for mechanical connections*

Mechanical connections shall:

(a) satisfy the requirements of 8.6.11 for mechanical anchors; A3

(b) when tested in tension or compression, as appropriate, to the application, exhibit a change in length at a stress of $0.7f_y$ in the bar, measured over the length of the coupler, of less than twice that of an equal length of unspliced bar;

(c) satisfy the requirements of 2.5.2.2 when used in situations where fatigue may develop.

8.7.5.3 *Use of welded splices and mechanical connections*

Welded splices in tension or compression shall meet the requirements of 8.7.4.1 (a) or (b).

Mechanical connections in tension or compression shall meet the requirements of 8.7.5.2.

8.7.5.4 *Use of welded splices and mechanical connections – an exception*

The requirements of 8.7.4.1(b) and 8.7.5.2, as appropriate, may be waived when splices:

(a) Are staggered at least 600 mm; and

(b) Can develop at least twice the calculated force in the bars to be spliced at the section; and

(c) Can develop equal to or greater than 0.7 f_y based on the total area of effective bars across the section; and

(d) Where the level of any resulting premature cracking is not likely to affect the performance of the structure, then the change of length shall be not more than six times that of an equal length of unspliced bar.

8.7.5.5 *Identification and marking*

Each coupler or coupling sleeve shall be legibly and durably marked with the identification of the manufacturer and the nominal bar size for which it is intended. Each coupler or coupling sleeve shall be traceable back to its production data and production batch.

8.7.5.6 *Installation*

The method of installation of mechanical connection systems shall be specified for all conditions that arise on a job site. This may be by reference to manufacturers’ written instructions. Connection systems that rely on a minimum length of engagement between the coupler or coupling sleeve and the bar for the development of the connection strength shall incorporate a system for positively locating the coupler or coupling sleeve and defining when adequate engagement has been achieved.

8.7.6 *Splices of welded plain or deformed wire fabric*

Lap splices shall be detailed by satisfying one of the following conditions:

(a) The overlap measurement between outermost cross wires of each fabric sheet is equal to or greater than the spacing of cross wires plus 50 mm, nor less than 1.5 L_d or 150 mm whichever is greater, where L_d is the development length for f_y as given in 8.6.8.2; or

(b) When cross wires are ignored or no cross wires are present within the lapped length and the lap is a contact or near contact lap splice, the splice length shall be equal to or greater than L_d, where L_d is the development length given by 8.6.8.3.

8.8 Shrinkage and temperature reinforcement

8.8.1 *Floor and roof slab reinforcement*

Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in structural floor and roof slabs where the principal reinforcement extends in one direction only. At all sections where it is required, such reinforcement shall be developed for its lower characteristic yield strength in conformance with 8.6.1 or 8.7.2. Such reinforcement shall provide at least the ratio of reinforcement area to gross concrete area of $0.7/f_y$, but equal to or greater than 0.0014.

8.8.2 *Large members*

In a large member whose size is not governed by stress considerations, or where exact analysis is impractical, minimum reinforcement on all surfaces should be the greater of 1000 mm^2 per metre width in each direction, with bars not further apart than 300 mm, or, where appropriate, as required by 2.4.4.8.

8.9 Additional design requirements for structures designed for earthquake effects

8.9.1 *Splices in reinforcement*

8.9.1.1 *Placement of splices*

Full strength welded splices meeting the requirements of 8.7.4.1(a) may be used in any location. For all other splices the following restrictions apply:

A3 (a) With the exception noted in (b) below, no portion of any splice shall be located within the beam-column joint region, or within one effective depth of the member from the critical section of a limited ductile or ductile plastic region in a beam where stress reversals in spliced bars could occur;

(b) Hooked lap splices which comply with 8.7.2.8 may be used in beam-column joints;

(c) In a column framing top and bottom into beams or other moment-resisting elements, the centre of the splice must be within the middle quarter of the storey height of the column unless it can be shown that a high level of protection is provided against the formation of plastic regions, as defined in Appendix D.

8.9.1.2 *Lap splices in region of reversing stresses*

Reinforcement in beams and columns shall not be spliced by lapping in a region where reversing stresses at the ultimate limit state may exceed $0.6f_y$ in tension or compression unless each spliced bar is confined by stirrup-ties so that:

$$\frac{A_{tr}}{s} \geq \frac{d_b f_y}{48 f_{yt}} \qquad \text{(Eq.8–18)}$$

except that where there is no alternative load path in a structure for the forces being carried by an element in the event of failure of the element, lap splicing shall not be permitted at all.

8.9.1.3 *Requirements for welded splices or mechanical connections*

A3 For welded splices or mechanical connections to be used in members that are subjected to seismic forces, such splices shall comply with 8.7.4.1 or 8.7.5.2. In addition to the requirements of 8.7.5.2, mechanical splices and anchorages shall satisfy the cyclic load performance requirements specified by ISO 15835-1 and ISO 15835-2 as follows:

(a) When tested in accordance with 5.6.2 of ISO 15835-2, the residual elongations after 4 cycles, u_4, shall be less than 0.3 mm, and after 8 cycles u_8 shall be less than 0.6 mm;

(b) Where high cycle fatigue is a consideration, the mechanical connection shall satisfy the requirements of 5.4 of ISO 15835-1. The testing shall comply with 5.5 of ISO 15835-2.

Splices not satisfying this stiffness requirement shall be used only if they are staggered so that no more than two-thirds of the reinforcement area is spliced within any 900 mm length of the member. A3

8.9.2 *Development length*

For calculation of development length, the reduction provisions of 8.6.3.3(a), 8.6.8.2 and 8.6.10.3.1 by α_b (equal to A_{sr}/A_{sp}) shall not apply. A3

NOTES

9 DESIGN OF REINFORCED CONCRETE BEAMS AND ONE-WAY SLABS FOR STRENGTH, SERVICEABILITY AND DUCTILITY

9.1 Notation

A_b area of longitudinal bar, mm^2
A_{cv} effective shear area, area used to calculate shear stress, mm^2 A2
A_g gross area of column cross section, mm^2
A_s area of flexural tension reinforcement, mm^2
A_s' area of compression reinforcement, mm^2
A_{sp} area of flexural reinforcement provided, mm^2 A3
A_{sr} area of flexural reinforcement required, mm^2
A_{te} area of one leg of stirrup-tie, mm^2
A_t area of reinforcement transverse to the web in the slab which lies within the distance X, mm^2 A3
A_v area of shear reinforcement perpendicular to the span within a distance s, mm^2
A_{vd} area of diagonal shear reinforcement, mm^2
A_{vh} area of shear reinforcement parallel to span, mm^2
b width of compression face of a member, mm
b_f effective overhanging flange width, mm A3
b_w width of web, mm
c_b distance from extreme compression fibre to neutral axis at balanced strain conditions, as defined in 7.4.2.8, mm
d distance from extreme compression fibre to centroid of tension reinforcement, mm A3

d_b nominal diameter of longitudinal reinforcing bar, mm
f_c' specified compressive strength of concrete, MPa
f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa
f_s' compression stress in the bar on one side of joint zone, MPa A2
f_y lower characteristic yield strength of longitudinal reinforcement, MPa
f_{yt} lower characteristic yield strength of transverse reinforcement, MPa

A2

h overall depth, mm
h_b overall depth of beam, mm A3
h_{b1}, h_{b2} beam depths used for determining effective flange widths, mm
h_c overall depth of column, mm
h_g overall depth of girder, mm A2
k_a factor allowing for the influence of aggregate size on shear strength
k_d factor allowing for the influence of member depth on shear strength

A3

ℓ_y potential plastic region ductile detailing length, mm
L_n clear span of member measured from face of supports, mm

A3

A3

p ratio of tension reinforcement = A_s/bd

A3

p_{max}, p_{min} maximum and minimum permitted values of the ratio of tension reinforcement computed using width of web
p_w A_s/b_wd
r factor defined in 9.4.4.1.4

s	spacing of transverse reinforcement in direction parallel to longitudinal reinforcement, mm
s_2	spacing of shear or torsional reinforcement in perpendicular direction to longitudinal reinforcement
T_{tc}	Tension force sustained by the reinforcement in the *in situ* concrete including the concrete topping above the precast units, N
v_b	basic shear stress, N
v_c	shear resisted by concrete, MPa
V_c	nominal shear strength provided by the concrete, N
V_{di}	design shear force to be resisted by diagonal shear reinforcement at the ultimate limit state, N
V_n	total nominal shear strength of cross section of beam, N
v_p	vertical shear stress per unit length, MPa/m
V_s	nominal shear strength provided by the shear reinforcement, N
$V_{s,\,min}$	minimum nominal shear strength provided by shear reinforcement for non-prestressed members, MPa
V^*	design shear force at section at the ultimate limit state, N
V_o^*	maximum shear force sustained when overstrength actions act in a member or adjacent member, N
α	angle between inclined stirrups or bent-up bars and longitudinal axis of members
α_d	factor in Equation 9–21
α_f	factor in Equations 9–21 and 9–22
α_o	factor in Equations 9–21 and 9–22
α_p	factor in Equation 9–23
α_s	factor in Equation 9–24
α_t	factor in Equation 9–22
γ	factor given by Equation 9–20
ΣA_b	sum of areas of longitudinal bars, mm^2
δ_c	calculated inter-storey deflection, mm
ϕ	strength reduction factor (see 2.3.2.2)
$\phi_{o,fy}$	overstrength factor depending on reinforcement grade, see 2.6.5.5.

A3 (margin amendment markers beside T_{tc}, v_p, $V_{s,\,min}$ and below δ_c)

9.2 Scope

A2 The provisions of this section shall apply to the design of reinforced concrete members for flexure and shear without axial force. The provisions for this and earlier sections are summarised in Table C9.3. The written requirements take precedence over Table C9.3. Beams containing plastic regions with sectional curvature ductility demands less than or equal to the limits for the nominally ductile plastic region defined in 2.6.1.3 shall meet the requirements of 9.3. Beams containing plastic regions designed for greater sectional curvature ductility than this shall meet the requirements of 9.3 as modified by 9.4.

9.3 General principles and design requirements for beams and one-way slabs

9.3.1 *General*

9.3.1.1 *Moments at supports for beams integral with supports*

For beams built integrally with supports, moments at faces of support may be used for the design of reinforcement.

A3 **9.3.1.2** *Effective width resisting compression of T-beams and L-beams*

In T and L beam construction where the slab and web are built integrally, or where they are effectively bonded together, the outstanding portion of slab acting as part of a flange on one or on both sides of a web may be assumed to act with the web in resisting flexural forces.

The maximum width of flange to one side of a beam that is effective in resisting flexural forces shall be equal to the smaller of: A3

(a) One-eighth of the span of the beam;
(b) Eight times the minimum thickness of the slab within the effective flange;
(c) The depth of the beam, h_{b1};
(d) The clear distance between adjacent beams times the factor $\left(\dfrac{h_{b1}}{h_{b1}+h_{b2}}\right)$

where h_{b1} is the depth of the beam being considered and h_{b2} is the depth of the adjacent beam;
(e) The clear distance from the beam to the parallel edge of the slab;
(f) At beam sections close to a slab edge that is perpendicular to the span of the beam, the maximum of:
(i) Half the distance from the beam section being considered to the free edge of the slab plus half the thickness of the slab edge beam if present; or
(ii) Half the distance from the beam section being considered to the external face of the column.

9.3.1.3 *Effective moment of inertia in T-beams and L-beams*

In calculating the effective moment of inertia of cracked sections, the effective width of the overhanging parts of flanged members shall be one-half of that given by either 9.3.1.2(a) or (b).

9.3.1.4 *Contribution of slab reinforcement to design strength of T and L-beams*

In T and L-beams the longitudinal reinforcement located in the portion of the flange identified in either (a) or (b) as appropriate may be assumed to contribute to the nominal flexural strength of the beam: A3

(a) The portion of longitudinal reinforcement located in half of the outstanding width of flange located closest to the beam web, as identified in 9.3.1.2, provided the total contribution of flexural tension force in each individual outstanding portion of a flange does not exceed 10 % of the total flexural tension force;
(b) The portion of longitudinal reinforcement located in the outstanding part of a flange identified in 9.3.1.2 provided that:
(i) A strut and tie analysis demonstrates that the shear flow in the flange associated with the longitudinal reinforcement is adequately connected to the beam web by transverse reinforcement that is anchored into the core of the web, or it passes on the web side of all or part of the flexural tension reinforcement in the web of the beam, and
(ii) Rational allowance is made for shear lag associated with diagonal cracking in the web and outstanding portion of the flange in the development and location of the termination points of the longitudinal and transverse reinforcement.

9.3.1.5 *Floor finishes*

When a separate floor finish is placed on a slab it shall be assumed that the floor finish is not included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with the requirements of Sections 13 and 18.

9.3.1.6 *Deep beams*

9.3.1.6.1 *Definition*

Deep beams are members loaded on one face and supported on the opposite face, so that compression struts can develop between the loads and supports, and have either:

(a) Clear spans, L_n, equal to or less than 3.6 times the effective depth for simply supported or continuous beams; or A2
(b) Clear spans equal to or less than 1.8 times the effective depth for cantilevered beams.

9.3.1.6.2 *Design requirements*

Deep beams shall be designed taking into account non-linear distribution of strains or by using strut-and-tie models. Possible lateral buckling shall be considered. Design of deep beams shall be in accordance with 9.3.10.

9.3.2 *Strength of beams and one-way slabs in bending*

The design of beams and one-way slabs for flexure at the ultimate limit state shall be based on the assumptions given in 7.4 and on the satisfaction of applicable conditions of equilibrium and compatibility of strains.

9.3.3 *Strength of beams and in shear*

The design of beams and for shear at the ultimate limit state shall be in accordance with 7.5 and 9.3.9.

9.3.4 *Strength of beams in torsion*

The design of beams for torsion, shear, and flexure at the ultimate limit state shall be in accordance with 7.6.

9.3.5 *Distance between lateral supports of beams*

9.3.5.1 *Limits on lateral support spacing*

Spacing of lateral supports for a beam shall not exceed 50 times the least width, b, of the compression flange or face.

9.3.5.2 *Effects of load eccentricity on lateral support spacing*

Effects of lateral eccentricity of load shall be taken into account in determining the spacing of lateral supports.

9.3.6 *Control of flexural cracking*

9.3.6.1 *General*

Members subjected to flexure shall be designed to control cracking in accordance with 2.4.4.

9.3.6.2 *Beams and one-way slabs*

In beams and one-way slabs, the flexural tension reinforcement shall be well distributed across the zone of maximum tension in the member cross section and shall satisfy 2.4.4.

9.3.6.3 *Skin reinforcement*

If the depth of a member exceeds 1.0 m, longitudinal skin reinforcement shall be placed along the side faces in accordance with 2.4.4.5.

9.3.7 *Control of deflections*

9.3.7.1 *Minimum thickness*

The minimum thickness specified in 2.4.3 shall apply unless the calculation of deflection according to 6.8 indicates that lesser thickness may be used without adverse effects.

9.3.8 *Longitudinal reinforcement in beams and one-way slabs*

9.3.8.1 *Maximum longitudinal reinforcement in beams and one-way slabs*

For beams and slabs the amount and distribution of longitudinal reinforcement provided shall be such that at every section, the distance from the extreme compression fibre to the neutral axis is less than $0.75c_b$. Where moment redistribution in accordance with 6.3.7 at a section is utilised, the neutral axis depth shall also comply with 6.3.7.2(f).

9.3.8.2 *Minimum longitudinal reinforcement in beams and one-way slabs*

A2 **9.3.8.2.1** *Minimum reinforcement in beams*

At every section of a beam, except as provided in one of 9.3.8.2.2, 9.3.8.2.3 or 9.3.8.2.4, (where tension reinforcement is required by analysis), the reinforcement area A_s provided shall be greater than that given by:

$$A_s = \frac{\sqrt{f_c'}}{4f_y} b_w d \quad \text{(Eq. 9–1)}$$

but equal to or greater than $1.4\, b_w d / f_y$.

For beams where the flexural strength of the member contributes to the lateral force resistance of the structure, the minimum negative moment reinforcement consisting of two or more bars shall extend right through the span. The area of this reinforcement shall be equal to or greater than the larger of one-quarter of the maximum negative moment reinforcement in the beam, or the value given by Equation 9–1. A2

9.3.8.2.2 *Minimum reinforcement in statically determinate T-beams*

For a statically determinate T-beam with the flange in tension, where A_s includes the area of longitudinal reinforcement in flanges in accordance with 9.3.1.4 the reinforcement area A_s shall be greater than the value given by Equation 9–1 with b_w replaced by either $2b_w$ or the width of the flange, whichever is smaller.

9.3.8.2.3 *Reduced minimum reinforcement* A2

For beams and slabs greater than 400 mm thick, where the flexural strength does not contribute to the lateral strength of the structure, the area of longitudinal reinforcement given by Equation 9–1 with its minimum limit of 1.4 $b_w d/f_y$, and by 9.3.8.2.2, may be reduced, provided that at every section of a beam or slab, for positive and negative moment the area of reinforcement shall be at least one-third greater than that required by analysis. A3 A3

9.3.8.2.4 *Minimum reinforcement in slabs and footings*

For structural slabs and footings of uniform thickness, the minimum area of principal reinforcement shall satisfy 9.3.8.2.1 and for reinforcement normal to the principal reinforcing and spacing of reinforcement shall be as required for shrinkage and temperature according to 8.8.

9.3.8.3 *Spacing of reinforcement in slabs*

The spacing of principal reinforcement in slabs shall not exceed the smaller of two times the slab thickness or 300 mm. For reinforcement perpendicular to the principal reinforcement, the maximum spacing of reinforcement shall not exceed the lesser of three times the slab thickness, 300 mm for bridges or 450 mm for buildings. The spacing of reinforcement in *in situ* concrete topping in floors containing precast units shall be equal to or less than 400 mm for reinforcement which is either above the precast units or parallel to the precast units. For reinforcement which crosses infills, with a width greater than 300 mm, the spacing shall be equal to or less than 200 mm and the reinforcement shall be fully developed on each side of the infill. A2 A2

9.3.8.4 *Maximum diameter of longitudinal beam bar in internal beam-column joint zones*

For nominally ductile structures the maximum diameter of longitudinal beam bars passing through beam-column joint zones shall not exceed the appropriate requirement given below for internal beam-column joints:

(a) Where the critical load combination for flexure in a beam at the a face of an internal column includes earthquake actions the ratio of bar diameter to column depth, d_b/h_c, shall not exceed:

$$\frac{d_b}{h_c} = 4\alpha_f \frac{\sqrt{f_c'}}{f_y} \qquad \text{(Eq. 9–2)}$$

where α_f is taken as 0.85 where the beam bar passes through a joint in a two-way frame and as 1.0 for a joint in a one-way frame.

(b) Where the critical load combination for flexure in a beam at the face of a column either, does not include earthquake actions, or, plastic regions cannot develop adjacent to the face of the column, the ratio of bar diameter to column depth shall not exceed: A2

$$\frac{d_b}{h_c} = 6\alpha_f \frac{\sqrt{f_c'}}{\left(f_y\left(1+\frac{f_s'}{f_y}\right)\right)} \qquad \text{(Eq. 9–3)}$$

The value of f_s' is the compression stress in the bar on one side of the joint zone, but need not be taken as greater than $0.5f_y$, and α_f is as defined in (a) above.

9.3.8.5 *Anchorage of beam bars using hooks in beam-column joints* A2

The bars shall be hooked and satisfy the requirements of 8.6.10, with the hooked end being bent towards the mid-height of the beam and the hook being located as close as possible to the face of the column

A2 furthest from the critical section, which is to be taken at the entry point of the bar into the column. The distance between outside edge of the hook and the point of entry into the column shall in all cases be equal to or greater than ¾ h_c.

9.3.9 ***Transverse reinforcement in beams and one-way slabs***

9.3.9.1 *General*

Transverse reinforcement shall be the maximum area required for shear combined with torsion or for control of bar buckling.

9.3.9.2 *Diameter and yield strength of transverse reinforcement*

Stirrup or tie reinforcement shall be at least 5 mm in diameter and the design yield strength shall not be taken greater than 500 MPa.

9.3.9.3 *Design for shear*

9.3.9.3.1 *Design shear force adjacent to supports*

A2 The design shear force in a beam at a support may be computed at a critical section a distance of d out from the edge of the support for reinforced concrete and at a distance of $h/2$ out for prestressed concrete, provided the conditions below are satisfied:

(a) The support reaction applies compression to the bottom surface of the beam;
(b) The loads are applied to the top or near the top of the beam; and
(c) No significant concentrated load occurs between the critical section and the support.

9.3.9.3.2 *Design of shear reinforcement*

The design of shear reinforcement shall be based on the assumptions given in 7.5 and be in accordance with 9.3.9.4.

9.3.9.3.3 *Maximum nominal shear stress and effective shear area*

A3 The maximum nominal shear stress, v_n, shall be equal to or less than $0.2f'_c$ or 10 MPa as given by 7.5.

The value of A_{cv} shall:

(a) For rectangular, T- and I- section shapes be taken as product of the web (b_w) width times the effective depth, (d);
(b) For octagonal, circular, elliptical, and similar shaped section A_{cv} shall be taken as the area enclosed by the transverse reinforcement;

A3 (c) For hollow sections A_{cv} shall be taken as $b_w d$ where b_w is the sum of the minimum web widths.

9.3.9.3.4 *Nominal shear strength provided by the concrete for normal density concrete, Vc*

The nominal shear strength resisted by concrete, V_c, shall be taken as:

$$V_c = v_c A_{cv} \qquad \text{(Eq. 9–4)}$$

where v_c is the shear resisted by concrete.

The value of v_c is given by:

$$v_c = k_d k_a v_b \qquad \text{(Eq. 9–5)}$$

A2 where v_b is equal to the smaller of $(0.07+10p_w)\sqrt{f'_c}$ or $0.2\sqrt{f'_c}$, but need not be taken as less than $0.08\sqrt{f'_c}$.

In the calculation for v_b the value of f'_c shall not be taken as greater than 50 MPa.

The factor k_a, in Equation 9–5, allows for the influence of maximum aggregate size on the shear strength. For concrete with a maximum aggregate size of 19 mm or more k_a shall be taken as 1.0. For concrete where the maximum aggregate size is of 10 mm or less, the value of k_a shall be taken as 0.85. Interpolation may be used between these limits. A3

The factor k_d allows for the influence of member depth on strength and it shall be calculated from any one of the appropriate conditions listed below:

(a) For members with shear reinforcement equal to or greater than the nominal shear reinforcement given in 9.3.9.4.15, $k_d = 1.0$;

(b) For members with a depth equal to or less than 400 mm and greater than 200 mm, the value of v_c may be calculated either by: A3
  (i) Taking k_d equal to 1.0 in (c), or
  (ii) By linear interpolation in terms of effective depth between equivalent members with the same p_w value and concrete strength as the member being considered:
    (A) v_c for a member with an effective depth of 400 mm using Equation 9–5, and
    (B) v_c for a member with an effective depth of 200 mm given by (e);

(c) For members with an effective depth greater than 400, $k_d = (400\,/d)^{0.25}$ where d is in mm;

(d) For members with longitudinal reinforcement in the web, with a ratio of 0.003 or more, for the area between the principal flexural tension reinforcement and the mid depth of the beam, and with a bar spacing which does not exceed 300 mm in any direction, k_d, is given by $k_d = (400/d)^{0.25}$, but with limits of $0.9 \le k_d \le 1.0$;

(e) For members with an effective depth equal to or less than 200 mm the value of v_c may be taken as the larger of $0.17\,k_a\sqrt{f_c'}$ or $(0.07 + 10\,p_w)k_a\sqrt{f_c'} \le 0.2k_a\sqrt{f_c'}$. A3

9.3.9.3.5 *Nominal shear strength provided by the concrete for lightweight concrete*

Provisions for the nominal shear strength provided by the concrete, apply to normal density concrete. Where lightweight aggregate concrete is used one of the following modifications shall apply:

(a) Where f_{ct} is specified and the concrete mix is designed in accordance with NZS 3152, provisions for v_b (in Equation 9–5 shall be modified by substituting 1.8 f_{ct} for $\sqrt{f_c'}$ but the value of 1.8 f_{ct} shall not exceed $\sqrt{f_c'}$;

(b) Where f_{ct} is not specified, all values of $\sqrt{f_c'}$ affecting v_b shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be applied when partial sand replacement is used.

9.3.9.3.6 *Nominal shear strength provided by shear reinforcement*

In accordance with 7.5 the shear reinforcement shall be computed using:

$$V_s = \frac{V^*}{\phi} - V_c \qquad \text{(Eq. 9–6)}$$

where V_c is the nominal shear strength provided by the concrete given in 9.3.9.3.4 and 9.3.9.3.5 and V_s is the shear strength provided by the shear reinforcement given in 9.3.9.4.

9.3.9.4 *Design of shear reinforcement in beams*

9.3.9.4.1 *General*

For beams a truss analogy shall be used to determine the nominal shear strength of members with web reinforcement. Either the strut and tie method may be used, in which case V_c shall be taken as zero, or V_c shall be calculated from 9.3.9.3.4 and 9.3.9.3.5 and V_s shall be calculated from 9.3.9.4.2 to 9.3.9.4.8. In either case the requirements of 9.3.9.4.5 to 9.3.9.4.8 shall be satisfied.

9.3.9.4.2 *Shear reinforcement perpendicular to longitudinal axis of the beams*

When shear reinforcement perpendicular to the longitudinal axis of beams is used and the applied shear is parallel to the legs of rectangular stirrups or ties:

$$V_s = A_v f_{yt} \frac{d}{s} \qquad \text{(Eq. 9–7)}$$

where A_v is the area of shear reinforcement within distance s.

9.3.9.4.3 *Bent-up bars or inclined stirrups of beams*

When bent-up bars or inclined stirrups are used as shear reinforcement in beams:

$$V_s = \frac{A_v f_{yt} (\sin\alpha + \cos\alpha) d}{s} \quad \text{(Eq. 9–8)}$$

where α is the angle between the bent-up bars or the inclined stirrups and longitudinal axis of the beam.

9.3.9.4.4 *Single bar or single group of parallel bars*

When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support:

$$V_s = A_v f_{yt} \sin\alpha \quad \text{(Eq. 9–9)}$$

but not greater than $0.25\sqrt{f_c'}\ b_w d$.

9.3.9.4.5 *Series or groups of parallel bent-up bars*

When shear reinforcement in beams consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, the required area shall be equal to or greater than that computed using Equation 9–8.

9.3.9.4.6 *Effective inclined portion of bent-up bar*

The centre three-quarters only of the inclined portion of any longitudinal bent-up bar in a beam shall be considered effective for shear reinforcement.

9.3.9.4.7 *More than one type of shear reinforcement*

Where more than one type of shear reinforcement is used to reinforce the same portion of the web of a beam the shear strength, V_s, shall be computed as the sum of the V_s values computed for the various types.

9.3.9.4.8 *Angle of shear reinforcement not parallel to applied shear*

In members, such as circular or elliptical members, where the angle made by the shear reinforcement intersecting a potential diagonal tension crack varies in direction, only the component of the shear reinforcement which is parallel to the shear force shall be included.

9.3.9.4.9 *Stirrups required where beam frames monolithically into side of girder*

Where a beam of depth h_b and width b, frames monolithically into a supporting girder of depth h_g, stirrups shall be provided in the supporting girder as follows:

(a) The design strength of the stirrups, $\phi \Sigma A_v f_{yt}$ shall equal or be greater than the total reaction transferred from the beams;

A3 (b) The stirrups specified in (a) shall be provided within a length of 2b centred about the centreline of the beam;

(c) The requirements of (a) and (b) are waived if the reaction from the beam is introduced within the compression zone of the girder, or the girder is supported below the beam girder joint.

9.3.9.4.10 *Stirrups required for non-monolithic beam-girder connections*

Where a beam frames into a supporting girder, and a monolithic connection is not provided, stirrups shall be provided at the end of the beam with a design strength, $\phi \Sigma A_v f_{yt}$, equal or greater than the total reaction transferred from the beam. These requirements are waived if the beam is supported by bearing on a seating and stirrups in the girder comply with 9.3.9.4.9.

9.3.9.4.11 *Location and anchorage of shear reinforcement*

Stirrups and other bars or wires used as shear reinforcement shall be anchored as required by 7.5.7.

9.3.9.4.12 *Spacing limits for shear reinforcement*

Spacing limits for shear reinforcement shall be as follows:

(a) Spacing of shear reinforcement measured along the axis of the member, shall be equal to or smaller than the smaller of 0.5d or 600 mm;

(b) Where the width of the web exceeds 0.5d the spacing between stirrup legs measured at right angles to the longitudinal axis the beam shall be equal to or smaller than 0.5d or 600 mm but need not be less than 250 mm; A2

(c) Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45° line, extending towards the reaction from mid-depth of member (i.e. 0.5d) to the longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement;

(d) When V_s exceeds $0.33\sqrt{f'_c}\,b_w d$, the maximum spacings given in 9.3.9.4.12(a) and (b) shall be reduced by one-half, except in (b) the spacing need not be less than 200 mm.

9.3.9.4.13 *Minimum area of shear reinforcement*

A minimum area of shear reinforcement shall be provided in all reinforced and prestressed concrete members as required by 7.5.10 when the design shear force exceeds one half of the design shear strength provided by concrete, ϕV_c, except: A3 A2

(a) In beams with a total depth equal to or less than 250 mm;

(b) In beams or ribs cast compositely with slabs, where the overall depth is equal to or less than 300 mm, and the clear spacing of beam webs is equal to or less than 750 mm; A3

(c) In reinforced slabs with a depth equal to or less than 250 mm. The shear strength provided by the concrete shall be calculated from 9.3.9.3.4 for one-way slabs and 12.7 for two-way slabs;

(d) For composite floors consisting of composite precast pretensioned units and *in situ* concrete topping in which the clear spacing between the webs is equal to or less than 750 mm, the depth of the precast unit is equal to or less than 300 mm and the overall depth is less than 400 mm. The shear force resisted by the concrete shall be taken as the smaller of that associated with positive or negative flexure. The shear strength provided by the concrete for positive flexure shall be calculated from 19.3.11.2.3 and the corresponding value for negative flexure shall be calculated from 19.3.11.2.4.

The exceptions (a), (b), (c) and (d) should not be used when highly repetitive loads occur inducing a shear force due to the variable component of the load exceeding V_c/3, or where either slabs are free to translate horizontally at their boundaries or load sharing possibilities in slabs, or between adjacent webs, do not exist. A2

9.3.9.4.14 *Minimum shear reinforcement waived by testing*

Minimum shear reinforcement requirements of 9.3.9.4.13 may be waived if shown by full scale testing that the required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

9.3.9.4.15 *Minimum nominal shear strength provided by shear reinforcement* A3

Where shear reinforcement is required by 9.3.9.4.13, and where 7.6.1.2 allows torsion to be neglected, the minimum nominal shear strength provided by shear reinforcement for non-prestressed members shall be computed by:

$$V_{s,\min} = \frac{1}{16}\sqrt{f'_c}\,b_w d \quad \text{(Eq. 9–10)}$$

9.3.9.5 *Torsional reinforcement*

Except for slabs and footings which are exempted from requirements for torsional reinforcement by 7.6.1.1, torsional reinforcement shall be provided in accordance with 7.6.

9.3.9.6 *Design of transverse reinforcement for lateral restraint of longitudinal bars*

9.3.9.6.1 *Extent of transverse reinforcement*

Stirrups or ties conforming to 9.3.9.6.2 and 9.3.9.6.3 shall be present throughout the length of a beam or slab where longitudinal compression reinforcement is required.

9.3.9.6.2 *Centre-to-centre spacing of transverse reinforcement*

Centre-to-centre spacing of stirrups or ties along the member shall not exceed the smaller of the least lateral dimension of the cross section of the member or 16 longitudinal bar diameters.

9.3.9.6.3 *Arrangement of stirrups or ties*

Stirrups or ties shall be arranged so that every corner and alternate longitudinal bar that is required to function as compression reinforcement shall have lateral support provided by the corner of a stirrup or tie. This lateral support shall be provided by an included angle of not more than 135° and no longitudinal bar shall be further than 150 mm clear on each side from a laterally supported bar. The requirements of 7.5.7.1 shall be satisfied.

9.3.9.6.4 *Enclosure of compression reinforcement*

Stirrup or tie reinforcement shall enclose the longitudinal compression reinforcement in the webs of beams.

9.3.10 ***Special provisions for deep beams***

9.3.10.1 *General*

A2 The provisions of 9.3.10 apply to members which satisfy the requirements of 9.3.1.6.1.

9.3.10.2 *Design methods*

Deep beams shall be designed using strut-and-tie models or by taking into account the non-linear distribution of strains. The minimum reinforcement in deep beams is to comply with 9.3.10.3 and 9.3.10.4.

9.3.10.3 *Minimum vertical shear reinforcement*

The area of shear reinforcement perpendicular to the span A_v, shall be equal to or greater than $0.0025b_w s$,
A2 and s shall not exceed $d/5$, nor 300 mm, but where d is less than 750 mm, s may be taken as 150 mm.

9.3.10.4 *Minimum horizontal shear reinforcement*

The area of shear reinforcement parallel to the span, A_{vh}, shall be equal to or greater than $0.0015b_w s_2$, and
A2 s_2 shall not exceed $d/5$, nor 300 mm, but where d is less than 750 mm, s_2 may be taken as 150 mm.

9.3.11 ***Openings in the web***

9.3.11.1 *General*

Adjacent openings for services in the web of flexural members shall be arranged so that potential failure planes across such openings cannot occur.

9.3.11.2 *Location and size of openings*

Small square or circular openings may be placed in the mid-depth of the web provided that cover requirements to longitudinal and transverse reinforcement are satisfied, and the clear distance between such openings, measured along the member, is equal to or greater than 150 mm. The size of small openings shall not exceed 1000 mm^2 for members with an effective depth less than or equal to 500 mm, or $0.004d^2$ when the effective depth is more than 500 mm.

9.3.11.3 *Larger openings*

Webs with openings larger than that permitted by 9.3.11.2 shall be subject to rational design to ensure that the forces and moments are adequately transferred at and in the vicinity of the openings.

9.3.11.4 *Location and size of large openings*

Whenever the largest dimension of an opening exceeds one-quarter of the effective depth of the member it is to be considered large. Such openings shall not be placed in the web where they could affect the flexural or shear capacity of the member, or where the design shear force exceeds $0.4\sqrt{f'_c}\, b_w d$, or closer than 1.5h to the critical section of a plastic region. In no case shall the height of the opening exceed 0.4d or its edge be closer than 0.33d to the compression face of the member.

9.3.11.5 *Reinforcement in chords adjacent to openings*

For openings defined by 9.3.11.4, longitudinal and transverse reinforcement shall be placed in the chords at both sides of the opening to resist $1^1/_2$ times the shear force and bending moment generated by the shear across the opening. Shear resistance shall be assigned to each chord in proportion of its stiffness taking into account the effects of cracking and axial compression and tension induced in the chords by the primary moment at the opening.

9.3.11.6 *Reinforcement in webs adjacent to openings*

Transverse web reinforcement, extending over the full depth of the web, shall be placed adjacent to both sides of a large opening over a distance not exceeding one-half of the effective depth of the member to resist twice the entire design shear force across the opening.

9.4 Additional design requirements for members designed for ductility in earthquakes

9.4.1 *Dimensions of beams*

9.4.1.1 *General*

For beams which sustain plastic regions in the ultimate limit state either an analysis based on first principles shall be made to demonstrate that the beam is stable or the dimension limits given in 9.4.1.2, 9.4.1.3 or 9.4.1.4 as appropriate shall be satisfied.

9.4.1.2 *Beams with rectangular cross sections*

The depth, width and clear length between the faces of supports of members with rectangular cross sections, to which moments are applied at both ends by adjacent beams, columns or both, shall be such that:

$$\frac{L_n}{b_w} \leq 25 \quad \text{(Eq. 9–11)}$$

and

$$\frac{L_n h}{b_w^2} \leq 100 \quad \text{(Eq. 9–12)}$$

9.4.1.3 *Cantilevered beams*

The depth, width and clear length from the face of support of cantilever members with rectangular cross sections, shall be such that:

$$\frac{L_n}{b_w} \leq 15 \quad \text{(Eq. 9–13)}$$

and

$$\frac{L_n h}{b_w^2} \leq 60 \quad \text{(Eq. 9–14)}$$

9.4.1.4 *T - and L - beams*

The width of web of T- and L- beams, in which the flange or flanges are integrally built with the web, shall be such that the values given by Equations 9–11 and 9–13 are not exceeded by more than 50 %.

9.4.1.5 *Width of compression face of members*

The width of the compression face of a member with rectangular, T-, L- or I- section shall be equal to or greater than 200 mm.

9.4.1.6 *Slab width effective in tension in negative moment regions of beams*

9.4.1.6.1 *Contribution of slab reinforcement to design strength of beams*

In T- and L- beams built integrally with slabs, slab reinforcement contained within the effective overhanging flange may be considered to contribute to the design flexural strength in ductile and limited ductile plastic regions in beams, as detailed for nominally ductile beams in 9.3.1.4. A3

9.4.1.6.2 *Contribution of slab reinforcement to overstrength of plastic region in a beam*

A3 Where T- and L-beams are built integrally with slabs that contain post-tensioned prestressed cables, both the width of the outstanding flange that contributes to overstrength and the stress level that may be sustained by the cables shall be determined by a special study.

In reinforced concrete T and L-beams built integrally with slabs, slab reinforcement in the overhanging portion of flanges, which are identified in (a), (b), (c) or (e) below, shall be assumed to contribute to the overstrength moment of resistance at the critical section of plastic regions in the beam being considered. A2 Where precast units are contained in a portion of slab within the effective overhanging flange width their contribution to strength shall be included as specified in (d), (e) and (f). In no case need the flexural tension force contribution of an overhanging flange exceed the value given in (g). A2

(a) Where a beam containing the potential plastic region is at right angles to the edge of the floor and it frames into an exterior column, but no transverse edge beam is present, the effective width of overhanging flange, b_f, shall be taken as the smaller of the distance at the critical section of the potential plastic region in the beam between the web and a line drawn at 45° from the intersection of a line drawn parallel to the web and touching the side of the column and the edge of the slab. Any reinforcement passing through this section shall be assumed to be stressed to 1.1 $\phi_{o,fy}$ f_y, where the value of $\phi_{o,fy}$ is given in 2.6.5.5. A2

(b) Where a beam containing the potential plastic region is at right angles to the edge of a slab frames into an external column and the slab is supported by a transverse beam, the effective overhanging flange width, b_f, shall be taken as the smaller of the width defined in (a) above plus twice the width of the web of the transverse beam. A2

The tension force sustained by the overhanging flange shall be calculated as in (a).

(c) Where a beam containing a potential plastic region or regions passes through a column the effective overhanging flange width on each side of the beam shall be taken as the smaller of:

(i) Three times the overall depth of the beam;

(ii) The clear distance between adjacent beams times the factor $\left(\dfrac{h_{b1}}{h_{b1}+h_{b2}}\right)$

Where h_{b1} is the depth of the beam being considered and h_{b2} is the depth of the adjacent beam.

(d) Where precast prestressed components, which are:

(i) Parallel or near parallel to the beam containing the potential plastic region

(ii) Span past these potential plastic regions, and

(iii) Are located within the effective overhanging flange width defined in (c) above, their contribution to the flexural overstrength of the plastic regions shall be calculated.

The contribution of an overhanging flange containing prestressed units consists of two parts:

(i) The tension force sustained by the reinforcement in the *in situ* concrete including the concrete topping above the precast units. This component, T_{tc}, shall be taken as the total area of the non-prestressed reinforcement within the overhanging flange, which is parallel to the web of the beam containing the plastic region, times 1.1 f_y.

(ii) The tension force, which acts at the mid-depth of the topping concrete, that can be sustained by the precast units located within the overhanging flange width. The determination of this force shall either be based on a rational analysis, or on the simplifying assumption that in the limiting condition the compression force in the precast unit is coincident with the prestressing force. With this assumption the tension force resisted by the precast units, T_p, is given by:

$$T_p = \frac{M_f}{e} \qquad \text{(Eq. 9–15)}$$

Where

e = the distance between the mid-depth of the topping concrete to the centroid of the prestressing force.

M_f = the bending in the effective over-hanging flange located in the plane containing the critical section of the plastic region being considered. This bending moment, M_f, is calculated, as detailed below, assuming precast units and topping concrete comprising this overhanging flange are supported at the ends of the precast units by transverse beams.

The bending moment, M_f, shall be calculated by summing the components due to:

(i) The total dead load of the overhanging flange and the associated long-term live load;

(ii) The positive flexural moments which can be transmitted to the precast units at their supports by reinforcement connecting the units to the transverse beams, with the smaller of the top or bottom reinforcement stressed to $\phi_{o,fy} f_y$;

(iii) The vertical shear forces, which can be transferred between the web of the beam and the first prestressed unit in the flange and between the face of any column located close to the potential plastic regions and the first precast unit. In both cases the shear forces per unit length, v_p, are calculated from the flexural resistance provided by the slab linking the beam web or column to the first precast unit slab by the equation:

$$v_p = \left(\frac{m_{\ell,w} + m_{\ell,p}}{L_\ell} \right) \qquad \text{(Eq. 9–16)}$$ A3

Where:

$m_{\ell,w}$ = the flexural strength of the linking slab per unit length at the face of the web or column;

$m_{\ell,p}$ = the flexural strength of the linking slab at the face of the first precast unit;

L_ℓ = the span of the linking slab, between the face of the web and the face of the first precast unit, or between the face of the column and first precast unit.

The flexural strengths of the linking slab per unit length, $m_{\ell,w}$ and $m_{\ell,p}$ shall be based on standard flexural ultimate strength theory but assuming the stress in the reinforcement is $1.1 f_y$ and a rectangular concrete stress block with a stress of $0.2\,f'_c$, where the span is between the web of the beam and the first precast unit and $0.8\,f'_c$ where the span is between the column face and first precast unit. A3 A2

(e) Where a beam containing a potential plastic region or regions passes through a column with a transverse beam framing into it, the effective flange width on the side or sides with the transverse shall be taken as:

(i) The smaller of the clear distance between the adjacent beams times the factor $\left(\frac{h_{b1}}{h_{b1} + h_{b2}} \right) \le 4\,h_{b1}$ for the case where the transverse beams support precast floor units;

(ii) For the case where transverse beam does not support precast floor units, the effective flange width is equal to the value given above, but the limit of $4h_{b1}$ replaced by $3h_{b1}$.
Where h_{b1} is the depth of the beam being considered and h_{b2} is the width of the adjacent beam.

The tension force contribution of the flange shall be taken as the area of reinforcement connecting the flange to the transverse beam times a stress in this reinforcement of $1.1\,\phi_{o,fy} f_y$.

(f) For situations which are not covered by (a), (b) (c) (d) or (e), the contribution of the flange shall be based on a rational extension of the effective flange widths and method of calculation in (a), (b), (c), (d) and (e).

(g) The contribution of a flange to the overstrength tension force, T_f, in a beam need not be taken greater than:

$$T_f = \phi_{o,fy} f_y A_\ell + 2.0 f_y A_t \quad \text{(Eq. 9–17)}$$

Where:

A_ℓ is the area of longitudinal reinforcement, (parallel to web) in the overhanging flange;

f_y is the design yield stress;

$\phi_{o,fy}$ is the overstrength factor for the reinforcement given in 2.6.5.6;

A3 A_t is the area of reinforcement transverse to the web in the slab which lies within the distance X;

X is the distance to the end of the prestressed unit from the critical section of the plastic region being considered;

The value of T_f at the section being considered may be taken as the smaller of the values calculated from each end of the precast unit.

In calculating the flexural overstrength in a plastic region, when the flexural compression force is on the flange side of the member, the effective width of slab on each side of the beam, which contributes to the resistance of the compression force, shall be taken as equal to 4 times the thickness of the slab adjacent to the web.

9.4.1.6.3 *Diameter and extent of slab bars*

The diameter of bars in that part of the slab specified in 9.4.1.6.1 shall not exceed one-fifth of the slab thickness. Such bars, when subjected to tension, shall extend by the horizontal distance from the position of the bar to the centre of the beam section beyond the point specified in 8.6.12.3.

9.4.1.7 *Narrow beams and wide columns*

Where narrow beams frame into wide columns, the width of a column that shall be assumed to resist the forces transmitted by the beam shall be in accordance with 15.4.6.

9.4.1.8 *Wide beams at columns*

Where wide beams frame into columns the width of beam that shall be assumed to resist the forces transmitted by the column shall be no more than the width of the column plus a distance on each side of the column equal to one-quarter of the overall depth of the column in the relevant direction.

9.4.2 *Ductile detailing lengths*

A2 Special detailing is required in ductile and limited ductile plastic regions, extending from the critical section of the plastic regions over a length equal to the ductile detailing length, ℓ_y. that are associated with ductile or limited ductile plastic regions. These regions and lengths shall be located as follows:

(a) Where the critical section is located at the face of a supporting column, wall or beam: over a length equal to twice the beam depth, measured from the critical section toward mid-span, at each end of the beam where a plastic region may develop;

(b) Where the critical section is located at a distance equal to or greater than either the beam depth h or 500 mm away from a column or wall face: over a length that commences between the column or wall face and the critical section, at least either 0.5h or 250 mm from the critical section, and extends at least 1.5h past the critical section toward mid-span;

(c) Where, within the span, yielding of longitudinal reinforcement may occur only in one face of the beam as a result of inelastic displacements of the frame: over the lengths equal to twice the beam depth on both sides of the critical section.

9.4.3 *Longitudinal reinforcement in beams containing ductile or limited ductile plastic regions*

9.4.3.1 *Development of beam reinforcement*

The distribution and curtailment of the longitudinal beam reinforcement shall be such that the flexural overstrength of a section can be attained at critical sections in potential plastic hinge regions.

9.4.3.2 *Anchorage of beam bars in columns or beam studs*

9.4.3.2.1 *Point of commencement of bar anchorage*

When longitudinal beam bars are anchored in cores of exterior and interior columns or beam stubs, the anchorage for tension shall be deemed to commence at one-half of the relevant depth of the column or 8 d_b, whichever is less, from the face at which the beam bar enters the column. Where it can be shown that the critical section of the plastic hinge is at a distance of at least the beam depth or 500 mm, whichever is less, from the column face, the development length may be considered to commence at the column face of entry.

9.4.3.2.2 *Reinforcement of beam stubs*

Where longitudinal beam bars are terminated in beam stubs, the development length of the bars in compression shall be assumed to commence at the smaller of 8d_b or h/2 from the face of the column where the bar enters the joint zone. Where the development length in compression is inadequate to anchor the bar, reinforcement shall be provided to constrain the vertical leg of the bar. Reinforcement providing this constraint shall have a tension capacity equal to or greater than one twelfth of the yield force in the bar or bars to be anchored, and it shall be located within a vertical distance of 8 times the diameter of the anchored bar or bars measured from the longitudinal axis of the bar or bars. A2

9.4.3.2.3 *Development length*

For calculation of the development length, the reduction provisions of 8.6.3.3, 8.6.8.2 and 8.6.10.3 by $\alpha_b = A_{sr} / A_{sp}$ shall not apply.

9.4.3.2.4 *Anchorage of diagonal bars in coupling beams*

When three or more diagonal or horizontal bars of a coupling beam are anchored in adjacent structural walls, the development length shall be 1.5 times the development length computed from 8.6.3 and 8.9.2. A2

9.4.3.2.5 *Bars to terminate with a hook or anchorage device*

Notwithstanding the adequacy of the anchorage of a beam bar in a column core or a beam stub, no bar shall be terminated without a vertical 90° standard hook or equivalent anchorage device as near as practically possible to the far side of the column core, or the end of the beam stub where appropriate, and not closer than three-quarters of the relevant depth of the column to the face of entry. Top beam bars shall only be bent down and bottom bars must be bent up.

9.4.3.3 *Maximum longitudinal reinforcement in beams containing ductile plastic regions*

At any section of a beam within a ductile detailing length, ℓ_y, as defined in 9.4.2, the tension reinforcement ratio, p, shall not exceed:

$$p_{max} = \frac{f'_c + 10}{6f_y} \leq 0.025 \qquad \text{(Eq. 9–18)}$$

where the reinforcement ratio, p, shall be computed using the width of the web.

9.4.3.4 *Minimum longitudinal reinforcement in beams containing ductile plastic regions*

When determining the longitudinal reinforcement in beams of ductile structures:

(a) Any section within the ductile detailing length, ℓ_y, as defined in 9.4.2, shall have an area of compression reinforcement, A'_s, which is restrained against buckling as required by 9.4.5, equal to or greater than the appropriate proportion of flexural tension reinforcement, A_s, given below: A2
 (i) $A'_s > 0.5A_s$ for ductile plastic regions defined in 2.6.1.3
 (ii) $A'_s > 0.38A_s$ for limited ductile plastic regions defined in 2.6.1.3

 The area A_s is the area of flexural tension reinforcement provided within the section defined in 9.4.1.6.1 to meet the ultimate strength requirements.

This requirement need not be complied with when the compression reinforcement is placed within the depth of a compression flange of a T- or L- beam formed as a cast-in-place concrete floor slab built integrally with the web at a section subjected to positive bending moment, or where a unidirectional positive moment plastic hinge forms in the span of a beam supporting precast concrete floor units which have a cast-in-place concrete topping of thickness 60 mm or more.

(b) At any section of a beam the minimum longitudinal reinforcement ratio, p, for both top and bottom reinforcement computed using the width of the web shall exceed that given by:

$$p_{min} = \frac{\sqrt{f_c'}}{4f_y} \quad \text{(Eq. 9–19)}$$

(c) At least one quarter of the larger of the top flexural reinforcement required at either end of a beam shall be continued throughout its length. At least two 16 mm diameter bars shall be provided in both the top and bottom throughout the length of the beam.

9.4.3.5 *Maximum diameter of longitudinal beam bars passing through interior joints of ductile structures*

9.4.3.5.1 *General*

A2 The maximum diameter of Grades 300 and 500 longitudinal beam bars passing through an interior joint shall be computed from either 9.4.3.5.2 or 9.4.3.5.3 below provided one of the conditions, (a) to (d), given below is satisfied:

(a) Grade 300 reinforcement is used;
(b) The inter-storey deflections divided by the storey height at the ultimate limit state does not exceed 1.8 % when calculated using the equivalent static or modal response spectrum methods;
(c) The beam-column joint zone is protected from plastic hinge formation at the faces of the column (as illustrated in Figure C9.19);
(d) The plastic hinge rotation at either face of the column does not exceed 0.016 radians.

If none of these conditions is satisfied the permissible diameter of Grade 500 beam reinforcement passing through an interior joint shall be determined by multiplying the diameter given by 9.4.3.5.2 or 9.4.3.5.3 below by γ, where:

$$\gamma = (1.53 - 0.29\delta_c), \text{ but not greater than } 1.0 \quad \text{(Eq. 9–20)}$$

where

δ_c is the inter-storey drift to inter-storey height expressed as a percentage calculated in accordance with NZS 1170.5.

9.4.3.5.2 *Basic ratio of maximum longitudinal beam bar diameter to column depth*

A2 For beam bars passing through a column at a beam-column joint, the ratio of maximum longitudinal bar diameter to column depth shall comply with the appropriate value given in (a) or (b) below:

(a) Where potential plastic regions exist at the column faces:

$$\frac{d_b}{h_c} \leq 3.3\alpha_f\alpha_d \frac{\sqrt{f_c'}}{1.25f_y} \quad \text{(Eq. 9–21)}$$

The value of f_c' in Equation 9–21 shall not exceed 70 MPa:

(i) When beam bars pass through a joint in two directions, as in two-way frames, α_f = 0.85. For beam bars in one-way frames, α_f = 1.0
(ii) When the potential plastic hinges are classed as:
(A) Ductile plastic regions α_d = 1.0
(B) Limited ductile plastic regions α_d = 1.2.

(b) When the beam potential plastic hinges are located at a distance of at least the smaller of h or 500 mm away from the column faces so that the beam reinforcement remains in the elastic range on each side of the joint zone, the requirements of 9.3.8.4 shall be satisfied.

9.4.3.5.3 *Alternative ratio of maximum longitudinal beam bar diameter to column depth*

Alternatively by considering additional parameters, the ratio of maximum longitudinal beam bar diameter to column depth may be determined by:

$$\frac{d_b}{h_c} \le 6\left(\frac{\alpha_t \alpha_p}{\alpha_s}\right)\alpha_f \alpha_d \frac{\sqrt{f_c'}}{1.25 f_y} \quad \text{(Eq. 9–22)}$$ A3

where the variables are defined as follows:

(a) Values of α_f and α_d are as in 9.4.3.5.2;

(b) α_t = 0.85 for a top beam bar where more than 300 mm of fresh concrete is cast below the bar
α_t = 1.0 for all other cases

(c) To allow for the beneficial effect of compression on a column:

$$\alpha_p = \frac{N_o}{2 f_c' A_g} + 0.95 \quad \text{(Eq. 9–23)}$$

with the limitation of $1.0 \le \alpha_p \le 1.25$.
N_o is the minimum design overstrength axial load determined by capacity design in accordance with appendix D.

(d) The coefficient, α_s, allows for the more severe bond stress conditions at overstrength acting on beam reinforcement passing through a beam-column joint, where the strength of the reinforcement in the compression zone is less than the flexural tension force resisted by the beam and tension flanges. The value of α_s is given by: A2

$$\alpha_s = [2.55 - R]\frac{1}{\alpha_d} \quad \text{(Eq. 9–24)}$$

where R is the ratio of $\phi_{o,fy} A_s' f_y$ to the flexural tension force sustained by the beam and flanges at overstrength, with the limitation $0.75 \le R \le 1.0$.

9.4.3.6 *Splices in longitudinal reinforcement of beams of ductile structures*

9.4.3.6.1 *General*

Splices in longitudinal reinforcement in beams and one-way slabs shall comply with 8.7 and 8.9.1.

9.4.3.6.2 *Location of splices*

Full strength welded splices meeting the requirements of 8.7.4.1(a) and 8.7.4.2 may be used in any location. For all other splices in beams no portion shall be located in a beam-column joint region, or within one effective depth of member from the critical section of a potential plastic region in a beam where stress reversals in spliced bars could occur, unless 8.9.1.2 is complied with.

9.4.4 *Transverse reinforcement in beams of ductile structures*

9.4.4.1 *Design for shear in beams of ductile structures*

9.4.4.1.1 *Design shear strength*

The design shear at a section in a beam, V_o^* shall be determined from consideration of the flexural overstrength being developed at the most probable location of critical sections within the member or in adjacent members, and the gravity load with load factors as specified in 2.6.5.2.

9.4.4.1.2 *Design of shear reinforcement*

Design of shear reinforcement shall be in accordance with 7.5 and 9.3.9.3.6 but with V_c in potential plastic regions being taken as defined in 9.4.4.1.3.

9.4.4.1.3 *Nominal shear strength provided by concrete in potential plastic hinge regions of beams*

In potential plastic hinge regions defined in 9.4.2 the nominal shear strength provided by the concrete, V_c, shall be taken from the appropriate criterion below:

(a) In potential ductile plastic regions, V_c, shall be taken as zero;

(b) In potential limited ductile plastic regions, V_c, shall be taken as not more than half the value given by 9.3.9.3.4.

A2

9.4.4.1.4 *Sliding shear in reversing plastic regions*

Ductile and limited ductile reversing plastic regions in beams shall be designed to resist sliding shear on sections normal to the axis of the beam, as detailed below:

(a) The maximum shear force, V_o^*, at the critical section of the plastic region shall be equal to or less than the smaller of 0.16 $f_c' A_{cv}$ or 0.85 $A_{cv}\sqrt{f_c'}$, unless the requirements of 11.4.9.4 are satisfied;

(b) Where the value of V_o^* exceeds 0.25 $(2+r)\sqrt{f_c'}\, A_{cv}$, diagonal reinforcement shall be provided to resist a shear force, V_{di}, equal to or greater than:

A3

$$V_{di} \geq 0.7\left[\frac{V_o^*}{A_{cv}\sqrt{f_c'}} + 0.4\right](-r)V_o^* \quad \leq V_o^* \qquad \text{(Eq. 9–25)}$$

A2

Whereby taking into account the sense of the reversing total shear forces resulting from the two directions of earthquake actions, V_{di},needs to be considered only where –l.0 < r < –0.2.

Where r is the algebraic ratio at the critical section of the plastic hinge of the numerically smaller to the larger shear force when the reversal of direction of shear force can occur, and is always taken as negative;

(c) The diagonal reinforcement required to resist V_{di} shall be taken as the transverse component of the force or forces acting in the diagonal reinforcement when it sustains its design yield stress in tension or compression. This reinforcement may be calculated from the expression:

$$V_{di} = f_y \sin\alpha\,(A_{vd} + A_{vd}')$$

where A_{vd} and A_{vd}' are the areas of diagonal reinforcement in two directions placed at an angle of α to the longitudinal axis of the beam. The diagonal reinforcement may be placed in one or two directions;

(d) Where diagonal reinforcement is required, it shall extend for a distance equal to or greater than the effective depth of the beam from the critical section of the plastic region;

(e) The angle, α, between the diagonal reinforcement and the longitudinal axis of the beam, shall be within the range of 30° to 60°;

(f) Where diagonal reinforcement is assumed to act in compression in contributing to V_{di}, and the reinforcement is bent within the clear span of the beam, stirrups normal to the axis of the beam shall be located within the region of the bend in the diagonal reinforcement to resist the vertical component of the diagonal compression force.

9.4.4.1.5 *Design for conventional shear in plastic hinge*

Design for shear reinforcement to maintain equilibrium across a diagonal tension crack shall be in accordance with 9.4.4.1 and 7.5. The vertical component of the tension force carried by the diagonal reinforcement which crosses the potential diagonal crack may be added to the shear resistance provided by stirrups to give the shear force resisted by web reinforcement.

9.4.4.1.6 *Minimum shear reinforcement*

A2 Stirrups which are anchored round the top and bottom reinforcement in the beam shall be provided over the clear span of the beam. The spacing of stirrups shall be equal to or less than the smaller of 12d_b or d/2, where d_b is the diameter of the smallest longitudinal bar in the corners of the stirrups near the top and bottom faces of the beam. The area of each stirrup, A_v, shall be equal to or greater than:

$$A_v = \frac{\sqrt{f_c'}}{12}\frac{b_w s}{f_y} \qquad \text{(Eq. 9–27)}$$ A2

9.4.4.1.7 *Diagonally reinforced coupling beams* A3

See 11.4.9.4.

9.4.5 *Buckling restraint of longitudinal bars in potential ductile and limited ductile plastic regions*

Transverse reinforcement in the form of stirrup-ties shall be placed in potential plastic regions of beams, as defined in 9.4.2 as follows:

(a) Stirrup-ties shall be arranged so that each longitudinal bar or bundle of bars in the upper and lower faces of the beam is restrained against buckling by a 90° bend of a stirrup-tie, except that where two or more bars at not more than 200 mm centres apart are so restrained, any bars between them are exempted from this requirement;

(b) The diameter of the stirrup-ties shall be equal to or greater than 5 mm, and the area of one leg of a stirrup-tie in the direction of potential buckling of the longitudinal bar shall be equal to or greater than:

$$A_{te} = \frac{\Sigma A_b f_y}{96 f_{yt}}\frac{s}{d_b} \qquad \text{(Eq. 9–28)}$$

where s is the spacing of stirrup-ties, ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 9.4.5(a) and f_{yt} shall not be taken larger than 800 MPa. Longitudinal bars centred more than 75 mm from the inner face of stirrup-ties need not be considered in determining the value of ΣA_b;

(c) If a horizontal layer of longitudinal bars is centred further than 100 mm from the inner face of the adjacent horizontal leg of stirrup-ties, the outermost bars shall be tied laterally as required in 9.4.5(b), unless this layer is situated further than h/4 from the compression edge of the section;

(d) In potential plastic hinge regions defined by 9.4.2(a) and (b) the centre-to-centre spacing of stirrup-ties for a ductile plastic region (DPR) shall not exceed the smaller of d/4 or 6 times the diameter of any longitudinal bar to be restrained in the outer layers. The centre-to-centre spacing in a limited ductile plastic region (LDPR) shall not exceed the smaller of d/4 or 10 times the diameter of any longitudinal bar to be restrained in the outer layers. Where 9.4.2(a) applies, the first stirrup-tie in a beam shall be as close as practicable to the column ties and shall be not further than 50 mm from the column face.

(e) In potential plastic hinge regions defined by 9.4.2(c) the centre-to-centre spacing of stirrup-ties shall not exceed either $d/3$ or ten times the diameter of any longitudinal compression bar to be restrained. The area of stirrup-ties need not satisfy Equation 9–28. When the potential plastic hinge region defined by 9.4.2(c) overlaps that defined by 9.4.2(a) or (b), the spacing and area of stirrup ties shall be governed by the requirements of 9.4.2(a) or (b), respectively.

(f) Stirrup-ties shall be assumed to contribute to the shear strength of the beam.

10 DESIGN OF REINFORCED CONCRETE COLUMNS AND PIERS FOR STRENGTH AND DUCTILITY

10.1 Notation

A_b area of a longitudinal bar, mm^2

A_c area of concrete core of section measured to outside of peripheral spiral or hoop, mm^2

A_{cv} area of concrete assumed to resist shear, (see10.3.10.2.1), mm^2 A2

A_g gross area of section, mm^2

A_h area of one leg of hoop or spiral bar at spacing, s, mm A3

A_{sh} total effective area of hoop bars and supplementary cross-ties in the direction under consideration within spacing s_h, mm^2

A_{st} total area of longitudinal reinforcement, mm^2

A_t area of structural steel shape or pipe, mm^2

A_{te} area of one leg of stirrup-tie, mm^2

A_{tr} smaller of area of transverse reinforcement within a spacing s crossing plane of splitting normal to concrete surface containing extreme tension fibres, or total area of transverse reinforcement normal to the layer of bars within a spacing, s, divided by n, mm^2. If longitudinal bars are enclosed within a spiral or circular hoop reinforcement, $A_{tr} = A_h$ when $n \leq 6$ (See C8.6.3.3) A3

A_{t1} minimum area of a single tie (one leg)

A_v area of shear reinforcement within a spacing s, mm^2

b width of compression face of member, mm

b_w web width or diameter of circular section, mm

C_m a factor relating actual moment diagram to an equivalent uniform moment diagram

d distance from extreme compression fibre to centroid of longitudinal tension reinforcement (for circular sections, d need not be taken less than the distance from extreme compression fibre to centroid of tension reinforcement in opposite half of member), mm A3

d'' depth of concrete core of column measured from centre-to-centre of peripheral rectangular hoop, circular hoop or spiral, mm

d_b diameter of reinforcing bar, mm

E_c modulus of elasticity of concrete, MPa, see 5.2.3

E_s modulus of elasticity of steel, MPa, see 5.3.4

EI flexural rigidity of a member. See Equations 10–6 and 10–7 for columns

f_{ct} average split cylinder tensile strength of lightweight aggregate concrete, MPa

f'_c specified compressive strength of concrete, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement or the yield strength of structural steel casing, MPa

f_{yt} lower characteristic yield strength of spiral, hoop, stirrup-tie or supplementary cross-tie reinforcement, MPa

h overall depth of member, mm

h_b overall depth of beam, mm

h'' dimension of concrete core of rectangular section, measured perpendicular to the direction of the hoop bars, measured to the outside of the peripheral hoop, mm

I_g moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4

I_{se} moment of inertia of reinforcement about centroidal axis of member cross section, mm^4

I_t moment of inertia of structural steel shape or pipe about centroidal axis of composite member section, mm^4

k effective length factor for a column or pier

ℓ_p effective length for determining curvatures in a plastic region, mm.

ℓ_y ductile detailing length, mm

L_n clear length of member measured from face of supports, mm

L_u unsupported length of a column or pier, mm

m $f_y/(0.85 f'_c)$

M_c moment to be used for design of a column or pier, N mm

M_1 value of smaller design end moment on a column or pier calculated by conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature, N mm

M_2 value of larger design end moment on a column or pier calculated by conventional elastic frame analysis, always positive, N mm

M^* design moment at section at the ultimate limit state, N mm

N_c critical load, see Equation 10–5, N

$N_{n,max}$ nominal axial load compressive strength of column when the load is applied with zero eccentricity, N

N_o^* design axial load derived from overstrength considerations (capacity design), N

N^* design axial load at ultimate limit state to be taken as positive for compression and negative for tension, N

n number of bars

p_s ratio of volume of spiral or circular hoop reinforcement to total volume of concrete core (outside to outside of spirals or hoops)

p_t ratio of non-prestressed longitudinal column reinforcement = A_{st}/A_g

A2 A3 p_w proportion of flexural tension reinforcement within one-quarter of the effective depth of the member closest to the extreme tension reinforcement to the shear area, A_{cv}. For circular or octagonal columns, p_w may be taken as 0.33 A_{st}/A_{cv}

r radius of gyration of cross section of a column or pier, mm

s centre-to-centre spacing of stirrup-ties along member, mm

s_h centre-to-centre spacing of hoop sets, mm

A3 s_t spacing of the ties that are normal to the longer side of the column

t_s thickness of steel encasing concrete in a composite member, mm

V shear force, N

A3 v_b shear resisted by concrete in an equivalent reinforced concrete beam, MPa

V_c nominal shear strength provided by the concrete mechanisms, N

V_E shear force derived from lateral earthquake forces for the ultimate limit state, N

A3 v_n total nominal shear strength of section, MPa

V_n total nominal shear strength of cross section of column or pier, N

V_s nominal shear strength provided by the shear reinforcement, N

V^* design shear force at the section at the ultimate limit state, N

A2

α_1 factor defined in 7.4.2.7

β_d ratio of design axial dead load to total design axial load of a column or pier

θ angle between the inclined crack and the horizontal axis of column or pier

ΣA_b sum of areas of longitudinal bars, mm^2

δ moment magnification factor, see 10.3.2.3.5

A3 δ_o relative displacement of the ends of a member, mm

ϕ strength reduction factor, see 2.3.2.2

10.2 Scope

The provisions of this section shall apply to the design of reinforced concrete members for flexure and shear with axial force. The provisions of this and earlier sections are summarised in Table C10.2. The written requirements take precedence over Table C10.2. Columns containing plastic regions with sectional curvature ductility demands less than or equal to the limits for nominally ductile plastic regions defined in Table 2.4 shall meet the requirements of 10.3. Columns containing plastic regions designed for greater sectional ductility demand than this and columns in frames with ductile or limited ductile beam hinges shall meet the requirements of 10.3 as modified by 10.4.

A3 Requirements for piers shall be the same as those for columns.

10.3 General principles and design requirements for columns A3

10.3.1 *Strength calculations at the ultimate limit state*

Columns shall be designed for the most unfavourable combination of design moment, M^*, design axial force, N^*, and design shear force V^*. The maximum design moment, M^*, shall be magnified for slenderness effects in accordance with 10.3.2. A3

10.3.1.1 *Dependable section cross section* A3

A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions.

10.3.2 *Slenderness effects in columns*

10.3.2.1 *Design considerations for columns*

The design of columns shall be based on forces and moments determined from a second-order analysis of the structure, or by simplified methods such as described in 10.3.2.2, or alternatively by complying with the section dimension limits given in 10.4.3. Such analysis shall take into account the influence of axial loads and variable moments of inertia due to cracking on member stiffness and end moments, the effect of deflections on moments and forces, and the effects of duration of loads, shrinkage and creep and interaction with the supporting foundations. A3 A2

10.3.2.2 *Evaluation of slenderness effects in columns braced against sidesway* A3

In lieu of the detailed procedure prescribed in 10.3.2.1, slenderness effects in columns braced against sidesway may be evaluated in accordance with the approximate procedure presented in 10.3.2.3.

10.3.2.3 *Approximate evaluation of slenderness effects*

The approximate method of evaluation of slenderness effects for columns braced against sidesway given by 10.3.2.3.1 to 10.3.2.3.6, may be used in lieu of that in 10.3.2.1 provided that: A3

(a) The member cannot form ductile or limited ductile plastic regions in the ultimate limit state;

(b) The relative displacement of the ends of the member δ_o, in the ultimate limit state is such that:

$$N^* \delta_o \le 0.05\ V^* L_u \qquad \text{(Eq. 10–1)}$$

(c) The slenderness ratio, which is given by $\frac{kL_u}{r}$, is equal to or less than 100; where L_u is defined in 10.3.2.3.1, k is defined in 10.3.2.3.2 and r is defined in 10.3.2.3.3. A3

10.3.2.3.1 *Unsupported length*

The unsupported length of a column shall be determined as follows: A3

(a) The unsupported length, L_u, shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support for that column or pier, in the direction being considered;

(b) Where column capitals or haunches are present, the unsupported length shall be measured to the lower extremity of the capital or haunch in the plane considered.

10.3.2.3.2 *Effective length factor*

The effective length factor, k, of a column braced against sidesway shall be taken as 1.0, unless analysis shows that a lower value may be used. A3

10.3.2.3.3 *Radius of gyration*

The radius of gyration, r, shall be taken equal to 0.30 times the overall dimension in the direction stability is being considered for a rectangular column, and 0.25 times the diameter for a circular column. For other shapes, r shall be computed for the gross concrete section. A3

10.3.2.3.4 *Consideration of slenderness*

In compression members braced against sidesway, slenderness effects may be ignored for compression members that satisfy:

$$\frac{kL_u}{r} \le 34 - 12\left(M_1 / M_2\right) \qquad \text{(Eq. 10–2)}$$

where the term [34 – 12 (M_1/M_2)] shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

10.3.2.3.5 *Design actions including slenderness effects*

Design actions including the effects of slenderness shall be determined as follows:

A3 (a) Columns shall be designed using the design ultimate load, N^*, from a first order analysis and a magnified ultimate moment, M_c, defined by:

$$M_c = \delta M_2 \quad \text{(Eq. 10–3)}$$

where

$$\delta = \frac{C_m}{1-\left(\dfrac{N^*}{0.75\,N_c}\right)} \geq 1.0 \quad \text{(Eq. 10–4)}$$

and

$$N_c = \frac{\pi^2 EI}{(kL_u)^2} \quad \text{(Eq. 10–5)}$$

In lieu of a more accurate calculation, EI in Equation 10–5 shall be taken either as:

$$EI = \frac{E_c I_g/5 + E_s I_{se}}{1+\beta_d} \quad \text{(Eq. 10–6)}$$

or conservatively as:

$$EI = \frac{(E_c I_g/2.5)}{1+\beta_d} \quad \text{(Eq. 10–7)}$$

A3 (b) In Equation 10–4 for columns braced against sidesway and without transverse loads between supports, C_m shall be taken as:

$$C_m = 0.6 + 0.4\,\frac{M_1}{M_2} \geq 0.4 \quad \text{(Eq. 10–8)}$$

A3 where M_1/M_2 is positive if the column is bent in single curvature, and negative if the column is bent in double curvature. For columns with transverse loads between supports, C_m shall be taken as 1.0.

In Equations 10–6 and 10–7 β_d shall be taken as the ratio of the maximum factored long-term axial load (permanent action) to the maximum factored design axial load, due to the same load combination, and shall not be taken as greater than 1.

(c) The moment M_2 in Equation 10–3 shall be taken not less than:

$$M_{2,min} = N^* (15 + 0.03h) \quad \text{(Eq. 10–9)}$$

about each axis separately. For columns for which $M_{2,min}$ exceeds M_2, the value of C_m in Equation 10–8 shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments M_1 and M_2. A3

10.3.2.3.6 *Bending about both principal axes*

For columns subject to bending about both principal axes, the moment about each axis shall be magnified by δ computed from the corresponding conditions of restraint about that axis. A3

10.3.3 *Design cross-sectional dimensions for columns* A3

10.3.3.1 *Compression member with multiple spirals*

Outer limits of the effective cross section of a column with two or more interlocking spirals shall be taken as the distance between the extreme limits of the spirals plus the minimum concrete cover around the peripheral spiral bars required by Sections 3 and 4. A3

10.3.3.2 *Equivalent circular compression member*

In lieu of using the gross area for design, a column with a square, octagonal, or other shaped cross section may be considered as a circular section with a diameter equal to the least lateral dimension of the actual shape. The required percentage of reinforcement, and design strength shall be based on that circular section. A3

10.3.4 *Strength of columns in bending with axial force* A3

10.3.4.1 *General assumptions for flexural and axial force design*

The design of columns for flexure and axial force at the ultimate limit state shall be in accordance with 7.4 and shall be based on satisfaction of applicable conditions of equilibrium and compatibility of strains. A3

10.3.4.2 *Limit for design axial force, N *, on columns*

For columns the ultimate axial load in compression, N^*, shall be less than 0.85 $\phi N_{n,max}$ for members where: A3

$$N_{n,max} = \alpha_1 f'_c (A_g - A_{st}) + f_y A_{st} \qquad \text{(Eq. 10–10)}$$

where α_1 is given by 7.4.2.7(c).

10.3.5 *Transmission of axial force through floor systems*

10.3.5.1 *Transmission of load through floor system*

When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be as provided for by one of 10.3.5.2, 10.3.5.3 or 10.3.5.4.

10.3.5.2 *Placement of concrete in floor*

Concrete of the strength specified for the column shall be placed in the floor at the column location. The top surface of the column concrete shall extend 600 mm into the slab from the face of the column. The column concrete shall be well integrated into the floor concrete.

10.3.5.3 *Strength of column through floor*

The strength of a column through a floor system shall be based on the lower value of concrete strength. To achieve a column strength through the floor system comparable with the column above and below the slab, supplementary reinforcement in the column, through the floor system, fully developed above and below the slab and adequately confined, may be provided.

10.3.5.4 *Strength of columns laterally supported on four sides*

For columns laterally supported on four sides by beams of approximately equal depth or by slabs, the strength of the column may be based on an assumed concrete strength in the column joint equal to 75 % of the column concrete strength plus 35 % of the floor concrete strength. In the application of this clause, the ratio of column concrete strength to slab concrete strength shall not be taken greater than 2.5 for design.

10.3.6 *Perimeter columns to be tied into floors*

A2 Columns at the perimeter of a floor shall be tied back into the floor by either reinforced concrete beams or tie reinforcement provided in the topping. The tie reinforcement shall be effectively anchored perpendicular to the frame and capable of resisting the larger of 5 % of the maximum total axial compression load acting on the column at the level being considered, or 20 % of the shear force induced by the seismic design actions in the column in the storey below the level being considered.

10.3.7 *Strength of columns in torsion, shear and flexure*

A3 The design of columns for torsion, shear and flexure at the ultimate limit state shall be in accordance with 7.5 and 7.6, and with 10.3.1, 10.3.4 and 10.3.10.2.

10.3.8 *Longitudinal reinforcement in columns*

10.3.8.1 *Limits for area of longitudinal reinforcement*

The area of longitudinal reinforcement for columns shall be greater than 0.008 times the gross area, A_g, of the section, and less than 0.08 times the gross area, A_g, at any location including lap splices.

10.3.8.2 *Minimum number of longitudinal bars*

A3 A2 The minimum number of longitudinal bars in a column shall be 8, except that this number may be reduced to 4 or 6 where the clear spacing between adjacent bars on the same side of the section is less than 150 mm and the axial load $N^* \leq 0.1\phi f'_c A_g$.

10.3.8.3 *Spacing of longitudinal reinforcement*

The centre-to-centre spacing of longitudinal bars in a circular column shall be less than or equal to the larger of one-quarter of the diameter of the section, or 200 mm.

In rectangular sections the maximum permissible centre-to-centre spacing of longitudinal bars, which are cross linked across the cross section, shall depend on the ratio of the longer side, h, to the shorter side, b, as set out in (a) and (b) below.

(a) Where the ratio of h/b < 2.0 the maximum permissible spacing shall be the larger of $b/3$ or 200mm.

(b) Where the ratio of h/b > 2.0 the maximum spacing shall be as for (a) except in the mid regions of the longer side. In the mid region lying between lines drawn at a distance of the larger of b or 1.5 times the depth to the neutral axis from the extreme fibres, the spacing may be increased to the smaller of $h/4$ or 300 mm.

10.3.8.4 *Cranking of longitudinal bars*

Where longitudinal bars are offset, the slope of the inclined portion of the bar with the axis of the column shall be less than or equal to 1 in 6, and the portions of the bar above and below the offset shall be parallel to the face of the column. Adequate horizontal support at the offset bends shall be provided by ties, spirals, other means of restraints or parts of the floor construction. These shall be placed so that the resultant force, providing the horizontal support for the bursting forces, acts through the centre of the bend. The horizontal thrust to be resisted shall be assumed as 1.5 times the horizontal component of the nominal force in the inclined portion of the bar, assumed to be stressed to f_y.

10.3.9 *Splices of longitudinal reinforcement*

10.3.9.1 *General*

A3 Splices in the longitudinal reinforcement of columns shall comply with 8.7.

10.3.9.2 *Offset column faces*

Where column faces are offset 75 mm or more, splices of vertical bars adjacent to the offset face shall be made by separate reinforcing bars lapped as required herein.

10.3.9.3 *Laps designed for full yield stress when stress exceeds 0.5 f_y.*

Where the stress in the longitudinal bars in a column calculated for any loading condition exceeds 0.5 f_y in tension, either lap splices designed for full yield stress in tension shall be used, or full strength welded splices in accordance with 8.7.4.1(a), high strength welded splices, or high strength mechanical connections in accordance with 8.7.4.1(b) and 8.7.5.2 respectively shall be provided.

10.3.10 *Transverse reinforcement in columns* A3

10.3.10.1 *General*

Transverse reinforcement shall satisfy the requirements of shear, torsion, confinement of concrete, and lateral restraint of longitudinal bars against premature buckling. The maximum area required for shear combined with torsion, confinement, or control of buckling of bars shall be used.

10.3.10.2 *Design for shear*

Design for shear reinforcement shall be in accordance with 7.5. Where the reaction, in the direction of applied shear, introduces compression to the end regions of continuous or cantilever columns, the maximum design shear force V * at the ultimate limit state for sections located at less than distance d from the face of the support may be taken as that computed at distance d from the face of the support.

10.3.10.2.1 *Maximum permissible nominal shear force and effective shear area*

The maximum total nominal shear stress, v_n, shall not exceed $0.2f'_c$ or 6 MPa. The value of the effective shear area, A_{cv}, shall: A3

(a) For rectangular, T- and I-section shapes be taken as the product of the web width, b_w, times the effective depth, d;

(b) For octagonal, circular, elliptical, and similarly shaped sections be taken as the area enclosed by the transverse reinforcement with the area of outstanding compression or tension flanges being neglected.

10.3.10.2.2 *Method of design for shear*

For columns a truss analogy shall be used to calculate the contribution of shear reinforcement to shear strength. Either: A3

(a) The strut and tie method may be used, in which case V_c shall be taken as zero; or

(b) V_c shall be calculated from 10.3.10.3 and V_s calculated from 10.3.10.4.

In either case the requirements of 10.3.10.4.3 to 10.3.10.4.4 shall apply.

10.3.10.3 *Shear strength provided by concrete*

10.3.10.3.1 *Nominal shear strength provided by the concrete for normal density concrete*

For normal density concrete V_c shall be taken as not greater than:

$$V_c = k_a k_n v_b A_{cv} \qquad \text{(Eq. 10–11)}$$

where:

k_a is equal to 1.0 for maximum aggregate size of 19 mm or more and equal to 0.85 for a maximum aggregate size of 10 mm. Interpolation may be used for intermediate sizes. A3

v_b is given by

$$v_b = (0.07 + 10\,p_w)\sqrt{f'_c} \qquad \text{(Eq. 10–12)}$$

with limits of

$$0.08\sqrt{f'_c} < v_b < 0.2\sqrt{f'_c} \qquad \text{(Eq. 10–13)}$$

In the calculation of v_b the value of f'_c shall not be taken greater that 50 MPa.

A3

A_{cv} is defined in 10.3.10.2.1;

k_n allows for the influence of axial load and it is given for members subjected to axial compression by:

$$k_n = \left[1 + \frac{3N^*}{A_g f'_c}\right] \quad \text{(Eq. 10–14)}$$

A3 where $\frac{N^*}{A_g f'_c}$ shall not be taken greater than 0.3.

and for members subjected to axial tension, where N^* is negative for tension, k_n is given by:

$$k_n = \left[1 + \frac{12N^*}{A_g f'_c}\right] \geq 0 \quad \text{(Eq. 10–15)}$$

10.3.10.3.2 *Change in shear strength in members where sides are not parallel to the longitudinal axis*

A2 A3 In members where the sides are not parallel to the longitudinal axis, allowance shall be made for any decrease in shear resistance due to the transverse component of the compression and tension forces due to flexure and axial load. The decrease in shear strength is associated with the depth increasing in the direction of decreasing moment.

10.3.10.3.3 *Nominal shear strength provided by the concrete for lightweight concrete*

Provisions for shear strength provided by the concrete, V_c, apply to normal density concrete. Where lightweight aggregate concrete is used one of the following modifications shall apply:

(a) Where f_{ct} is specified and the concrete mix is designed in accordance with NZS 3152, provisions for V_c shall be modified by substituting $1.8f_{ct}$ for $\sqrt{f'_c}$ but the value of $1.8f_{ct}$ shall not exceed $\sqrt{f'_c}$;

(b) Where f_{ct} is not specified, all values of $\sqrt{f'_c}$ affecting V_c shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be applied when partial sand replacement is used.

10.3.10.4 *Shear reinforcement*

10.3.10.4.1 *Required nominal shear strength from reinforcement*

In accordance with 7.5, the required shear strength shall be computed using:

$$V_s = \frac{V^*}{\phi} - V_c \quad \text{(Eq. 10–16)}$$

where V_c is the nominal shear strength provided by the concrete given in 10.3.10.3 and V_s is the shear strength provided by the shear reinforcement given in 10.3.10.4.2.

10.3.10.4.2 *Nominal shear strength provided by shear reinforcement*

When shear reinforcement perpendicular to the longitudinal axis of columns is used:

(a) for rectangular hoops or ties:

$$V_s = \frac{A_v f_{yt} d}{s} \quad \text{(Eq. 10–17)}$$

(b) For circular hoops or spirals:

$$V_s = \frac{\pi}{2}\frac{A_h f_{yt} d''}{s} \quad \text{(Eq. 10–18)}$$

where d'' is the depth of the core dimension from centre-to-centre of peripheral hoop or spiral, and where A_h is the area of one leg of hoop or spiral bar at spacing, s. A3

(c) For other sections where the angle of the shear reinforcement intersecting a potential diagonal tension crack varies in direction, only the component of the shear reinforcement which is parallel to the shear force shall be included.

10.3.10.4.3 *Maximum spacing of shear reinforcement*

Where the nominal shear resisted by the reinforcement, V_s, exceeds a value of $0.33\sqrt{f'_c}\,A_{cv}$, the spacing shall not exceed $d/4$.

10.3.10.4.4 *Minimum shear strength provided by shear reinforcement*

The area of shear reinforcement shall be equal to or greater than:

A3

(a) For rectangular and T-shaped sections:

$$A_v = \frac{1}{16}\sqrt{f'_c}\,\frac{b_w s}{f_{yt}} \quad \text{(Eq. 10–19)}$$

A3

(b) For circular sections:

$$A_h = \frac{1}{32}\sqrt{f'_c}\,\frac{d'' s}{f_{yt}} \quad \text{(Eq.10-19(a))}$$

10.3.10.5 *Design of spiral or circular hoop transverse reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.3.10.5.1 *Confinement and anti-buckling reinforcement*

The volumetric ratio, p_s, shall be equal to or greater than that given by the greater of Equation 10–20 or Equation 10–21 for confinement of concrete and lateral restraint of longitudinal bars:

(a) For confinement of concrete:

A3

A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions;

$$p_s = \frac{(1 - p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084 \quad \text{(Eq. 10–20)}$$

where N^* is the maximum design axial load for load combinations involving wind or seismic actions, or any other load case in which significant lateral force is applied to the structure as a whole;

The value of $p_t m$ used in the equation shall not be taken greater than 0.4.

A3

(b) For lateral restraint of longitudinal bars against premature buckling:

$$p_s = \frac{A_{st}}{155\,d''}\frac{f_y}{f_{yt}}\frac{1}{d_b} \quad \text{(Eq. 10–21)}$$

In Equations 10–20 and 10–21, f_{yt} shall not exceed 500 MPa. A3

10.3.10.5.2 *Spacing of spirals or circular hoops*

A3 The centre-to-centre spacing of spirals or circular hoops along the member shall be less than or equal to the smaller of one-third of the diameter of the cross section of the member or ten times the diameter of the smallest longitudinal bar to be restrained. Clear spacing shall be equal to or greater than 25 mm.

10.3.10.6 *Design of rectangular hoop and tie transverse reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.3.10.6.1 *Confinement and anti-buckling reinforcement*

A3 The total effective area of reinforcement in each of the principal directions of the cross section within spacing s_h shall be greater than that given by Equation 10–22 or 10–23:

For confinement of concrete:

A3 A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions.

$$A_{sh} = \frac{(1-p_t m)s_h h''}{3.3}\frac{A_g}{A_c}\frac{f_c'}{f_{yt}}\frac{N^*}{\phi f_c' A_g} - 0.0065 s_h h'' \quad \text{(Eq. 10–22)}$$

A3 The value of $p_t m$ used in the equation shall not be taken greater than 0.4.

N^* is the maximum design axial load for load combinations involving wind or seismic actions, or any other load case in which significant lateral force is applied to the structure as a whole.

For lateral restraint of longitudinal bars against premature buckling:

A3 No individual leg of a stirrup-tie shall have an area less than that given by Equation 10–23.

$$A_{te} = \frac{\sum A_b f_y}{135 f_{yt}}\frac{s_h}{d_b} \quad \text{(Eq. 10–23)}$$

where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars between longitudinal bars restrained in accordance with 10.3.8.3.

A3 In Equations 10–22 and 10–23, f_{yt} shall not exceed 500 MPa.

10.3.10.6.2 *Spacing of tie sets*

A3 The centre-to-centre spacing of the tie sets along the member shall be less than or equal to the smaller of one-third of the least lateral dimension of the cross section, or ten times the diameter of the smallest longitudinal bar to be restrained.

10.3.10.6.3 *Support of longitudinal bars*

Each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following two cases of bars are exempt from this requirement:

(a) Bars or bundles of bars which lie between two laterally supported bars or bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed the larger of one-third of the lateral dimension of the cross section in the direction of the spacing or 200 mm;

A3 (b) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inside face of hoop bars.

10.3.10.7 *Minimum diameter of transverse reinforcement*

10.3.10.7.1 *Minimum diameters for rectangular hoops and ties*

Rectangular hoop or tie reinforcement shall be at least 5 mm in diameter for longitudinal bars less than 20 mm in diameter, 10 mm in diameter for longitudinal bars from 20 to 32 mm in diameter and 12 mm in diameter for longitudinal bars larger than 32 mm in diameter and for bundled longitudinal bars.

10.3.10.7.2 *Minimum diameters for spiral and circular hoops*

Spiral or circular hoop reinforcement shall be of a size and assembly that permits handling and placing without distortion from its designed dimensions. Spiral or circular hoop bars shall be equal to or greater than 5 mm in diameter. A3

10.3.10.8 *Anchorage of transverse reinforcement*

(a) Except where permitted by 8.7.2.8, transverse reinforcement shall not be anchored by lap splicing; A2
(b) Spirals shall be anchored by either welding to the previous turn, in accordance with 8.7.4.1(b) or by terminating the spiral with at least a 135° stirrup hook, engaging a longitudinal bar and with the stirrup hook being a clear distance away from the previous turn of not more than 25 mm;
(c) Circular or rectangular hoops shall be anchored by either a mechanical connection or welded splice in accordance with 8.7.4.1(b), or by terminating each end of the hoop with at least a 135° stirrup hook, overlapping the other end and engaging a longitudinal bar. Each end of a cross tie shall engage a longitudinal bar with at least a 135° stirrup hook.

10.3.10.9 *Set out of transverse reinforcement at column ends*

At the ends of columns, the spacing of transverse reinforcement shall be: A3

(a) Located vertically not more than 75 mm above the top of the footing or slab in any storey, and not more than 75 mm below the lowest horizontal reinforcement in members supported above;
(b) Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spirals or circular hoops to bottom of slab or drop panel;
(c) In columns with capitals, transverse reinforcement shall extend to a level at which the diameter or width of capital is twice that of the column; A3
(d) For column bars that are not restrained against buckling by beams, the distance between the first tie in the column and that within the beam-column joint shall not exceed six times the diameter of the column bar to be restrained.

10.3.11 ***Composite compression members***

10.3.11.1 *General*

Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipes, or tubing with or without longitudinal bars.

10.3.11.2 *Strength*

Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

10.3.11.3 *Axial load strength assigned to concrete*

Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

10.3.11.4 *Axial load strength not assigned to concrete*

All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.3.11.5 *Slenderness effects*

Slenderness effects shall be provided for by methods based on a fundamental analysis. Alternatively they may be based on a radius of gyration of a composite section given by:

A3

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t}{(E_c A_g / 5) + E_s A_t}} \qquad \text{(Eq. 10–24)}$$

and, as an alternative to a more accurate calculation, EI in Equation 10–5 shall be taken either as Equation 10–6 or:

$$EI = \frac{(E_c I_g / 5)}{1 + \beta_d} + E_s I_t \qquad \text{(Eq. 10–25)}$$

10.3.11.6 *Structural steel encased concrete core*

10.3.11.6.1 *Steel encased concrete core*

For a composite member with a concrete core encased by structural steel, the thickness of the steel encasement shall be equal to or greater than:

$t_s > b\sqrt{\frac{f_y}{3E_s}}$ for each face of width b

or

$t_s > h\sqrt{\frac{f_y}{8E_s}}$ for circular sections of diameter h

where, for the purpose of this clause, f_y is the yield stress of the structural steel casing.

10.3.11.6.2 *Longitudinal bars*

Longitudinal bars located within the encased concrete core may be used in computing A_t and I_t.

10.4 Additional design requirements for members designed for ductility in earthquakes

10.4.1 *Strength calculations at the ultimate limit state*

The design of cross sections subjected to flexure with or without axial load shall be consistent with 7.4.2.

10.4.2 *Protection of columns at the ultimate limit state*

For frames where sidesway mechanisms with plastic hinges forming only in columns are not permitted at the ultimate limit state, the design moments and axial loads on columns shall include the effect of possible beam overstrength, concurrent seismic forces, and magnification of column moments due to dynamic effects, in order to provide a high degree of protection against the formation of a column sway mechanism.

10.4.3 *Dimensions of columns*

A3 **10.4.3.1** *General*

A3 For columns, which sustain plastic regions in the ultimate limit state in load combinations involving seismic actions, either an analysis based on first principles shall be made to demonstrate that the member is
A3 stable, or the dimensional limits given in 10.4.3.2, 10.4.3.3 and 10.4.3.4 shall be satisfied.

10.4.3.2 *Columns in framed structures*

The depth, width and clear length between the faces of supports of members with rectangular cross sections, to which moments are applied at both ends by adjacent beams, columns or both, shall be such that:

$$\frac{L_n}{b_w} \le 25 \qquad \text{(Eq. 10–26)}$$

and

$$\frac{L_n h}{b_w^2} \leq 100 \quad \text{(Eq. 10–27)}$$

10.4.3.3 *Cantilevered columns*

The depth, width and clear length from the face of support of cantilever members with rectangular cross sections, excluding bridge piers, shall be such that:

$$\frac{L_n}{b_w} \leq 15 \quad \text{(Eq. 10–28)}$$

and

$$\frac{L_n h}{b_w^2} \leq 60 \quad \text{(Eq. 10–29)}$$

10.4.3.4 *Web width of T - and L - member*

The width of web of T- and L- members, in which the flange or flanges are integrally built with the web, shall be such that the values given by Equations 10–26 and 10–28 are not exceeded by more than 50 %.

10.4.3.5 *Compression face width of T-, L- or I- members*

The width of the compression face of a member with rectangular, T-, L- or I- section shall be greater than or equal to 200 mm.

10.4.3.6 *Narrow beams and wide columns*

Where narrow beams frame into columns, the width of a column that shall be assumed to resist the forces transmitted by the beam shall be in accordance with 15.3.4.

10.4.4 ***Limit for design axial force on columns*** A3

For columns the maximum design load in compression, N_o^*, shall be less than $0.7N_{n,max}$ where:

$$N_{n,max} = \alpha_1 f'_c (A_g - A_{st}) + f_y A_{st} \quad \text{(Eq. 10–30)}$$

where α_1 is given by 7.4.2.7(c).

10.4.5 ***Ductile detailing length***

Ductile detailing lengths, ℓ_y, of end regions in columns adjacent to moment-resisting connections shall be the greater of: A3

(a) (i) $\ell_y = h$ for $N_o^* \leq 0.25\ f'_c A_g$,

(ii) $\ell_y = 2.0h$ for $0.25\ f'_c A_g < N_o^* \leq 0.5\ f'_c A_g$

(iii) $\ell_y = 3.0h$ for $0.5\ f'_c A_g < N_o^* \leq 0.7\ N_{n,max}$

where

h is the diameter of a circular cross section or the dimension in the direction resisting the applied moment of a rectangular section

or

(b) The length from the joint over which the design moment, taking into account dynamic magnification and overstrength actions, is greater than the following proportions of the moment at the end of the member:

(i) 0.8 for $N_o^* \leq 0.25\ f'_c A_g$ A3

(ii) 0.7 for $0.25\ f'_c A_g < N_o^* \leq 0.5\ f'_c A_g$

(iii) 0.6 for $0.5\ f'_c A_g < N_o^* \leq 0.7\ N_{n,max}$

10.4.6 ***Longitudinal reinforcement in columns***

10.4.6.1 *Longitudinal reinforcement*

Longitudinal reinforcement in columns shall be as required by 10.4.6.2 to 10.4.6.7. A3

10.4.6.2 *Maximum area of longitudinal reinforcement*

The area of longitudinal reinforcement shall be not greater than $18A_g/f_y$ except that in the region of lap splices the total area shall not exceed $24A_g/f_y$.

A3 **10.4.6.3** *Spacing of longitudinal bars in plastic hinge regions*

The spacing of longitudinal bars in potential plastic regions, as defined in 10.4.5, shall satisfy the appropriate limits given in (a) and (b) below.

(a) For a member with a circular cross section, the centre-to-centre spacing between longitudinal bars shall be less than or equal to the larger of one-quarter of the diameter of the section or 200 mm.

(b) In rectangular sections the maximum permissible centre-to-centre spacing of longitudinal bars, which are cross linked across the cross section, shall depend on the ratio of the longer side, *h*, to the shorter side, *b*, as set out in (i) and (ii);

  (i) Where the ratio of h/b < 2.0 the maximum permissible spacing shall be the larger of $b/4$ or 200mm;

A3 (ii) Where the ratio of $h/b \geq 2.0$ the maximum spacing shall be as for (i) except in the mid-region of the longer dimension, *h*, which lies between lines that are normal to the longer dimension and located at a distance from the extreme fibres of the column equal to the larger of *b* or 1.5 times the depth to the neutral axis for bending about the major axis. In the mid-region the maximum spacing may be increased to be equal to or smaller than the smaller of $h/4$ or 300 mm.

10.4.6.4 *Spacing of longitudinal reinforcement in columns*

The spacing of longitudinal bars given in 10.4.6.3 may be relaxed to those in 10.3.8.3 for the regions of the column defined below:

A3 (a) for regions of column located outside the ductile detailing lengths, ℓ_y, identified in 10.4.5.

(b) for columns designed using method A of Appendix D within the ductile detailing lengths defined in D3.1 (Method A) as providing a high level of protection against the formation of plastic regions.

10.4.6.5 *Anchorage of column bars in beam-column joints*

10.4.6.5.1 *Termination of bars in potential plastic hinge regions*

Where column bars terminate in beam-column joints or joints between columns and foundation members and where a plastic hinge in the column may be expected, the anchorage of the longitudinal column bars into the joint region shall be assumed to commence at one-half of the depth of the beam or $8d_b$, whichever is less, from the face at which the column bar enters the beam or foundation member. When it is shown that a column plastic hinge adjacent to the beam face cannot occur, the development length shall be considered to commence from the beam face of entry.

10.4.6.5.2 *Termination of bars in joint*

Notwithstanding the adequacy of the anchorage of a column bar into an intersecting beam, no column bar shall be terminated in a joint area without a horizontal 90° standard hook or equivalent anchorage device as near the far face of the beam as practically possible, and not closer than three-quarters of the depth of the beam to the face of entry. Unless a column is designed to resist only axial forces, the direction of the horizontal leg of the bend must always be towards the far face of the column.

10.4.6.6 *Maximum longitudinal column bar diameter in beam-column joint zones*

The maximum diameter of longitudinal bars passing through a beam-column joint zone shall satisfy the appropriate requirement of (a) or (b) given below:

(a) Where columns have been designed by Method B in Appendix D, or by Method A in Appendix D and the joint zone being considered is below the mid-height of the second storey:

$$\frac{d_b}{h_b} \leq 3.2\frac{\sqrt{f_c'}}{f_y} \qquad \text{(Eq. 10–31)}$$

(b) Where columns have been designed by Method A and the joint zone being considered is above the mid-height of the second storey, the maximum diameter is given by:

$$\frac{d_b}{h_b} \leq 4.0\frac{\sqrt{f_c'}}{f_y} \qquad \text{(Eq. 10–32)}$$

This requirement need not be met if it is shown that stresses in extreme column bars during an earthquake remain in tension or compression over the whole bar length contained within the joint.

10.4.6.7 *Detailing of column bars passing through beam-column joints*

Longitudinal column bars passing through the joint must be extended straight through joints of the type covered by 10.4.6.6(a). Where longitudinal column bars within or near joints of the type covered by 10.4.6.6(b) are offset, the slope of the inclined bars with the axis of the column shall not exceed 1 in 6, and horizontal ties at the bend, in addition to those otherwise required by 15.4.4, shall be provided to carry 1.5 times the horizontal thrust developed by the column bars at yield stress.

10.4.6.8 *Splices of longitudinal reinforcement*

10.4.6.8.1 *General*

Splices in the longitudinal reinforcement of columns shall comply with 8.7. A3

10.4.6.8.2 *Location of splices in reinforcement*

Full strength welded splices meeting the requirements of 8.7.4.1(a) may be used in any location. For all other splices the following restrictions apply:

(a) In a column in a building the centre of the splice must be within the middle quarter of the storey height of the column unless it can be shown that there is a high degree of protection against the formation of hinges adjacent to the beam faces;

(b) In a bridge column or pier no part of a splice shall be located within a distance of the member depth from a section where the reinforcement may reach $0.9f_y$ when overstrength moments act in the column;

(c) Reinforcement in columns or piers shall not be spliced by lapping in a region where stresses at the ultimate limit state may exceed $0.6f_y$ in tension or compression unless each spliced bar is confined by stirrup-ties so that: A3

$$\frac{A_{tr}}{s} \geq \frac{d_b f_y}{48 f_{yt}} \qquad \text{(Eq. 10–33)}$$

A3

10.4.7 Transverse reinforcement in columns

10.4.7.1 *Transverse reinforcement quantity*

Transverse reinforcement shall satisfy the requirements of shear, confinement of concrete and lateral restraint of longitudinal bars against premature buckling. The maximum area required for shear combined with torsion, or for confinement, or for control of buckling of bars, shall be used. A3

10.4.7.2 *Design for shear*

10.4.7.2.1 *Design shear force*

The design shear force of columns subjected to combined flexure and axial load shall be determined from the consideration of forces on the member, with the combination of maximum likely end moments which gives the maximum shear. A3

The minimum nominal shear strength permitted in a column, at the ultimate limit state, shall be equal to or greater than: A3

(a) For a building with more than one storey, 1.6 times the shear force V_E;

(b) For a building with one storey or a bridge, 1.5 times V_E;

(c) In the first storey of a building with two or more storeys, or in any structure where lateral seismic forces or elongation can cause plastic hinges to form at both ends of the member, the design shear shall not be less than the sum of the overstrength moments at each end divided by the clear distance between the critical sections of the plastic hinges.

A3 (d) As required by method A or B (in Appendix D) being used in the design of the columns.

A2

10.4.7.2.2 *Types of potential plastic hinges in columns*

Detailing in the ductile detailing length shall be suitable for the type of potential plastic region defined in (a) or (b) below as appropriate:

(a) Columns shall be designed with ductile potential plastic regions where any of the following applies:
   (i) The columns have been designed with a low level of protection against the formation of plastic hinges as in Method B in Appendix D;
   (ii) Where the section ductility exceeds the limit for limited ductile plastic regions;
   (iii) In the lower one and a half storeys of multi-storey buildings designed with a high level of protection against the formation of plastic hinges as in Method A in Appendix D.

(b) Columns shall be designed with ductile or limited ductile plastic regions where either:
   (i) The section ductility is equal to or less than the limit for limited ductile plastic regions, or
   (ii) In columns in multi-storey buildings designed with a high level of protection against the formation of plastic hinges in levels above the first one and a half storeys as in Method A in Appendix D.

10.4.7.2.3 *Design of shear reinforcement*

The design of shear reinforcement in columns shall comply with 7.5 and 10.3.10.2, 10.3.10.3 and 10.3.10.4, except where specifically noted in (a) or (b) as appropriate below:

(a) In regions outside ductile detailing lengths, 10.3.10.4.4 shall be replaced by 10.4.7.2.7;

(b) Within the ductile detailing lengths of columns the design of shear reinforcement shall be either by:
   (i) The strut and tie method with 10.3.10.2.2(a) being replaced by 10.4.7.2.4 and 10.3.10.4.4 replaced by 10.4.7.2.7; or
   (ii) Satisfying the requirements of 10.4.7.2.5.

10.4.7.2.4 *Strut and tie method for shear design*

A2

Where the strut and tie method is used within ductile detailing lengths, the following conditions shall apply:

(a) The shear resistance of the concrete in the flexural tension zone shall be taken as zero;

(b) The angle between the diagonal compression struts and the flexural tension reinforcement shall be equal to or greater than 45°.

10.4.7.2.5 *Conventional method of shear design*

Shear resistance provided by concrete shall be computed from 10.3.10.3, but with 10.3.10.3.1 being replaced by 10.4.7.2.6.

Shear resistance provided by reinforcement shall satisfy the requirements of 10.3.10.4, but 10.3.10.4.4 shall be replaced by 10.4.7.2.7.

A3

10.4.7.2.6 *Nominal shear stress provided by the concrete in columns*

A2

The nominal shear strength provided by concrete in columns shall be taken as the product of the shear area, A_{cv}, times the nominal shear stress, v_c, given in (a), (b) or (c) as appropriate.

(a) In regions outside ductile detailing lengths, the nominal shear stress resistance of concrete, v_c, is given by 10.3.10.3.1;

A3

(b) Within the ductile detailing length for ductile plastic regions, the nominal shear resistance provided by concrete, v_c, is given by:

$$v_c = 3v_b\left[\frac{N_o^*}{A_g f_c'} - 0.1\right] \geq 0.0 \qquad \text{(Eq. 10–34)}$$

where $\left[\frac{N_o^*}{A_g f_c'}\right] \leq 0.3$

(c) Within the ductile detailing length for limited ductile plastic regions, the nominal shear stress resistance of concrete, v_c, is given by: A2

$$v_c = v_b\left[0.5 + 3\left(\frac{N_o^*}{A_g f_c'} - 0.1\right)\right] \geq 0.0 \qquad \text{(Eq.10–35)}$$

where $\left[\frac{N_o^*}{A_g f_c'}\right] \leq 0.3$ A3

$$v_c = 0.0 \text{ for } \frac{N_o^*}{A_g f_c'} \leq -0.067 \qquad \text{(Eq.10–36)}$$

where v_b is given by 10.3.10.3.1 and N_o^* is taken as positive for compressive axial loads. A2

10.4.7.2.7 *Minimum shear reinforcement*

The area of shear reinforcement shall be equal to or greater than:

$$A_v = \frac{1}{12}\sqrt{f_c'}\,\frac{b_w s}{f_{yt}} \qquad \text{(Eq. 10–37)}$$

10.4.7.3 *Alternative design methods for concrete confinement and lateral restraint of longitudinal bars*

In lieu of the methods specified in 10.4.7.4 and 10.4.7.5, moment-curvature analysis may be conducted in order to achieve the calculated curvature ductility factor in the potential plastic hinge regions at the ultimate limit state. Such an analysis shall be conducted using stress-strain relations for confined concrete and reinforcing steel, satisfying the requirements of equilibrium and compatibility of strains with allowance for reversed cyclic loading, to determine the transverse reinforcement required for concrete confinement and lateral restraint of longitudinal bars against premature buckling.

10.4.7.4 *Design of spiral or circular hoop reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.4.7.4.1 *In ductile potential plastic hinge regions*

In the ductile detailing lengths of potential plastic regions as defined in 10.4.5 and 10.4.7.2.2, where spirals or circular hoops are used, the volumetric ratio, p_s, shall be equal to or larger than that given by the greater of Equation 10–38 or Equation 10–39. A2

(a) For confinement of concrete A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions. The required confinement reinforcement is given by:

$$p_s = \frac{(1.3 - p_t m)}{2.4}\frac{A_g}{A_c}\frac{f_c'}{f_{yt}}\frac{N_o^*}{f_c' A_g} - 0.0084 \qquad \text{(Eq. 10–38)}$$

Where the value of $p_t m$ used in the equation shall not be taken greater than 0.4.

(b) For lateral restraint of longitudinal bars against premature buckling:

$$p_s = \frac{A_{st}}{110d''}\frac{f_y}{f_{yt}}\frac{1}{d_b} \qquad \text{(Eq. 10–39)}$$

In Equations 10–38 and 10–39 f_{yt} shall not exceed 500 MPa. A3

A3 **10.4.7.4.2** *In limited ductile potential plastic hinge regions*

A2 Where spiral or circular hoops are used in the ductile detailing lengths defined in 10.4.5, for limited ductile plastic regions as defined in 10.4.7.2.2(b), the quantity of transverse reinforcement provided shall be equal to or larger than the greater of that given by Equation 10–39, or 70 % of that required by Equation 10–38.

10.4.7.4.3 *In columns protected against plastic hinging*

A2 In frames where columns are designed with sufficient strength to provide a high degree of protection against plastic hinging, the required quantity of transverse reinforcement placed in the regions of columns defined as ductile detailing lengths in potential plastic hinge regions in 10.4.5 shall be the larger of that required by Equation 10–21, or 70 % of that required by Equation 10–38. This reduction in the quantity of transverse reinforcement in potential plastic hinge regions shall not be permitted at the top and bottom of the columns below the mid height of the first elevated storey (one and a half storeys above the primary plastic hinges at the base of the columns), nor in a column in any other storey where the column may form a primary plastic hinge at the top and/or bottom of the column in the storey.

10.4.7.4.4 *In regions outside potential plastic hinge regions*

A2 Outside the plastic hinge ductile detailing lengths defined in 10.4.5, transverse reinforcement shall be as required by 10.3.10.5.

A3 **10.4.7.4.5** *Spacing of spirals or circular hoop reinforcement in columns*

When spiral or circular hoop reinforcement is used the spacing shall be as follows:

A2 (a) In a ductile detailing length of a ductile potential plastic hinge region as defined by 10.4.5, and 2.6.1.3 the centre-to-centre spacing of spirals or circular hoops along the member shall be equal to or less than the smaller of one-quarter of the diameter of the cross section of the member or six times the diameter of the longitudinal bar to be restrained;

A2 (b) In a ductile detailing length of a limited ductile potential plastic hinge regions and in potential plastic hinge regions with a high degree of protection against plastic hinging, the centre-to-centre spacing of spirals or circular hoops along the member shall be equal to or less than the smaller of one-quarter of the diameter of the cross section of the member or ten times the diameter of the longitudinal bar to be restrained;

A2 (c) Outside the ductile detailing lengths of potential plastic hinge regions and in potential plastic hinge regions with a high degree of protection against plastic hinging, the centre-to-centre spacing of transverse reinforcement along the member shall be equal to or less than the smaller of one-third of the diameter of the column or ten times the diameter of the longitudinal bar to be restrained.

10.4.7.5 *Design of rectangular hoop or tie reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.4.7.5.1 *In ductile potential plastic hinge regions*

A2 In the ductile detailing lengths as defined in 10.4.5 of potential ductile plastic regions as defined in 10.4.7.2.2, where rectangular hoops or ties are used the total effective area of hoop bars and supplementary cross-ties in each of the principal directions of the cross section within spacing, s_h, shall be the greater of Equation 10–40 or Equation 10–41.

(a) For confinement of concrete A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions. The required confinement reinforcement is given by:

A3

$$A_{sh} = \frac{(1.3 - p_t m)s_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*_o}{f'_c A_g} - 0.006 s_h h'' \quad \text{(Eq. 10–40)}$$

Where the value of $p_t m$ used in the equation shall not be taken greater than 0.4.

(b) For rectangular columns where $h/b > 2.0$ and $\frac{N_o^*}{f'_c A_g} \leq 0.25$ and the seismic forces induce bending moments about the strong axis, the rectangular hoop and tie reinforcement normal to the longer side, h, shall be equal to or greater than the appropriate value given below (where A_{sh} is given by Equation 10–40, and s_t is the spacing of the ties which are normal to the longer side of the column): A3

- (i) In the end regions of the column, which are outside the mid-region defined in 10.4.6.3, the minimum area of a single tie (one leg), A_{t1}, is given by $A_{t1} \geq \frac{A_{sh}}{h''} s_t$
- (ii) In the mid-region defined in 10.4.6.3 the minimum area of a tie leg may be reduced to $A_{t1} > 0.5 \frac{A_{sh}}{h''} s_t$
- (iii) In all cases the total area of transverse reinforcement shall equal or exceed the area specified in Equation 10–22 of 10.3.10.6.1
- (iv) Equation 10–41 shall be satisfied everywhere;

(c) For lateral restraint of longitudinal bars against premature buckling:

The area of each leg of a hoop bar or cross tie in the direction of potential buckling of the longitudinal bar shall be given by:

$$A_{te} = \frac{\Sigma A_b f_y}{96 f_{yt}} \frac{s_h}{d_b} \quad \text{(Eq. 10–41)}$$

where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 10.4.7.6. Longitudinal bars centred more than 75 mm from the inner face of stirrup-ties need not be considered in determining the value of ΣA_b.

In Equations 10–40 and 10–41 f_{yt} shall not exceed 500 MPa.

10.4.7.5.2 *In limited ductile potential plastic hinge regions*

In the ductile detailing length as defined in 10.4.5 of potential limited ductile plastic regions as defined in 10.4.7.2.2, the transverse reinforcement provided shall be the larger of that required by Equation 10–41 or 70 % of that required by Equation 10–40. A2

10.4.7.5.3 *In potential plastic hinge regions with protection against plastic hinging*

In frames where columns are designed with sufficient strength to provide a high degree of protection against plastic hinging (method A in Appendix D, above mid-height of second storey), the required quantity of transverse reinforcement placed in the regions of columns defined as potential plastic hinge regions in 10.4.5 shall be the larger of that required by Equation 10–23 or 70 % of that required by Equation 10–40. This reduction in the quantity of transverse reinforcement in potential plastic hinge regions shall not be permitted at the top and bottom of the columns of the first storey nor in any storey in which a column sidesway mechanism could occur with plastic hinges forming in the columns. A2 A3

10.4.7.5.4 *In regions outside the potential plastic hinge regions*

Outside the ductile detailing lengths defined in 10.4.5, transverse reinforcement shall be as required by 10.3.10.6 as appropriate. A2

10.4.7.5.5 *Spacing of rectangular hoop or tie reinforcement in columns* A3

When rectangular hoop or tie reinforcement is used the spacing shall be as follows:

(a) In ductile plastic regions defined in 10.4.7.2.2 in the associated ductile detailing lengths as defined in 10.4.5, the centre-to-centre spacing of stirrup-ties shall not exceed the smaller of one-quarter of the least lateral dimension of the cross section of the member or six times the diameter of the smallest longitudinal bar to be restrained in the outer layers; A2 A3

A2 (b) In limited ductile plastic regions defined in 10.4.7.2.2 in the associated ductile detailing lengths as defined in 10.4.5, the centre-to-centre spacing of stirrup ties shall not exceed the smaller of one-
A3 quarter of the least lateral dimension of the member or ten times the diameter of any longitudinal bar to be restrained in the outer layers;

A3 (c) Outside the ductile detailing lengths of potential plastic hinge regions of a column and in potential plastic hinge regions with a high degree of protection against plastic hinging, over the length of the column between the potential plastic hinge regions, the centre-to-centre spacing of transverse reinforcement along the member shall not exceed the smaller of one-third of the least lateral dimension or ten times the diameter of the smallest longitudinal bar to be restrained.

10.4.7.6 *Support of longitudinal bars*

In potential plastic hinge regions, each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following two cases of bars are exempt from this requirement:

(a) Bars or bundles of bars which lie between two laterally supported bars or bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed the larger of one-quarter of the adjacent lateral dimension of the cross section or 200 mm between centres;

A3 (b) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inside face of hoop bars.

11 DESIGN OF STRUCTURAL WALLS FOR STRENGTH, SERVICEABILITY AND DUCTILITY

11.1 Notation

A_b area of a longitudinal bar, mm^2 A3

A_c^* area of concrete core extending over the outer c' length of the neutral axis depth which is subjected to compression, measured to centre of peripheral hoop legs, mm^2

A_{cv} area used to calculate shear stress, taken as dt, where d may be taken as $0.8L_w$, mm^2 A3

A_g gross area of section, mm^2

A_g^* gross area of concrete section extending over outer c length of the neutral axis depth that is subjected to compression, mm^2 A3

A_r The aspect ratio for the wall, taken as h_w/L_w for single storey walls and taken as $\frac{M_e}{V_e L_w}$ for two or more storeys where the M_e/V_e ratio is for first mode or equivalent static analysis for seismic actions

A_s area of longitudinal (vertical) reinforcement at a horizontal spacing of s_v along the wall, mm^2

A_{se} effective area of reinforcement, mm^2

$A_{s,f}$ area of reinforcement orientated parallel to the plane containing the coupled walls within the width or widths b_f, mm^2

A_{sh} total effective area of hoop bars and supplementary cross ties distributed over length h'' in the direction under consideration, within vertical spacing s_h, mm^2

A_t total area of longitudinal reinforcement at a section in a wall, mm^2 A2

A_{te} area of one leg of stirrup-tie, mm^2

A_{tr} area of transverse reinforcement within a spacing $s_{\ell t}$ crossing plane of splitting normal to concrete surface containing extreme tension fibres, mm^2 A3

A_v area of in-plane horizontal shear reinforcement within a vertical spacing of s_2, mm^2

A_{wb} gross area of boundary element, mm^2

A_{wv} the area of vertical reinforcement with a horizontal spacing of s_1, mm^2 A3

B_f width of floor contributing to the restraint against elongation of the coupling beam one side of the coupled walls, mm

c distance of neutral axis from the extreme compression fibre of the wall section at the flexural strength for ultimate limit state, mm

c' length of wall section defined by Equation 11–27 to be confined by transverse reinforcement, mm

c_b distance from extreme compression fibre to neutral axis at balanced strain conditions

c_c a limiting depth given by Equation 11–25, mm A3

d distance from extreme compression fibre to centroid of tension force in longitudinal reinforcement, which may be taken as defined in 11.3.11.3.3 for shear strength calculations, mm

d_b diameter of the longitudinal bar, mm

E_c modulus of elasticity of concrete, MPa A2

EI flexural rigidity of a member, MPa, mm^4 A3

E_s modulus of elasticity of reinforcing steel, MPa

f_c' specified compressive strength of concrete, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

$f_{y,end}$ lower characteristic yield strength of non-prestressed vertical reinforcement in the end zone of the wall, MPa A3

f_{yh} lower characteristic yield strength of non-prestressed hoop or supplementary cross tie reinforcement, MPa

f_{yn} lower characteristic strength of vertical non-prestressed reinforcement, MPa

f_{yt} transverse reinforcement yield strength, MPa

A3 $f_{y,web}$ lower characteristic yield strength of non-prestressed vertical reinforcement in the portion of the wall between end zones, MPa

h'' dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop, mm

A3 h_b overall depth of coupling beam, mm

h_n clear vertical height between floors or other effective lines of lateral support, mm

h_w total height of wall from base to top, mm

A3 I moment of inertia of a section, mm^4

A2 I_{cr} second moment of area of transformed cracked section, mm^4

k proportion of the neutral axis depth to the effective depth of member in elastically responding transformed section, calculated neglecting the axial load

A3 k_d a factor used to define material strain limits as given in 2.6.1.3.4

k_e effective length factor for Euler buckling

k_{ft} effective length factor for flexural torsional buckling

A3 k_m factor for determining t_m

L_b length of flexural member, mm

L_c length of compression member, mm

A3 L_{cb} clear span of coupling beam, mm

L_d development length, mm

A3 L_n the clear distance between floors or other effective lines of lateral support, or clear span, mm

L_p effective plastic hinge length, mm

L_s lap splice length, mm

L_w horizontal length of wall, mm

A3 M_e moment from earthquake actions obtained in either an equivalent static analysis or the first mode action in a response spectrum analysis, N mm

M^* ultimate limit state design moment, N mm

M^*_a the moment at the mid-height section of the wall due to factored loads, N mm

M^*_e design moment at the base of the wall due to lateral loads, N mm

M^*_o overstrength moment of resistance at the critical section of the plastic region of a wall, N mm

M_n nominal flexural strength, N mm

N^* design axial load at the ultimate limit state, N

A3 N^*_o maximum axial load in potential plastic regions, N

n modular ratio E_s/E where E_s is the modulus of elasticity of steel

A3 $N_{o,c}$ the axial load induced in a coupling beam due to restraint provided by floor slabs connected to the coupled walls, N

A2

p_ℓ the ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section = A_s/ts_v

A3 $p_{\ell e}$ the ratio of vertical wall reinforcement area in the end zone to the area of the end zone

A2

A3

s_1 centre-to-centre spacing of vertical shear reinforcement, mm

s_2 centre-to-centre spacing of horizontal shear reinforcement, mm

s_h centre-to-centre spacing of horizontal hoop sets, mm

A3 $s_{\ell t}$ centre-to-centre spacing of lapped splice ties, mm

s_p structural performance factor

s_{tv} vertical centre-to-centre spacing of transverse tie sets, mm

s_v horizontal spacing of vertical reinforcement along the length of a wall, mm

A3 t smallest thickness of the wall, mm

t_f thickness of flange, mm

t_m thickness of boundary region of wall at potential plastic hinge region, mm

t_w web thickness, mm

V^*	design shear force, N
v_c	shear stress resisted by concrete, MPa A3
V_c	concrete shear strength, N
V_e	shear force from earthquake actions obtained in either an equivalent static analysis or the first mode action in a response spectrum analysis, N A3
v_{max}	maximum nominal shear stress, MPa
V_n	total nominal shear strength, N
v_s	shear stress resisted by horizontal web reinforcement, MPa A3
V_s	nominal shear strength provided by shear reinforcement, N
α	the angle between the diagonal reinforcement and the axis of the coupling beam A3
α_k	a factor accounting for ductility of plastic hinge for determining confinement requirements
α_m	factor for determining wall slenderness
α_q	a factor accounting for ductility of platic hinge for determing shear requirements
α_r	factor for determining thickness of boundary section of wall
β	factor for determining ductility factor
ε_c	extreme fibre compression strain
ϕ	strength reduction factor (see 2.3.2.2)
ϕ_{max}	limiting curvature, radians/mm
ϕ_{ow}	ratio of moment of resistance at overstrength to moment resulting from specified earthquake actions, where both moments refer to the critical section of the potential plastic region
$\phi_{o,fy}$	overstrength coefficient for reinforcement as given in 2.6.5.5
λ	factor accounting for ductility of plastic hinge for determining shear strength provided by the concrete
λ_s	factor for determining wall slenderness
λ_v	factor for determining shear strength provided by concrete
μ	displacement ductility factor
Δ_u	deflection at the mid-height section of the wall, mm A3
ΣA_b	sum of area of longitudinal bars, mm^2
ξ	factor for determining thickness of boundary section of wall
ψ	ratio of $\Sigma EI/L_c$ of compression members to $\Sigma EI/L_b$ of flexural members in a plane at one end of a compression member
ψ_{min}	the smaller of ψ_A or ψ_B which represents the ψ ratio at each end, A and B, of a compression member

11.2 Scope

11.2.1 *Application*

Provisions of this section shall apply to the design of walls subjected to axial load, with or without flexure, and shear. The provisions of this and earlier sections are summarised in Table C11.3. The clauses within Section 11 take precedence over Table C11.3.

11.2.2 *Requirements determined by curvature ductility*

Walls containing plastic regions with material strain limits demands at the ultimate limit state less than or equal to the limits for nominally ductile plastic regions defined in 2.6.1.3 shall meet the requirements of 11.3 whilst walls in structures designed for greater curvature ductility than this shall be designed to meet the requirements of 11.3 as modified by 11.4. A3 A2

11.2.3 *Axial load limit on walls*

When the limits in 11.3.1.6 and 11.4.1.1 are exceeded, the design of a wall shall proceed according to the provisions of Section 10. A3

11.3 General principles and design requirements for structural walls

11.3.1 *General design principles*

11.3.1.1 *General*

A3 Walls shall be designed for any vertical loading and/or lateral in-plane and out-of-plane forces to which they may be subjected. The design moment, M^*, for bending about the minor axis of the wall, shall include consideration of the additional moment caused by the eccentricity of the applied axial load to the deflected shape. If bending in the out-of-plane direction is to be relied upon for strength, the wall shall be designed as a column, according to the provisions of Section 10.

A3 **11.3.1.2** *Provision for eccentric loads for out-of-plane actions*

The design of a wall shall take account of the actual eccentricity of the vertical force but in no case shall
A3 the design bending moment (M^*) be taken as less than N^* times $0.05t_w$.

A3 **11.3.1.3** *Flange reinforcement effective in resisting flexure*

Only the vertical reinforcement placed within a flange width, each side of the web, equal to one-half the distance from the section under consideration to the top of the wall, but not greater than $4t_f$, shall be considered effective in resisting flexure.

11.3.1.4 *Interaction of flanges, boundary members and webs*

Cantilever or coupled structural walls shall be considered as integral units. The strength of flanges, boundary members and webs shall be evaluated on the basis of compatible interaction between these elements using rational analysis. Due allowance for openings in components shall be made.

11.3.1.5 *Singly reinforced walls*

Buildings where singly reinforced walls form part of the primary lateral load resisting system shall be designed for nominally ductile actions, with the appropriate strength reduction factor defined in 2.3.2.2. The area of vertical reinforcement for singly reinforced walls shall be equal to or less than the limit given in 11.3.12.3(a).

11.3.1.6 *Maximum axial actions*

For walls the ultimate axial load in compression, N^*, shall be less than $0.3\phi f'_c A_g$.

11.3.2 *Minimum wall thickness*

Structural walls shall have a thickness, t, equal to or greater than 100 mm.

11.3.3 *Maximum wall thickness for singly reinforced walls*

Basement walls more than 250 mm thick and other walls more than 200 mm thick shall have the reinforcement placed in two layers parallel with the faces of the wall.

11.3.4 *Design for stability*

11.3.4.1 *Design by rational analysis*

Walls shall be designed to ensure stability at the ultimate limit state due to:

A3 (a) P-delta effects associated with bending about the minor axis of the wall
(b) Euler buckling
(c) Flexural torsional buckling

Stability may be determined by rational analysis using the assumption in 11.3.4.2, or by simplified methods outlined in 11.3.4.3.

11.3.4.2 *Design assumptions for rational stability analysis*

A2 The design moment determined by rational analysis shall consider the eccentricity of the applied load, degree of support fixity, and a wall stiffness calculated from transformed sectional properties neglecting concrete in tension, multiplied by a stiffness reduction factor of 0.75 (shown in brackets in Equations 11–3 and 11–4).

11.3.4.3 *Simplified methods of stability analysis*

11.3.5 provides simplified methods of ensuring stability at the ultimate limit state for slender walls with a single layer of centrally placed reinforcement.

11.3.6 provides a simplified method of ensuring stability in doubly reinforced walls subjected to eccentric axial load without face loads.

11.3.5 *Simplified stability assessment for slender singly reinforced walls*

11.3.5.1 *Design for actions causing bending about the minor axis* A3

11.3.5.1.1 *Limitations on use of method*

Walls designed using the requirements of 11.3.5.1.2 shall:

(a) Have a vertical stress N^*/A_g at the mid-height of the section of less than 0.06 f'_c, for the load case causing bending about the minor axis. A3

(b) The walls shall be supported at the top and bottom. The method is not applicable to cantilevered walls bent about the minor axis. A3

11.3.5.1.2 *Design moment and P-delta effects – simplified method*

The design moment strength ϕM_n for combined flexure and axial loads at the mid-height cross section shall satisfy:

$$\phi M_n \geq M^* \quad \text{(Eq. 11–1)}$$

where

$$M^* = M_a^* + N^* \Delta_u \quad \text{(Eq.11–2)}$$

M_a^* is the moment at the mid-height section of the wall due to factored loads, and Δ_u is:

$$\Delta_u = \frac{5M^* h_n^2}{(0.75)48E_c I_{cr}} \quad \text{(Eq. 11–3)}$$ A3

M^* shall be obtained by iteration of deflections, or by direct calculation using Equation 11–4

$$M^* = \frac{M_a^*}{1 - \dfrac{5N^* h_n^2}{(0.75)48E_c I_{cr}}} \quad \text{(Eq. 11–4)}$$ A3

Where

For short-term loads E_c is given by 5.2.3, and for long-term loads E_c shall be modified to consider creep.

The value of I_{cr} shall either be calculated by rational analysis based on elastic analysis, or the approximation given by Equation 11–5 may be used where $N^*/A_g f'_c \leq 0.06$. A2

$$I_{cr} = nA_{se}(d - kd)^2 + \frac{L_w (kd)^3}{3} \quad \text{(Eq. 11–5)}$$

where k is determined by elastic theory,

$$n = \text{modular ratio} = \frac{E_s}{E_c} \quad \text{(Eq. 11–6)}$$

and A_{se} may be taken as:

$$A_{se} = \frac{N^* + A_t f_y}{f_y} \quad \text{(Eq. 11–7)}$$

11.3.5.2 *Design for actions causing bending about the strong axis*

11.3.5.2.1 *Limitation on use of method*

A2 Walls shall be designed to the requirements of 11.3.5.2.2 and shall:

(a) Have an axial load at the base of the wall of $N^* < 0.015 f'_c A_g$ for the load case causing bending about the strong axis.

(b) The eccentricity of the axial load from the longitudinal axis shall be less than the wall thickness.

(c) Singly reinforced walls that are designed to be part of the primary lateral load resisting system for in-plane loads, shall be designed to ensure that mid-height hinges shall not form in the walls due to face loading. Singly reinforced walls designed to resist face loads by cantilever action shall be designed to ensure that plastic hinges do not form at the base of the wall due to face loads.

11.3.5.2.2 *Prevention of flexural torsional buckling of walls loaded in-plane with low axial loads*

The limiting effective height to thickness ratio to prevent flexural torsional buckling shall be determined from the lesser of Equations 11–8, 11–9 and 11–10.

A3 $$\frac{k_{ft} h_n}{t} \le 12\sqrt{\frac{h_n / L_w}{\lambda s}} \qquad \text{(Eq. 11–8)}$$

and

A3 $$\frac{h_n}{t} \le 75 \qquad \text{(Eq. 11–9)}$$

and

A3 $$\frac{k_{ft} h_n}{t} \le 65 \qquad \text{(Eq. 11–10)}$$

where the effective length factor for flexural torsional buckling, k_{ft} is given by 11.3.5.2.3, and λ_s = the lesser of:

A3 (a) $\dfrac{N^*}{f'_c A_g} + p_\ell \dfrac{f_y}{f'_c}$

(b) $\dfrac{2.2 M_e^*}{L_w A_g f'_c}$

A2 where M_e^* is the design moment at the base of the wall due to lateral loads, for seismic load cases this shall be the action corresponding with μ = 1.25 and S_p = 0.9.

11.3.5.2.3 *Effective height between lines of lateral support*

A3 The effective height between lines of lateral support for flexural torsional bucking shall be taken as $k_{ft} h_n$, where k_{ft} is given by Table 11.1.

Table 11.1 – Effective wall height co-efficient k_{ft} for walls with a potential nominally ductile plastic region under in-plane loading A3

Case	Support condition at		Potential plastic region classification for in-plane loads[(4)]	k_{ft}
	Base of wall	Top of wall		
1	Fixed	Pinned	NDPR	0.85 or 1.0 where minor axis hinge forms at base
2	Fixed	Nil	NDPR	1.4
3	Pinned	Pinned	NDPR	1.0

NOTE –
(1) Fixed, means rotational, lateral, and torsional support are provided.
(2) Pinned, means torsional and lateral restraint is provided, but not rotational restraint.
(3) Nil, means none of torsional, rotational, or lateral restraint is provided
(4) Abbreviations for potential plastic region classifications (see 2.6.1.3):
Nominally ductile plastic region, NDPR.

11.3.6 *Simplified stability assessment for doubly reinforced concrete walls*

11.3.6.1 *Limitation on the use of the method* A3

Walls designed using the requirements of 11.3.6.2 shall:

(a) Be doubly reinforced;

(b) Not be subjected to face loads for the action combination being considered. A2

11.3.6.2 *Design for actions causing bending about the minor axis* A3

Design of walls for loads eccentric to the wall longitudinal axis without face loads shall include the effects of slenderness using the method outlined in 10.3.2 when the unsupported height, (h_n), to wall thickness, t, ratio exceeds the following limits: A2

$$\frac{k_e h_n}{t} \geq \frac{\alpha_m}{\sqrt{\dfrac{N^*}{f'_c A_g}}} \quad \text{(Eq. 11–11)}$$ A3

where

(a) α_m = 6.5 for walls braced against sidesway and pinned at each end; A2

(b) α_m = 8 for braced walls rotationally fixed at one end.

The effective length factor for Euler buckling, k_e, is given by:

(a) k_e = 0.85 + 0.05ψ_{min} for walls braced against lateral sidesway; and

(b) k_e = 2.0 + 0.3ψ for cantilevered walls not prevented from sidesway;

where

ψ = ratio of $\Sigma EI/L_c$ of compression members to $\Sigma EI/L_b$ of flexural members in a plane at one end of a compression member

ψ_{min} = the smaller of ψ_A or ψ_B which represent the ψ ratio at each end, A and B, of a compression member.

11.3.7 *Walls with high axial loads*

The ratio of effective height to thickness ($k_e h_n/t$) shall be equal to or less than 20 where $N^* > 0.2\phi f'_c A_g$. A3

11.3.8 *Minimum thickness for compression flanges of walls*

Where the length of a flange that is required to resist compression extends for a distance of more than three times the flange thickness from the face of the web, the ratio of the flange thickness to the height between lateral support of the flange, given by (h_n/t_f), shall be equal to or less than 20.

11.3.9 *Flexural crack control*

Walls subject to flexure shall be designed to control cracking in accordance with 2.4.4.

A3 **11.3.10 *Strength of walls in flexure***

The design of walls for flexure at the ultimate limit state shall be based on the assumptions given in 7.4 and on the satisfaction of conditions of equilibrium and compatibility of strains.

11.3.11 *Strength of walls in shear*

11.3.11.1 *General*

The design of walls for shear at the ultimate limit state shall be in accordance with 7.5.

A3 **11.3.11.2** *Shear design of face loaded walls*

Design for shear forces perpendicular to face of a wall shall be in accordance with the provisions for slabs in 12.7 and 9.3.9.

A3 **11.3.11.3** *Design for shear in the plane of a wall*

Design for horizontal shear forces in the plane of a wall shall be in accordance with 11.3.11.3.1 to 11.3.11.3.8.

A3 **11.3.11.3.1** *Design horizontal section for shear*

Design of a horizontal section for shear in the plane of a wall shall be based on 7.5, where concrete shear strength, V_c, shall be in accordance with 11.3.11.3.4 or 11.3.11.3 and shear reinforcement shall be in accordance with 11.3.11.3.8.

A3 **11.3.11.3.2** *Maximum shear stress*

The shear stress at any horizontal section due to in-plane actions in a wall and based on the minimum effective wall thickness shall be equal to or less than the value given by 7.5.2.

A3 **11.3.11.3.3** *Definition of d*

For design for horizontal shear forces in the plane of a wall, d shall be taken as equal to $0.8L_w$. A larger value of d, equal to the distance from the extreme compression fibre to the centre of force of all
A2 reinforcement in tension, may be used when determined by a strain compatibility analysis prior to first yielding of longitudinal reinforcement. For face loading, d shall be taken as the distance from the extreme compression fibre to the centroid of longitudinal tension reinforcement.

A3 **11.3.11.3.4** *Concrete shear strength – simplified*

The shear resistance provided by the concrete may be calculated by the simplified method given below in lieu of the more detailed method in 11.3.11.3.5. This simplified method may only be used where the ratio, p_ℓ, of longitudinal reinforcement to area of concrete for any part of the wall exceeds a value of 0.003 and the spacing of reinforcement does not exceed 300 mm in any direction. Where this condition is satisfied V_c shall be taken to be the smaller of:

A2 $$V_c = 0.17\sqrt{f_c'}A_{cv} \quad \text{(Eq. 11–12)}$$

or

A2 $$V_c = 0.17\left[\sqrt{f_c'} + \frac{N^*}{A_g}\right]A_{cv} \quad \text{(Eq. 11–13)}$$

where N^* is taken as negative for axial tension.

A3 **11.3.11.3.5** *Concrete shear strength – detailed*

Concrete shear strength, v_cA_{cv}, shall be computed by Equation 11–17 where v_c shall be the lesser of that calculated from Equations 11–14 and 11–15:

$$v_c = \left(0.27\sqrt{f_c'} + \frac{N^*}{4A_g}\right) \qquad \text{(Eq.11–14)}$$

or

$$v_c = 0.05\sqrt{f_c'} + \frac{L_w\left(0.1\sqrt{f_c'} + 0.2\frac{N^*}{A_g}\right)}{\frac{M^*}{V^*} - \frac{L_w}{2}} \qquad \text{(Eq.11–15)}$$

where N^* is negative for tension. When $(M^* / V^* - L_w /2)$ is zero or negative, Equation 11–15 shall not apply.

11.3.11.3.6 *Shear design of sections near base of walls* A3

Sections located closer to the wall base than a distance $L_w /2$ or one-half the wall height, whichever is less, shall be designed for the same V_c as that computed at the smaller of $L_w /2$ or one-half the height.

11.3.11.3.7 *Shear reinforcement always to be provided* A3

Irrespective of whether the total nominal shear strength, V_n, is more or less than $V_c /2$, reinforcement shall be provided in accordance with 11.3.11.3.8.

11.3.11.3.8 *Design of horizontal shear reinforcement* A3

Design of shear reinforcement for walls shall satisfy the following requirements:

(a) Where the total design shear force, V^*, exceeds the concrete shear strength, V_c, horizontal shear reinforcement shall be computed from:

$$V_s = \frac{V^*}{\phi} - V_c \qquad \text{(Eq. 11–16)}$$

where

$$V_c = v_c A_{cv} \qquad \text{(Eq. 11–17)}$$

and

$$V_s = A_v f_{yt} \frac{d}{s_2} \qquad \text{(Eq. 11–18)}$$

where A_v is the area of horizontal shear reinforcement within a distance s_2.

(b) Irrespective of the requirements of (a) above the area of horizontal shear reinforcement in a wall shall be equal to or greater than:

$$A_v = \frac{0.7 t_w s_2}{f_{yt}} \qquad \text{(Eq. 11–19)}$$ A3

(c) Vertical spacing of horizontal reinforcement, s_2, shall be equal to or less than the lesser of $L_w/5$, $3t$ or 300 mm; and

(d) Horizontal bar laps shall be in accordance with 8.7.2.8.

A3 **11.3.11.3.9** *Detailing of vertical shear reinforcement*

The ratio p_ℓ of vertical shear reinforcement area to unit concrete area of horizontal section shall be equal to or greater than $0.7/f_{yn}$, and the horizontal spacing of vertical reinforcement s_1 shall be equal to or less than the lesser of $L_w/3$, $3t$, or 300 mm.

Vertical shear reinforcement shall not be additional to the vertical flexural (longitudinal) reinforcement as required in 11.3.12, except where required in squat walls.

11.3.12 *Wall reinforcement*

11.3.12.1 *General*

All concrete walls shall have reinforcement placed in two directions at an angle of approximately 90°. Bars shall not be bent round re-entrant angles unless special provisions are made for positive resistance of bursting forces at bends of bars.

A3 **11.3.12.2** *Placement of reinforcement in walls*

(a) Basement walls more than 250 mm thick and other walls more than 200 mm thick shall have the reinforcement for each direction placed in two layers parallel with the faces of the wall.
(b) Bars shall be equal to or larger than 10 mm in diameter.
A3 (c) Bars shall be spaced at no more than three times the thickness of the wall or 300 mm on centres, whichever is the least.
(d) The diameter of the bar in the wall shall not exceed one seventh of the wall thickness.
A3 (e) The horizontal reinforcement shall be anchored as close as practicable to the end face of the wall.

11.3.12.3 *Minimum and maximum area of reinforcement*

A3 The ratio of vertical reinforcement in any section of a wall shall satisfy the limitations given in (a), (b) and (c) below:

(a) For actions causing bending about the minor axis of singly reinforced walls, the area of vertical reinforcement shall be such that at every section the distance from the extreme compression fibre to the neutral axis shall be equal to or less than $0.75c_b$;
(b) The ratio of vertical reinforcement to concrete unit area, p_ℓ, in a rectangular wall, or in a boundary element (11.4.3.3), or any other rectangular element in a wall, shall be equal to or less than $16/f_y$, except in regions where lapped splices in boundary elements are unavoidable, in which case the total ratio including the area of splices shall not exceed $21/f_y$;
(c) The ratio of vertical reinforcement to unit cross section area, p_ℓ, shall be equal to or greater than $\dfrac{\sqrt{f_c'}}{4f_y}$.

11.3.12.4 *Reinforcement around openings*

In addition to the minimum as prescribed in 11.3.12.3 there shall be reinforcement with a yield strength equal to or greater than 600 N per mm of wall thickness, around all window or door openings. Such bars shall extend at least 600 mm beyond the corners of the openings.

A3 **11.3.12.5** *Anchorage of shear reinforcement at wall ends*

For walls, excluding singly reinforced walls, the horizontal reinforcement shall be anchored at the end of the wall using one of the methods outlined in (a) to (c):

(a) Horizontal reinforcement shall be bent around both of the corner end bars to form a continuous U-shaped bar; bend diameters shall comply with 8.4.2;

(b) Horizontal reinforcement shall be bent around a corner bar at the ends of the wall with a 135° or 180° hook. The two corner bars at the end of the wall shall be linked transversely with the tie having a cross-sectional area and spacing equal to that of the horizontal bars. These links shall be bent around the vertical bars with 135° hooks complying with 8.6.10;

(c) Horizontal reinforcement shall be anchored as close as practical to the end of the wall with 90° or semicircular hooks, as per 8.6.10, and these hooks shall be terminated within a cage that comprises horizontal stirrups enclosing at least four vertical bars at the end of the wall. These four vertical bars shall comprise the two corner bars and at least the next group of vertical bars along the horizontal plane of the wall. The ratio of cross-sectional area of closed stirrups making up the cage to their spacing (A/s) shall be the same as the ratio of the area of the horizontal reinforcement being anchored within the closed cage to its spacing (A/s). The vertical spacing of the stirrups shall not be greater than the vertical spacing of the horizontal reinforcement. The distance between the innermost face of the closed cage and the hook end shall be at least the development length (see 8.6.10). A3

Horizontal shear reinforcing bars shall be lapped in compliance with 8.7.2.8.

11.3.12.6 *Anchorage of shear reinforcement at intersections of walls*

At wall intersections of L-, T- or C-shaped walls the horizontal reinforcement shall be anchored using either (a) or (b) below:

(a) With 90° or 180° hooks, as per 8.6.10, these hooks shall be terminated within a cage that comprises closed stirrups enclosing at least four vertical bars at the end of the wall. These four vertical bars shall comprise the two corner bars and at least the next group of vertical bars along the horizontal plane of the wall. The ratio of cross-sectional area of stirrups enclosing the cage to their spacing (A/s) shall be the same as the ratio of the area of the horizontal reinforcement being anchored within the cage to its spacing (A/s). The vertical spacing of the stirrups shall not be greater than the vertical spacing of the horizontal reinforcement. The distance between the innermost face of the closed cage and the hook end shall be at least the development length (see 8.6.10);

(b) Horizontal reinforcement shall be bent around vertical bars on the far face of the intersection bars to form a continuous U-shaped bar; bend diameters shall comply with 8.4.2.

11.3.12.7 *Curtailment of flexural reinforcement*

Curtailment of flexural reinforcement shall comply with 8.6.12.

11.3.12.8 *Vertical reinforcement in squat walls*

11.3.12.8.1 *Shear resistance*

In structural walls where the $M_e/(V_e L_w)$ ratio, found from the first mode or from an equivalent static analysis, is less than 0.75, or where the aspect ratio of height divided by length h_w/L_w is equal to or less than 0.5, the minimum area of vertical reinforcement, A_{wv}, for shear resistance, at any location along the length of the wall shall be determined by Equation 11–19(a) or shall be based on the strut and tie method.

$$\frac{A_{wv}}{s_1 t_w} \geq \frac{v_s}{f_y} \qquad \text{(Eq. 11–19a)}$$

where v_s is the shear stress resisted by horizontal web reinforcement, which is given by:

$$v_s = \frac{V_s}{0.8 L_w t}.$$

If the area provided by flexural reinforcement at the location being considered exceeds A_{wv} from Equation 11–19(a), no additional vertical reinforcement need be added for shear resistance.

11.3.12.8.2 *Flexural resistance*

In structural walls where the $M_e/(V_e L_w)$ ratio is less than 0.75 the strut and tie method of analysis may be used for flexure and shear instead of the standard approach based on plane sections remaining plane. Where the $M_e/(V_e L_w)$ value is less than 0.5 the strut and tie method shall be used for the design of flexure and shear, with the angle between the critical section and the compression struts being equal to or greater than 30°.

11.4 Additional design requirements for members designed for ductility in earthquakes

11.4.1 *General seismic design requirements*

A3

11.4.1.1 *Maximum axial actions for ductile walls*

For ductile and limited ductile walls the maximum axial load in potential plastic regions (N_o^*) shall not exceed $N_o^* \leq 0.3A_g f'_c$.

11.4.1.2 *Design of ductile walls*

In the design of ductile walls subjected to seismic forces at the ultimate limit state, the requirements of 2.6.8 shall be satisfied.

11.4.1.3 *Effective flange projections for walls with returns*

For determining the nominal moment strength, M_n, of a wall the provisions of 11.3.1.3 shall apply.

A3 A2 When the overstrength moment of resistance, M_o^*, is required, the effective width of the flange acting in tension, either side of the web, shall be equal to the distance from the section under consideration to the top of the wall but not greater than flange width. When the flange is in compression the requirements of 11.3.1.3 apply.

A3

11.4.2 *Ductile detailing lengths*

The ductile detailing length measured from the critical section for in-plane actions shall be taken as the larger of:

(a) $\dfrac{0.25M_e}{V_e} \leq 2L_w$; or

(b) 1.5 L_w.

where $\dfrac{M_e}{V_e}$ at the critical section is the moment to shear force ratio for seismic actions found from an equivalent static analysis or a first mode value from a modal response spectrum analysis.

Where the critical section in coupled walls is located immediately above the foundation beam the ductile detailing length shall extend from the critical section for a distance of two storeys.

11.4.3 *Dimensional limitations*

11.4.3.1 *Prevention of buckling of thin walls loaded in-plane*

To safeguard against premature out-of-plane buckling in the potential plastic hinge region of walls, the limitations of 11.4.3.2 to 11.4.3.4 shall apply.

11.4.3.2 *Minimum thickness for prevention of instability within plastic hinge region*

To safeguard against out-of-plane buckling in the potential plastic hinge regions of ductile walls, the following limitations shall apply for walls with axial force levels greater than 0.05 $f'_c A_g$.

The thickness in the boundary region of the wall section, extending over the lesser of the ductile detailing length or the full height of the storey containing the potential plastic region, shall be equal to or greater than:

$$t_m = \frac{\alpha_r k_m \beta (A_r + 2) L_w}{1700\sqrt{\xi}} \quad \text{......... (Eq. 11–20)}$$

where

α_r = 1.0 for doubly reinforced walls
β = 5 for limited ductile plastic regions
β = 7 for ductile plastic regions
k_m = 1.0, unless it can be shown that for long walls:

$$k_m = \frac{h_n}{(0.25 + 0.055 A_r) L_w} < 1.0 \quad \text{(Eq. 11–21)}$$ A3

and

$$\xi = 0.3 - \frac{p_\ell f_y}{2.5 f_c'} > 0.1 \quad \text{(Eq. 11–22)}$$

In Equation 11–22 p_ℓ shall be calculated for the vertical reinforcement in the boundary region only (see 11.3.12).

11.4.3.3 *Dimensions of enlarged boundary element*
Where 11.4.3.2 controls the thickness of the wall in the boundary region, an enlarged boundary element shall be provided with gross area, A_{wb}, satisfying the following limitations:

$$t_m^{\,2} \le A_{wb} \ge \frac{t_m L_w}{10} \quad \text{(Eq. 11–23)}$$

11.4.3.4 *Flange thickness for prevention of instability within plastic hinge region*
Where the effective width of a flange located at a distance of more than three times the flange thickness from the face of the web is required to resist flexure, the effective height to flange thickness ratio ($k_e h_n / t_f$) shall be equal to or less than 15 for ductile plastic regions and shall be equal to or less than 20 for limited ductile plastic regions.

11.4.4 *Reinforcement limits*

11.4.4.1 *Reinforcement diameters*

In ductile detailing lengths in a wall, the diameter of vertical reinforcement bars shall not exceed:

(a) In ductile plastic hinge regions, one-tenth of the thickness; and

(b) In limited ductile plastic hinge regions, one-eighth the wall thicknesss.

11.4.4.2 *Minimum area of reinforcement*
In the end zone of the wall, which is defined as having a length extending a distance of 15 % of the wall length in the direction of loading from the extreme tension fibre, and a width equal to the thickness of the end of the wall at the location being considered, the ratio of the area of vertical reinforcement in the end zone to the area of the end zone, $p_{\ell e}$, shall be equal to or greater than $\dfrac{\sqrt{f_c'}}{2 f_y}$.

In the portion of the wall between end zones, the ratio of the area of vertical reinforcement to unit cross section area, p_ℓ, shall be equal to or greater than $\dfrac{\sqrt{f_c'}}{4 f_y}$, and greater than 0.3 $p_{\ell e}(f_{y,end}/f_{y,web})$.

11.4.5 *Transverse reinforcement*

11.4.5.1 *Transverse reinforcement requirements*
The requirements for minimum reinforcement ratio, placing of reinforcement, diameter of transverse bars used and their spacing shall be in accordance with 11.3.12.

A3

11.4.5.2 *Transverse tie reinforcement for lateral restraint in plastic hinge regions – compression region of wall*

In the ductile detailing length, transverse tie reinforcement complying with (a) to (c) shall be provided within the compression neutral axis depth. The spacing of ties along the longitudinal bars shall be equal to or less than $6d_b$ in ductile plastic regions and $10d_b$ in limited ductile plastic regions, where d_b is the diameter of the longitudinal bar to be restrained.

(a) Ties suitably shaped shall be arranged so that each longitudinal bar or bundle of bars, placed close to the wall surface, is restrained against buckling by a 90° bend or at least a 135° standard hook of a tie. Where two or more bars at not more than 200 mm centres apart are so restrained, any bars between them are exempted from this requirement;

(b) The diameter of transverse reinforcement ties shall be greater than or equal to 6 mm;

(c) The area of one leg of a tie, A_{te}, in the direction of potential buckling of the longitudinal bar, shall be calculated from:

$$A_{te} = \frac{\Sigma A_b f_y}{96 f_{yt}} \frac{s_{th}}{d_b} \quad \text{(Eq. 11–24)}$$

where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 11.4.5.2(a), and s_{tv} is the vertical spacing of transverse tie sets. Longitudinal bars centred more than 75 mm from the inner face of stirrup-ties need not be considered in determining the value of ΣA_b.

11.4.5.3 *Transverse tie reinforcement for lateral restraint in plastic hinge regions – central region of wall*

In ductile detailing lengths, transverse tie reinforcement complying with 11.4.5.2(a) to (c) shall be provided around longitudinal reinforcement within the central portion of the wall outside the neutral axis depth, c, when any of (a) to (d) occur. The spacing of transverse ties along the longitudinal bars, when required, shall be equal to or less than the lesser of $L_w/3$, t or 300 mm.

(a) $\phi_{ow} V^*$ exceeds $0.075 f_c' \times 0.8 L_w t$;

(b) The average spacing of vertical bars in the central 70 % of the wall is less than $5d_b$ over more than 10 % of the wall length;

(c) Clear cover between the vertical bar and face of the wall is less than or equal to $1.5d_b$;

(d) The ductile detailing length contains a plastic region where the curvature demand exceeds the maximum permitted curvature, defined as $K_d \phi_y$, for limited ductile plastic regions; see 2.6.1.3.4.

11.4.5.4 *Transverse reinforcement for lateral restraint of longitudinal bars outside plastic hinge regions*

Outside the plastic hinge regions defined by 11.4.2, transverse reinforcement shall be in accordance with 11.3.

11.4.5.5 *Confinement requirements in plastic hinge region*

Where the neutral axis depth in the potential yield regions of a wall, computed for the appropriate design forces for the ultimate limit state, exceeds:

$$c_c = 0.1 \phi_{ow} L_w \quad \text{(Eq. 11–25(a))}$$

for limited ductile regions; and

$$c_c = 0.05 \phi_{ow} L_w \quad \text{(Eq. 11–25(b))}$$

for ductile plastic regions as defined by 2.6.1.3, the following requirements shall be satisfied in that part of the wall section which is subjected to compression strains due to the design forces:

(a) Rectangular or polygonal closed hoops, surrounding longitudinal bars, shall be used so that:

$$A_{sh} = \alpha_k s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07 \right) \quad \text{(Eq. 11–26)}$$

where

α_k = 0.25 for ductile plastic regions

α_k = 0.175 for limited ductile plastic regions defined by 2.6.1.3 A2

(b) The length of the confined region of the compressed wall section c' shall be equal to or greater than c. A3

(c) Longitudinal bars shall be restrained against possible buckling in accordance with 11.4.5.2;

(d) The centre-to-centre spacing of hoops along longitudinal bars in fully ductile plastic regions shall not exceed six times the diameter of the longitudinal bar, or one half of the wall thickness in the confined region. For limited ductile plastic regions the centre-to-centre spacing shall not exceed $10d_b$, or the thickness of the wall in the confined region;

(e) Each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following two cases of bars are exempt from this requirement:

   (i) Bars or bundles of bars that lie between two laterally supported bars or bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed one-half of the adjacent lateral dimension of the cross section, or 200 mm; A3

   (ii) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inside hoops.

(f) The ductile detailing length of the wall, over which the requirements for hoops in accordance with (a) to (d) is satisfied, shall be as defined in 11.4.2; A3

(g) The region to be confined shall contain more than one layer of longitudinal reinforcement;

(h) Confinement reinforcement shall not be additional to transverse reinforcement in the plastic hinge region required in 11.4.5.2 and 11.4.5.3;

(i) Consideration shall be given to forces at directions other than aligned with the wall axes, and consideration shall be given to the consequential effects on the neutral axis.

11.4.6 *Shear strength*

11.4.6.1 *General*

The evaluation of shear strength and the determination of shear reinforcement for walls shall be in accordance with 7.5. A2

11.4.6.2 *Maximum design shear force* A3

In the estimation of the maximum shear demand on a wall of limited ductility, the maximum shear need not exceed that corresponding to a nominally ductile wall element derived using μ = 1.25 and S_p = 0.9.

11.4.6.3 *Shear strength provided by the concrete*

In walls subjected to an axial load N^*, the shear resisted by concrete, V_c, in the ductile detailing length defined in 11.4.2 shall not exceed:

$$V_c = \left(0.27\lambda\sqrt{f_c'} + \frac{N^*}{4A_g}\right)t_w d \geq 0.0 \quad \text{(Eq. 11–28)}$$

where

λ = 0.25 for ductile plastic regions

λ = 0.5 for limited ductile plastic regions defined by Table 2.4

N^* shall be taken as negative for walls subject to axial tension A3

The total design shear force, V^*, shall not exceed:

A3

$$\frac{V^*}{\phi} \le \left(\frac{\phi_{ow}}{\alpha_q} + 0.15\right)\sqrt{f_c'}\, A_{cv} \le v_{\max} A_{cv} \quad \text{(Eq. 11–29)}$$

where

α_q = 3.0 for limited ductile plastic regions

α_q = 6.0 for ductile plastic regions defined by Table 2.4.

Linear interpretation between these values may occur when the calculated curvature ductilities lie between the limits provided in Table 2.4 for limited ductile plastic regions and ductile plastic regions.

A2 $v_{\max}$ is given by 7.5.2.

A3 **11.4.6.4** *Sliding shear of squat walls*

Squat walls having adequate foundations to enable a plastic hinge to develop at the base shall be designed so as to ensure that no sliding shear failure along the base section could occur before a displacement ductility capacity assigned to such walls can be fully developed (see 2.6.1.3.4(e)).

A3

11.4.7 *Walls with openings*

Openings in structural walls shall be so arranged that unintentional failure planes across adjacent openings, do not reduce the shear or flexural strength of the structure. For ductile cantilever walls with irregular openings appropriate analyses such as based on strut-and-tie models shall establish rational paths for the internal forces. Capacity design procedures shall be used to ensure that the horizontal shear reinforcement will not yield before the flexural strength of the wall is developed.

A3

11.4.8 *Special splice and anchorage requirements*

11.4.8.1 *Splicing of flexural tension reinforcement*

The splicing of the principal vertical flexural tension reinforcement in the ductile detailing length in ductile walls shall be avoided if possible. Not more than one-third in ductile plastic regions, and one-half for limited ductile plastic regions of such reinforcement shall be spliced at the same location where yielding can occur.

A3 **11.4.8.2** *Staggering and confining of lapped splices*

The stagger between the lapped splices in the ductile detailing length shall be equal to or greater than one-half the splice length, L_s, and lateral ties, spaced at equal to or less than 10 times the diameter of the longitudinal bar, satisfying the requirements of Equation 11–30, shall surround the lapped bars.

$$\frac{A_{tr}}{s_{lt}} \ge \frac{d_b f_y}{48 f_{yt}} \quad \text{(Eq. 11–30)}$$

11.4.8.3 *Welded and mechanical splices*

Mechanical connections and welded splices satisfying the requirements of 8.7.4.1 may be used in ductile detailing lengths in walls, provided that not more than one-half of the reinforcement shall be spliced at one section, and the stagger shall be equal to or greater than 600 mm. Welded splices satisfying 8.7.4.1(a) or mechanical connections meeting the additional testing requirements for stiffness of 8.9.1.3 need not be staggered.

A3 **11.4.8.4** *Welded splices in areas where yielding can not occur*

When by capacity design procedure or otherwise it can be shown that yielding of wall reinforcement could not occur, only the requirements of 8.7.5.4 need be satisfied.

A3

11.4.9 *Coupled shear walls*

11.4.9.1 *General*

Ductile coupled walls shall be designed to restrict inelastic deformation to potential ductile plastic regions at the base of each wall and in the coupling beams. The critical section in the wall shall be detailed for ductility. Additionally, the region of the wall immediately above the lowest coupling beam and below the next coupling beam shall be detailed for ductility. In the area above the potential plastic regions, the walls shall be designed to restrict inelastic deformation to the coupling beams as detailed in 11.4.9.2. The designer shall ensure elastic behaviour of other areas of the walls by ensuring that walls are designed to sustain 1.2 times the maximum overstrength moment from each individual coupling beam acting in addition to the overall seismic actions on the walls. The factor of 1.2 need not be applied to the simultaneous overstrength actions of all the beams. A3

The deformation of the coupling beams in the ultimate limit state shall comply with the material strain limits (see 2.6.1).

11.4.9.2 *Overstrength of coupling beams*

The overstrength of the coupling beams shall be calculated on the basis of material strengths detailed in 2.6.5.5 and on the assumption that the coupling beams are subjected to an axial force of $N_{o,c}$, which arises from restraint against elongation provided by floors in the vicinity of the walls.

The proportion of floor providing restraint to elongation of each coupling beam shall be equal to the overstrength tension capacity of the floor, or floors where there is a floor on each side of the coupled walls. The width of floor, B_f, contributing to the restraint against elongation of the coupling beam on one side of the coupled walls shall be equal to the smaller of:

(a) Half the span or distance of the floor to the next line of support; or
(b) Four times the depth of the coupling beam (h_b).

For floors that are either cast *in situ*, cast on metal trays, or constructed on precast units that span in a direction normal to the plane of the coupled walls, the axial restraining force shall be taken as:

$$N_{o,c} = \phi_{o,fy} f_y A_{s,f} \quad \text{(Eq. 11–31)}$$

where $A_{s,f}$ is the area of reinforcement orientated parallel to the plane containing the coupled walls within the width or widths B_f and $\phi_{o,fy}$ is defined in 2.6.5.5.

For floors containing precast units that are parallel to the plane containing the coupled walls, the axial force shall be calculated from the axial strength of the floor within the length B_f by the method outlined in 9.4.1.6.2.

In situations that are not covered above, such as where the precast units span at an angle that is not close to parallel to or normal to the plane of the coupled walls, the restraining force in the floor shall be evaluated from first principles.

For conventionally reinforced coupling beams the overstrength moments at each end of the beams shall be calculated by standard theory using strengths of reinforcement of $\phi_{o,fy} f_y$ and concrete strength of $f'_c + 15$ when the sections are subjected to the axial force $N_{o,c}$.

For diagonally reinforced coupling beams the overstrength shear force in the coupling beam shall be calculated as:

$$V_{o,c} = 2\,\phi_{o,fy} f_y A_{s,d} \sin\alpha + N_{o,c} \tan\alpha \quad \text{(Eq. 11–32)}$$

Where A_{sd} is the area of diagonal reinforcement in one diagonal and α is the angle the diagonal sustains with the axis of the beam.

11.4.9.3 *Diagonally reinforced coupling beams*

Diagonally reinforced coupling beams shall not be used where L_n/h_b exceeds 4, where h_b is the overall depth of the beam.

(a) Diagonally reinforced coupling beams shall be used where either:
   (i) The shear force exceeds the allowable limit given in 9.4.4.1.4(a) or (b), or
   (ii) The seismic induced deformation exceeds the limit that can be sustained by flexural rotation in plastic regions (2.6.1) but is less than the shear deformation limit for diagonally reinforced coupling beams (2.6.1);
(b) In diagonally reinforced coupling beams, the seismic design moment and shear shall be designed to be resisted by two sets of diagonal reinforcement which intersect at the mid-section of the coupling

A3

beam. Each of these two sets of reinforcement shall consist of four or more bars provided in two or more layers, and each set shall be separately enclosed by rectangular ties (or equivalent spirals), and satisfy the following:

(i) Stirrup-ties shall be arranged so that each bar within the length L_n is restrained against buckling by a 90° bend in a stirrup-tie at a spacing of equal to or less than $6d_b$, except where two or more bars at not more than 200 mm centres are so restrained; any bars between them are exempt from this requirement;

(ii) Diagonally reinforced coupling beams shall contain longitudinal reinforcement that satisfies 9.4.3.4(b) and stirrups enclosing the top and bottom reinforcement that satisfies Equation 9–27, with a spacing that is equal to or less than the smaller of 12 times the diameter of the longitudinal bars or $h_b/4$;

(iii) The diagonal reinforcement shall be anchored in adjacent members by a length equal to or greater than 1.5 times its development length in tension (L_d) in accordance with 8.6.3.

11.4.9.4 *Reinforcement details for diagonally reinforced coupling beams*

Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid-span shall satisfy either (a) or (b):

(a) Each group of diagonal bars shall be enclosed by transverse reinforcement having overall dimensions not less than $0.5t$ across the beam and not less than $t/5$ parallel to the plane of the beam, where t is the web thickness of the coupling beam. The transverse reinforcement shall satisfy 9.3.9.6 and 9.4.5, and shall have spacing measured parallel to the diagonal bars satisfying the lesser of 150 mm or six times the diameter of the diagonal bars and shall have spacing of cross ties or of legs of hoops measured perpendicular to the diagonal bars not exceeding 300 mm. The transverse reinforcement shall continue through the intersection of the diagonal bars. The longitudinal bars at the top and bottom of the beam shall satisfy 9.4.3.4(b) and the stirrups shall satisfy 9.4.4.1.6. The longitudinal bars shall terminate short of face of the wall so that they do not displace the diagonal bars or anchor into the walls. The cage of longitudinal bars shall have sufficient stiffness to prevent it distorting during placement of concrete;

(b) Transverse reinforcement shall be provided for the entire beam cross section, satisfying 9.3.9.6 and 9.4.5, with longitudinal spacing not exceeding the lesser of 150 mm and six times the diameter of the diagonal bars. The spacing of cross ties or legs of hoops both vertically and horizontally in the plane of the beam cross section shall be less than 200 mm. Each cross tie and each hoop leg shall engage a longitudinal bar of equal or larger diameter.

11.4.9.5 *Conventionally reinforced coupling beams*

Conventional coupling beams may be used where the conditions in (a) or (b) below are satisfied:

(a) The ultimate limit state shear force (V^*) acting on the coupling beam neglecting the presence of any axial force ($N_{o,c}$) shall be equal to or less than $0.3\phi\sqrt{f_c'}\,A_{cv}$;

(b) The material strains are equal to or less than the limiting value given in 2.6.1.3.4.

Equal areas of longitudinal reinforcement shall be placed at the top and bottom of the beam. This reinforcement shall be detailed and restrained against buckling as required in 9.4.5. Stirrups shall be designed to resist the shear force over the full length of the beam assuming no shear is resisted by the concrete (V_c is zero), and the axial force $N_{o,c}$ is not acting. Intermediate longitudinal reinforcement, which need not be confined against buckling, shall be spaced at regular intervals between the top and bottom longitudinal reinforcement, which complies with 2.4.4.5, and has an area equal to or greater than one-sixth of the sum of the longitudinal reinforcement areas at the top and bottom of the beam.

12 DESIGN OF REINFORCED CONCRETE TWO-WAY SLABS FOR STRENGTH AND SERVICEABILITY

12.1 Notation

a	larger side of rectangular contact area, mm
A_s	area of non-prestressed tension reinforcement, mm^2
A_v	area of shear reinforcement within a distance s, mm^2
b	width of compression face, or smaller side of rectangular contact area, mm
b_o	perimeter of critical section for slabs and foundations, mm
b_x	is the length of the side of the perimeter, b_o, being considered in design for shear reinforcement, mm
b_1	width of critical section defined in 12.7.1(b) measured in the direction of the span for which moments are determined, mm
b_2	width of the critical section defined in 12.7.1(b) measured in the direction perpendicular to b_1, mm
c_1	size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm
c_2	size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, mm
d	distance from extreme compression fibre to centroid of tension reinforcement, mm
f'_c	specified compressive strength of concrete, MPa
f_{yt}	lower characteristic yield strength of vertical (stirrup) reinforcement, MPa
h	overall thickness of member, mm
h_v	total depth of shearhead cross section, mm
L_1	support centre to support centre span of slab not supported by a beam or wall, mm
L_n	clear span, in the direction moments are being determined, measured face-to-face of supports, mm
L_s	span of slab, mm
L_v	length of shearhead arm from centroid of concentrated load or reaction mm
M^*	design moment at section at the ultimate limit state, N mm
M_p	required plastic moment strength of shearhead cross section, N mm
M_v	moment resistance contributed by shearhead reinforcement, N mm
p	ratio of tension reinforcement = A_s/bd
p_b	value of p for balanced strain conditions derived by 7.4.2.8
s	centre-to-centre spacing of shear or torsional reinforcement measured in the direction parallel to the longitudinal reinforcement, mm
t	thickness of surfacing and filling material, mm
u	larger side of rectangular loaded area allowing for load spread, mm
v	smaller side of rectangular loaded area allowing for load spread, mm
v_c	shear stress resisted by concrete, MPa
V_c	nominal shear strength provided by concrete mechanisms, MPa
V_n	nominal shear strength of section, N
v_n	total nominal shear stress, MPa
V_s	nominal shear strength provided by the shear reinforcement, N
V^*	design shear force at section at the ultimate limit state, N
α_s	factor accounting for columns
α_v	ratio of stiffness of shearhead arm to surrounding composite slab section
β_c	ratio of long side to short side of concentrated load or reaction area
η	number of arms in shearhead connection
ϕ	strength reduction factor (see 2.3.2.2)
θ	skew angle
γ_f	fraction of unbalanced moment considered to be transferred by flexure

γ_v fraction of unbalanced moment considered to be transferred by eccentricity of shear

12.2 Scope

The provisions of this section shall apply to the design of reinforced concrete two-way slab systems subject predominantly to loading acting at right angles to the plane of the slab.

All references in this section to loads, moments, shear forces and torsions refer to actions at the ultimate limit state unless specifically noted otherwise.

12.3 General

12.3.1 *Slab systems*

A slab system may be supported on columns or walls. If supported by columns, no portion of a column capital shall be considered for structural purposes that lies outside the largest inverted right circular cone or pyramid with a 90° vertex that can be included within the outline of the column capital.

12.3.2 *Floor finishes*

When a separate floor finish is placed on a slab it shall be assumed that:

(a) A floor finish is not included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with the requirements of Sections 13 and 18;

(b) All concrete floor finishes may be taken as part of the required cover or total thickness for non-structural considerations.

12.3.3 *Recesses and pockets*

Solid slabs and slabs with recesses or pockets made by permanent or removable fillers between ribs or joists in two directions are included within the scope of this section.

12.3.4 *Panelled ceilings*

Slabs with panelled ceilings are included within the scope of this section, provided the panel of reduced thickness lies entirely within middle strips, and is equal to or greater than the larger of two-thirds of the thickness of the remainder of the slab, excluding the drop panel, nor less than 100 mm thick.

12.3.5 *Prestressed concrete slabs*

For the design of prestressed concrete slabs refer to Section 19.

12.4 Design procedures

12.4.1 *General*

A slab system may be designed by any procedure satisfying conditions of equilibrium and geometrical compatibility if shown that the design strength is at least that required at the ultimate limit state by either AS/NZS 1170 or other referenced loading standard, and that all serviceability conditions are investigated and satisfied at the serviceability limit state.

12.4.2 *Design methods*

The design moments and shears resulting from distributed or concentrated loads shall be determined using one of the following:

(a) Linear elastic analysis for thin plates as in 12.5.3 and 6.3; or

(b) Non-linear analysis as in 12.5.4 and 6.4; or

(c) Plastic analysis as in 6.5.3 and 12.5.5; or

(d) Idealised frame method of analysis as C6.3.8; or

(e) Simplified method of analysis as in 6.7; or

(f) Empirical method for bridge slabs as in 12.8.2.

12.5 Design for flexure

12.5.1 *General*

The slabs and beams (if any) between supports may be proportioned for the moments at the ultimate limit state prevailing at every section. Design for flexure shall be in accordance with Sections 7 and 9. The range of stresses permitted in the reinforcement due to service live load shall also satisfy the limitation specified under 2.5.2 if appropriate. A3

12.5.2 *Effective area of concentrated loads*

The peak moments in slabs induced in the immediate vicinity of concentrated loads may take into account the spread of load from the contact area. For a rectangular contact area with sides of length a and b, the sides of the effective rectangular spread shall not exceed the values for u and v given by: A3

$$u = a + 2t + 3h \qquad \text{(Eq. 12–1)}$$

$$v = b + 2t + 3h \qquad \text{(Eq. 12–2)}$$

Where the load areas derived from Equations 12–1 and 12–2 overlap, the total load shall be considered as uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab.

12.5.3 *Design moments from elastic thin plate theory*

The design bending moments and torsional moments may be determined assuming that the slabs act as thin elastic plates in accordance with 6.3. The assumptions adopted for computing flexural and torsional rigidities of sections shall be consistent throughout the analysis.

The torsional moment in a slab changes with the direction of the axes assumed in the analysis. The reinforcement shall be provided to sustain the principal moments. A3

12.5.4 *Design moments from non-linear analysis*

The design bending moments and torsional moments may be determined taking into account all relevant non-linear and inelastic effects of the materials in accordance with 6.4.

12.5.5 *Design moments from plastic theory*

The design moments may be determined by a plastic theory such as Johansen's yield line theory or Hillerborg's strip method, provided that the ratios between negative and positive moments used are similar to those obtained by the use of elastic thin plate theory. The maximum value for the tension reinforcement ratio, p, used shall not exceed 0.4 of the ratio producing balanced conditions as defined by 7.4.2.8.

12.5.6 *Slab reinforcement*

12.5.6.1 *Size of drop panels*

Where a drop panel is used to reduce the amount of negative moment reinforcement over the column of a flat slab, the size of drop panel shall be in accordance with the following:

(a) The drop panel shall extend in each direction from centreline of the support a distance equal to or greater than one-eighth of the span length measured from centre-to-centre of supports in that direction;

(b) The projection of the drop panel below the slab shall be at least one-quarter of the slab thickness beyond the drop;

(c) In computing the required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed greater than one-quarter of the distance from the edge of the drop panel to the edge of the column or column capital.

12.5.6.2 *Area of reinforcement*

The area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections but shall be equal to or greater than required by 8.8 or more than the limiting value given by the area required to control crack widths as required by 2.4.4.

12.5.6.3 *Spacing of flexural reinforcement*

Spacing of flexural reinforcement shall not exceed the smallest of two times the slab thickness or 300 mm, except for cellular or ribbed construction. In the slab over cellular spaces, or between ribs, the maximum spacing shall be three times the slab thickness.

12.5.6.4 *Extent of positive moment reinforcement at edge*

Positive moment reinforcement perpendicular to a discontinuous supported edge shall extend to the edge of the slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns, or walls.

12.5.6.5 *Anchorage of negative moment reinforcement at edge*

Negative moment reinforcement perpendicular to a discontinuous supported edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, to be developed at the face of the support according to the provisions of Section 8.

12.5.6.6 *Anchorage at edge*

Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

12.5.6.7 *Reinforcement for torsional moments*

In slabs supported on beams or walls, reinforcement shall be provided in the corners to resist the combined actions due to torsion and flexure found from a rational analyses, or the provisions (a), (b) and (c) shall be satisfied:

(a) Torsional reinforcement shall be provided at any corner where the slab is discontinuous at both edges meeting at that corner. It shall consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers, per unit width of slab, shall be at least three-quarters of the area per unit width required for the maximum mid-span positive moment per unit width in the slab;

(b) Torsional reinforcement equal to half that described in (a) shall be provided at a corner contained by edges over only one of which the slab is continuous;

(c) Torsional reinforcement need not be provided at any corner contained by edges over both of which the slab is continuous.

12.5.6.8 *Slabs supported on columns*

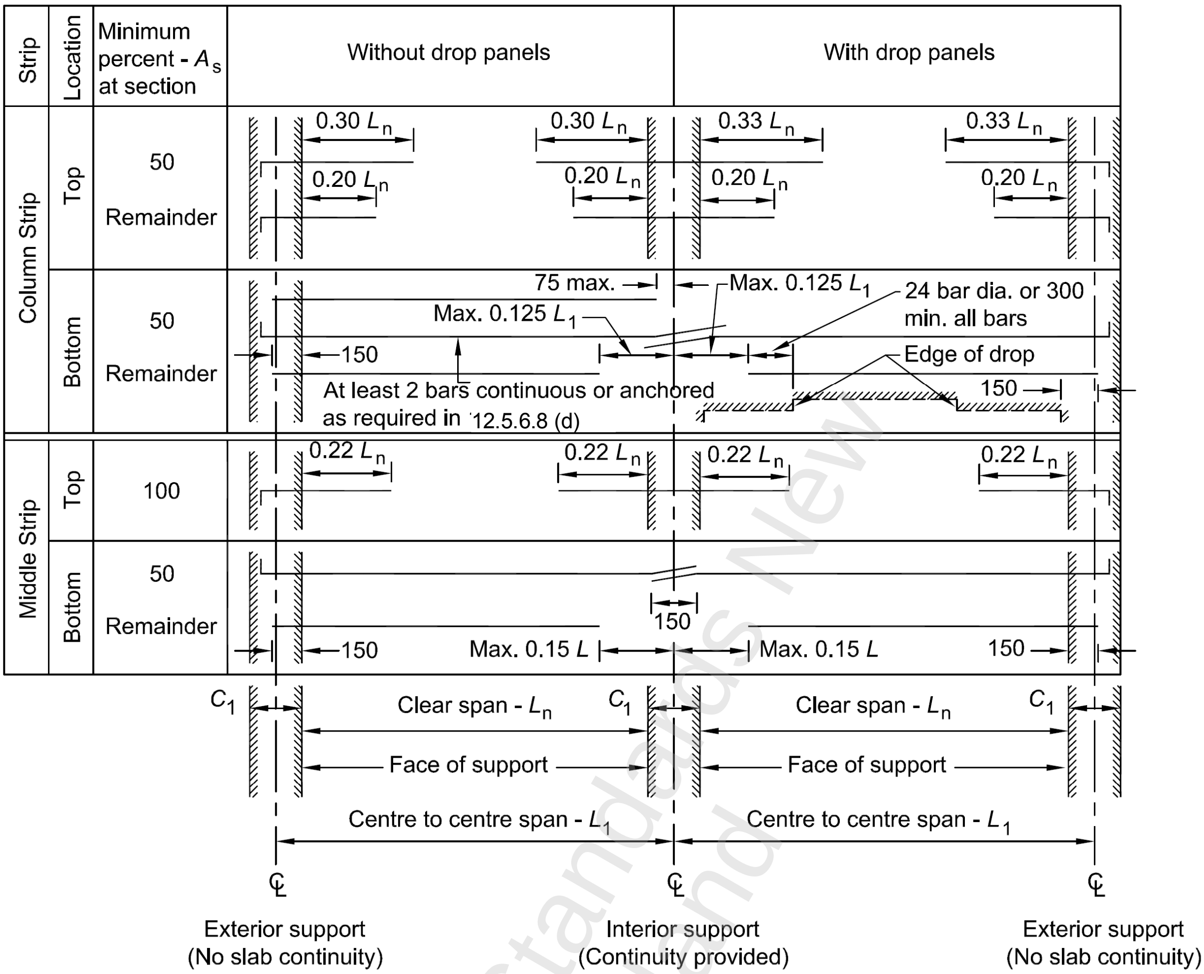
In slabs supported on columns, reinforcement for moments induced by gravity loading shall comply with all the following requirements:

(a) The minimum extensions for reinforcement shall be as prescribed in Figure 12.1;

(b) Where adjacent spans are unequal, extension of negative moment reinforcement beyond the face of the support as prescribed in Figure 12.1 shall be based on requirements of the longer span;

(c) Bent bars shall be used only when the depth-span ratio permits use of bends 45° or less;

(d) Integrity reinforcement shall be provided as required by 12.5.6.9.



A2

Figure 12.1 – Minimum extensions for reinforcement in slabs without beams or walls

12.5.6.9 *Integrity reinforcement for slabs supported on columns*

Slabs supported on columns shall satisfy either (a) or (b) as appropriate.

(a) Where slabs are supported on columns reinforcement in the bottom of the slab, with an area A_{bs}, shall pass through or be anchored in the columns and extend into the slab for a minimum distance of a development length. The area of this reinforcement crossing the interface between the column and the slab, A_{bs}, shall be given by: A3

$$A_{bs} > \frac{2V^*}{\phi f_y} \quad \text{(Eq. 12–3)}$$

(b) In lift slab construction the slab shall be supported on the lower surface by a component or components, which are tied into the column. At least two column strip bottom bars shall be placed in each direction which either pass through the shear head or lifting collar or pass as close to the column as practical. At exterior columns this reinforcement shall be anchored at the shearhead or lifting collar. At all columns this reinforcement shall be extended into the slab beyond collar. At all columns this reinforcement shall be extended into the slab beyond the face of the column for a minimum distance of a development length.

12.6 Serviceability of slabs

12.6.1 *General*

Slabs shall be designed so that the cracking and deflections at the serviceability limit state do not exceed specified limits.

12.6.2 *Cracking*

Flexural cracking slab reinforcement shall comply with the requirements of 12.5.6.2.

12.6.3 *Deflections*

To control deflections the minimum thickness specified in 2.4.3 shall apply unless the calculation of deflection according to 6.8 indicates the lesser thickness may be used without adverse effects.

12.7 Design for shear

12.7.1 *Critical sections for shear*

Shear strength of slabs and footings in the vicinity of concentrated loads or reactions is governed by the more severe of two conditions:

(a) Beam action for the slab or footing, with a critical section perpendicular to the plane of the slab extending across the entire width and located at a distance, d, from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with 7.5 and 9.3.9.3 and 9.3.9.4;

(b) Two-way action for a slab or a footing, with a critical section perpendicular to the plane of the slab and located so that its perimeter, b_o, is a minimum, but need not approach closer than $d/2$ to edges or corners of columns, concentrated loads, reaction areas or changes of slab thickness such as edges of capitals or drop panels. For this condition, the slab or footing shall be designed in accordance with 12.7.2 to 12.7.7. For square or rectangular columns, concentrated loads, or reaction areas, the critical sections may have four straight sides. A circular area may be replaced by a square of equal area.

12.7.2 *Design for two-way action*

The design of a slab or footing for two-way action shall be based on 7.5. V_c shall be computed in accordance with 12.7.3.2, V_s shall be computed in accordance with 12.7.4 except that for slabs with shear heads, V_n shall be in accordance with 12.7.5.4. When moment is transferred between slab and column 12.7.7 shall apply.

12.7.3 *Shear strength*

12.7.3.1 *Nominal shear strength for punching shear*

The nominal shear strength for any portion of the critical perimeter, V_n is given by:

$$V_n = V_s + V_c \qquad \text{(Eq. 12–4)}$$

Where $V_c = v_c b_o d$ and V_s is given by 12.7.4.

and

$$\frac{V^*}{\phi} \le V_n \qquad \text{(Eq. 12–5)}$$

12.7.3.2 *Nominal shear stress resisted by the concrete*

For non-prestressed slabs subject to punching shear the shear stress resisted by the concrete, v_c, shall be the smallest of:

A2 (a) $$v_c = \frac{1}{6} k_{ds}\left(1+\frac{2}{\beta_c}\right)\sqrt{f'_c} \qquad \text{(Eq. 12–6)}$$

where β_c is the ratio of the long side to the short side of the concentrated load or reaction area; or

A2 (b) $$v_c = \frac{1}{6} k_{ds}\left(\frac{\alpha_s d}{b_o}+1\right)\sqrt{f'_c} \qquad \text{(Eq. 12–7)}$$

where α_s = 20 for interior columns, 15 for edge columns, 10 for corner columns; or

(c) $v_c = \frac{1}{3} k_{ds} \sqrt{f'_c}$ ....(Eq. 12–8) A2

where k_{ds} allows for the influence of size on v_c and it is given by $k_{ds} = \sqrt{\frac{200}{d}}$ with the limits of $0.5 \leq k_{ds} \leq 1.0$, where d is the average effective depth round the critical perimeter. A3

12.7.3.3 *Nominal shear stress, v_n for punching shear*

The nominal shear stress for punching shear shall be taken as the sum of:

(a) the shear stress due to the force normal to the slab, as given V_n/b_od

(b) the shear stress due to the transfer of moment to the slab from a column or beam, as given in 12.7.7.

The shear stress shall be based on the perimeter b_o, as defined in 12.7.1(b), with deductions for free edges and openings in the slab as defined in 12.7.6, and the effective depth d.

12.7.3.4 *Maximum nominal shear stress*

The maximum nominal shear stress for punching shear, on any part of the perimeter shall not exceed $0.5\sqrt{f'_c}$.

12.7.3.5 *Shear to be resisted by shear reinforcement for punching shear*

When the nominal shear stress, v_n, on any part of the critical perimeter, b_o, exceeds the critical value of v_c given in 12.7.3.2, the value of v_c round the complete perimeter shall be taken as the smaller of that given by Equations 12–6, 12–7, 12–8 or 12–9. A2

$v_c = \frac{1}{6}\sqrt{f'_c}$ .... (Eq. 12–9)

Shear reinforcement shall be provided to sustain the shear force V_s given by: A2

$V_s = (v_n - v_c)\, b_x d$ .... (Eq. 12–10)

Where b_x is the length of side being considered and d is the effective depth over that length.

12.7.4 *Shear reinforcement consisting of bars or wires or stirrups*

12.7.4.1 *Design requirements*

Shear reinforcement consisting of effectively anchored bars, wires or single or multiple-leg stirrups is permitted in slabs and footings where the effective depth d is greater than or equal to 150 mm and greater than or equal to 16 times the diameter of the shear reinforcement.

12.7.4.2 *Area of shear reinforcement*

Shear reinforcement required on any side to resist V_s given by Equation 12–10, shall be calculated from appropriate expression below:

(a) Where the shear reinforcement is provided by stirrups, with a yield stress f_{yv}, placed at a spacing s, measured on the perimeter b_o, for a length b_x: A2

$A_v f_{yv} \frac{d}{s} \geq V_s$ and $s \leq \frac{d}{2}$ .... (Eq. 12–11)

(b) Where the shear reinforcement is provided by stirrups or bent up bars, which make an angle of α to the axis of the slab and are spaced at a distance, s:

$A_v f_{yv} (\sin\alpha + \cos\alpha) \frac{d}{s} \geq V_s$ and $s \leq \frac{d}{2}$ .... (Eq. 12–12) A2

For the inclined reinforcement to contribute to A_v, the angle this reinforcement makes to the axis of the member, measured from the direction of decreasing flexural tension, shall be 90° or less.

(c) Where only one line of reinforcement is used.

A2 $A_v f_{yv} \sin\alpha \geq V_s$ .... (Eq. 12–13)

12.7.4.3 *Minimum shear reinforcement for punching shear*

Where shear reinforcement is required over any part of the critical perimeter by 12.7.3.5, shear reinforcement shall be equal to or greater than that required to resist a shear force of:

A3

$$V_s = \frac{1}{16}\sqrt{f_c'}\,b_o d \qquad \text{(Eq. 12–14)}$$

12.7.4.4 *Placement of shear reinforcement in the form of vertical stirrups*

A3 The distance between the column face and the first line of vertical stirrup legs that surround the column shall be less than or equal to $d/2$. The spacing between adjacent stirrup legs in the first line of shear reinforcement shall be less than or equal to $2d$ measured in a direction parallel to the column face. The spacing between successive lines of shear reinforcement that surround the column shall be less than or equal to $d/2$ measured in a direction perpendicular to the column face.

12.7.4.5 *Anchorage requirements of shear reinforcement in the form of bars or wires*

Slab shear reinforcement in the form of bars or wires shall engage the longitudinal flexural reinforcement in the direction being considered. A 135° stirrup hook shall be used rather than a 90° hook where there is the possibility of cover concrete being lost at the development of the strength of the member. Closed stirrups shall be used in regions that are likely to reach yield stress (ultimate limit state). Shear reinforcement consisting of vertical bars anchored at each end with plates having an area of at least 10 times the cross-sectioned area of the bars can be used.

12.7.5 Shear reinforcement consisting of structural steel I or channel-shaped sections and other equivalent devices

12.7.5.1 *General*

Shear reinforcement consisting of steel I or channel shapes (shearheads) or other equivalent devices proven by tests to be equally effective may be used in slabs. Provisions of 12.7.5 shall apply where shear due to gravity load is transferred to interior columns. Where moment is transferred 12.7.7.3(c) shall apply.

12.7.5.2 *Details of shearheads*

Details of shearheads shall be as follows:

(a) Each shearhead shall consist of steel shapes fabricated by welding with full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section;

(b) The shearhead shall not be deeper than 70 times the web thickness of the steel shape;

(c) The ends of each shearhead arm may be cut at angles equal to or greater than 30° with the horizontal, provided the plastic moment strength of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead;

(d) All compression flanges of steel shapes shall be located within $0.3d$ of the compression surface of the slab;

(e) The ratio α_v between the flexural stiffness for each shearhead arm and that for the surrounding composite cracked slab section of width $(c_2 + d)$ shall be equal to or greater than 0.15;

(f) The plastic moment strength, M_p, required for each arm of the shearhead shall be computed by:

$$M_p = \frac{V^*}{2\phi\eta}\left[h_v + \alpha_v\left(L_v - \frac{c_1}{2}\right)\right] \qquad \text{(Eq. 12–15)}$$

where ϕ is the strength reduction factor for flexure, η is the number of arms and L_v is the minimum length of each shearhead arm required to comply with the requirements of 12.7.5.3 and 12.7.5.4.

12.7.5.3 *Critical slab section for shear*

The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-quarters of the distance $[L_v - (c_1/2)]$ from the column face to the end of the shearhead arm. The critical section shall be located so that its perimeter, b_o, is a minimum, but need not be closer than the perimeter defined in 12.7.1(b).

12.7.5.4 *Limit on nominal shear strength*

The nominal shear strength, V_n shall not be taken greater than $0.33\sqrt{f'_c}\ b_o d$ on the critical section defined in 12.7.5.3. When shearhead reinforcement is provided it shall not be taken greater than $0.6\sqrt{f'_c}\ b_o d$ on the critical section defined in 12.7.1(b).

12.7.5.5 *Moment of resistance contributed by shearhead*

A shearhead may be assumed to contribute a moment of resistance, M_v, to each slab column strip computed by:

$$M_v = \frac{\phi \alpha_v V^*}{2\eta}\left(L_v - \frac{c_1}{2}\right) \quad \text{(Eq. 12–16)}$$

where ϕ is the strength reduction factor for flexure, η is the number of arms and L_v is the length of each shearhead arm actually provided. However, M_v shall not exceed the smallest of:

(a) 30 % of the total factored moment required for each slab column strip;

(b) The change in the column strip moment over length L_v;

(c) The value of M_p computed by Equation 12–15;

when unbalanced moments are considered, the shearhead must have adequate anchorage to transmit M_p to the column.

12.7.6 *Openings in slabs*

When openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in Section 6, the critical slab sections for shear defined in 12.7.1(b) and 12.7.5.3 shall be modified as follows:

(a) For slabs without shearheads, that part of the perimeter of the critical section that is enclosed by straight lines projecting from the centroid of the column, concentrated load or reaction area and tangent to the boundaries of the openings shall be considered ineffective;

(b) For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in (a) above.

12.7.7 *Transfer of moment and shear in slab column connections*

12.7.7.1 *General*

When gravity load, wind or other lateral forces cause transfer of unbalanced moment M^* between a slab and a column, a fraction $\gamma_f M^*$ of the unbalanced moment shall be transferred by flexure in accordance with 12.7.7.2. The remainder of the unbalanced moment given by $\gamma_v M^*$ shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in 12.7.1(b) where:

$$\gamma_v = 1 - \gamma_f \quad \text{(Eq. 12–17)}$$

12.7.7.2 *Unbalanced moment transferred by flexure*

The fraction of the unbalanced moment $\gamma_f M^*$ shall be considered to be transferred by flexure within an effective slab width between lines that are one and one-half slab or drop panel thicknesses (1.5h) outside opposite faces of the column or capital, where M^* is the moment to be transferred and:

$$\gamma_f = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} \quad \text{(Eq. 12–18)}$$

For unbalanced moment about an axis parallel to the edge at exterior supports the value of γ_f by Equation 12–18 may increase up to 1.0 provided that V^* at an edge support does not exceed $0.75\phi V_c$ or at a corner support does not exceed $0.5\phi V_c$. For unbalanced moments at interior supports, and for unbalanced moments about an axis transverse to the edge at exterior supports, the value of γ_f in Equation 12–18 may be increased by 25 % provided that V^* at the support does not exceed $0.4\phi V_c$. The reinforcement ratio p within the effective width defined in 12.7.7.2 shall not exceed $0.375p_b$, where p_b is the balanced reinforcement ratio. Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in 12.7.7.2.

12.7.7.3 *Unbalanced moment transferred by eccentricity of shear*

A fraction of the unbalanced moment $\gamma_v M^*$, considered to be transferred by eccentricity of shear results in a shear stress which shall be assumed to vary linearly about the centroid of the critical section defined in 12.7.1(b). The maximum shear stress due to the design shear force V^* and moment $\gamma_v M^*$ shall not exceed ϕv_n, where:

(a) For members without shear reinforcement:

$$\phi v_n = \frac{\phi V_c}{b_o d} \quad \text{(Eq. 12–19)}$$

(b) For members with shear reinforcement other than shearheads:

$$\phi v_n = \frac{\phi (V_c + V_s)}{b_o d} \quad \text{(Eq. 12–20)}$$

where V_c and V_s are defined in 12.7.3.2, 12.7.3.5 and 12.7.4. The design shall take into account the variation of shear stress around the column. The shear stress shall not exceed $0.17\sqrt{f'_c}$ at the critical section located $d/2$ outside the outermost line of stirrup legs that surround the column;

(c) When shear reinforcement consisting of shearheads is used the sum of the shear stresses due to vertical load acting on the critical section defined by 12.7.5.3 and the shear stress resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in 12.7.1(b) shall not exceed $\phi 0.33\sqrt{f'_c}$.

12.8 Design of reinforced concrete bridge decks

12.8.1 *Design methods*

Two methods of design may be used for reinforced concrete bridge deck slabs supported on beams or girders.

(a) Empirical design based on assumed membrane action, in accordance with 12.8.2; or

(b) Elastic plate bending analysis in accordance with 12.8.3;

where the dimensional and structural limitations of the empirical design method are not met, or for deck cantilevers, the elastic plate bending analysis design method shall be used.

A2

12.8.2 *Empirical design based on assumed membrane action*

12.8.2.1 *General*

A3

Slabs intended to carry highway vehicle loads as specified by the New Zealand Transport Agency's Bridge Manual, satisfying the requirements below, and designed in accordance with this method need not be analysed for bending moments and shears in the slab due to traffic loading, and the requirements of Section 2 and 9 shall be waived. This method is not applicable to the design of slabs for oversize wheel loads (e.g. for the loads from the oversize wheels of large earth-moving, plant or agricultural machinery). This method is not applicable to cantilevered slabs.

12.8.2.2 *Conditions*

The empirical design method shall only be used if all of the following conditions are satisfied: A2

(a) The supporting components are made of steel or concrete;

(b) The deck is fully cast-in-place and water cured;

(c) The deck is of uniform depth except for haunches at girder flanges and other local thickening;

(d) The deck is made composite with the supporting structural components. In the case of the negative moment region of continuous steel girders, two or more shear connectors at a spacing of less than 600 mm shall be provided;

(e) Cross-frames or diaphragms are used throughout the cross section at lines of support;

(f) For cross sections involving torsionally stiff units such as individual separated box beams, either intermediate diaphragms between the boxes are provided at a spacing equal to or less than 8.0 m, or A3 the need for supplemental reinforcement over the webs to accommodate transverse bending between A2 the box units is investigated, and reinforcement is provided if necessary;

(g) The ratio of the span length, L_s, to design depth does not exceed 18.0 and is between 6.0 and 18.0, A3 where the design depth is the slab thickness excluding any sacrificial wearing surface; A2

(h) The span length, L_s, as specified in 12.8.4 is equal to or less than 4.100 m; A3

(i) The minimum slab thickness is equal to or greater than 175 mm, excluding any sacrificial wearing surface; A2

(j) The core depth of the slab is equal to or greater than 100 mm. The core depth is defined as the slab thickness less any wearing surface and the top and bottom cover thicknesses; A3

(k) There is an overhang beyond the centreline of the outside girder of at least five times the slab thickness. This condition may be considered satisfied if the overhang is at least three times the slab thickness and a structurally continuous concrete kerb or barrier is made composite with the overhang; A2

(l) The specified 28 day strength of the deck concrete is equal to or greater than 30 MPa;

(m) The design vehicle axle and wheel loads being no greater in their effects than those imposed by the New Zealand Transport Agency Bridge Manual design vehicle loadings. A3

12.8.2.3 *Reinforcement*

For slabs meeting the above conditions, the deck reinforcement shall comprise:

(a) Layers of reinforcement in two directions at right angles in the top and bottom of the slab, placed as close to the outside surfaces as possible, as permitted by cover requirements;

(b) The reinforcing steel shall have a yield strength greater than or equal to 420 MPa;

(c) The minimum amount of reinforcement shall be 570 mm^2/m of steel in each direction in the bottom layer, 380 mm^2/m of steel in each direction in the top layer;

(d) All reinforcement shall be straight bars except that hooks may be provided where required;

(e) The maximum spacing of the reinforcement may be 300 mm;

(f) The bars shall be spliced by lapping or by butt welding, or by mechanical connections satisfying 8.7.5.2 only;

(g) For skew angles, θ, greater than 25°, the specified reinforcement in both directions shall be doubled in the end regions of the deck. The span end regions are as defined in Figure 12.2.

A3

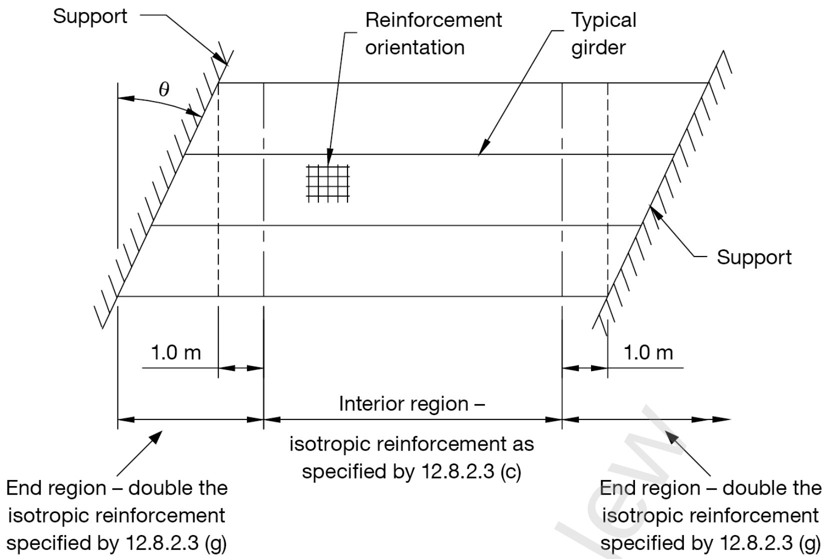


Figure 12.2 – Reinforcement of skewed slabs by the empirical method

A2

12.8.2.4 *Longitudinal negative moments in continuous structures*

The longitudinal bars of the isotropic reinforcement may participate in resisting negative moments at an internal support in a continuous structure.

12.8.2.5 *Minimum slab thickness*

For deck slabs designed by the empirical method of 12.8.2, the minimum slab thickness requirements of 12.8.2 shall take precedence over the other requirements of NZS 3101.

12.8.3 *Design based on elastic plate bending analysis*

12.8.3.1 *Determination of moments*

The moments in deck slabs due to the local effects of wheels shall be determined by an elastic analysis, assuming the slab to act as a thin plate. Adequate allowance shall be made for the effects of the rotation of the edges monolithic with beams, due to torsional rotation of the beams, and the effects of relative displacement of beams.

12.8.3.2 *Deck slab also functioning as a flange*

Where the deck slab resists the effects of live load by the top flange of a box girder also functioning as the deck slab, or a transverse distribution member integral with the deck slab the slab, shall be designed for the sum of the effects of the appropriate loading for each condition.

12.8.3.3 *Haunched slabs*

Where slabs are haunched at fixed edges, allowance for the increase in support moment due to the haunch shall be made either by modifying the moments determined for slabs of uniform thickness, or by a rational analysis that takes into account the varying section.

A2

12.8.4 *Span length of reinforced concrete bridge deck slabs*

12.8.4.1 *General*

The following requirements apply to the determination of slab span length, L_s, as used in TTable 2.3 and in the application of the empirical design method specified in 12.8.2.

12.8.4.2 *Slab span for a uniform slab monolithic with webs*

For a uniform concrete slab, monolithic with concrete webs, the slab span length, L_s, shall be taken as the clear span.

12.8.4.3 *Span length for a haunched slab*

For a slab monolithic with concrete webs or tied down to steel girders locally haunched adjacent to its supports, where the thickness at the root of the haunch is at least 1.5 times the thickness at the centre of the slab and the length of the haunch is less than twice the slab thickness, the slab span length, L_s, shall be taken as the distance between the mid-points of opposite haunches. A2

12.8.4.4 *Span length of slab on steel girders*

For a uniform slab on steel girder, the slab span, L_s, shall be taken as the average of the distance between webs and the clear distance between flange edges.

12.8.4.5 *Span length of slab spanning between non-uniformly spaced supports*

Where the spacing of supporting components varies, the span length, L_s, shall be taken as the larger of the deck lengths at the two locations shown in Figure 12.3.

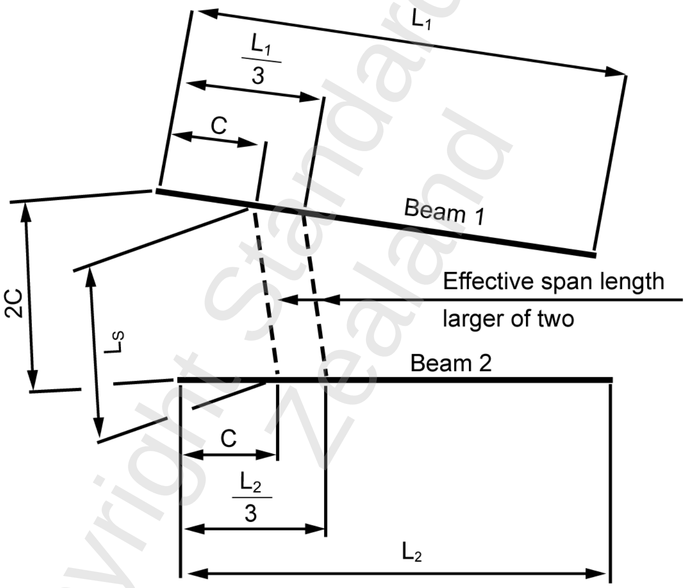


Figure 12.3 – Effective span length for non-uniform spacing of beams

NOTES

13 DESIGN OF DIAPHRAGMS

13.1 Notation

A3

G permanent action (self-weight or 'dead' action)
Q imposed action (due to occupancy and use, 'live' action)
ψ_c combination factor for imposed action

13.2 Scope and definitions

Provisions of this section apply to diaphragms in buildings. They are defined as relatively thin but stiff horizontal or nearly horizontal structural systems which transmit in-plane lateral forces to, or between lateral force-resisting elements.

Diaphragms which are not designed to dissipate energy at the ultimate limit state shall meet the requirements of 13.3. Diaphragms designed to dissipate energy shall meet the requirements of 13.3 as modified by 13.4.

13.3 General principles and design requirements

13.3.1 *Functions of diaphragms*

Diaphragms may be required to function simultaneously as floors subjected to gravity loads and as diaphragms to transfer in-plane actions due to lateral forces. Seismically induced actions shall be calculated in accordance with NZS 1170.5, or another referenced loading Standard. A3

13.3.2 *Analysis procedures*

The ultimate limit state design of diaphragms shall be based upon strut and tie models in accordance with Appendix A.

Diaphragm stiffness and the effects of creep, shrinkage and thermal gradients shall be considered at the serviceability limit state.

13.3.3 *Openings*

Penetrations of diaphragms by openings shall not impair the feasible transmission of internal forces. Analysis and design shall be based on strut-and-tie models simulating admissible and effective in-plane load paths between and around openings.

13.3.4 *Stiffness*

Analysis for the internal forces transmitted between diaphragms and their supports shall account for the stiffness of the chosen load path as dictated by the presence of openings.

13.3.5 *Reinforcement shall be anchored*

Reinforced concrete slabs cast with the supporting beams, columns or walls designed to carry gravity loads in one-way or in two-ways in accordance with Section 12 shall be reinforced in two orthogonal directions with an amount in each direction equal to or greater than that required by 8.8. For diaphragm actions such reinforcement shall be developed beyond the edges of slab panels within boundary beams or walls.

13.3.6 *Changes in depth*

Where changes in the depth of the diaphragm are provided, the impact of this on force transfer shall be considered.

13.3.7 *Diaphragms incorporating precast concrete elements*

13.3.7.1 *Composite concrete flexural members*

Where composite action of precast concrete floor elements and a cast-in-place concrete topping is relied on, the requirements of 18.5 shall be satisfied.

13.3.7.2 *Requirements for toppings transferring diaphragm forces*

A cast-in-place reinforced concrete topping over precast floor systems may be used to transfer diaphragm forces, provided that:

(a) The cast-in-place concrete topping is at least 50 mm thick; and

(b) Minimum reinforcement in two principal directions in accordance with 8.8 is placed in the topping slab; and

(c) Class E reinforcing bars to AS/NZS 4671 are provided around the perimeter of the floor span in accordance with 13.3.7.3; and

(d) Either:

   (i) The requirements of 18.5.4.1 relating to the interface between the *in situ* topping and precast units are satisfied; or

   (ii) Connections between precast elements and the cast-in-place topping are provided in accordance with 13.4.3.

13.3.7.3 *Starter and continuity bars*

At the perimeter of the floor, Class E starter bars to AS/NZS 4671 shall be provided to anchor the topping to the supporting element.

At interior supports, where the floor is continuous over the supports, Class E continuity bars shall be provided in the topping above and perpendicular to the supporting member.

The required area and the length of the this reinforcement shall be determined by analysis but shall have a capacity in excess of 100 kN/m and extend into the topping beyond the end/edge of the precast by at least 600 mm. The curtailment of these bars shall be staggered to ensure that no more than 50 % of the bars are curtailed at the same location.

13.3.7.4 *Transfer of diaphragm forces across joints in untopped systems*

Diaphragm action of precast and cast-in-place systems without an effective cast-in-place concrete topping shall be assumed only if the transfer of in-plane forces across appropriately formed joints between concrete components, consistent with diaphragm action in both principal axes of the structural system, is equivalent to that of a cast-in-place concrete slab with reinforcement satisfying at least the requirements of 8.8.

13.3.7.5 *Connection of diaphragm to primary lateral force-resisting system*

Connections by means of reinforcement from precast or cast-in-place concrete diaphragms to components of the primary force-resisting systems shall be adequate to accommodate the expected deformations at the interface while maintaining load paths.

13.3.8 *Reinforcement detailing for elastically responding diaphragms*

When diaphragms are designed for earthquake forces in accordance with NZS 1170.5, or other referenced loading standard, no special requirements for detailing of reinforcement for ductility need be satisfied.

13.3.9 *Strength of diaphragms in shear*

The strength design of diaphragms for shear shall be based upon strut and tie models in accordance with Appendix A.

13.3.10 *Columns to be tied to diaphragms*

Columns are to be tied to diaphragms in accordance with 10.3.6.

13.4 Additional design requirements for elements designed for ductility in earthquakes

13.4.1 *Design forces for designed to dissipate energy diaphragms*

Diaphragms designed to dissipate energy from earthquake induced forces shall only be permitted when justified by special theoretical and experimental studies.

13.4.2 *Reinforcement detailing*

In diaphragms designed as permitted by 13.4.1, inelastic regions must be clearly identified and appropriate detailing of the reinforcement corresponding to the relevant requirements of Sections 7 and 11 shall be provided.

13.4.3 *Diaphragms incorporating precast concrete elements*

13.4.3.1 *Precast shall be tied to topping*

In potential plastic hinge regions, the consequences of delamination of the topping from the precast members shall be assessed. Where composite action is required to support $G + \Psi_c Q$ as defined in AS/NZS 1170 Part 0, connectors shall be provided between the topping and precast to satisfy:

(a) Ties with an effective area of 40 mm^2 per m^2 of floor area, or equivalent connectors, shall connect the topping to the precast element;

(b) Spacing of connectors shall not exceed 1500 mm, and the tributary area of topping reliant on each connector shall not exceed 2.25 m^2;

(c) Connectors shall engage horizontal reinforcement, or shall be otherwise effectively anchored into both the topping and the precast element, or into the joints between precast elements where contact surfaces are in accordance with 18.5.4.1.

NOTES

14 FOOTINGS, PILES AND PILE CAPS

14.1 Notation

A_g gross area of section, mm^2
A_s area of non-prestressed reinforcement, mm^2 A3

d the effective depth of the pile cap, footing or spread footing, mm
d_p diameter or side dimension of pile at footing base, mm A3

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa
p_t ratio of area of non-prestressed tension reinforcement to gross section area, A_s/A_g A3

ϕ strength reduction factor (see 2.3.2.2)

14.2 Scope

The provisions of this section shall apply for the structural design of isolated and combined footings. Basic principles for the structural design of piles are also included.

Footings, piles and pile caps containing plastic regions with material strain demands less than or equal to the limits for nominally ductile plastic regions defined in Table 2.4 shall meet the requirements of 14.3. Footings, piles and pilecaps which are designed for ductility in response to earthquake effects shall meet the requirements of 14.3 as modified by 14.4.

14.3 General principles and requirements

14.3.1 *Serviceability and ultimate limit state design*

The base area of footings or the number and arrangement of piles shall be determined from the greater of:

(a) The external forces and moments resulting from ultimate limit state loads (transmitted by the foundation element to the soil or piles) and ultimate soil pressure or the ultimate pile capacity selected through principles of soil mechanics; or
(b) The footing area or number and arrangement of piles necessary to ensure overall and differential settlement criteria are met at the serviceability limit state.

14.3.2 *Design of pile caps*

Pile caps shall be designed using either flexural theory or a strut-and-tie approach.

14.3.3 *Moment in footings*

14.3.3.1 *Moment on a section*

The moment on any section of a footing shall be determined by passing a vertical plane through the footing and computing the moment of the forces acting over the entire area of footing on one side of that vertical plane.

14.3.3.2 *Critical design section*

The maximum design moment for an isolated footing shall be computed as prescribed in 14.3.3.1 at critical sections located as follows:

(a) At the face of a column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall;
(b) Halfway between the middle and edge of a wall, for footings supporting a masonry wall;
(c) Two times the base plate thickness out from the column face for a footing supporting a column with an unstiffened base plate;
(d) By rational analysis for a column supported on a base plate with stiffeners.

14.3.3.3 *Strength of footings in flexure*

Footings that resist imposed actions as beams or one-way slabs, shall be designed based on the assumptions of 7.4 and 9.3.6 and 9.3.8.

Footings that resist imposed actions as two-way slabs, shall be designed in accordance with 12.5.

14.3.3.4 *Foundation elements supporting circular or regular polygon shaped columns or pedestals*

Circular or regular polygon shaped concrete columns or pedestals may be treated as square members of the same area for determining the location of critical sections for moment, shear, and development of reinforcement in foundation or elements.

14.3.4 *Shear in footings*

14.3.4.1 *General*

The shear design of footings that resist imposed actions as beams or one-way slabs, shall be designed based on the assumptions of 9.3.9 and 12.7.

The shear design of footings that resist imposed actions as two-way slabs, shall be designed in accordance with 12.7.

14.3.4.2 *Spread footings and footing supported by piles*

The location of the critical section for shear in accordance with 7.5 shall be measured from the face of a column, pedestal, or wall, for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from the location defined in 14.3.3.2 (c) and (d).

14.3.4.3 *Shear in pile caps*

The computation of shear on any section through a pile cap shall be in accordance with the following:

A3 (a) The entire reaction from any pile whose centre is located $d/2$ or more outside the section shall be considered as producing shear on that section;

A3 (b) The reaction from any pile whose centre is located $d/2$ or more inside the section shall be considered as producing no shear on that section;

(c) For intermediate positions of pile centre, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between the full value at $d/2$ outside
A3 the section and zero value at $d/2$ inside the section.

14.3.5 *Development of reinforcement in footings*

14.3.5.1 *General*

Detailing for the development of reinforcement in footing shall be in accordance with Section 8.

14.3.5.2 *Development of reinforcement*

The calculated tension or compression in reinforcement at each section shall be developed on each side of that section by sufficient embedment length, end anchorage, hooks (tension only), or a combination thereof, and in the case of mesh, by overlapping grids.

14.3.5.3 *Critical sections for development*

Critical sections for the development of reinforcement shall be assumed at the same locations as defined in 14.3.3.2 for maximum design moment, and at all other vertical planes where changes of section or reinforcement occur.

14.3.5.4 *Curtailment of reinforcement*

Curtailment of flexural reinforcement shall be in accordance with 8.6.12, 8.6.13 and 8.6.14.

14.3.6 *Piled foundations*

14.3.6.1 *General*

The design of piles shall in addition to the requirements of Section 2, include due consideration of the loads associated with installation of the piles.

14.3.6.2 *Strength of piles in axial load and flexure*

Reinforced concrete piles shall be designed for axial load and flexure in accordance with 7.4 and 10.3.4. A3

Prestressed concrete piles shall be designed for axial load and flexure in accordance with 19.3.6 and 19.3.7.2.

14.3.6.3 *Details for upper ends of piles*

It shall be assumed that plastic hinges form in the upper ends of piles, except where it can be established that movement of the structure relative to the ground, or ground deformation, will not cause yielding of the longitudinal reinforcement. Such potential plastic hinges shall be reinforced as potential plastic hinge regions.

14.3.6.4 *Longitudinal reinforcement in reinforced concrete piles*

In regions where yielding of the reinforcement is expected at the ultimate limit state, and over a length defined by 14.3.6.10, the minimum amount of reinforcement, and detailing, shall be as specified by 10.3.8.

In regions where yielding of the reinforcement is not expected at the ultimate limit state, the minimum amount of reinforcement shall be as specified by 14.3.6.5.

14.3.6.5 *Minimum longitudinal reinforcement in reinforced concrete piles*

The minimum longitudinal reinforcement ratio, p_t, in piles shall be as follows:

(a) For piles having a gross area of section, A_g, equal to, or less than 0.5×10^6 mm^2, p_t shall be equal to or greater than $2.4/f_y$;

(b) For piles having a cross-sectional area, A_g, equal to or greater than 2×10^6 mm^2, p_t shall be equal to or greater than $1.2/f_y$;

(c) For piles having a cross-sectional area, A_g, between 0.5×10^6 mm^2, and 2×10^6 mm^2, p_t shall be equal to or greater than given by Equation 14–1;

$$p_t = \frac{2400}{f_y\sqrt{2A_g}} \qquad \text{(Eq. 14–1)}$$

14.3.6.6 *Maximum longitudinal reinforcement in reinforced concrete piles*

The area of longitudinal reinforcement in piles shall be less than 0.08 times the gross area, A_g, at any location including lap splices. A3

14.3.6.7 *Longitudinal reinforcement in prestressed concrete pile*

Members with average prestress f_{ps} less than 1.5 MPa shall have minimum reinforcement in accordance with 14.3.6.4.

In regions where yielding of the reinforcement is expected at the ultimate limit state, and over a length defined by 14.3.6.10, the minimum amount of reinforcement and detailing shall be as specified by 19.4.4.1 to 19.4.4.4.

14.3.6.8 *Strength of piles in shear*

Reinforced concrete piles shall be designed for shear based on the assumptions of 7.5 and 10.3.10. For piles smaller than 250 mm square or circular, the minimum shear reinforcement requirements of 10.3.10.4.4 may be waived if the design shear force, V^*, is less than one-half of the shear strength provided by the concrete (ϕV_c).

Prestressed concrete piles shall be designed for shear in accordance with 19.3.11. For piles smaller than 250 mm square or circular, the minimum shear reinforcement requirements of 10.3.10.4.4 may be waived if the design shear force, V^*, is less than one-half of the shear strength provided by the concrete (ϕV_c).

14.3.6.9 *Piled foundations with permanent casing*

For piled foundation systems the permanent shell or casing of a pile may be considered as providing a proportion of the strength of the pile. For steel casings an appropriate allowance shall be made for loss of wall thickness by corrosion during the specified intended life of the structure.

14.3.6.10 *Transverse reinforcement for confinement and lateral restraint of longitudinal bars*

A3 Where yielding of the longitudinal reinforcement is expected, transverse reinforcement complying with 10.3.10.5 for circular piles, and 10.3.10.6 for square piles shall be provided. Where this yielding occurs at the junction of the pile with a pile cap or other overlying structure this transverse reinforcement shall be provided over the length defined by 10.4.5. Where yielding of the longitudinal reinforcement may occur at depth in the ground, this transverse reinforcement shall be provided either side of the level of maximum moment, taking into account the possible variability of this level due to such factors as the variability in the soil stiffness, liquefaction of soil layers, and for river crossings the variability in the depth of scour, over a length which shall be the greater of:

(a) The length defined in 10.4.5 plus three times the pile section depth;
(b) Twice the length defined in 10.4.5.

Also allowance shall be made for migration of the location of maximum moment upwards towards the ground surface as plasticity in the pile develops with the final depth being approximately 70 % of that predicted by elastic analysis.

14.4 Additional design requirements for members designed for ductility in earthquakes

14.4.1 *Designing for ductility*

14.4.1.1 *General*

The foundation system shall maintain its ability to support the design gravity loads while sustaining the chosen earthquake energy dissipating mechanisms in the structure at the development of the relevant overstrength actions of the structure.

14.4.1.2 *Compliance with additional requirements*

All members shall comply with the additional requirements for members designed for seismic forces as set down in the relevant sections of this Standard. However, flexural members, other than piles, which have a nominal strength greater than the greatest total seismic action that can be transmitted to them from the superstructure, need not comply with these requirements.

14.4.1.3 *Longitudinal reinforcement*

Within the region defined by 14.3.6.10, longitudinal reinforcement for reinforced concrete piles shall comply with 10.4.6.

For prestressed concrete piles, longitudinal reinforcement shall comply with19.4.4.1 to 19.4.4.3 within the region defined by 14.3.6.10.

14.4.1.4 *Transverse reinforcement*

Within the region defined by 14.3.6.10, transverse reinforcement for reinforced concrete piles shall comply A1 with 10.4.7.

For prestressed concrete piles, transverse reinforcement shall comply with 19.4.4.4 and 19.4.4.5 within the region defined by 14.3.6.10.

14.4.2 *Pile caps*

Where earthquake induced moments are to be transmitted at the intersection of columns and pile caps, design of this region as a beam-column joint shall be in accordance with 15.4.

15 DESIGN OF BEAM-COLUMN JOINTS

15.1 Notation

A_g gross area of column section, mm^2

A_{jh} total area of effective horizontal joint shear reinforcement in the direction being considered, mm^2

A_{jv} total area of effective vertical joint shear reinforcement, mm^2

A_s area of non-prestressed tension beam reinforcement including bars in effective tension flanges, where applicable, mm^2

A3

A_s^* greater of the area of top or bottom beam reinforcement passing through a joint, mm^2

b_c overall width of column, mm

b_j effective width of joint, mm (see 15.3.4)

b_w web width, mm

C_j $\dfrac{V_{jh}}{V_{jx} + V_{jz}}$

A3

e eccentricity between the centrelines of the webs of a beam and a column at a joint, mm

f'_c specified compressive strength of concrete, MPa

A3

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

f_{yh} lower characteristic yield strength of horizontal joint shear reinforcement, MPa

f_{yv} lower characteristic yield strength of non-prestressed vertical joint shear reinforcement, MPa

h_b overall depth of beam, mm

h_c overall depth of column in the direction of the horizontal shear to be considered, mm

P_{cs} force after all losses in prestressing steel that is located within the central third of the beam depth, N A3

N^* design axial column load at ultimate limit state, N

N_o^* minimum design axial column load at the ultimate limit state, consistent with capacity design principles where relevant and including vertical prestressing where applicable, taken positive when causing compression occurring simultaneously with V_{jh}, N

V_{ch} nominal horizontal shear force transferred across a joint by the diagonal compression strut mechanism, N

V_{cv} nominal vertical shear force transferred across a joint by the diagonal compression strut mechanism, N

V_{jh} nominal horizontal shear force transferred across a joint in the direction being considered, N

V_{jx} nominal horizontal joint shear force transferred in x direction, N A3

V_{jv} nominal vertical shear force transferred across a joint, N

V_{jz} nominal horizontal joint shear force transferred in z direction, N

V_{sh} nominal horizontal shear force transferred across a joint by the truss mechanism, N

V_{sv} nominal vertical shear force transferred across a joint by the truss mechanism, N

V_{jh}^* design horizontal shear force across a joint, N

V_{ojh}^* design horizontal shear force across a joint at overstrength, N

V_{jv}^* design vertical shear force across a joint, N

V_v^* design vertical shear force, N A3

α_i factor for determining V_{ch}

α_v factor for determining V_{cv}

β ratio of area of compression beam reinforcement to that of the tension beam reinforcement at exterior beam-column joint, not to be taken larger than unity

ΔA_{jh} permitted reduction in horizontal joint shear reinforcement, mm^2

ϕ strength reduction factor for beam-column joints, 0.75 for 15.3 and 1.0 for 15.4 A3

15.2 Scope

15.2.1 *General*

A3 Provisions of this section apply to the design of beam-column joints subject to shear where a minimum of 70 % of the top beam reinforcement and 70 % of the bottom beam reinforcement contributing to nominal flexural strength either pass through the column or are anchored in the column. Where this does not apply, design shall be based on strut and tie analysis.

A2 Where all regions immediately adjacent to a beam-column joint remain elastic or contain nominally ductile plastic regions, the joint zone shall be designed to meet the requirements of 15.3. Where any of the members framing into the joint zone contain ductile or limited ductile plastic regions adjacent to the joint zone, it shall be designed to meet the requirements of 15.3 as modified by 15.4. The written requirements take precedence over Table C15.1.

15.2.2 *Alternative methods*

In lieu of the methods specified in 15.3 and 15.4, principles of mechanics based on strut-and-tie models or equivalent may be used to determine the internal forces and hence the required shear reinforcement and anchorages in beam-column joints.

15.3 General principles and design requirements for beam-column joints

15.3.1 *Design criteria*

Beam-column joints shall satisfy the following criteria:

(a) At the serviceability limit state, a joint shall perform at least as well as the members that it joins;

(b) At the ultimate limit state, a joint shall have a design strength sufficient to resist the most adverse load combinations sustained by the adjoining members, as specified by AS/NZS 1170 or other referenced loading standard.

15.3.2 *Design forces*

A3 The design forces acting on a beam-column joint shall be evaluated from the maximum internal forces introduced by all members meeting at the joint with the joint in equilibrium, subjected to the most adverse combination of ultimate limit state loads as required by AS/NZS 1170 or other referenced loading standard. The forces acting on the joint shall be determined based on the nature of the critical load combination as follows:

(c) Where the most adverse combination of loads results from non-seismic actions, the forces acting on the joint shall be those obtained from a suitable analysis;

(d) Where a nominally ductile plastic region may form in a member connected to a joint, the shear force in the joint zone shall be calculated on the basis that the reinforcement in the plastic region is at yield.

15.3.3 *Consideration of concurrency*

Where beams frame into the joint from two directions, these forces need only be considered in each direction independently. A2

15.3.4 *Maximum horizontal joint shear force*

The horizontal design shear force across a joint, V^*_{jh}, shall not exceed the smaller of 0.20 $f'_c b_j h_c$, or $10 b_j h_c$ where h_c is the overall depth of the column in the direction of the horizontal shear to be considered and the effective joint width, b_j, shall be taken as:

A2 (a) Where $b_c \geq b_w$:
either $b_j = b_c$, or $b_j = b_w + 0.5\, h_c$, whichever is the smaller;

(b) Where $b_c < b_w$:
either $b_j = b_w$, or $b_j = b_c + 0.5\, h_c$, whichever is the smaller.

15.3.5 *Design principles, mechanisms of shear resistance*

The joint shear shall be assumed to be resisted by a concrete mechanism and a truss mechanism, comprising horizontal and vertical stirrups or bars and diagonal concrete struts.

Superposition of the two mechanisms for horizontal and vertical joint shear transfer results in nominal shear forces being transferred across the joint core as follows:

$$V_{jh} = V_{ch} + V_{sh} = V_{ch} + A_{jh} f_{yh} \quad \text{(Eq. 15–1)}$$

$V_{jv} = V_{cv} + V_{sv} = V_{cv} + A_{jv}f_{yv}$ .... (Eq. 15–2)

Where V_{ch} and V_{cv} are the nominal horizontal and vertical shear forces transferred across the joint core by the diagonal compression strut mechanism respectively, and V_{sh} and V_{sv} are the nominal horizontal and vertical shear forces transferred across the joint core by the truss mechanism, across the corner to corner potential diagonal failure plane, respectively.

15.3.6 *Horizontal joint shear reinforcement*

15.3.6.1 *Design basis for horizontal shear*
The design for horizontal shear force is based on:

$V^*_{jh} \le \phi V_{jh}$ .... (Eq. 15–3)

15.3.6.2 *Area of horizontal joint shear reinforcement*

(a) The area of total effective horizontal joint shear reinforcement corresponding with each direction of horizontal joint shear force shall be:

$$A_{jh} = \frac{V^*_{jh} - \phi V_{ch}}{\phi\, f_{yh}} \quad \text{(Eq. 15–4)}$$

where

$$\phi V_{ch} = V^*_{jh}\left(0.5 + \frac{C_j N^*}{A_g f'_c}\right) \quad \text{(Eq. 15–5)}$$

In Equation 15–5 the value of $\left(N^*/A_g f'_c\right)$ shall not be taken greater than 0.2. A3

The distribution of this reinforcement within the joint shall be as required by 15.4.4.3. A1

(b) In external beam column joints where the beam reinforcement is terminated by 90° hooks as specified in 8.6.12.6, joint shear reinforcement equal to or greater than $A_{jh} = 0.25\dfrac{V_{jh}}{\phi f_{yh}}$ shall enclose the vertical leg of the hooked bars and the longitudinal column reinforcement. The criteria applies separately to both positive and negative terminating beam reinforcement. A3

15.3.7 *Vertical joint shear reinforcement*

15.3.7.1 *Design basis for vertical shear*
The design for vertical shear force is based on:

$V^*_v \le \phi\, V_{jv}$ .... (Eq. 15–6)

15.3.7.2 *Area of vertical joint shear reinforcement*
The area of total effective vertical joint shear reinforcement corresponding with each direction of horizontal joint shear force shall be:

$$A_{jv} = \frac{V_{jh}^{*}\frac{h_b}{h_c} - \phi V_{cv}}{\phi f_{yv}} \quad \text{(Eq. 15–7)}$$

where

$$\phi V_{cv} = 0.6 V_{jh}^{*}\frac{h_b}{h_c} + C_j N^{*} \quad \text{(Eq. 15–8)}$$

A3 Where N^{*} shall be taken as negative for tension.

The distribution of this reinforcement within the joint shall be as required by 15.4.5.2.

15.3.8 *Confinement*

The horizontal transverse confinement reinforcement in beam-column joints shall be equal to or greater than that required by 10.3.10.5 and 10.3.10.6, with the exception of joints connecting beams at all four column faces in which case the transverse joint reinforcement may be reduced to one-half of that required in 10.3.10.5 and 10.3.10.6. In no case shall the stirrup-tie spacing in the joint core exceed 10 times the diameter of the smallest column bar or 200 mm, whichever is less.

15.4 Additional design requirements for beam-column joints with ductile, including limited ductile, members adjacent to the joint

15.4.1 *General*

Special provisions are made in this section for beam-column joints that are subjected to forces arising from the formation of ductile plastic regions in the adjacent members. Joints must be designed in such a way that the required energy dissipation occurs in potential plastic hinges of adjacent members and not in the joint core region.

15.4.2 *Design forces*

15.4.2.1 *Forces acting on beam-column joint*

The design forces acting on a beam-column joint core shall be evaluated from the maximum internal forces generated by all the members meeting at the joint in equilibrium. The forces shall be those induced when the overstrengths of the beam or beams are developed, except in cases when a column is permitted to be the weaker member. Where a plastic hinge can develop in a beam adjacent to a joint, all tension reinforcement of the beam section, including that placed in flanges in accordance with 9.4.1.6, shall be taken into account. Where plastic hinges are to develop in columns rather than in beams, nominal joint shear strength shall be based on the overstrength of the columns.

Where detailing ensures that plastic regions are located away from the joint face, the design joint shear forces shall be calculated from the forces occurring at the joint face at the overstrength of the plastic regions.

15.4.2.2 *Horizontal design shear force*

The magnitude of the horizontal design shear force in the joint, V_{jh}^{*}, shall be evaluated from a rational analysis taking into account the effect of all forces acting on the joint.

15.4.2.3 *Consideration of concurrency*

At columns of two-way frames where beams frame into the joint from two directions, these forces need only be considered in each direction independently. However, axial column forces caused by beam plastic hinges in two directions should be considered.

15.4.3 *Design assumptions*

15.4.3.1 *The role of shear reinforcement*

The design of the shear reinforcement in the joint shall be based on the prevention of premature bond failure and effective control of a potential tension failure plane that extends from one corner of the joint to the diagonally opposite edge.

15.4.3.2 *Maximum horizontal design shear force*

The horizontal design shear force across a joint for seismic stress reversals shall not exceed the smaller of $0.2f'_c b_j h_c$, or 10 $b_j h_c$ where b_j is defined in 15.3.4.

15.4.3.3 *Determination of shear resistance of joint*

The shear strength of joints shall be assessed as follows:

(a) The shear resistance of beam-column joints shall be based on a mechanism consisting of a single diagonal concrete strut and a truss mechanism with horizontal and vertical stirrups, hoops or bars adequately anchored at the boundaries of the joint capable of sustaining a diagonal concrete compression field;

(b) Other forms of joint shear reinforcement such as beam bars bent diagonally across the joint in one or both directions, or large diameter hoops placed outside the joint core where horizontal beam haunches allow this to be done, may also be used if it can be shown by rational analysis or tests or both that the required joint shear and anchorage forces can be adequately transferred.

15.4.3.4 *Horizontal joint shear reinforcement*

The requirements of 15.4.4 and 15.4.5 shall apply when limited ductile or ductile plastic regions can develop in the beams at the column face. Where plastic regions in columns adjacent to a joint are permitted, application of 15.4.4 and 15.4.5 shall be correspondingly interchanged. Where plastic hinges can develop in the beams, but these are remote from the column face, the design joint shear forces shall be calculated in accordance with 15.4.2.1, and joint reinforcement and detailing shall be in accordance with 15.3.5 to 15.3.8. As forces are derived from overstrengths, a ϕ value of 1.0 shall be used in accordance with 2.3.2.2. A3

15.4.3.5 *Placement of shear reinforcement*

The required horizontal and vertical joint shear reinforcement shall be placed within the effective width of the joint, b_j; defined in 15.3.4, relevant to each direction of loading.

15.4.3.6 *Design yield strength of shear reinforcement*

The design yield strength of shear reinforcement, f_{yh} and f_{yv}, shall not exceed 500 MPa.

15.4.4 *Horizontal joint shear reinforcement*

15.4.4.1 *Area of horizontal joint shear reinforcement*

The area of total effective horizontal joint shear reinforcement corresponding to each direction of horizontal joint shear force shall be:

(a) *For interior joints*

A_{jh} shall be determined from Equation 15–9

$$A_{jh} = \frac{6V^*_{ojh}}{f'_c b_j h_c}\left(\frac{\alpha_i f_y A^*_s}{f_{yh}}\right) \quad \text{(Eq. 15–9)}$$

where

A2

$$0.85 \le \left[\frac{6V^*_{\mathrm{ojh}}}{f'_{\mathrm{c}} b_{\mathrm{j}} h_{\mathrm{c}}}\right] \le 1.20$$

and

(i) $\alpha_{\mathrm{i}} = 1.4\alpha_{\mathrm{n}}$

or, where the beneficial effects of axial compression loads acting above the joint are included;

$$\alpha_{\mathrm{i}} = \left(1.4 - 1.6\frac{C_{\mathrm{j}} N^*_o}{f'_{\mathrm{c}} A_{\mathrm{g}}}\right)\alpha_{\mathrm{n}}$$

where

A3

$\alpha_{\mathrm{n}} = 0.85$ where the material strain of the plastic region adjacent to the joint is equal to or less than that for LDPR (see 2.6.1.3)

$\alpha_{\mathrm{n}} = 1.0$ where the material strain of the plastic region adjacent to the joint is equal to or less than that for DPR (see 2.6.1.3)

(ii) A^*_{s} is the greater of the area of top or bottom beam reinforcement passing through the joint. It excludes bars in effective tension flanges.

(b) *For exterior joints*

A_{jh} shall be determined from Equation 15–10

A2

$$A_{\mathrm{jh}} = \frac{6V^*_{\mathrm{ojh}}}{f'_{\mathrm{c}} b_{\mathrm{j}} h_{\mathrm{c}}}\left(\frac{\beta f_{\mathrm{y}} A_{\mathrm{s}}}{f_{\mathrm{yh}}}\right)\left(0.7 - \frac{C_{\mathrm{j}} N^*_{\mathrm{o}}}{f'_c A_{\mathrm{g}}}\right) \quad \text{(Eq. 15–10)}$$

where

A2

$$0.85 \le \left[\frac{6V^*_{\mathrm{ojh}}}{f'_c b_{\mathrm{j}} h_{\mathrm{c}}}\right] \le 1.20$$

and N^*_{o} is taken negative for axial tension in which case $C_{\mathrm{j}} = 1$ must be assumed, and β = ratio of area of compression beam reinforcement to area of tension beam reinforcement, not to be taken larger than unity.

(c) The area A_{jh} to be provided in accordance with 15.4.4.1 (a) and (b) shall be equal to or greater than $0.4V^*_{\mathrm{ojh}}/f_{\mathrm{yh}}$.

15.4.4.2 *Prestressed beams*

Where beams are prestressed through the joint, the horizontal joint shear reinforcement required by 15.4.4.1 may be reduced by:

$$\Delta A_{\mathrm{jh}} = \frac{0.7P_{\mathrm{cs}}}{f_{\mathrm{yh}}} \quad \text{(Eq. 15–11)}$$

where P_{cs} is the force after all losses in the prestressing steel that is located within the central third of the beam depth.

15.4.4.3 *Distribution of horizontal joint shear reinforcement* A1

The effective horizontal joint shear reinforcement crossing the potential diagonal failure plane shall consist of sets of stirrups or hoops or intermediate ties or equivalent reinforcement placed between but not immediately adjacent to the innermost layers of the top and bottom beam reinforcement, and shall be distributed as uniformly as practicable. Any tie leg bent around column bars that does not cross the potential diagonal failure plane, shall be neglected.

15.4.4.4 *Minimum horizontal joint reinforcement* A1

The quantity of horizontal joint reinforcement, placed as required by 15.4.4.3, shall be equal to or greater than that required by 10.4.7.4 and 10.4.7.5 for confinement of concrete and lateral restraint of bars in the end regions of columns immediately above or below a joint. The vertical spacing of sets of ties or hoops within a joint shall not exceed 10 times the diameter of the smallest column bar or 200 mm, whichever is less.

15.4.5 *Vertical joint shear reinforcement*

15.4.5.1 *Area of vertical joint reinforcement* A2

Where columns are designed so that primary plastic hinges do not form against the beam face of the joint zone, the total area of effective vertical joint shear reinforcement shall be determined for each of the two directions for interior and exterior joints from Equation 15–12.

$$A_{jv} = \alpha_v A_{jh} \frac{f_{yh}}{f_{yv}} \frac{h_b}{h_c} \quad \text{(Eq. 15–12)}$$

where

$$\alpha_v = \frac{0.7}{1 + \dfrac{N_o^*}{f_c' A_g}} \quad \text{(Eq. 15–13)}$$

Where a primary plastic hinge may form in the column against the beam face, the vertical joint zone reinforcement should be designed on the same basis as the horizontal joint shear reinforcement for a joint zone with beams, which may form plastic regions against the column face or faces. A2

15.4.5.2 *Vertical joint shear reinforcement*

The vertical joint shear reinforcement shall consist of intermediate column bars, placed in the plane of bending between corner bars, or vertical stirrup ties or special vertical bars, placed in the column and adequately anchored to transmit the required tensile forces within the joint. The total area of effective vertical joint shear reinforcement shall be placed within the effective joint area, $b_j h_c$, as defined by 15.3.4.

15.4.5.3 *Spacing of vertical joint reinforcement*

The horizontal spacing of vertical joint reinforcement in each plane of any beam framing into a joint shall not exceed the larger of one-quarter of the adjacent lateral dimension of the section or 200 mm, and in all cases there shall be at least one intermediate bar in each side of the column in that plane.

15.4.6 *Joints with wide columns and narrow beams*

Where the width of the column is larger than the effective joint width specified in 15.3.4 or 15.4.7, all flexural reinforcement in the column that is required to interact with the narrow beam shall be placed within the effective joint area, $b_j h_c$. Additional longitudinal column reinforcement shall be placed outside of this effective joint area in accordance with 10.4.6.2. Transverse reinforcement outside of the effective joint area shall be in accordance with the confinement provisions of 10.4.7.4 and 10.4.7.5.

15.4.7 *Eccentric beam-column joints*

The effective joint width, b_j, shall not exceed $0.5(b_w + b_c + 0.5\, h_c) - e$, where e is the eccentricity of a beam relative to the column into which it frames and equals the distance between the centrelines of the webs of the beam and the column.

15.4.8 *Maximum diameter of longitudinal beam bars passing through joints*

The diameter of longitudinal beam bars passing through the beam-column joint shall be in accordance with 9.4.3.5.

15.4.9 *Maximum diameter of column bars passing through joint*

The diameter of longitudinal column bars passing through the beam-column joint shall be in accordance with 10.4.6.6.

16 BEARING STRENGTH, BRACKETS AND CORBELS

16.1 Notation

a	shear span, distance between the centroid of the concentrated load on a corbel and the critical section for flexure, mm (A3)
A_1	loaded area, mm^2
A_2	the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of one vertical to two horizontal, mm^2
A_f	area of reinforcement in bracket or corbel resisting moment ($V^*a + N^*_c c_t$), mm^2 (A3)
A_h	area of closed corbel stirrups or ties, mm^2
A_n	area of reinforcement in bracket or corbel resisting tensile force N^*_c, mm^2
A_s	area of primary reinforcement adjacent to the bearing surface of the bracket or corbel resisting the axial tension force, N^*_c, and the bending moment of ($V^*a + N^*_c c_t$) about primary reinforcement at the critical section, mm^2 (A3)
A_{vf}	area of fully developed shear-friction reinforcement, mm^2
b	width of corbel at right angles to shear span, a, mm
c_t	cover distance from bearing surface in a corbel to centroid of the reinforcement closest to the tension surface of the corbel, mm
d	distance from extreme compression fibre to centroid of longitudinal flexural tension reinforcement at the springing of a corbel, mm
f'_c	specified compressive strength of concrete, MPa
f_ℓ	average confining stress at the perimeter of a loaded area, MPa
f_y	lower characteristic yield strength of steel reinforcement, MPa
h	overall depth of corbel, mm
N^*_c	design tensile force applied at top of bracket or corbel acting simultaneously with V^* to be taken as positive if in tension, N (A3)
p	proportion of flexural reinforcement, $A_s/b_w d$
V^*	design shear force, N
V_n	nominal shear strength of section, N
α	the angle between the critical section and the inclined strut (A3)
μ_f	coefficient of friction at critical section of corbel
ϕ	strength reduction factor

16.2 Scope

The provisions of this section apply to bearing strength and the design of corbels, brackets and ledges. A3

16.3 Bearing strength

16.3.1 *General*

The maximum bearing stress of concrete shall not exceed $\phi(0.74\ f'_c)$, except where the supporting surface is wider on all sides than the loaded area. In this event, the design bearing strength of the loaded area may be multiplied by $\sqrt{A_2 / A_1} \leq 2.0$, provided that the requirements of 16.4.3 are satisfied. A3

16.3.2 *Exclusions*

The maximum bearing stress given in 16.3.1 may be exceeded where either: A3

(a) Extensive tests have shown the bearing pressure can be sustained without any reduction in safety index.

(b) The loaded area A_1 is confined, by reinforcement or some other means, by a confining stress f_ℓ, in which case the maximum bearing stress may be taken as: A3

A3 $\phi(0.95f'_c + 4.5f_\ell)A_1 \leq \phi(40)\, A_1$ .... (Eq. 16–1)

where ϕ is the strength reduction factor and f_ℓ may be taken as the average confining stress at the perimeter of the loaded area, but f_ℓ shall not exceed 1.5 times the minimum confining pressure.

16.3.3 *Strength reduction factor*

A3 The strength reduction factor shall be taken as 0.75 for design bearing stress, strut and tie forces and design using the empirical design method for corbels in accordance with clause 16.5.

The same strength reduction factor shall be used in design calculations that include allowance for displacements at maximum considered earthquake level.

16.3.4 *Calculation of A_1 and A_2*

In the calculation of A_1 the area shall not cross the appropriate plane defined in (a) and (b).

The value of A_2 shall be taken as the area of the largest frustum of a pyramid, cone, or tapered wedge having for its base the loaded area A_1 and having side slopes of one vertical to two horizontal contained wholly within the support or member, but not crossing the appropriate vertical plane defined in (a) and (b):

(a) For a corbel or bracket, a vertical plane drawn through one of the following as appropriate:
   (i) the inside face of the anchor bar for the primary reinforcement
   (ii) the inside face of armouring
   (iii) the end of the straight portion of the primary reinforcement where there is no armouring or anchor bar;

(b) For the supported concrete unit, a vertical plane drawn through one of the following:
   (i) for a slab or hollow-core unit, a distance of 30 mm from the back face of the member
   (ii) for a member supported on a web or rib, a distance of 30 mm from the back face of the member
   (iii) for a member where the end of the unit is armoured, the face of the armouring.

16.4 Design of brackets and corbels

A3 16.4.1 *Critical section for flexure*

The critical section for flexure in a corbel or bracket shall be taken as:

(a) Where it is supported by a wall or a column, the face of the wall or the column;

(b) Where it is supported by a beam, at the centroid of the vertical reinforcement closest to the bracket or corbel.

16.4.2 *Design actions and limiting dimensions for a corbel*

The corbel or bracket shall be designed to resist simultaneously the vertical load, V^*, and a lateral force, N^*_c, equal to the vertical load, V^*, times the appropriate coefficient of friction. In no case shall the coefficient of friction be less than 0.7.

The length of the corbel or bracket shall be such that the minimum bearing area required for the load-factored actions can be maintained for the sum of the displacements defined in 18.6.7 for nominally ductile structures, of for 18.6.7 and 18.8 for ductile and limited ductile structures or for regions of nominally ductile structures which contain limited ductile plastic regions.

16.4.3 *Bearing area and bearing stresses*

16.4.3.1 *Bearing stresses*

The bearing stress acting on a supporting member shall be equal to or smaller than the smallest of the maximum permitted bearing stress:

(a) On the bearing strip or pad;

(b) Permitted on the corbel;

(c) On the supported member.

16.4.3.2 *Bearing areas relied on for corbels and brackets* A3

For a corbel or bracket, the bearing area to be relied upon shall not extend beyond:

(a) The inside face of the anchor bar for the primary reinforcement; or

(b) Where there is no armouring or anchor bar, the end of the straight portion of the primary reinforcement.

If armouring that effectively confines the outside edge is present, the bearing area may be assumed to extend to the outside edge of the armouring.

16.4.4 *Method of design*

Brackets and corbels shall be designed by either the empirical method in 16.5 or the strut and tie method in 16.6, depending on the ratio of the shear span divided by the effective depth, a/d, as defined in (a) and (b): A3

(a) Where the ratio is less than 1.5 the strut and tie method may be used;

(b) Where the ratio is equal to or less than 1.0, and N_c^* is less than V^*, the empirical method may be used.

Where the ratio exceeds 1.5, the member shall be designed as a cantilever beam.

16.5 Empirical design of corbels or brackets

16.5.1 *Depth at outside edge*

The depth at the outside edge of the bearing area shall be equal to or greater than $0.5d$.

16.5.2 *Design actions at the critical section* A3

The critical section shall be designed to resist simultaneously a shear V^*, a moment ($V^*a + N_c^* c_t$), and a horizontal tensile force N_c^*, acting at the level of the primary tension reinforcement, which is closest to the bearing side of the corbel.

16.5.3 *Shear-friction reinforcement*

Design of shear-friction reinforcement A_{vf} to resist shear V^* shall be in accordance with 7.7.

16.5.4 *Maximum shear stress*

The nominal shear stress at the support face shall be equal to or less than the smaller of: A3

(a) $0.2f'_c$ *or* 6.0 MPa for normal weight concrete;

(b) $(0.2 - 0.07a/d)\ f'_c$ or $(6 - 2a/d)$ MPa for all lightweight or sand lightweight concrete.

16.5.5 *Reinforcement for flexure*

Reinforcement A_f to resist moment $V^*a + N_c^* c_t$ shall be computed in accordance with 7.4. A3

16.5.6 *Reinforcement for axial tension force*

Reinforcement A_n to resist tensile force N_c^* shall be determined from $N_c^* < \phi A_n f_y$.

16.5.7 *Primary tension reinforcement*

The area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or $(2A_{vf}/3 + A_n)$.

16.5.8 *Closed stirrups or ties*

Closed stirrups or ties parallel to A_s, with a total area A_h equal to, or greater than $0.5(A_s - A_n)$, shall be uniformly distributed within the upper two-thirds of the effective depth adjacent to A_s. A3

16.5.9 *Minimum ratio for p*

Ratio $p = A_s/bd$ shall be equal to or greater than $\dfrac{\sqrt{f'_c}}{4 f_y}$. A3

16.5.10 *Reinforcement A_s*

At the critical section of a bracket or corbel, the primary tension reinforcement A_s shall be anchored to develop the lower characteristic yield strength of the bar acting at the face of the support by one of the following: A3

(a) By a structural weld to a transverse bar of at least equal size; the weld is to be designed to develop lower characteristic yield strength f_y of A_s bars;

A3 (b) By bending primary tension bars A_s back to form a horizontal loop provided the bearing area is located a distance of $3d_b$ behind the inner face of the end of the primary reinforcement, where d_b is the diameter of the bar; or

(c) By some other means of positive anchorage.

16.6 Design requirement by strut and tie method

16.6.1 *Principal reinforcement A_s*

The primary reinforcement, A_s, nearest to the bearing surface of a corbel, shall at the critical section be designed to resist a bending moment of $[V^* a + N_c^* c_t]$, and a horizontal tensile force N_c^*, with both actions acting at the level of the primary reinforcement A_s. The internal lever-arm of the flexural forces shall not exceed *0.87d*, where *d* is the effective depth. The minimum area of A_s shall be equal to or greater than

$$\frac{\sqrt{f_c'}}{4 f_y}\ bd.$$

16.6.2 *Shear friction reinforcement*

The inclined compression force balancing the primary tension force and forces V^* and N_c^* is referred to as the primary compression force. Where the angle, α, between the centroid of the primary compression force and the critical section is less than 25°, shear friction reinforcement in the upper two-thirds of the effective depth of the corbel shall be provided. The minimum area of shear friction reinforcement shall be equal to or greater than:

$$A_{vf} = \frac{V^*}{\phi f_{vy}}\left(\frac{1}{\mu_f} - \tan\alpha\right) \geq 0 \qquad \text{(Eq. 16–2)}$$

where μ_f is the coefficient of shear friction (given in 7.7.4.3), and α is the angle between the primary compression force and the critical section. Where the corbel is not cast at the same time as the support member, the coefficient of friction, μ_f, appropriate for the surface at the interface to the corbel shall be used (7.7.4).

16.7 Design requirements for beams supporting corbels or brackets

The deformation of columns or walls subjected to lateral forces can result in twisting of beams that frame into them. The structural members supported on corbels or brackets can provide restraint to this twisting action, which can lead to significant torsional actions being induced in the beams. To ensure that a non-ductile failure cannot be induced, the torsional reinforcement complying with 7.6.1.4 shall be provided over a minimum distance of h_b from the wall or column supporting the beam, where h_b is the overall depth of beam.

16.8 Design requirements for ledges supporting precast units

16.8.1 *The ledge support of precast floor units*

A ledge supporting precast concrete floor units (hollow-core, flat slabs, ribs, or tee units) is either the top of a reinforced concrete beam on to which fresh concrete shall be cast to form a composite beam, or a preformed shelf to support the floor units.

16.8.2 *Design of ledges*

Ledges may be designed by either the empirical method in 16.5 or the strut and tie method in 16.6, according to 16.4.4.

16.8.2.1 *Detailing of reinforcement in ledges*

16.8.2.1.1 *Empirical method of 16.5*

The distribution of transverse reinforcement required by the empirical method:

(a) Hollow-core and flat slab units: uniformly distribute the transverse reinforcement across the width of the unit;

(b) Rib and tee units: place the transverse reinforcement within the width of the rib or web of the tee unit. A3

16.8.2.1.2 *Strut and tie method of 16.6*

The distribution of transverse reinforcement shall satisfy the requirements of Appendix A.

NOTES

17 EMBEDDED ITEMS, ANCHORS AND SECONDARY STRUCTURAL ELEMENTS A3

17.1 Notation

A_{brg} bearing area of the head of stud or anchor, mm^2

A_n projected rectilinear area on the free surface of the concrete member due to the assumed 35° concrete failure surface of a single anchor or a group of anchors, mm^2 A3

A_{no} projected concrete failure area of one anchor when not limited by edge distance, mm^2

A_{se} effective cross-sectional area of an anchor, mm^2

A_v projected concrete failure area of an anchor or group of anchors in shear, mm^2

A_{vo} projected concrete failure area of an anchor or group of anchors in shear, when not limited by corner influences, spacing, or member thickness, mm^2

A3

c_1 distance from the centre of an anchor shaft to the edge of the concrete in the direction in which the load is applied, mm. If tension is applied to the anchor then $c_1 = c_{min}$. Where anchors subject to shear are located in narrow sections of limited thickness, c_1 shall not exceed the largest of $c_2/1.5$, $h/1.5$, or $s/3$

c_2 distance from centre of shaft to the edge of the concrete, perpendicular to c_1, mm

c_{max} largest edge distance, mm

c_{min} smallest edge distance, mm

d_o outside diameter or shaft diameter of the anchor. Shall be taken as 0.8 times the width of a hooked steel plate, mm

e_h distance from the inner surface to the outer tip of a hooked bolt, mm A3

e'_n the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension (always taken as positive), mm

e'_v distance between the point of shear force application and the centroid of the group of anchors resisting the shear in the direction of the applied shear, mm

f'_c specified compressive strength of concrete, MPa

f_{ut} ultimate tensile strength of the anchor, but shall not be taken greater than $1.9f_y$, MPa A3

f_y specified yield strength of anchor steel, MPa

h overall thickness or height of member measured parallel to anchor axis, mm A3

h_{ef} effective anchor embedment depth, mm

ℓ load-bearing length of anchors for shear, equal to h_{ef} for anchors with constant stiffness over the full length of the embedded section but less than $8d_o$. Shall be taken as 0.8 times the effective embedment depth for hooked metal plates, mm

k coefficient for basic concrete breakout strength in tension

k_{cp} coefficient of pry-out strength

k_1 multiplier for edge distance

k_2 0.6 for cast-in headed studs, headed bolts, or hooked bolts, or hooked steel plates

n number of anchors in a group

N_b basic concrete breakout strength in tension of a single anchor in cracked concrete, N

N_{cb} nominal concrete breakout strength in tension of a single anchor or group of anchors, N A3

N_n lower characteristic strength in tension, N

N_p pullout strength in tension of a single anchor in cracked concrete, N

N_{pn} lower characteristic pullout strength in tension of a single anchor, N

N_s nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, N

N_{sb} side blowout strength of a single anchor, N

N^* design tension, N

s centre-to-centre spacing of the anchors, mm

ϕ strength reduction factor

A3 λ factor for determining the average modulus of rupture for lightweight concrete

V_b basic concrete breakout strength for a single anchor in shear, N
V_{cb} lower characteristic concrete breakout strength in shear of a single or group of anchors, N
V_{cp} lower characteristic concrete pry-out strength, N
V_n lower characteristic shear strength, N
V_s nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, N
V^* design shear force, N
Ψ_1 modification factor, for strength in tension, to account for anchor groups loaded eccentrically
Ψ_2 modification factor, for strength in tension, to account for edge distances smaller than 1.5 h_{ef}
Ψ_3 modification factor, for strength in tension, to account for cracking
Ψ_4 modification factor for pullout strength, to account for cracking
Ψ_5 modification factor, for strength in shear, to account for anchor groups loaded eccentrically
Ψ_6 modification factor, for strength in shear, to account for edge distances smaller than 1.5c_1
Ψ_7 modification factor, for strength in shear, to account for cracking

17.2 Scope

A3 The requirements of this section apply to conduits or pipes embedded within structural concrete and to cast-in ductile headed studs, headed bolts, hooked bolts and hooked steel plates that are likely to transmit forces to a concrete structure or between elements of a structure.

See 17.5.5 for requirements for post-installed mechanical anchors and post-installed adhesive anchors.

17.3 Design procedures

Elements within the scope of this section shall be designed in accordance with the procedures of 2.2, 2.3 and 2.4 as appropriate.

17.4 Embedded items

Conduits or pipes shall not significantly impair the strength of the construction.

A3 17.5 Anchors

17.5.1 *General*

A3 Anchors, including holding-down bolts, inserts and ferrules and associated hardware, shall comply with 17.5.2 to 17.5.9. Anchors subjected to seismic actions shall satisfy the requirements of 17.6.

17.5.2 *Design forces*

A3 Anchors and anchor groups shall be designed to transmit all the actions set out in AS/NZS 1170 or other referenced loading standard for the ultimate limit state. The design actions shall also include forces induced in the connection due to creep, shrinkage, temperature effects and relative deformation between the attached items.

17.5.3 *Inserts for lifting*

Design of inserts for lifting shall be in accordance with the Approved Code of Practice for the Safe Handling, Transportation and Erection of Precast Concrete published by the Department of Labour.

A3 17.5.4 *Strength of anchors by testing*

The nominal strength of anchors may be based upon tests to evaluate the 5 percentile fracture, or by calculation, of the following:

(a) Steel strength of anchor in tension;
(b) Steel strength of anchor in shear;
(c) Concrete breakout strength of anchor in tension;
(d) Concrete breakout strength of anchor in shear;
(e) Pullout strength of anchor in tension;

(f) Concrete side-face blowout strength of anchor in tension;

(g) Concrete pry-out strength of anchor in shear.

17.5.5 *Strength of anchors by calculation*

A3

For ductile steel headed studs, headed bolts, hooked bolts and hooked steel plates with diameters less than 50 mm and embedment depths less than 635 mm, calculations for the capacity of cast-in-place mechanical fasteners without supplementary reinforcement in cracked and uncracked concrete shall be determined in accordance with 17.5.6. All other cast-in-place anchors are outside the scope of this document. The effect of supplementary reinforcement on the restraint of concrete breakout may be included by rational analysis.

Post-installed mechanical anchors and post-installed adhesive anchors shall pass the prequalification testing stipulated in ETAG 001, Annex E and be designed in accordance with EOTA TR045.

17.5.6 *Strength of cast-in anchors*

17.5.6.1 *Scope*

The design method outlined in this section applies to cast-in ductile steel anchors, without supplementary reinforcement. Speciality inserts, through bolts, multiple anchors connected to a single plate at the embedded end of the anchors, adhesive or grouted anchors, and direct anchors such as powder or pneumatic actuated nails or bolts are not included. A3

17.5.6.2 *Load application*

Load application involving high cycle fatigue or impact loads are not covered by this section. A3

17.5.6.3 *Strength requirements*

In the design of anchors

$$N^* \leq \phi N_n \quad \text{(Eq. 17–1)}$$

and

$$V^* \leq \phi V_n \quad \text{(Eq. 17–2)}$$

where N_n is the lower characteristic strength in tension and is given by 17.5.7 and V_n is the lower characteristic shear strength given by 17.5.8.

17.5.6.4 *Strength reduction factors*

The strength reduction factors for strength of ductile steel element shall be: A3

(a) Tension: $\phi = 0.75$ .... (Eq. 17–3)

(b) Shear: $\phi = 0.65$ .... (Eq. 17–4)

The strength reduction factors for concrete breakout or side-face blowout strength where supplementary reinforcement is not present shall be:

(c) Tension: $\phi = 0.65$ .... (Eq.17–4(a))

(d) Shear: $\phi = 0.65$ .... (Eq.17–4(b))

17.5.6.5 *Interaction of tension and shear*

Resistance to combined tensile and shear actions shall be considered in design using interaction expressions that result in computation of strength in substantial agreement with results of comprehensive test. This requirement shall be considered satisfied by 17.5.6.6.

17.5.6.6 *Interaction of tension and shear – simplified procedures*

Unless determined in accordance with 17.5.6.5, anchors or groups of anchors shall be designed to satisfy the following:

(a) Where $V^* \leq 0.2\ \phi V_n$ then full strength in tension is permitted ($\phi N_n \geq N^*$)

(b) Where $N^* \leq 0.2\ \phi N_n$ then full strength in shear is permitted ($\phi V_n \geq V^*$)

(c) Where $V^* > 0.2\ \phi V_n$ and $N^* > 0.2\ \phi N_n$ then:

$$\frac{N^*}{\phi N_n} + \frac{V^*}{\phi V_n} \leq 1.2 \qquad \text{(Eq. 17–5)}$$

17.5.7 *Lower characteristic strength of anchor in tension*

The calculated lower characteristic strength of an anchor in tension, N_n, shall be the smaller of the following:

(a) The lower characteristic tensile strength of the steel of the anchor, N_s, 17.5.7.1;

(b) The lower characteristic concrete breakout strength of the anchor in tension, N_{cb}, 17.5.7.2;

(c) The lower characteristic pullout strength of the anchor in tension, N_{pn}, 17.5.7.3;

(d) The lower characteristic concrete side face blowout strength of the anchor in tension, N_{sb}, 17.5.7.4.

17.5.7.1 *Steel strength of anchor in tension*

The lower characteristic tensile strength of an anchor as governed by the steel, N_s, shall be given by:

$$N_s = nA_{se}f_{ut} \qquad \text{(Eq. 17–6)}$$

where

n = number of anchors in a group,

A_{se} = effective cross-sectional area of an anchor, mm²

A3 f_{ut} = ultimate tensile strength of the anchor, but shall not be taken greater than $1.9f_y$, MPa.

17.5.7.2 *Strength of concrete breakout of anchor*

The lower characteristic breakout strength of an anchor, or group of anchors, in tension, N_{cb}, without supplementary reinforcement and normal weight concrete is given by:

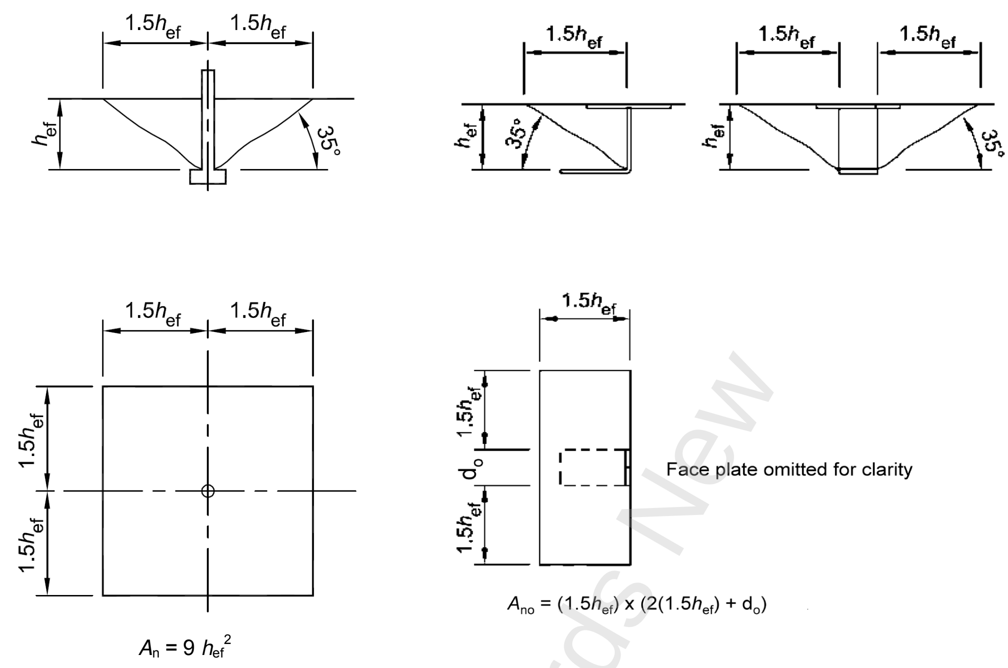
$$N_{cb} = \Psi_1\Psi_2\Psi_3 \frac{A_n}{A_{no}} N_b \qquad \text{(Eq. 17–7)}$$

where

A3 A_n = projected rectilinear area on the free surface of the concrete member due to the assumed 35° concrete failure surface taken from the outside of the head of the anchor or group of anchors (shall be taken as less than or equal to nA_{no}). Where the perimeter of the group of anchors is closer than $1.5h_{ef}$ to any edge, consideration shall be given to the overlap of failure surfaces with the edge and corners of concrete panels, mm²

A_{no} = projected concrete failure area of one anchor when not limited by edge distance, as shown in Figure 17.1, mm².

A2



A3

A2

(a) Headed anchor (b) Hooked plate anchor

Figure 17.1 – Typical failure surface areas of individual anchors, not limited by edge distances

Ψ_1 = modification for anchor groups comprising of more than one anchor. Equal to 1.0 for a single anchor and where $e'_n < s/2$ it is given by:

$$\Psi_1 = \frac{1}{\left(1 + \frac{2e'_n}{3h_{ef}}\right)} \leq 1.0 \quad \text{(Eq. 17–8)}$$

where

e'_n = the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension (always taken as positive), mm

h_{ef} = effective anchor embedment depth, mm A3

s = centre-to-centre spacing of the anchors, mm

Ψ_2 = modification factor for edge distances, given by:

(a) $\Psi_2 = 1.0$ when $c_{min} \geq 1.5h_{ef}$

or,

(b) $\Psi_2 = 0.7 + 0.3\,\frac{c_{min}}{1.5h_{ef}}$ when $c_{min} < 1.5h_{ef}$

Ψ_3 = modification for cracking of concrete, equals:

(a) Ψ_3 = 1.25 for cast-in anchors in uncracked concrete,

(b) Ψ_3 = 1.0 for concrete which is cracked at service load levels. Cracking in the concrete shall be controlled by reinforcement distributed in accordance with 2.4.4.4 and 2.4.4.5.

N_b = basic concrete breakout strength in tension of a single anchor in concrete cracked at service load levels but with the extent of cracking controlled by reinforcement distributed in accordance with 2.4.4.4 and 2.4.4.5, given by:

$$N_b = k\lambda\sqrt{f_c'}\,h_{ef}^{1.5} \quad \text{(Eq. 17–9)}$$

where

f_c' shall not be taken greater than 70 MPa

k = 10 for cast-in anchors

h_{ef} = effective anchor embedment depth, mm, however if three or more edges are closer than 1.5 h_{ef} to the anchor, h_{ef} shall be replaced by c_{max}/1.5 in Equation 17–9, where c_{max} is the largest edge distance of the influencing edges.

A3 Alternatively, for cast-in headed studs and headed bolts with 280 mm ≤ h_{ef} ≤ 635 mm, N_b shall not exceed:

$$N_b = 3.9\lambda\sqrt{f_c'}\,h_{ef}^{5/3} \quad \text{(Eq. 17–9(a))}$$

17.5.7.3 *Lower characteristic tension pullout strength of anchor*

The lower characteristic pullout strength of an anchor or group of anchors in tension shall be given by:

$$N_{pn} = \Psi_4 N_p \quad \text{(Eq. 17-10)}$$

where

N_{pn} = lower characteristic pullout strength in tension of a single anchor, N

Ψ_4 = modification factor for pullout strength

N_p = pullout strength of a single anchor in cracked concrete, N, given by:

(a) For a headed stud or headed bolt:

$$N_p = 8f_c' A_{brg} \quad \text{(Eq. 17–11)}$$

(b) For a hooked bolt where $3d_o \le e_h \le 4.5d_o$:

$$N_p = 0.9\, f_c'\, e_h d_o \quad \text{(Eq. 17–12)}$$

where

A_{brg} = bearing area of the head of stud or anchor, mm^2

d_o = outside diameter of anchor or shaft diameter of a headed stud, headed bolt or hooked bolt, mm

e_h = distance from the inner surface to the outer tip of a hooked bolt, mm

Ψ_4 = 1.0 for concrete cracked at service load levels but with the extent of cracking controlled by reinforcement distributed in accordance with 2.4.4.4 and 2.4.4.5

Ψ_4 = 1.4 for concrete with no cracking at service load levels.

17.5.7.4 *Lower characteristic concrete side face blowout strength*

The side face blowout strength of a headed anchor with deep embedment close to an edge ($c < 0.4h_{ef}$) shall not exceed:

A3

$$N_{sb} = 13.3k_1 c_1 \lambda\sqrt{A_{brg} f_c'} \quad \text{(Eq. 17–13)}$$

where

N_{sb} = side blowout strength of a single anchor, N

c_1 = distance from the centre of an anchor shaft to the edge of the concrete in the direction in which the load is applied, mm. If tension is applied to the anchor then $c_1 = c_{min}$. Where anchors subject to shear are located in narrow sections of limited thickness, c_1 shall not exceed the largest of $c_2/1.5$, $h/1.5$, or $s/3$ A3

k_1 = multiplier for edge distance, given by:

(a) When $c_2 \geq 3c_1$, $k_1 = 1.0$

(b) When $c_2 < 3c_1$, $k_1 = \dfrac{1+\dfrac{c_2}{c_1}}{4}$ A3

where

c_2 = distance from centre of shaft to the edge of the concrete, perpendicular to c_1, mm.

17.5.8 ***Lower characteristic strength of anchor in shear***

The calculated lower characteristic strength of an anchor in shear, V_n, shall be the smallest of the following;

(a) The lower characteristic shear strength of the steel of the anchor, V_s, 17.5.8.1;
(b) The lower characteristic concrete breakout strength of the anchor in shear, V_{cb}, 17.5.8.2, or 17.5.8.3;
(c) The lower characteristic concrete pry-out strength of the anchor in shear, V_{cp}, 17.5.8.4.

17.5.8.1 *Lower characteristic shear strength of steel of anchor*

The lower characteristic strength of an anchor, or group of anchors, in shear governed by the steel shall not exceed:

(a) For cast-in headed stud anchors:

$$V_s = n A_{se} f_{ut} \qquad \text{(Eq. 17–14)}$$

(b) For cast-in headed bolts and hooked bolt anchors:

$$V_s = n 0.6 A_{se} f_{ut} \qquad \text{(Eq. 17–15)}$$

where f_{ut} shall be less than $1.9f_y$ or 860 MPa.

17.5.8.2 *Lower characteristic concrete breakout strength of the anchor in shear perpendicular to edge*

The concrete breakout strength of an anchor, or group of anchors, in shear when loaded perpendicular to an edge shall not exceed:

$$V_{cb} = \frac{A_v}{A_{vo}} \Psi_5 \Psi_6 \Psi_7 V_b \qquad \text{(Eq. 17–16)}$$

For anchors or anchor groups located at or near corners the shear strength shall be determined in each direction.

where

V_{cb} = lower characteristic concrete breakout strength in shear of a single or group of anchors, N.

A_v = projected concrete failure area of an anchor or group of anchors in shear, mm^2 as shown in Figure 17.2(b) A2

A_{vo} = projected concrete failure area of an anchor in shear, when not limited by corner influences, spacing, or member thickness, mm^2 as shown in Figure 17.2(a)

V_b = basic concrete breakout strength for a single anchor in shear, N, for anchors at centre-to-centre spacing greater than 65 mm, is given by the lesser of: A3

A3

$$V_b = k_2\left(\frac{\ell}{d_o}\right)^{0.2} \lambda\sqrt{d_o f_c'}(c_1)^{1.5} \quad \text{(Eq. 17–17(a))}$$

$$V_b = 3.8\lambda\sqrt{f_c'}(c_1)^{1.5} \quad \text{(Eq. 17–17(b))}$$

where

k_2 = 0.6 for cast-in headed studs, headed bolts, or hooked bolts, or hooked steel plates.

ℓ = load-bearing length of anchors for shear, equal to h_{ef} for anchors with constant stiffness over the full length of the embedded section but less than $8d_o$. Shall be taken as 0.8 times the effective embedment depth for hooked metal plates.

d_o = outside diameter or shaft diameter of the anchor. Shall be taken as 0.8 times the width of a hooked steel plate, mm.

c_1 = distance from the centre of resistance of an anchor to the edge of the concrete in the direction which the load is applied, mm. A2 For hooked bars or hooked plates, c_1 is the lesser of the distance from either the centre of the anchor or the centre of the bend radius to the nearest edge in the direction of the applied shear force. A3 For the special case of anchors influenced by three or more edges, c_1 shall not exceed the greater of c_2/1.5 in either direction, h/1.5, and one third the maximum spacing between anchors within the group.

Ψ_5 = modification factor for anchor groups, where $e_v' < \frac{s}{2}$ and Ψ_5 is given by:

$$\Psi_5 = \frac{1}{1+\frac{2e_v'}{3c_1}} \le 1.0 \quad \text{(Eq. 17–18)}$$

where

e_v' = the distance between the point of shear force application and the centroid of the group of anchors resisting the shear in the direction of the applied shear, mm

Ψ_6 = modification factor for edge distance given by:

(a) For $c_2 \ge 1.5c_1$ $\Psi_6 = 1.0$ .... (Eq. 17–19)

or

(b) For $c_2 < 1.5c_1$ $\Psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1}$ .... (Eq. 17–20)

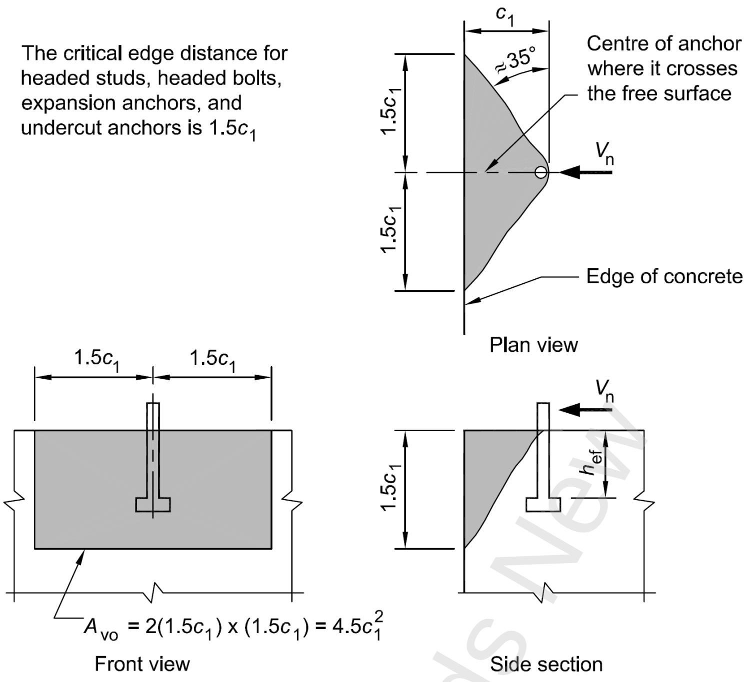
Ψ_7 = modification factor for cracked concrete, given by:

A3

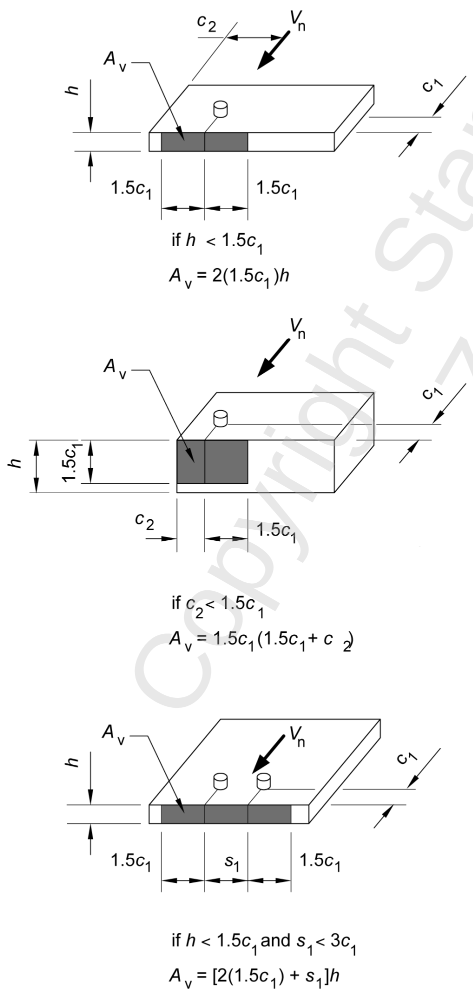
Ψ_7 = 1.0 for anchors in cracked concrete with no supplementary reinforcement or with smaller than 12 mm diameter reinforcing bar as supplementary reinforcement

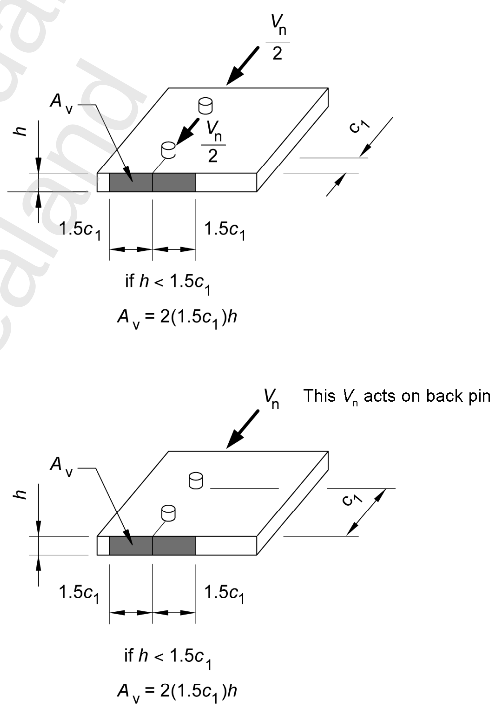
Ψ_7 = 1.2 for anchors in cracked concrete with a 12 mm diameter reinforcing bar or greater as supplementary reinforcement

Ψ_7 = 1.4 for concrete that is not cracked at service load levels.



(a) Calculation of A_{vo}





NOTE -

(1) One assumption if the distribution of forces indicates that half the shear would be critical on front anchor and its projected area.

(2) Another assumption of the distribution of forces (that applies only where anchors are rigidly connected to the attachment) indicates that the total shear would be critical on the rear anchor and its projected area.

(3) h is the thickness of the member, but for calculating A_v shall not be taken greater than $1.5c_1$.

(b) Projected area for single anchors and groups of anchors and calculation of A_v

Figure 17.2 – Determination of A_v and A_{vo} for anchors

A2

17.5.8.3 *Lower characteristic concrete breakout strength of the anchor in shear parallel to edge*

The concrete breakout strength of an anchor or group of anchors, in shear when loaded parallel to an edge shall not exceed:

$$V_{cb} = 2\frac{A_v}{A_{vo}}\Psi_5\Psi_7 V_b \quad \text{(Eq. 17–21)}$$

A2 where A_v, A_{vo}, Ψ_5 and Ψ_7 and V_b are defined in 17.5.8.2.

17.5.8.4 *Lower characteristic concrete pry-out of the anchor in shear*

The nominal pry-out strength of an anchor shall not exceed:

$$V_{cp} = k_{cp}N_{cb} \quad \text{(Eq. 17–22]}$$

where

V_{cp} = lower characteristic concrete pry-out strength, N

A3 N_{cb} = nominal concrete breakout strength in tension of a single anchor or group of anchors, N

k_{cp} = coefficient of pry-out strength, given by:

(a) For $h_{ef} < 65$ mm, $k_{cp} = 1.0$;

(b) For $h_{ef} \geq 65$ mm, $k_{cp} = 2.0$.

17.5.9 *Durability and fire resistance*

The cover for durability and fire resistance shall be in accordance with Sections 3 and 4 respectively.

A3 17.6 Additional design requirements for anchors designed for earthquake effects

17.6.1 *Anchor design philosophy*

Anchors shall be designed to prevent failure in an earthquake. The design philosophy adopted to achieve this shall be one of (a) to (d), or a combination of these:

(a) Anchors shall be designed to accommodate relative seismic movement by separation (17.6.2);

(b) The strengths of the anchors are greater than the actions associated with ductile yielding of the attachment (17.6.3);

(c) Anchors shall be designed to remain elastic (17.6.4);

(d) The anchor is designed to accommodate the expected seismic actions and deformations in a ductile manner (17.6.5).

A3 17.6.2 *Anchors designed for seismic separation*

When seismic deflection of the structure results in relative movement between an element and the points on the structure to which it is fixed, the anchors shall be designed to give clearance for the relative movements at these anchoring points, corresponding to 1.5 times the seismic deflection at the ultimate limit state computed from NZS 1170.5. Frictional forces that may be present on sliding surfaces shall be allowed for in the design of the anchor.

A3 17.6.3 *Anchors stronger than the overstrength capacity of the attachment*

When this design philosophy is adopted, the components of the anchor shall be designed so that the capacity of the anchor exceeds the development of overstrength yielding of the attachment. The design actions on the anchors shall be determined assuming overstrength actions determined in accordance with 2.6.5.4 and shall include consideration of plastic hinge elongation, creep, shrinkage and temperature effects. The capacity of the anchors shall be determined in accordance with 17.5.4 or 17.5.5 using the strength reduction factors of 17.5.6.4.

A3 17.6.4 *Anchors designed to remain elastic*

Anchors designed to remain elastic shall be designed for the actions described in Clause 8.7 of NZS 1170.5. The capacity of the anchors shall be determined in accordance with 17.5.4 or 17.5.5 using 0.75 times the strength reduction factors of 17.5.6.4.

A3 17.6.5 *Anchors designed for ductility*

Anchors may be designed for ductility for the actions described in Clause 8.7 of NZS 1170.5, when the actions and relative movement do not require deformations in the anchors in excess of twice their yield deformations. The calculation of the deformation in the connection shall include consideration of inter-storey drift, plastic hinge elongation, creep, shrinkage and temperature effects. The connection shall be designed to prevent non-ductile failure modes.

17.6.6 *Anchors in plastic hinge regions*

A3

In regions of potential plastic hinging, the contribution of the cover concrete to the anchorage of anchors shall be ignored.

NOTES

18 PRECAST CONCRETE AND COMPOSITE CONCRETE FLEXURAL MEMBERS

18.1 Notation

A_g	gross area of section. For hollow section, A_g is the area of concrete only and does not include the area of voids, mm^2
b_v	width of cross section or effective width of interface in a section between precast and cast *in situ* concrete (see 18.5.4.2), mm
d	distance from extreme compression fibre to centroid of tension reinforcement, mm
f_y	lower characteristic yield strength of non-prestressed reinforcement, MPa
f'_c	specified compressive strength of concrete, MPa
h	overall depth of member, mm
h_b	overall depth of a beam, mm
I	moment of inertia of composite section, mm^4
j_d	internal level arm of the flexural forces in the section, mm
ℓ_y	potential plastic region ductile detailing length, mm
Q	first moment of area beyond the shear plane, being considered about the axis of bending, mm^3
S_p	structural performance factor
$v_{d\ell}$	nominal longitudinal shear stress at any cross section or the nominal shear stress on the interface between the precast concrete shell and the cast-in-place core of the beam, MPa
v_ℓ	maximum permissible longitudinal shear stress, MPa
V_L	longitudinal shear force, N
V_n	nominal shear strength of section, N
V^*	design shear force at section at the ultimate limit state, N
ϕ	strength reduction factor, see 2.3.2.2

A2 A3 A3 A3 A3 A3 A2 A2 A3 A2

18.2 Scope

18.2.1 *Precast concrete defined*

Provisions of this section apply for design of precast concrete members or structures, defined as those with structural elements that have been cast at a location other than in their final position in the structure.

18.2.2 *Composite concrete flexural members defined*

Also covered in this section are composite concrete flexural members comprising precast or cast-in-place concrete flexural members constructed in separate placements, but interconnected so that all elements respond to loads and forces as a single unit. A3

18.2.3 *Composite concrete and structural steel not covered*

Composite compression members of mixed concrete and structural steel sections shall be designed in accordance with 10.3.11. Other composite members of mixed concrete and structural steel shall be designed in accordance with NZS 3404. A3

18.2.4 *Section 18 in addition to other provisions of this Standard*

All provisions of this Standard, not specifically excluded and not in conflict with the provisions of section 18 shall also apply to structures incorporating precast and composite concrete structural members.

18.3 General

18.3.1 *Design to consider all loading and restraint conditions*

The design of precast members and connections shall consider all loading and restraint conditions from initial fabrication to completion of the structure, including those resulting from removal from the mould, storage, transportation, erection and propping. Design for temporary load cases during construction shall take account of the actual concrete strength at the relevant ages or stages of construction.

18.3.2 ***Include forces and deformations at connections***

When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.

18.3.3 ***Consider serviceability and ultimate limit states***

A3 Deflections, end rotations and lateral displacements shall be considered in the ultimate and serviceability limit states, and for a maximum considered earthquake seismic event where specifically required in a clause. Peak deformations shall be taken as $1/S_p$ times the corresponding design deformation given in NZS 1170.5 or other referenced loading standard for the limit state being considered, and the corresponding maximum considered earthquake value is 1.5 times the design ultimate limit state value. Hence a peak maximum considered earthquake deformation is $1.5/S_p$ times the design ultimate limit state deformation given in NZS 1170.5.

18.3.4 ***Tolerances***

Tolerances for dimensions of members and for their locations in the structure shall be specified by the designer. Design of precast members, connections and supports, shall include the effects of these tolerances. Combinations of secondary effects and construction tolerances shall be considered in designing bearing and/or hanger supports for precast concrete members.

A3 **18.3.5** ***Effects to be taken into consideration***

Long-term creep, shrinkage, temperature effects, differential settlement of foundations and restraint conditions shall be considered in the design and detailing of precast concrete members and their supports and connections.

18.4 Distribution of forces among members

A3 **18.4.1** ***Forces perpendicular to the axis of the member***

Distribution of forces that are perpendicular to the axis of the member shall be established by analysis or by test.

18.4.2 ***In-plane forces***

Where the system behaviour requires in-plane forces to be transferred between the members of a precast floor or wall system, then the following shall apply:

(a) In-plane force paths shall be continuous through both connections and members; and

(b) Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

18.5 Member design

18.5.1 ***Prestressed slabs and wall panels***

A3 In one-way prestressed slabs and in one-way prestressed wall panels, all not wider than 2.4 m and where members are not mechanically connected so as to cause restraint in the transverse direction, the shrinkage and temperature reinforcement requirements of 8.8 for the precast unit in the direction normal to the flexural reinforcement may be waived. This waiver shall not apply to members that require reinforcement to resist transverse flexural stresses or to untopped precast floor units. In the context of this clause "prestressed" is defined as having equal to or greater than 1.5 MPa average residual concrete compression.

18.5.2 ***Composite concrete flexural members***

18.5.2.1 *Shored and unshored members*

No distinction shall be made between shored and unshored members in the design for flexural strength of composite members for the ultimate limit state.

18.5.2.2 *Design of constituent elements*

Constituent elements shall be designed to support all loads that may be introduced prior to full development of the design strength of composite members.

18.5.2.3 *Reinforcement for composite members*

Reinforcement shall be provided as required for strength; and to control cracking and to prevent separation or slippage of individual elements of composite members.

18.5.3 *Shear resisted by composite and non-composite sections*

Concrete elements prior to being made composite, and as composite members, shall be designed for the shear forces they may sustain at the ultimate limit state. A2

18.5.4 *Longitudinal shear in composite members*

18.5.4.1 *Transfer of interface shear between precast and in situ concrete* A3

The maximum permissible longitudinal shear stress, v_ℓ, shall be 0.55 MPa where:

(a) The contact surfaces are clean, free of laitance, and roughened with a peak to trough amplitude equal to or greater than 5 mm; or

(b) The contact surfaces are clean, laitance is removed and minimum ties are provided in accordance with 18.5.5; or

(c) In the case of precast floor units produced by a dry mix extrusion type process, followed by surface treatment that leaves the top surface with a peak to trough roughness equal to or greater than 2 mm, clean, and free of laitance, and provides shear keys equal to or greater than 40 mm wide at not greater than 1200 mm centres are used with an *in situ* topping for composite construction;

(d) The contact interface between *in situ* and precast concrete is subject to specialised processes to ensure complete bonding by curing regimes and/or chemical processes such as wet to dry epoxy. The validity of such bonding processes shall be proven by cross-joint shear tests

Where both (a) and (b) are satisfied, the maximum permissable longitudinal shear stress, v_ℓ, shall be 2.4 MPa.

Shear transfer between *in situ* concrete and structural members which have been previously constructed, such as walls, beams, etc., shall comply with shear friction requirements in 7.8.

18.5.4.2 *Interface shear stress*

The longitudinal shear may be evaluated by computing the actual compressive or tensile force in any segment, with provisions made to transfer that force as longitudinal shear to the reacting element. Shear stress so derived shall not exceed values given by 18.5.4.1. A3

The interface shear stress, $v_{d\ell}$, may be calculated at any cross section, except in potential plastic regions, by: A3

(a) For uncracked concrete sections, or sections where concrete in tension is neglected in calculating the section properties: A2

$$v_{d\ell} = \frac{V^* Q}{\phi I b_v} \quad \text{(Eq. 18–1)}$$

where b_v is the width of the section at the level being investigated, while in shell beams it may be taken as the length of the interface between the *in situ* concrete and the shell, that is the width of the interface at the bottom of the shell plus the sum of the interfaces of the upstanding sides of the shell, and ϕ = 0.75. A3

(b) For cracked reinforced concrete with zero axial load

$$v_{d\ell} = \frac{V^*}{\phi j d b_v} \quad \text{(Eq. 18–2)}$$

where *jd* is the internal level arm of the flexural forces in the section and ϕ is 0.75. A2

(c) For members subjected to axial load or prestress that sustain flexural cracks, the interface shear stresses, where required, shall be found from the difference in longitudinal stresses at closely spaced section, using standard theory. A3

18.5.4.3 *VOID* A3

18.5.4.4 *Transfer of shear where tension exists*

Where tension exists perpendicular to any surface, shear transfer by contact shall be assumed only when the minimum tie requirements of 18.5.5 are satisfied.

18.5.4.5 *Requirements for bridge superstructures*

For the main flexural members of bridge superstructures, ties equal or in excess of that required by 18.5.5 shall always be provided. Contact surfaces shall always be adequately roughened.

18.5.4.6 *Bridge deck overlays*

For the rehabilitation of an existing bridge deck through the application of an overlay, as an alternative to the requirements of 18.5.4.5, the design approach outlined in C18.5.4.6 of the Commentary may be adopted.

18.5.5 *Ties for longitudinal shear*

18.5.5.1 *Minimum anchorage into composite topping*

Adequately extended and anchored shear reinforcement may be included as contributing toward the resistance of longitudinal shear. The minimum thicknesses of normal density, composite topping concrete with compression strength of at least 25 MPa that stirrups, ties or spirals with 20 mm cover may effectively be anchored in are:

6 mm stirrups, ties or spirals ....50 mm minimum topping
10 mm stirrups, ties or spirals......................75 mm minimum topping
12 mm stirrups, ties or spirals......................90 mm minimum topping
16 mm stirrups, ties or spirals....................105 mm minimum topping

If cover greater than 20 mm is required, the thickness of topping indicated above shall be increased by the amount of additional cover.

18.5.5.2 *Minimum area and spacing of ties*

A3 Where transverse bars or stirrups, ties or spirals are used to transfer longitudinal shear, the tie area shall be equal to or greater than that required by 9.3.9.4.15 and the spacing of the ties shall not exceed four times the least dimension of the supported element or 600 mm, except when the following apply:

(a) For precast prestressed tee units, the spacing in the transverse direction can be increased to a maximum of 1200 mm to permit the ties to be located above the webs;
(b) For other precast prestressed floor units the spacing in one direction may be increased to 600 mm.

18.5.5.3 *Types of ties*

Ties for longitudinal shear may consist of single bars, multiple leg stirrups, spirals, headed studs, or the vertical legs of welded wire fabric. All ties shall be fully anchored into the components in accordance with 7.5.7.

18.5.6 *Precast shell beam construction*

18.5.6.1 *Section and material properties*

The designer shall take into account the various actions of the section as a whole and whether fully composite behaviour or that of only the cast-in-place core of the beam is expected in determining the section and material properties of beams incorporating precast shells.

18.5.6.2 *Requirements for fully-composite action*

Fully-composite action of the precast shell and cast-in-place core of the beam may be assumed only when the shear stresses along the interface between the precast shell and the cast-in-place core comply with 18.5.4.

In determining the nominal shear stresses on the interface, account shall be taken of transverse and longitudinal shear stresses that may occur.

18.5.6.3 *Design of precast shell*

When designing the precast shell in accordance with 18.5.2 and 18.5.3 consideration shall be given as to whether composite action of the beam can be relied on to resist some or all of the forces applied to the shell or whether by design or through other effects the precast shell is required to carry the applied forces alone. A3

18.5.6.4 *Shear strength of composite beam*

In determining the shear strength of the fully composite beam, rational analysis shall be used to evaluate the contributions to shear resistance of all stirrups and ties and the combined concrete of the precast shell and of the cast-in-place core of the beam.

18.6 Structural integrity and robustness

18.6.1 *Load path to lateral force-resisting systems*

Precast concrete elements shall be connected to other precast elements, cast-in-place concrete or steel elements or to the foundation structure in a manner that ensures that effective load paths for the transfer of forces to primary lateral force-resisting systems can be developed. For the purposes of this clause, a floor system consisting of precast elements and a cast-in-place topping shall be regarded as being precast.

18.6.2 *Diaphragm action*

Where precast components participate in the transfer of horizontal forces by means of diaphragm action, the requirements of 13.3.7.4 shall also be satisfied.

18.6.3 *Wall structures*

For precast concrete systems supported on precast wall structures the following provisions shall apply: A3

(a) A continuous load path in floor and roof members as required in 18.6.1 shall provide a tensile capacity by way of longitudinal and transverse ties continuous over internal wall supports and between members and external walls. A nominal strength equivalent equal to or greater than 22 kN per metre shall be separately provided along and across the building. Ties parallel to slab spans shall be spaced at not more than 3 m centres. Provisions shall be made to transfer forces around openings;

(b) In addition, continuous reinforcement shall be placed around the perimeter and within 1.2 m of the edges of each floor and the roof, to resist the design forces and to have a nominal strength in tension equivalent to greater than 70 kN.

18.6.4 *Joints between vertical members*

Vertical tension reinforcement across horizontal joints of essential vertical precast structural members shall be provided in accordance with the following requirements:

(a) Precast columns shall have a nominal strength in tension equal to or greater than that corresponding to a reinforcement ratio of $1.5/f_y$;

(b) For columns with a cross-sectional area larger than that required by consideration of loading, the use of a proportionally reduced effective area equal to or greater than one-half of the total area, A_g, may be used to satisfy 18.6.4 (a);

(c) Precast panels shall have continuous vertical tension reinforcement over the full height of the building capable of transmitting the design forces, and shall have a nominal tensile strength equivalent to at least 45 kN per metre of horizontal wall length. Two or more vertical ties shall be provided in each wall panel.

18.6.5 *Connections*

Connections between precast elements, and between precast and cast-in-place concrete elements, shall be designed to meet the following requirements:

(a) To control cracking due to restraint of volume change, temperature changes, and differential temperature gradients; A3

(b) To develop a failure mode by yielding of steel reinforcement or other non brittle mechanism.

18.6.6 *Frames supporting precast floors*

Frames supporting precast floors shall be tied to the floor in accordance with 10.3.6.

18.6.7 *Deformation compatibility of precast flooring systems*

18.6.7.1 *General*

Precast floor systems shall be designed and detailed to meet the requirements of 2.6.1.1. The implications of the deformation of the primary structure for the seating of the floor system and the integrity of the topping slab shall be considered so that these elements meet their performance requirements.

A3 Design may be based on rational calculation or on methods proved through testing. Calculations or tests shall demonstrate that detailing of the support will permit deformation due to relative rotation and displacement between the precast floor unit and the support consistent with the peak inter-storey drift associated with an maximum considered earthquake event (see 18.3.3), and elongation demands in accordance with 7.8. Calculations or tests shall also make allowance for construction tolerance and deformations associated with creep, shrinkage and thermal effects. Refer also to 19.3.11.2.4 for shear strength of pretensioned floor units near supports.

A3 18.6.7.2 *Precast flooring parallel to beams, walls and other structural elements*

Where hollow-core flooring is adjacent to a beam, wall or other structural element, and where deflections under seismic action may result in different vertical displacements between the hollow-core flooring and the adjacent structural element, either:

A2 (a) The hollow-core unit shall be linked to the parallel beam by the reinforced topping slab only, and shall be placed no closer than the larger of 600 mm or 6 times the thickness of the linking slab to the beam; or

A3 (b) Calculations shall be conducted to demonstrate that plastic hinges cannot form in an adjacent beam and any restraint against incompatible vertical displacement between hollow-core units and adjacent structural elements will not cause longitudinal cracking in the webs and failure of the hollow-core unit.

18.7 Connection and bearing design

18.7.1 *Transfer of forces between members*

Forces may be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing bar connections, welded or bolted connections reinforced topping, or a combination of these means.

A3 18.7.2 *Void*

18.7.3 *Connections using different materials*

When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.

18.7.4 *Floor or roof members supported by bearing on a seating*

A3

A3

18.7.4.1 *General requirements*

For precast floor or roof members supported by bearing onto a seating, with or without the presence of a cast-in-place topping or continuity reinforcement, the support of precast units shall be designed to prevent the transmission of forces to the units that may endanger the structural integrity of the unit or supporting ledge. The requirements in 16.8.2.1, 19.3.11.2.3 and 19.3.11.2.4 shall also be considered.

18.7.4.2 *Bridges*

Bridge spans composed of simply supported precast concrete superstructure elements shall satisfy the connection and support requirements of the New Zealand Transport Agency's Bridge Manual.

18.7.4.3 *Minimum seating requirements for floor or roof members in buildings*

In nominally ductile buildings where there are no limited ductile or ductile plastic regions and the seating is the primary mechanism for transfer of gravity loads, each floor or roof member and its supporting systems shall have design dimensions selected so that, after allowance is made for a reasonable combination of unfavourable construction tolerances, the seating dimension from the end of the precast member to the edge of the support in the direction of the span shall be greater than or equal to the largest of (a), (b) or (c) below:

(a) At least 1/180 of the clear span of the supported member;

(b) A value of:
   (i) 50 mm for solid or composite slabs up to 190 mm total thickness
   (ii) 75 mm for all other precast floor or roof units;

(c) The summation of values described in (i) to (iv) below:
   (i) The greater of the bearing length calculated according to clause 16.3, or 10 mm
   (ii) Potential spalling from the end of the supported member, which shall be taken as 30 mm for solid slabs, hollow-core units and units supported by webs or ribs. No allowance for spalling need be made where armouring is used
   (iii) Potential spalling from the front face of the supporting member, which shall be taken as the greater of the cover to the longitudinal bars, or 30 mm. No allowance for spalling need be made where armouring is used
   (iv) An appropriate allowance for movement due to creep, shrinkage and thermal movement.

Where precast floor or roof members are supported on a seating, the members shall be mounted at both ends on low friction bearing pads or strips with an in-service coefficient of friction of less than 0.7. The pad or strip shall have a minimum width of two-thirds of the width of web or rib, and the full width of a hollow-core unit or flat slab, and a minimum length in the direction of the span of 50 mm.

Where steel plate armouring of the seating is used in a floor or roof member, or its supporting member, the armouring detail shall resist all applied forces and shall be fully engaged with the reinforcement in the supporting member.

The design of the seating, including for serviceability limit state actions, shall make allowance for rotations arising from creep and shrinkage of concrete, variations in temperature and temperature gradients and associated rotations which are particularly significant where units are exposed to the sun.

18.7.4.4 *Detailing requirements for support of hollow-core floors*

Floors incorporating precast hollow-core units shall meet the requirements of either (a) or (b):

(a) Calculations from first principles or tests shall demonstrate that the detailing of the support can sustain the rotation and lateral displacement between the units and supporting beam or wall associated with $1.5/S_p$ times the inter-storey drift;

(b) The support detail described in C18.6.7(e) shall be used.

A3 **18.7.5 *Development of positive moment reinforcement***

The requirements of 8.6.13.1 shall not apply to the positive bending moment reinforcement for statically determinate precast members, but at least one-third of such reinforcement shall extend to the centre of the bearing length, taking into account tolerances described in 18.3.

A3 **18.7.6 *Precast stairs and ramps***

The supports of stairs and access ramps shall satisfy 2.2.3(d) and 2.6.10 and be capable of sustaining the inter-storey drift specified for a sliding ledge support in NZS 1170.5, in addition to an appropriate allowance for construction tolerance and potential elongation effects. Allowance is also to be made for friction forces induced by sliding from thermal movements and movements resulting from seismic and other effects.

18.8 Additional requirements for ductile structures designed for earthquake effects

A3 **18.8.1 *Seating requirements for ductile structures***

Where seating is the primary mechanism for the transfer of gravity loads in ductile and limited ductile structures, or in nominally ductile structures that contain ductile or limited ductile plastic hinges, the length of the seating on the supporting member without edge armouring shall be greater than the summation of lengths (a) to (g), but not less than required by 18.7.4:

(a) The bearing length required by 16.3, but not less than 10 mm;

(b) Cover loss due to spalling of the support beam ledge equal to:

  (i) In the ductile detailing length, ℓ_y, the distance from the outside face of the support to the centreline of the longitudinal reinforcing bar that is anchored by ties

  (ii) Outside the ductile detailing length, the cover to the longitudinal bars in the supporting member;

(c) Peak maximum considered earthquake plastic hinge elongation, which is $1.5/S_p$ times the elongation at the ultimate limit state design inter-storey drift, calculated in accordance with 7.8 at the mid-depth of the beam, but not greater than $0.036h_b$ for each hinge. Where the precast unit spans past multiple bays in a frame, the total elongation is the sum of the total elongation in those bays;

(d) Any reduction in seating length due to the twisting of the supporting beam due to inter-storey drift of $1.5/S_p$ times the peak ultimate limit state inter-storey drift;

(e) Potential spalling of the back face of the supported member in accordance with 18.7.4.3(c)(ii);

(f) An allowance for shrinkage, creep, variations in temperature and temperature gradients;

(g) An allowance for construction tolerances.

18.8.2 *Detailing requirements for support of rib and infill floors*

Rib and infill floor systems shall not be restrained by the concrete of the supporting element against rotation at the supports that would cause tension in the bottom of the precast floor system.

18.8.3 *Composite concrete flexural members*

18.8.3.1 *Diaphragm action*

Where diaphragm action is to be provided by means of a cast-in-place topping, the requirements of 13.4.3 shall be satisfied.

A3 **18.8.3.2** *Frame dilatancy*

Adequate support shall be provided to precast flooring units to take account of inelastic actions of ductile frames including the effects of frame dilatancy.

A3 **18.8.3.3** *Precast shell beams*

A3 **18.8.3.3.1** *Length of potential plastic hinge regions in moment resisting frames*

For beams of moment resisting frames that are constructed incorporating precast shells and are expected to form plastic hinges, the ductile detailing lengths shall be taken to be equal to or greater than twice the depth of the cast-in-place cores of the beams.

A3 **18.8.3.3.2** *Flexural strength in potential plastic regions*

In potential plastic regions the nominal and design flexural strengths shall be determined from the cast-in-place concrete beam core alone.

A3 **18.8.3.3.3** *Flexural overstrength*

The flexural overstrength of the potential plastic regions shall be determined as follows:

(a) For moments that induce tension stresses in the bottom fibres of the precast shell:
   (i) When the critical section of a plastic region occurs at the column face, the flexural overstrength shall be calculated from the section and materials of the section that is effective at the interface;
   (ii) When the critical section of a potential plastic region occurs at a distance greater than the depth of the core along the beam, the flexural overstrength shall be calculated from the section and material properties of the beam assuming fully composite behaviour.

(b) For moments that induce tension stresses in the top fibres of the beam, the flexural overstrength shall be calculated from the section and material properties of the beam assuming fully composite behaviour.

18.8.3.3.4 *Design strength of shell beam in potential plastic region* A3

In potential plastic hinge regions it shall be assumed that there is no composite action between the cast-in-place core and the adjacent shell when designing the shell in accordance with 18.5.6.3, unless detailing is provided to ensure composite action is maintained by ties which are anchored around longitudinal reinforcement in the shell beam and anchored in the cast in place core. The quantity of this reinforcement shall maintain the shear transfer across the interface by satisfying the requirements of either 18.5.4 or 7.7. A2

18.8.3.3.5 *Flexural design of shell beams between potential plastic hinge regions* A3

In the region between the potential plastic hinge regions, the flexural design may be undertaken assuming fully composite action only when the shear stresses, along the interface between the precast shell and the cast-in-place core comply with 18.5.4.

18.8.4 ***Broad categories of precast concrete seismic systems*** A3

18.8.4.1 *Construction incorporating precast concrete*

The construction of seismic moment resisting frames and structural walls incorporating precast concrete elements generally fall into two broad categories, either "equivalent monolithic" systems or, "jointed" systems. The distinction between these types of construction is based on the design of the connections between the precast concrete elements as provided by 18.8.4.2 to 18.8.4.3. A3

18.8.4.2 *Equivalent monolithic systems*

18.8.4.2.1 *Definition*

A precast concrete structural system satisfying the requirements of this clause shall have strength and toughness equivalent to that provided by a comparable monolithic reinforced concrete structure.

18.8.4.2.2 *Connections in monolithic systems* A3

The connections between precast concrete elements of equivalent monolithic systems (cast-in-place emulation) can be subdivided into two categories:

(a) *Strong connections of nominal ductility*

   In moment resisting frames and structural walls these connections are protected by a capacity design approach which ensures that flexural yielding occurs away from the connection region;

(b) *Ductile connections*

   Ductile connections of equivalent monolithic systems typically comprise longitudinal reinforcing bars in the connection which are expected to enter the post-elastic range in a severe earthquake.

In moment resisting frames yield penetration may occur into the connection end-region. The potential plastic hinge region may extend a distance along the end of the member as in cast-in-place construction.

18.8.4.3 *Jointed systems* A3

18.8.4.3.1 *Definition*

In jointed systems the connections are weaker than the adjacent precast concrete elements. Jointed systems do not emulate the performance of cast-in-place concrete construction. The post-elastic deformations of these systems during an earthquake are typically concentrated at the interfaces of the precast concrete elements where a crack opens and closes.

18.8.4.3.2 *Connections in jointed systems* A3

The connections between precast concrete elements of jointed systems can be subdivided into three categories:

(a) *Connections of limited ductility*

Connections of limited ductility in jointed systems are usually dry connections formed by welding or bolting reinforced bars or plates or steel embedments and dry-packing and grouting. These connections do not behave as if part of a monolithic construction and generally have limited ductility. An example of a jointed system with connections of limited ductility involving structural walls is tilt up construction. Generally such structures are designed for limited ductility or nominally ductile behaviour;

(b) *Ductile jointed connections*

Ductile connections of jointed systems are generally dry connections in which unbonded post-tensioned tendons are used to connect the precast concrete elements together. The non-linear deformations of the system are concentrated at the interfaces of the precast concrete elements where a crack opens and closes. The unbonded post-tensioned tendons remain in the elastic range. These connections have the advantage of reduced damage and of being self-centring (i.e., practically no residual deformation) after an earthquake;

(c) *Ductile hybrid connections*

Hybrid systems have connections which combine both unbonded post-tensioned tendons and longitudinal steel reinforcing bars (tension/compression yield) or other energy dissipating devices (e.g., flexing steel plates or friction devices).

Appendix B of Part 1 provides some guidance and further references for the design of ductile-jointed hybrid precast concrete systems.

A3 **18.8.4.4** *Precast shell beam construction*

18.8.4.4.1 *Length of potential plastic hinge regions in moment resisting frames*

For beams of moment resisting frames that are constructed incorporating precast shells and are expected to form plastic hinges, the ductile detailing lengths shall be taken to be equal to or greater than twice the depth of the cast-in-place cores of the beams.

A3 **18.8.4.4.2** *Flexural strength in potential plastic regions*

In potential plastic regions the nominal and design flexural strengths shall be determined from the cast-in-place concrete beam core alone.

A3 **18.8.4.4.3** *Flexural overstrength*

The flexural overstrength of the potential plastic regions shall be determined as follows:

(a) For moments that induce tension stresses in the bottom fibres of the precast shell:

   (i) When the critical section of a plastic region occurs at the column face, the flexural overstrength shall be calculated from the section and materials of the section that is effective at the interface;

   (ii) When the critical section of a potential plastic region occurs at a distance greater than the depth of the core along the beam, the flexural overstrength shall be calculated from the section and material properties of the beam assuming fully composite behaviour.

(b) For moments that induce tension stresses in the top fibres of the beam, the flexural overstrength shall be calculated from the section and material properties of the beam assuming fully composite behaviour.

A3 **18.8.4.4.4** *Design strength of shell beam in potential plastic region*

A2 In potential plastic hinge regions it shall be assumed that there is no composite action between the cast-in-place core and the adjacent shell when designing the shell in accordance with 18.5.6.3, unless detailing is provided to ensure composite action is maintained by ties which are anchored around longitudinal reinforcement in the shell beam and anchored in the cast in place core. The quantity of this reinforcement shall maintain the shear transfer across the interface by satisfying the requirements of either 18.5.4 or 7.7.

A3 **18.8.4.4.5** *Flexural design of shell beams between potential plastic hinge regions*

In the region between the potential plastic hinge regions, the flexural design may be undertaken assuming fully composite action only when the shear stresses, along the interface between the precast shell and the cast-in-place core comply with 18.5.4.

19 PRESTRESSED CONCRETE

19.1 Notation

a	depth of equivalent rectangular stress block as defined in 7.4.2.7, or depth of compression force against post-tension anchor, mm
A	area of concrete between extreme tension fibre and centroid of uncracked section, mm^2
A_c	area of concrete at the cross section considered, mm^2, or area of core or spirally confined compression zone measured to outside of spiral, mm^2
A_{cf}	larger gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, mm^2
A_{cv}	effective shear area, area used to calculate shear stress, mm^2 A2
A_g	gross area of section, mm^2
A_{ps}	area of prestressed reinforcement in flexural tension zone, mm^2
A_s	area of non-prestressed tension reinforcement, mm^2
A_s'	area of non-prestressed compression reinforcement, mm^2
A_v	area of shear reinforcement within a distance s, mm^2
b	width of compression face of member, mm
b_o	is perimeter of critical section, mm
b_w	web width, mm
c	distance from extreme compression fibre to neutral axis, mm
c_c	clear cover from the nearest surface in tension to the surface of the flexural tension steel, mm
d	distance from extreme compression fibre to centroid of flexural tension reinforcement, but for prestressed members need not be taken as less than 0.8h, mm
d_c	the distance from extreme compression fibre to the centroid of the prestressed reinforcement, mm
d_p	distance from extreme compression fibre to centroid of prestressing reinforcement, or to combined centroid of the area of reinforcement when non-prestressing tension reinforcement is included, mm
d'	distance from extreme compression fibre to centroid of compression reinforcement, mm
e	base of Napierian logarithm
E_c	modulus of elasticity of concrete, MPa
E_p	modulus of elasticity of prestressing steel, MPa
f_c'	specified compressive strength of concrete, MPa

A2

f_{ci}'	compressive strength of concrete at time of initial prestress, MPa
f_{dc}	stress in a reinforcing bar before concrete cracks when the stress in the concrete surrounding the bar is zero, MPa

A2

f_{pc}	average pre-stress, MPa A3
f_{px}	stress in tendon at distance L_{px} measured from the jacking end, MPa

A2

f_{pe}	compressive stress in concrete due to effective prestressing forces only (after allowing for all prestress losses) at extreme fibre of section where tensile stress is caused by externally applied loads, MPa
f_{pi}	stress in tendon immediately after transfer, MPa
f_{pj}	stress in tendon at the jacking end, MPa
f_{ps}	stress in prestressed reinforcement at nominal strength, MPa
f_{pu}	tensile strength of prestressing steel being the quotient of the characteristic minimum breaking force and the nominal cross section area, MPa A2
f_{py}	specified yield strength of prestressing steel, or the 0.2 % proof stress, MPa
f_s	stress in non-prestressed bonded reinforcement at service loads, MPa
f_{se}	effective stress in prestressed reinforcement (after allowance for all prestress losses), MPa

f_{ss}	stress induced on extreme tension fibre due to self strain action, MPa
f_{sw}	stress sustained at neutral axis due to self strain action, MPa
f_t	extreme fibre stress in tension in the precompressed tensile zone, computed using gross or transformed section properties, MPa
f_y	lower characteristic yield strength of non-prestressed longitudinal reinforcement, MPa
g_s	distance from centre of reinforcing bar to a point on surface of concrete where crack width is being assessed, mm
h	overall thickness of member, mm
I	second moment of area of section resisting externally applied loads, mm^4
j	the time after prestressing, days
k_b	coefficient based on bond characteristics of reinforcement
k_4	coefficient dependent on duration of prestressing force
k_5	coefficient dependent on stress in tendon
k_6	function dependent on average annual temperature
L_{px}	the length of the tendon from the jacking end to a point at a distance a from that end, mm
M_{cr}	bending moment causing flexural cracking at section due to externally applied loads, N mm
M_{max}	maximum design bending moment at section due to externally applied loads, N mm
M_o	bending moment sustained at decompression of extreme tension fibre, N mm
M^*	design bending moment at section at ultimate limit state, N mm
N_c	tensile force in the concrete due to service dead load plus live load, N
$N_{n,max}$	axial load strength of member when the external load is applied without eccentricity, that is, when uniform strain exists across section, N
N^*	design axial load at the ultimate limit state, N
p	ratio of non-prestressed tension reinforcement, A_s/bd
p_p	ratio of prestressed reinforcement, A_{ps}/bd_p
P_{su}	factored prestressing force at the anchorage device, N
p'	ratio of non-prestressed compression reinforcement, A_s'/bd
R	a coefficient equal to the ratio of loss of prestress force due to relaxation of the prestressed tendon to the initial prestress force in the tendon at anchorage or after transfer
R_b	basic relaxation of tendon, MPa
R_{sc}	ratio of loss of prestress force due to relaxation of tendon to the initial prestress force modified to account for the effects of creep and shrinkage in the concrete
s	centre-to-centre spacing of flexural tension steel near the extreme tension face, mm. Where there is only one bar or tendon near the extreme tension face, s is the width of the extreme tension face
T	temperature, °C
v_c	shear stress resisted by concrete, MPa
V	shear force, N
V_b	shear resisted by concrete in an equivalent reinforced concrete beam, N
V_c	nominal shear strength provided by concrete, N
V_{ci}	nominal shear strength provided by the concrete when diagonal tension cracking results from combined shear and moment, N
V_{cw}	nominal shear strength provided by the concrete when diagonal tension cracking results from principal tensile stress in web, N
V_p	vertical component of effective prestressing force at section, N
V_s	nominal shear strength provided by the shear reinforcement, N
V^*	design shear force at section at ultimate limit state, N
y_t	distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension, mm
α_{tot}	sum of the absolute values of successive angular deviations of the prestressing tension over length L_{px}, radians
α_c	linear coefficient of expansion of concrete, $°C^{-1}$
α_s	factor for determining the shear carried by concrete at columns of two-way prestressed slabs and footings

β_1 factor defined in 7.4.2.7

β_p constant and to compute V_c in prestressed slabs, or an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects

μ coefficient of friction between post-tension cable and prestressing duct

γ_p factor for type of prestressing tendon
= 0.55 for f_{py}/f_{pu} not less than 0.80
= 0.40 for f_{py}/f_{pu} not less than 0.85
= 0.28 for f_{py}/f_{pu} not less than 0.90

Δf_{ps} stress in prestressing steel at service loads based on cracked section analysis less decompression stress, f_{dc} in prestressing steel, MPa

ε_{cc} creep strain in concrete

ε_{cs} shrinkage strain in concrete

λ factor to provide for lightweight concrete (see 7.7.4.3)

ϕ strength reduction factor (see 2.3.2.2)

ϕ_{cc} design creep factor

ω $p\, f_y / f'_c$

ω' $p'\, f_y / f'_c$

19.2 Scope

19.2.1 *General*

Provisions in this section apply to structural members prestressed with wires, strands or bars meeting the requirements of NZS 3109, AS 1311 or AS 1313 or AS/NZS 4672.1 for prestressing steels.

19.2.2 *Other provisions for prestressed concrete*

The following provisions shall be applied to the design of prestressed concrete members;

(a) Sections 1 to 7 inclusive;
(b) The spacing of non-prestressed and pretensioned reinforcement in 8.3 but excluding 8.3.5;
(c) The development of non-prestressed reinforcement in 8.6;
(d) The detailing of non-prestressed reinforcement in respect to bar bending, welding, development and splicing in 8.4, 8.5, 8.6 and 8.7 respectively
(e) Other provisions where they are specifically noted in chapter 19.

19.3 General principles and requirements

19.3.1 *General design assumptions*

19.3.1.1 *Design requirements*

Members shall meet the requirements for the serviceability and ultimate limit states specified in this Standard. Design shall be based on strength at the ultimate limit state and on behaviour at the serviceability limit state at all stages that may be critical during the life of the structure from the time the prestress is first applied.

19.3.1.2 *Concrete stresses at the serviceability limit state*

Concrete stresses at the serviceability limit state shall not exceed the values given in 19.3.3.5 unless it can be shown by analysis or test that performance of the member will not be impaired.

19.3.1.3 *Secondary prestressing moments*

The moments due to the reactions which are induced by the prestressing forces shall be:

(a) Included in the calculation of stresses and deflections for the serviceability limit state; and
(b) May be excluded in the calculations relating to the required flexural strength of sections at the ultimate limit state where it can be shown that the member has sufficient ductility to accommodate the associated inelastic deformation;
(c) In the design for shear, load cases both with and without secondary moments shall be considered.

19.3.1.4 *Effect of deformations*

Provision shall be made for the effects on parts of the structure or adjoining structure of elastic and plastic deformation and the effects of volume change due to temperature variation, creep and shrinkage of the concrete.

19.3.1.5 *Possibility of buckling*

The possibility of buckling in a member between points where the concrete and the prestressing steel are in contact and of buckling in thin webs and flanges shall be considered.

19.3.1.6 *Section properties*

In computing section properties before bonding of prestressing steel the effect of loss of area due to open ducts shall be considered.

19.3.1.7 *Tendons deviating from straight lines*

Where tendons are subjected to deviations from a straight line, allowance shall be made for the forces caused by these deviations.

19.3.1.8 *Reinforcement for shrinkage and temperature stresses*

Reinforcement for shrinkage and temperature stresses normal to the direction of prestress shall be provided, where appropriate, in accordance with 8.8 or 18.5.1.

19.3.1.9 *Stress concentrations*

Stress concentrations due to prestressing shall be considered in the design.

19.3.1.10 *Unbonded tendons*

Where unbonded tendons are used:

(a) The use of unbonded tendons is permitted provided they are in accordance with NZS 3109, are adequately protected from corrosion in accordance with 19.3.15, and the exposure classification as defined in Table 3.1 is not C or U;
(b) Serviceability requirements shall be in accordance with 19.3.3;
(c) Bonded reinforcement shall be provided in accordance with 19.3.6.7;
(d) The flexural strength shall be computed in accordance with 19.3.6.

19.3.2 *Classification of prestressed members and sections*

Prestressed concrete flexural members and sections shall be classified by their condition at the serviceability limit state as uncracked (Class U), transitional between cracked and uncracked (Class T), or cracked (Class C) based on the computed extreme fibre stress, f_t, at service loads in the precompressed tensile zone assuming an uncracked section as follows:

(a) Class U: Buildings........................ $f_t < 0.7\sqrt{f_c'}$; Bridges............................... $f_t < 0.0\sqrt{f_c'}$
(b) Class T: Buildings ....$0.7\sqrt{f_c'} < f_t \le \sqrt{f_c'}$; Bridges....................... $0.0 < f_t < 0.5\sqrt{f_c'}$
(c) Class C: Buildings.............................. $f_t > \sqrt{f_c'}$. Bridges............................... $f_t > 0.5\sqrt{f_c'}$

A2 Where, on prestressed two-way slab systems in buildings, uniformly distributed loading creates the critical bending moments, the slab system shall be designed as Class U.

Where f_t exceeds $0.7\sqrt{f_c'}$ and the area of a flange on the tension side of the member exceeds the area of a corresponding flange on the compression side of the member, the member shall be designed as Class C.

The location where a member contains a construction joint and $f_t > 0.0$ shall be considered to be class C.

19.3.3 *Serviceability limit state requirements – flexural members*

19.3.3.1 *General*

Members shall meet the requirements at the serviceability limit state for permissible stresses and deflections.

19.3.3.2 *Calculation of stresses in the elastic range*

For investigation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with the following assumptions:

(a) Strains vary linearly with depth through the entire load range;

(b) At cracked sections, concrete resists no tension.

19.3.3.3 *Section properties*

For the calculation of stresses under service loads, for Class U and Class T flexural members, either gross or transformed uncracked section properties may be used. For Class C flexural members and the sections at construction joints, cracked transformed section properties shall be used.

19.3.3.4 *Deflection*

Deflections of prestressed members at the serviceability limit state shall be calculated in accordance with 6.8.4 and shall satisfy the requirements of 2.4.2.

19.3.3.5 *Permissible stresses in concrete*

19.3.3.5.1 *Permissible stresses in compression*

For prestressed concrete members, stresses in concrete at service loads shall not exceed the following: A2

(a) For class U, T and C flexural members, compression stresses in concrete in the extreme fibres, calculated on the basis of uncracked sections, shall immediately after transfer of prestress not exceed the stress 0.6 f'_{ci};

(b) For class U and class T flexural members, compression stresses in concrete in the extreme fibres, calculated on the basis of uncracked sections, shall not exceed the following:

  (i) After losses of prestress due to prestress plus sustained service load, or normal live load for bridges .... 0.45 f'_c

  (ii) After losses of prestress due to prestress plus total service load, or traffic overload for bridges .... 0.6 f'_c

  (iii) Where a differential temperature case associated with solar radiation on a member is considered, the stress limit given in (a) or (b) above may be increased to the smaller of 0.75 f'_c or the value given in (a) or (b) above with the addition of 0.67 $\alpha_c E_c T$, where T is the increase in temperature on the surface being considered, E_c is the elastic modulus of the concrete, and α_c is the coefficient of thermal expansion of the concrete.

(c) For class C compression stresses in concrete in the extreme fibres after prestress loss, calculated on the basis of transformed cracked sections, shall not exceed the following:

  (i) 0.52 f'_c under sustained service load or normal live load for bridges

  (ii) 0.65 f'_c under total service load conditions or traffic overload for bridges

  (iii) 0.75 f'_c where differential temperature associated with solar radiation is included in the load combination.

19.3.3.5.2 *Permissible stresses in tension – Class U and Class T members*

(a) The extreme fibre tensile stresses in Class U and Class T members under service loads shall not exceed the tensile stress limits for these member classes respectively given in 19.3.2; A2

(b) The tensile stresses in the concrete of Class U and Class T members immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following, unless reinforcement is added as specified in (c): A2

  (i) Extreme fibre stress in tension, except as permitted in (ii) A1

  Buildings: $0.25\sqrt{f'_{ci}}$ Bridges: $0.25\sqrt{f'_{ci}} < 1.4$ MPa

  (ii) Extreme fibre stress in tension at the ends of simply supported members

  Buildings: $0.5\sqrt{f'_{ci}}$ Bridges: $0.5\sqrt{f'_{ci}} < 2.8$ MPa;

(c) Where the extreme fibre tensile stress in a section immediately after transfer exceeds the appropriate limit given in (b) above, bonded reinforcement shall be provided to resist the total tensile force carried A2

A2 by the concrete, calculated on the basis of uncracked section analysis. This reinforcement may take the form of:

(i) Additional bonded non-prestressed reinforcement, in which case the area shall be sufficient to sustain the force at a stress equal to, or less than, the smaller of 200 MPa or 0.5 f_y.

(ii) Bonded pretension strands or wires, with an area such that the stress increment is equal to or less than 200 MPa, and the resultant stress in the strands or wires does not exceed the limit in 19.3.3.6.1(d).

This force shall be calculated on the basis of uncracked section properties, and the area of this reinforcement shall be sufficient to resist this force based on a stress equal to or less than the smaller of 210 MPa or 0.5f_y. This reinforcement shall be distributed relatively uniformly across the width of the tensile face of the member and positioned as close to the extreme tensile fibre of the member as practical.

A2 **19.3.3.5.3** *Crack widths for Class C members*

Crack widths for members subjected to serviceability limit state load combinations, but excluding wind or earthquake, shall be controlled by satisfying either (a) or (b) or (c) below:

(a) The tensile stress in the concrete calculated on the basis of uncracked section, is equal to or less than 0.0 at a construction joint, or $0.4\sqrt{f_c'}$ at other locations;

(b) The following two conditions are met:

(i) The tensile stress increment, Δf_s, is less than 250 MPa; and

(ii) The spacing, s, of bonded reinforcement nearer the extreme tension fibre shall not exceed that given by:

$$s = k_b\left[\frac{90000}{\Delta f_s - 50} - 2.5c_c\right] \qquad \text{(Eq.19–1)}$$

or

$$s = k_b\left[\frac{70000}{\Delta f_s - 50}\right] \qquad \text{(Eq.19–2)}$$

where

c_c is the clear cover distance between the surface of the reinforcement and the surface of the tension member

Δf_s is the change in stress of reinforcement that occurs between the value sustained in the serviceability design load case being considered and the value when the surrounding concrete is decompressed (at zero stress) after all long-term losses have occurred

k_b is equal to 1 for deformed reinforcing bars, $^2/_3$ for strands and $^5/_6$ where a mixture of deformed bars and strands are used

(c) Where limitations are placed on an acceptable crack width, w, in the flexural tension zone of a member, the crack width may be assessed from 2.4.4.6, in which g_s is replaced by g_s/k_b, where k_b is 1.0 for deformed bars and $^2/_3$ for strands, and f_s is replaced by $(\Delta f_s - 50)$. Where Δf_s is less than 150 MPa, the crack width may be assumed to be satisfactory without calculation.

19.3.3.6 *Permissible stresses in prestressed and Non-Prestressed Reinforcement*

A2 **19.3.3.6.1** *Permissible stresses in prestressed and non-prestressed reinforcement*

Tensile stress in prestressing tendons shall not exceed the following:

(a) Due to jacking force .... 0.94 f_{py}

but not greater than the lesser of 0.80 f_{pu} or the maximum value recommended by the manufacturer of prestressing tendons and anchorages;

(b) Immediately after prestress transfer .... 0.82 f_{py}
but not greater than 0.74 f_{pu};

(c) Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage .... 0.70 f_{pu}

(d) Pretensioned and post-tensioned strands after all losses........ 0.8 f_{py} A2
or non-prestressed longitudinal reinforcement .... 0.8 f_y

19.3.3.6.2 *Permissible service load stress ranges in prestressed and non-prestressed reinforcement*

(a) The stress range due to frequently repetitive live loading in straight prestressing strands or tendons shall not exceed 150 MPa unless justified by a special study;

(b) The stress range due to infrequent live loading in straight prestressed tendons shall not exceed 200 MPa, unless justified by a special study; A2

(c) The stress range in non-prestressed reinforcement shall comply with 2.5.2.

19.3.3.7 *Reinforcement on sides of beams*

If the total depth of a beam is equal to or greater than 1.0 m, skin reinforcement consisting of deformed reinforcement or bonded tendons shall be provided in the side faces as required by 2.4.4.5.

19.3.4 *Loss of prestress in tendons*

19.3.4.1 *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 19.3.4.2 and 19.3.4.3 respectively.

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

19.3.4.2 *Loss of prestress due to creep and shrinkage*

The loss of stress in prestressing tendons due to shortening of the cables as a result of elastic strains, creep strains in the concrete and shrinkage strains in the concrete, shall be calculated.

19.3.4.2.1 *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 may be applicable.

19.3.4.2.2 *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.

19.3.4.2.3 *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 as follows in order to obtain an estimate of the prestressing forces at the critical sections considered in the design.

19.3.4.2.4 *Determination of losses*

The extension of the tendon shall be calculated allowing for the variation in tension along its length.

(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, (f_{px}), at a distance L_{px}, measured from the jacking end, may be taken as:

$$f_{px} = f_{pj}e^{-\mu(\alpha_{tot}+\beta_p L_{px})} \quad \text{(Eq. 19–3)}$$

where

f_{pj} is the stress in the tendon at the jacking end

e is the base of Napierian logarithms

μ is the 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, may 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
(iv) Plastic ducts 0.14

α_{tot} is the sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length, (L_{px})

β_p is an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects, which as a first approximation may be taken as:

(i) For sheathing containing tendons other than bars and having an internal diameter:
(A) <50 mm .... 0.024 to 0.016 rad/m
(B) 50 mm but <90 mm .... 0.016 to 0.012 rad/m
(C) 90 mm but ≤140 mm .... 0.012 to 0.008 rad/m
(ii) For flat metal ducts containing tendons other than bars .... 0.024 to 0.016 rad/m
(iii) For sheathing containing bars and having an internal diameter of 50 mm or less .... 0.016 to 0.008 rad/m
(iv) For bars of any diameter in a greased-and-wrapped coating .... 0.008 rad/m
(v) Plastic ducts .... 0.001 rad/m

L_{px} is the length of the tendon from the jacking end to the point being considered

19.3.4.2.5 *Verification of friction losses*

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

19.3.4.2.6 *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 adjustment correspondingly required shall be made.

19.3.4.2.7 *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 curing 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) Relaxation of the tendon prior to transfer.

19.3.4.3 *Time-dependent losses of prestress*

19.3.4.3.1 *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 may be applicable.

19.3.4.3.2 *Loss of prestress due to shrinkage of the concrete*

The loss of stress in the tendon due to shrinkage of the concrete shall be based on the free shrinkage strain, ε_{cs}, determined in accordance with 5.2.10. In the absence of more detailed calculations, such as outlined in Appendix CE, the loss of prestress force shall be taken as $E_p\varepsilon_{cs}A_{ps}$, where ε_{cs} may be modified to allow for the effects of reinforcement.

19.3.4.3.3 *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, such as outlined in Appendix CE, and provided that the sustained stress in the concrete at the level of the tendons at no time exceeds 0.5 f'_c, the loss of prestress force due to creep of the concrete may be taken as $E_p\varepsilon_{cc}A_{ps}$, in which ε_{cc} is given by:

$\varepsilon_{cc} = \phi_{cc}\,(f_{cc}/E_c)$ .... (Eq. 19–4) A2

where

ϕ_{cc} is the design creep factor, calculated in accordance with 5.2.11

f_{cc} is the 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, together with the sustained long-term loads.

19.3.4.3.4 *Loss of prestress due to tendon relaxation*

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

(a) *Basic relaxation*

The basic relaxation coefficient, R_b, of a tendon after one thousand hours at 20 °C and (0.8 f_{pu}) shall be determined in accordance with AS/NZS 4672.1; A2

(b) *Design relaxation*

The design relaxation coefficient of a tendon, R, shall be determined from:

$R = k_4\, k_5\, k_6\, R_b$ .... (Eq. 19–5)

where

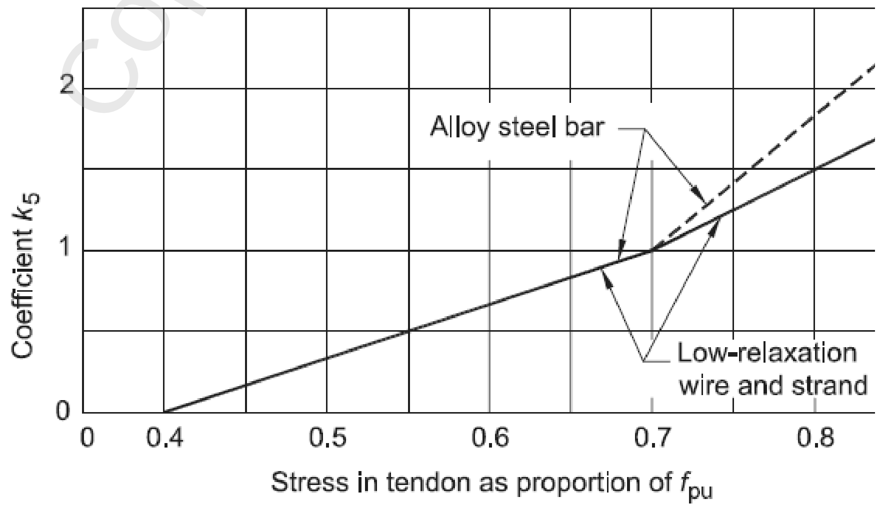
k_4 is a coefficient dependent on the duration of the prestressing force $= \log\,[5.4(j)^{1/6}]$

j is the time after prestressing, in days

k_5 is a coefficient, dependent on the stress in the tendon as a proportion of f_{pu}, determined from Figure 19.1 A3

k_6 is a function, dependent on the average annual temperature (T) in °C, taken as T/20 but equal to or greater than 1.0.

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



A2

Figure 19.1 – Coefficient k_5

(c) *Determination of loss due to relaxation*

The proportion of loss of prestress in a tendon due to relaxation of the tendon in the member shall be determined by modifying the stress loss 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 proportional loss of stress in the tendon, R_{sc}, in the member may be taken as:

$$R_{sc} = R\left(1 - \frac{\text{the loss of stress due to creep and shrinkage}}{f_{pi}}\right) \quad \text{(Eq. 19–6)}$$

where

f_{pi} is the stress in the tendon immediately after transfer.

19.3.4.4 *Loss of prestress due to other considerations*

Account shall be taken, if applicable, of losses due to:

(a) Deformations in the joints of precast structures assembled in sections; and

(b) The effects of any increase in creep caused by frequently repeated loads.

19.3.5 *Ultimate limit state design requirements*

Members shall meet the requirements at the ultimate limit state for flexure axial load and shear.

19.3.6 *Flexural strength of beams and slabs*

19.3.6.1 *Design flexural strength*

The design flexural strength of members containing prestressed reinforcement shall be taken as the nominal strength times the strength reduction factors, given in 2.3.2.2.

19.3.6.2 *Nominal flexural strength*

The nominal flexural strength shall be determined from basic assumptions in 7.4.2 with allowance being made for the additional strain in prestressed reinforcement due to prestressing. The stress in the prestressing tendons at the flexural strength, f_{ps}, shall be determined in accordance with 19.3.6.3, or alternatively where appropriate, it may be determined by the method given in 19.3.6.4.

19.3.6.3 *Strain compatibility analysis*

The stress in prestressed reinforcement in all cases may be determined from strain compatibility analysis using an appropriate stress-strain relationship for the prestressing tendons. In calculating the strain in the prestressing tendons allowance shall be made for strains imposed by prestressing.

19.3.6.4 *Alternative method*

As an alternative to a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} may be used where all the prestressed reinforcement is in the tension zone and if f_{se} is equal to or greater than $0.5f_{pu}$.

(a) For members with bonded tendons:

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[p_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]\right\} \quad \text{(Eq. 19–7)}$$

If any compression reinforcement is taken into account when calculating f_{ps} by Equation 19–7, the term

$$\left[p_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]$$

shall be taken equal to or greater than 0.17 and d' shall be no greater than $0.15d_p$;

(b) For members with unbonded tendons and with a span-to-depth ratio of 35 or less:

$$f_{ps} = f_{se} + 70 + \frac{f'_c}{100\,p_p} \quad \text{(Eq. 19–8)}$$

but f_{ps} in Equation 19–8 shall not be taken greater than f_{py} nor greater than $(f_{se} + 420)$;

(c) For members with unbonded tendons and with a span-to-depth ratio greater than 35:

$$f_{ps} = f_{se} + 70 + \frac{f_c'}{300 p_p} \quad \text{(Eq. 19–9)}$$

but f_{ps} in Equation 19–9 shall not be taken greater than f_{py}, nor greater than (f_{se} + 200).

19.3.6.5 *Non-prestressed reinforcement*

Non-prestressed reinforcement if used with prestressing steel, may be considered to contribute to the internal force and to be included in moment strength computations at a stress equal to that determined by strain compatibility analysis.

19.3.6.6 *Limits for longitudinal reinforcement*

19.3.6.6.1 *Maximum amount of reinforcement*

For beams and slabs the amount and distribution of longitudinal prestressed and non-prestressed reinforcement provided shall be such that when the nominal moment of resistance is developed the distance of the extreme compression fibre to the neutral axis shall not exceed the limiting value given in 19.3.6.6.2.

19.3.6.6.2 *Limiting neutral axis depth*

The limiting neutral axis depth shall be calculated from strain compatibility assuming the strain in the concrete in the extreme compression fibre is 0.003 and the increase in tensile strain in the prestressed reinforcement above that sustained when it was initially prestressed, or the strain in non-prestressed reinforcement closest to the extreme tension fibre is 0.0044.

19.3.6.6.3 *Minimum cracking moment*

The design moment in flexure for any section at the ultimate limit state shall be equal to or greater than 1.2 times the moment at first cracking computed on the basis of a modulus of rupture of $0.6\sqrt{f_c'}$. This provision may be waived for:

(a) Two-way, unbonded post-tensioned slabs; and

(b) Flexural members, where the flexural strength is at least twice that required by the ultimate limit state requirements of AS/NZS 1170 and NZS 1170.5 or other referenced loading standard.

19.3.6.6.4 *Placement of bonded reinforcement*

Part or all of the bonded reinforcement consisting of bars or tendons shall be provided as close as practicable to the extreme tension fibre in all prestressed flexural members, except that in members prestressed with unbonded tendons, the minimum bonded reinforcement consisting of bars or tendons shall be as required by 19.3.6.7.1 and 19.3.6.7.2.

19.3.6.7 *Minimum bonded reinforcement*

19.3.6.7.1 *Minimum bonded reinforcement with unbonded tendons*

Except for two-way flat slab systems and structures designed in accordance with 19.4.6 the minimum amount of bonded reinforcement, A_s, in members containing unbonded prestressing tendons shall be:

$$A_s = 0.004A \quad \text{(Eq. 19–10)}$$

where A is the area of concrete between the extreme flexural tension face of the member and the centroid of the uncracked section.

The bonded reinforcement shall be uniformly distributed over the pre-compressed tension zone and as close as practicable to the extreme tension fibre. This bonded reinforcement shall be provided regardless of the serviceability limit state stress condition.

19.3.6.7.2 *Minimum bonded reinforcement in two-way flat slab systems with unbonded tendons*

In two-way flat slab systems containing unbonded prestressing tendons, the minimum amount and the distribution of bonded reinforcement, A_s, shall be as follows:

(a) Bonded reinforcement is not required in positive moment areas where the computed concrete tensile stress at the serviceability limit state, after all prestress losses, does not exceed $0.17\sqrt{f'_c}$;

(b) In positive moment areas, where the computed concrete tensile stress at serviceability limit state is greater than $0.17\sqrt{f'_c}$ the minimum area of bonded reinforcement, A_s, shall be:

$$A_s = \frac{N_c}{0.5 f_y} \quad \text{(Eq. 19–11)}$$

and f_y shall not exceed 500 MPa. The bonded reinforcement shall be uniformly distributed over the pre-compressed tension zone as close as practicable to the extreme tension fibre;

(c) In negative moment areas at column supports, the minimum area of bonded reinforcement, A_s, in each direction shall be:

$$A_s = 0.00075\,A_{cf} \quad \text{(Eq. 19–12)}$$

The bonded reinforcement shall be distributed within a slab width between lines that are 1.5h outside opposite column faces, and shall be spaced not greater than 300 mm. At least four bars or wires shall be provided in each direction.

19.3.6.7.3 *Lengths of bonded reinforcement*

Bonded reinforcement required by 19.3.6.7.1 and 19.3.6.7.2 shall have minimum lengths as follows:

(a) Negative moment areas: Sufficient to extend to one sixth of the clear span on each side of the support;

(b) Positive moment areas: One-third of clear span length, centred in the positive moment area;

(c) Where bonded reinforcement is required for flexural strength in accordance with 19.3.6.6 or for tensile stress conditions in accordance with 19.3.6.7.2(b) the anchorage details and development of this reinforcement shall also conform to the provisions of Section 8.

19.3.7 *Compression members – combined flexure and axial loads*

19.3.7.1 *General*

Prestressed concrete members subject to combined flexure and axial load, with or without non-prestressed reinforcement, shall be proportioned by the strength design methods of this Standard. Effects of prestress, creep, shrinkage, and temperature change shall be included.

19.3.7.2 *Axial load limit*

For prestressed columns the design axial load N^*, shall not be taken greater than $0.85\phi N_{n,max}$, where $N_{n,max}$ is the axial load strength at zero eccentricity. In calculating the value of $N_{n,max}$ the strain in the concrete shall not exceed 0.003.

19.3.7.3 *Limits for reinforcement in prestressed compression members*

19.3.7.3.1 *Minimum longitudinal reinforcement*

Members with average prestress f_{pc} less than 1.5 MPa shall have minimum reinforcement in accordance with 10.3.8 and 10.3.9 for columns, or 11.3.12 for walls.

19.3.7.3.2 *Minimum transverse reinforcement*

Columns with an average prestress f_{pc} equal to or greater than 1.5 MPa shall have all tendons enclosed by either spirals or lateral ties in accordance with (a) through (d):

(a) Where spirals are used they shall conform to 10.3.10.5;

(b) Where lateral ties are used they shall be at least 10 mm in diameter and they shall conform to 10.3.10.6;
(c) Ties shall be located longitudinally not more than half a tie spacing above top of footing or slab in any storey, and not more than half a tie spacing below the lowest horizontal reinforcement in members supported above;
(d) Where beams or brackets frame into all sides of a column, ties shall be terminated not more than the smaller of half a tie spacing or 75 mm below lowest reinforcement in such beams or brackets.

19.3.7.3.3 *Minimum transverse reinforcement in walls*

For walls with average prestress f_{pc} equal to or greater than 1.5 MPa, minimum reinforcement required by 11.3.12 need not be applied where structural analysis shows that adequate strength, ductility and stability can be achieved.

19.3.8 *Statically indeterminate structures*

19.3.8.1 *General*

Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at the serviceability limit state and for adequate strength at the ultimate limit state.

19.3.8.2 *Serviceability limit state*

Performance at the serviceability limit state shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces produced by prestressing, (including secondary prestressing moments) together with serviceability loading cases which shall include any significant self strain loading conditions.

19.3.8.3 *Ultimate limit state*

Design calculations for the ultimate limit state shall ensure the design flexural strength exceeds the design flexural actions. In determining the design actions for flexure, moments may be redistributed as specified in 19.3.9. However, in determining design shear forces, critical actions shall be determined both with and without redistribution of moments.

19.3.9 *Redistribution of design moments for ultimate limit state*

19.3.9.1 *General*

In design calculations for the ultimate limit state design flexural actions in indeterminate prestressed concrete structures, bending moments found from an elastic analysis may be redistributed to the extent indicated in 19.3.9.2, 19.3.9.3 and 19.3.9.4.

19.3.9.2 *Fundamental analysis for moment redistribution*

19.3.9.2.1 *Where moment redistribution permitted*

Bending moments at supports obtained in an elastic analysis may be reduced or increased where an analysis demonstrates that there is adequate ductility in the potential plastic regions to sustain the inelastic rotations associated with the redistribution.

19.3.9.2.2 *Determining rotational capacity*

In determining the rotational capacity of the potential plastic regions, account shall be taken of:
(a) The properties of the concrete, as defined in 5.2;
(b) The stress strain characteristics of prestressed and non-prestressed reinforcement including their strain capacities; as defined in 5.4;
(c) If the prestressed reinforcement is bonded or unbonded.

19.3.9.3 *Exclusion of secondary moments*

Secondary moments may be neglected in determining ultimate limit state design moments where all the reinforcement is bonded and the depth of the neutral axis at the critical section is equal to or less than 0.2 times the effective depth when acted on by the ultimate limit state moment ignoring secondary moments.

19.3.9.4 *Deemed to apply approach for prestressed concrete members*

Design moments obtained from an elastic analysis, which includes secondary moments may be redistributed in accordance with all the following provisions:

(a) The moment at any section in a member derived from an elastic analysis due to a particular combination of design loads may be reduced by up to 20 % of the numerically largest moment given anywhere by the moment envelope for that particular member, covering all appropriate combinations of design load;

(b) Where, as a result of redistribution, the design moment at a support is reduced, the neutral axis depth, c, shall be smaller than:

$$c < \left(0.5 - \frac{\Delta M}{M_{\max}}\right)d \quad \text{(Eq. 19–13)}$$

where

ΔM is the change in moment at the support between the value found from an elastic analysis, including the secondary prestress moment, and the redistributed value.

$M_{\max}$ is the numerically largest bending moment due to the applied loads anywhere in the particular span being considered covering all appropriate combinations of design loads.

(c) The prestressed reinforcement is bonded.

19.3.9.5 *Design moments*

(a) Where moments at the supports of a structure are changed by redistribution, as permitted in 19.3.9.2, 19.3.9.3 or 19.3.9.4, intermediate values shall be adjusted to maintain equilibrium of both vertical and horizontal forces.

(b) The design strength at any section shall not be less than 80 % of the maximum bending moment at that section (including secondary moments) found in an elastic analysis.

19.3.9.6 *Redistribution in members with unbonded prestressed reinforcement*

Where unbonded prestressed reinforcement is used, redistribution of moments shall not be used unless the requirements of 19.3.9.2 are satisfied and any non-prestressed reinforcement is Grade E.

19.3.10 *Slab systems*

19.3.10.1 *Design actions*

The design moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with the provisions of Section 6.

19.3.10.2 *Design strengths*

Design flexural strength of prestressed slabs at every section shall be equal to or greater than the required strength. Design shear strength of prestressed slabs at columns shall be equal to or greater than the required strength.

19.3.10.3 *Service load conditions*

At service load conditions, all serviceability limitations, including limits on deflections, shall be met, with appropriate consideration of the factors listed in 19.3.8.2.

19.3.10.4 *Tendon layout*

In slabs, which are designed for uniformly distributed loads, where prestressed tendons provide the primary flexural reinforcement, the spacing of the tendons required for flexural reinforcement shall be equal to or smaller than the smaller of eight times the slab thickness or 1.5 m. Tendons shall provide a minimum average prestress (after allowance for all prestress losses) of 0.9 MPa on the slab section tributary to the tendon or tendon group. A minimum of two tendons shall be provided in each direction through the critical shear section over columns. Special consideration of tendon spacing shall be provided for slabs with concentrated loads.

19.3.10.5 *Bonded reinforcement*

In slabs with unbonded tendons, bonded reinforcement shall be provided in accordance with 19.3.6.6.4 and 19.3.6.7.

19.3.10.6 *Lift slabs*

In lift slabs with bonded bottom reinforcement and shearheads or lifting collars where it is not practical to pass bottom reinforcement through columns, at least two bonded bottom bars or wires in each direction shall pass through the shearhead or lifting collar as close to the column as practicable and be continued or spliced. At exterior columns the reinforcement shall be anchored at the shearhead or lifting collar.

19.3.11 ***Shear strength***

The design of beams and slabs for shear at the ultimate limit state shall be in accordance with 7.5.

19.3.11.1 *Beams and one-way slabs*

The nominal shear stress, calculated from Equation 7–5 in 7.5.1, shall be equal to or smaller than the smaller of $0.2f'_c$ or 10 MPa, and in calculating the shear stress resisted by concrete f'_c shall not be taken greater than 50 MPa. A3

A2

The area used to calculate the shear stress, A_{cv}, shall:

(a) For rectangular T- and I- section shapes, be taken as the product of the web width, b_w and the effective depth, d;

(b) For octagonal, circular, elliptical and similar shaped sections, be taken as the area of concrete enclosed by the transverse reinforcement;

(c) For hollow-core sections, be taken as the effective depth for the reinforcement resisting the flexural tension force times the minimum web width.

Where composite construction is used to build up a section, the value of A_{cv} and other section properties for any particular increment of loading should be appropriate to the section that exists when the load is applied. Where appropriate, the influence of redistribution of actions within the section due to creep and or shrinkage of concrete on shear strength should be considered.

19.3.11.2 *Nominal shear strength provided by the concrete*

The nominal shear strength provided by concrete shall be assessed either from 19.3.11.2.1 or 19.3.11.2.2, as appropriate, or v_c shall be taken as zero if a strut and tie analysis is used to design the shear reinforcement.

19.3.11.2.1 *Simplified method for determining nominal shear strength of concrete in beams and one-way slabs*

This method of calculating the nominal shear strength of concrete may be used as an alternative to the method given in 19.3.11.2.2, where:

(a) The effective prestress force provides 40 % or more of the nominal flexural strength of the member; A2

(b) The member is not subjected to axial tension or self strain actions, such as differential temperature, which can induce significant tensile stresses over part of the member.

The shear strength provided by the concrete, V_c, is given by:

$$V_c = \left(\frac{\sqrt{f'_c}}{20} + 5\frac{V^* d_c}{M^*} \right) A_{cv} \qquad \text{(Eq. 19–14)}$$

A2

where

The quantity $(V^* d_c / M^*)$ shall not be taken greater than 1.0, where M^* and V^* are the design moment and shear force occurring simultaneously at the section considered, and d_c is the distance from extreme compression fibre to centroid of the prestressed reinforcement;

V_c need not be taken less than $0.14\sqrt{f'_c}\ A_{cv}$, and shall not be taken greater than $0.4\sqrt{f'_c}\ A_{cv}$, except where the critical section lies within the transfer length of a strand or a single wire. Where this occurs the value of V_c shall be based on the calculated prestress level at the critical section assuming the transfer lengths are 50 or 100 diameters for the strand and wire respectively, and the value of V_c shall not exceed the value given by 19.3.11.2.3. A2

19.3.11.2.2 *General method for determining Vc in beams and one-way slabs*

The nominal shear strength provided by the concrete, V_c, shall be the lesser of the shear force sustained at flexural shear cracking, V_{ci}, as given in (a) and the web-shear force sustained at web-shear cracking, V_{cw}, as given in (b).

(a) The value of V_{ci} is given by:

$$V_{ci} = V_b + \frac{V^* M_o}{M^*} \quad \text{(Eq. 19–15)}$$

and M_o is the bending moment corresponding to decompression of the extreme tension fibre under the action of the applied loading, which is given by:

$$M_o = \left(\frac{I}{y_t}\right)\left(f_{pe} + f_{ss}\right) \quad \text{(Eq. 19–16)}$$

where

A2 V_{ci} need not be taken less than $0.14\sqrt{f_c'}\ A_{cv}$

The value of V_b is equal to the value of V_c for a reinforced concrete beam of the same size and reinforcement content as given by 9.3.9.3.4

V^* and M^* are the critical combinations of design shear force and bending moment at the section being considered

f_{ss} is the self strain stress induced on the extreme tension fibre, taken as negative for tension.

(b) The value of V_{cw} is given by:

A2 $$V_{cw} = 0.3(\sqrt{f_c'} + f_{pc} + f_{sw})\ A_{cv} + V_p \quad \text{(Eq. 19–17)}$$

where f_{sw} is the self strain stress sustained at the neutral axis, and f_{pc} is the corresponding longitudinal prestress at the neutral axis, both taken as negative for tension.

Alternatively, V_{cw} may be taken as the shear force that is sustained when the principal tensile stress in the load case being considered, is equal to $0.33\sqrt{f_c'}$ at the centroidal axis of the member, or at the intersection of the flanges with the web when the centroidal axis is in the flange.

19.3.11.2.3 *Shear strength in transfer length*

In pretensioned members:

A2 (a) Where a section at a distance of $h/2$ from the face of support is closer to the end of the member than the transfer length of the prestressing reinforcement; or

(b) Where bonding of some of the tendons does not extend to the end of the member and the critical section for shear is within the transfer length of the strand or wire.

When computing the value of V_c, which is taken as the lesser of V_{ci} or V_{cw}, the magnitude of the prestress force at the critical section shall be calculated assuming that the force in the pretensioned reinforcement increases linearly over the development length. The development length shall be taken as 50 diameters for strands and 100 diameters for single wires.

At the critical distance of $h/2$ from the face of simple supports where negative moments cannot develop, the value of V_c shall be taken as the smaller of V_{ci} given by Equation 19–15, or the smaller value of V_{cw} given by either Equation 19–17 or $0.4\sqrt{f_c'}\ A_{cv}$.

Specific requirements for precast floor units are given in 19.3.11.2.4.

19.3.11.2.4 *Shear strength of pretensioned floor units near supports* A2

Floors containing precast units, in which negative moments and axial tension can be applied to the units through reinforcement in the concrete topping and/or reinforcement in filled cores, shall be designed to sustain the shear associated with both positive and negative moments acting at the supports.

(a) Pretensioned precast floor units which contain pretensioned strands close to both the upper and lower surfaces of the units, shall be designed to satisfy either the requirements of 19.3.11.2.3, with the value of V_{ci} equal to the larger of that given by 19.3.11.2.1 or 19.3.11.2.2, or the value given in (b) below;

(b) Precast pretensioned floor units which do not contain pretensioned strands close to the top surface of the unit, shall satisfy the requirements of 19.3.11.2.3 for actions associated with positive moments close to the support, and the requirements given below for the region of the floor where reinforcement in the concrete topping and/or close to the top surface of the precast units is subjected to tension due to negative moment and axial tension:

(i) For units supported by bearing on their lower face, the critical sections for shear in the negative moment region shall be taken as a distance, d, away from the face of the support and at the section at the end of filled cells

(ii) The design shear strength of the concrete shall be taken as $\phi v_c A_{cv}$, where v_c is given below;

(A) For hollow-core units with near circular voids and a depth equal to or less than 350 mm $v_c = 0.2\sqrt{f_c'} \le 1.41$ MPa A3

(B) For hollow-core units where the web width is uniform over a height of ¼ of the depth of the units or more, or the overall depth exceeds 350 mm, v_c is given by 9.3.9.3.4, but with $k_a v_b$ in Equation 9–5 replaced with $k_a v_b = (0.10 + 10 p_w)\sqrt{f_c'} \le 0.2\sqrt{f_c'}$ A2

where p_w is equal to the area of negative moment reinforcement divided by A_{cv}

(C) For other forms of pretensioned units or composite unit and *in situ* concrete topping, the value of v_c shall be found from 9.3.9.3.4

(D) For part (b), d is the effective depth for negative moments taken as the distance from the centroid of non-prestressed reinforcement near the top surface of the floor to the bottom surface of the precast unit.

f_c' is the concrete strength in the precast unit, except where cores are filled in hollow-core units, f_c' shall be taken as the strength of *in situ* concrete

p_w is the proportion of non-prestressed reinforcement near the top surface of the floor divided by A_{cv}.

19.3.11.2.5 *Shear strength in two-way prestressed concrete slabs*

The shear strength of two-way slabs shall be calculated as for 12.7.1, 12.7.2, 12.7.3 and 12.7.4, except the modifications given below may be made:

The value of v_c given in 12.7.3.2 for punching shear may be replaced by the value of v_c given in Equation 19–18.

At columns on slabs or footings where the requirements of 19.3.6.7 are satisfied, the shear stress resisted by the concrete when shear reinforcement is not required, given in 12.7.4.2, may be replaced by the value of v_c given in Equation 19–18.

$$v_c = k_d \beta_p \sqrt{f_c'} + 0.3 f_{pc} + \frac{V_p}{b_o d} \quad \text{(Eq. 19–18)}$$ A2

where

β_p is the smaller of 0.29 or $(\frac{\alpha_s d}{b_o} + 1.5)/12$

α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns

b_o is perimeter of critical section defined in 12.7.1(b)

f_{pc} is the average value of f_{pc} for the two directions; and

A2 k_d allows for the influence of depth on v_c, given by: $k_d = \sqrt{\dfrac{200}{d}}$ with limits of $1.0 \le k_d \le 0.5$

V_p is the vertical component of all effective prestress forces crossing the critical section.

v_c may be computed by Equation 19–18 if the following are satisfied; otherwise, 12.7.3.2 shall apply:

(a) No portion of the column cross section shall be closer to a discontinuous edge than four times the slab thickness;

(b) f'_c in Equation 19–18 shall not be taken as greater than 35 MPa; and

(c) f_{pc} in each direction shall be equal to or greater than 0.9 MPa, nor be taken greater than 3.5 MPa.

Where shear reinforcement is required the shear stress resisted by the concrete shall be equal to $0.17\sqrt{f'_c}$.

A2 **19.3.11.2.6** *Shear resisted by beam type action in two-way slab*

The shear strength of a prestressed footing slab under beam type action in the vicinity of concentrated loads or reactions shall be calculated for a critical section perpendicular to the plane of the slab, extending across the entire width and located at a distance of $h/2$ from the face of the concentrated load or reaction. For this condition, the slab or footing shall be designed in accordance with 7.5 and 19.3.11.1, 19.3.11.2 and 19.3.11.3.

19.3.11.3 *Nominal shear strength provided by shear reinforcement*

19.3.11.3.1 *Details of shear reinforcement in slabs*

Shear reinforcement consisting of bent up bars or stirrups, shall not be assumed to contribute to shear strength in one- or two-way slabs unless either:

(a) The effective depth of the slab is equal to or greater than the smaller of 150 mm or 16 times the diameter of the stirrup; or

(b) The stirrup is anchored mechanically on the compression surface of the slab.

19.3.11.3.2 *Critical section for shear in prestressed members*

For prestressed members, sections located at less than a distance $h/2$ from face of support shall be designed for the same shear, V^*, as that computed at a distance $h/2$ provided the conditions in 9.3.9.3.1 are satisfied. A2

19.3.11.3.3 *Shear strength provided by reinforcement*

Shear reinforcement in prestressed concrete members shall be designed in accordance with 7.5.5, 7.5.6, 7.5.7, 7.5.8 and 7.5.9 and in accordance with the appropriate clauses given in (a), (b), (c) or (d) below: .

(a) Beams and one-way slabs shall satisfy Equation 9–6 and 9.3.9.3 with the modifications noted in 19.3.11.3.4;

(b) For columns and piers, 10.3.10.4;

(c) For walls,11.3.11.3.8;

(d) Two-way slabs, 12.7.4.

19.3.11.3.4 *Modification of design of shear reinforcement in beams and one-way slabs due to prestress*

In the design of shear reinforcement in beams and one-way slabs where the effective prestress force is equal to or greater than 40 % of the tensile strength of flexural tension reinforcement, the following modifications shall be made to 9.3.9.4.12 and 9.3.9.4.15:

(a) When 9.3.9.4.12 is applied the maximum spacing limits for shear reinforcement in part (a) may be increased to the smaller of $0.75h$ or 600 mm.

(b) When 9.3.9.4.15 is applied and shear reinforcement is required by 9.3.9.4.13, the minimum area of shear reinforcement, A_v, shall be the smaller of that given by Equation 9–10 or by Equation 19–19 below.

$$A_v = \frac{A_{ps} f_{pu} s}{80 f_y d} \sqrt{\frac{d}{b_w}} \qquad \text{(Eq. 19–19)}$$ A2

where s is the spacing of transverse reinforcement in direction parallel to the longitudinal reinforcement, mm.

19.3.12 *Torsional strength*

Design for torsional resistance in prestressed members shall be based on 7.6 with the prestress force after losses replacing the axial load N^*. A3

19.3.13 *Anchorage zones for post-tensioned tendons*

19.3.13.1 *General*

19.3.13.1.1 *Definition of anchorage zone*

The anchorage zone is the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section. Its extent is generally equal to the largest dimension of the cross section. For anchorage devices located away from the end of the member, the anchorage zone includes the disturbed regions both ahead of and behind the anchorage device.

19.3.13.1.2 *Design of anchorage zones*

Anchorage zones shall be designed to sustain local compression stresses bearing against the anchor, and tension forces, which are associated with the transmission of the force in the cable into the member, as detailed in 19.3.13.4.

In the design the following effects shall be considered;

(a) The effect of abrupt changes in section in the anchorage zone;

(b) The three dimensional aspect to the flow of forces requires splitting and spalling forces to be sustained in two planes at right angles;

(c) The sequence of stressing of the cables.

19.3.13.2 *Design forces in prestress tendons*

Design of anchorage zones shall be based upon the factored prestressing force, P_{su}, taken as 1.2 times the maximum prestressing jacking force and a strength reduction factor ϕ of 0.85.

19.3.13.3 *Design material strengths*

19.3.13.3.1 *Tensile strength of bonded reinforcement*

Tensile strength of bonded reinforcement is limited to f_y for non-prestressed reinforcement and to f_{py} for prestressed reinforcement. Tensile stress of unbonded prestressed reinforcement for resisting tensile forces in the anchorage zone shall be limited to $f_{ps} = f_{se} + 70$.

19.3.13.3.2 *Bearing stress against anchors*

The bearing stress in the concrete against the prestressed anchors shall comply with 16.3, except that the A2 concrete strength at the time the tendons are stressed, f'_{ci}, shall be used in place of the 28 day design strength, f'_c. The required concrete strength at the time the cables are stressed shall be given on the drawings.

19.3.13.3.3 *Tensile strength of concrete*

In the design of reinforcement to carry bursting and spalling tension forces the tensile strength of concrete shall be neglected.

19.3.13.4 *Design methods*

19.3.13.4.1 *Permitted methods*

The following methods shall be permitted for the design of anchorage zones provided that the specific procedures used result in prediction of strength in substantial agreement with results of comprehensive tests:

(a) Equilibrium based plasticity models (strut-and-tie models);
(b) Linear stress analysis (including finite element analysis or equivalent); or
(c) Simplified methods where applicable.

19.3.13.4.2 *Simplified and linear elastic methods*

Simplified methods may only be used where the method specifically allows for the cross section shape and any change in this shape, which occurs within the anchorage zone. Two-dimensional linear elastic methods (finite element) may be used provided allowance is made for any changes in section dimensions within the anchorage zone. Where simplified or two-dimensional elastic methods are used, analyses shall be made of actions on two axes at right angles to determine reinforcement required to sustain the spalling and bursting forces in each direction.

19.3.13.4.3 *Reinforcement required for tension forces in anchorage zones*

Reinforcement shall be provided to:

(a) Resist bursting forces in anchorage zones;
(b) Control spalling cracks, where these are induced by compatibility;
(c) Resist spalling forces where these are required for equilibrium;
(d) Resist splitting forces in anchorage zones located away from the end of a member, as specified in 19.3.13.4.4.

19.3.13.4.4 *Anchorage devices away from end of members*

For anchorage devices located away from the end of the member, bonded reinforcement with a nominal strength equal or greater than $0.35P_{su}$ shall be provided to transfer the force into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage devices and shall be fully developed both behind and ahead of the anchorage devices.

19.3.13.4.5 *Minimum reinforcement for spalling*

Except where extensive testing or analysis indicates that spalling reinforcement is not required, a minimum reinforcement with a nominal tensile strength equal to 2 % of each factored prestressing force shall be provided in orthogonal directions parallel to the back face of all anchorage zones to control spalling cracks.

19.3.13.5 *Detailing requirements*

Selection of reinforcement sizes, spacings, cover, and other details for anchorage zones shall make allowances for tolerances on the bending, fabrication, and placement of reinforcement, for the size of aggregate, and for adequate placement and consolidation of the concrete.

19.3.14 *Curved tendons*

Where tendons are curved in either plan or elevation, the influence of the radial force that the cable applies to the concrete shall be considered. Where required by analysis reinforcement shall be provided to resist the tensile forces associated with local bending, shear and bursting in the concrete.

19.3.15 *Corrosion protection for unbonded tendons*

19.3.15.1 *General*

Unbonded prestressing steel shall be encased with sheathing. The prestressing steel shall be completely coated and the sheathing around the prestressing steel filled with suitable material to inhibit corrosion.

19.3.15.2 *Watertightness*

Sheathing shall be watertight and continuous over the entire length to be unbonded.

19.3.15.3 *Corrosive environments*

For applications in corrosive environments, the sheathing shall be connected to all stressing, intermediate and fixed anchorages in a watertight fashion.

19.3.16 *Post-tensioning ducts*

19.3.16.1 *General*

Ducts for grouted tendons shall be mortar-tight and non-reactive with concrete, prestressing steel, grout, and corrosion inhibitor.

19.3.16.2 *Single wire, strand or bar*

Ducts for grouted single wire, single strand, or single bar tendons shall have an inside diameter at least 5 mm larger than the prestressing steel diameter.

19.3.16.3 *Multiple wire, strand or bar*

Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area of at least two times the cross-sectional area of the prestressing steel.

19.3.17 *Post-tensioning anchorages and couplers*

19.3.17.1 *Strength of anchorages and couplers*

Anchorages and couplers for bonded and unbonded tendons shall develop at least 95 % of the specified breaking strength of the prestressing steel, when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100 % of the specified breaking strength of the prestressing steel shall be developed at the critical sections after the prestressing steel is bonded in the member.

19.3.17.2 *Location of couplers*

Couplers shall be placed in areas approved by the design engineer and enclosed in housing long enough to permit necessary movements. A2

19.3.17.3 *Fatigue of anchorages and couplers*

In unbonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in anchorages and couplers.

19.3.17.4 *Protection against corrosion*

Anchorages, couplers, and end fittings shall be permanently protected against corrosion.

19.3.18 *External post-tensioning*

19.3.18.1 *General*

Post-tensioning tendons may be external to any concrete section of a member. The strength and serviceability design methods of this code shall be used in evaluating the effects of external tendon forces on the concrete structure.

19.3.18.2 *Flexural strength*

External tendons shall be considered as unbonded tendons when computing flexural strength unless provisions are made to effectively bond the external tendons to the concrete section along its entire length.

19.3.18.3 *Attachment to member*

External tendons shall be attached to the concrete member in a manner that maintains the desired eccentricity between the tendons and the concrete centroid throughout the full range of anticipated member deflection.

19.3.18.4 *Protection against corrosion*

External tendons and tendon anchorage regions shall be protected against corrosion, and the details of the protection method shall be indicated on the drawings or in the project specifications.

19.4 Additional design requirements for earthquake actions

19.4.1 *General*

This clause covers the design of prestressed and partially prestressed concrete members of ductile moment resisting frames and joints between such members.

19.4.2 *Materials*

19.4.2.1 *Prestressing steel*

The strain in the prestressed reinforcement at ultimate limit state, allowing for the required curvature calculated using the effective plastic hinge length in 2.6.1.3.3, shall not exceed the minimum specified ultimate strain.

19.4.2.2 *Concrete*

The value of f'_c used in design shall not exceed 70 MPa.

19.4.2.3 *Grouting of tendons*

Post-tensioned tendons in moment resisting frame members shall be grouted, except as allowed by 19.4.5.2, or for hybrid structures designed in accordance with 19.4.6.

A2 ### 19.4.3 *Beams and floor slabs*

19.4.3.1 *Dimensions*

Dimensions of prestressed beams shall be in accordance with the provisions of 9.4.1.

19.4.3.2 *Redistribution of moments*

Provided the limits to flexural steel are in accordance with 19.4.3.3(b) or (c), bending moments derived from elastic analyses may be redistributed in accordance with the provisions of 19.3.9.4 and 19.3.9.5 and secondary moments may be neglected.

19.4.3.3 *Nominally ductile, limited ductile and ductile plastic regions*

Design of regions of various ductility classification shall be limited as follows:

(a) *Nominally ductile plastic regions*

Permissible curvature in nominally ductile plastic regions shall be calculated from the material strain limits given in 2.6.1.3.4(a) by replacing f_y by (f_{py}-f_{so}), where f_{so} is the stress after losses in the prestress tendon closest the extreme tension fibre when the stress in that fibre is zero. A2

(b) *Limited ductile plastic regions*

In limited ductile plastic regions the following criteria shall be satisfied:

(i) The depth of the neutral axis at ultimate shall not exceed 0.2h;

(ii) In rectangular beams, or in T- or L- beams where the compression zone is on the opposite side the flange, the area of compression reinforcement times its yield stress ($A'_s f_y$) shall be equal to or greater than 0.15 times the compression force;

(iii) Transverse reinforcement shall comply with 9.4.5 but with a spacing equal to or less than six times the diameter of the longitudinal bar being held against buckling.

(c) *Ductile plastic regions*
An analysis shall be made based on engineering principles to demonstrate that the curvature ductility is equivalent to that of a similar sized reinforced concrete beam with a ductile plastic region.

19.4.3.4 *Contribution of reinforcement in flanges to strength of beams*
The contribution of reinforcement in a flange of a beam to the design strength shall be determined as set out in 9.4.1.6.1.

In the determination of flexural overstrength the reinforcement in a flange of a beam shall be determined as set out in 9.4.1.6.2.

Where less than 70 % of the longitudinal reinforcement required for the design tensile strength passes through the column the joint zone shall be designed using a strut and tie model. A2

19.4.3.5 *Transverse reinforcement*
Stirrup ties shall be provided in potential plastic hinge regions in accordance with the provisions of 9.4.4.

In potential plastic hinge regions the shear strength provided by the concrete shall be assumed to be zero. Stirrup ties shall be equal to or greater than 10 mm diameter and the distance between vertical legs of stirrup ties across the section and along the beam shall not exceed 200 mm between centres in each set of stirrup ties.

19.4.3.6 *Floors with precast pretensioned units* A2
The precast floor units shall be designed by capacity design to sustain the over-strength actions associated with seismic actions (both horizontal and vertical), together with associated gravity loads, when the reinforcement connecting the units to the supporting structure sustains a stress of $\phi_{o,fy} f_y$ where $\phi_{o,fy}$ is defined in 2.6.5.5.

The supports to the precast units shall be detailed to prevent the transmission of positive moments into the precast unit, due to relative rotation between the support and unit which could lead to flexural failure within the development length of pretensioned or non-prestressed reinforcement.

19.4.4 *Design of columns and piles*

19.4.4.1 *Confinement and anti-buckling reinforcement*
Design of nominally ductile prestressed piles and columns, and prestressed piles and columns containing potential limited ductile plastic regions, shall satisfy the provisions of 10.3.10.5 or 10.3.10.6, as appropriate. Prestressed concrete piles and columns containing potential ductile plastic regions shall satisfy the provisions of 10.4.7.

19.4.4.2 *Minimum reinforcement content*
In a column containing a ductile or limited ductile plastic region the proportion of reinforcement, p, which includes both the prestressed reinforcement and non-prestressed reinforcement, shall be equal to or greater than $0.5\frac{\sqrt{f_c'}}{f_y}$, where f_y is the yield stress of reinforcement, but with prestressed reinforcement f_y shall not be taken greater than 500 MPa.

19.4.4.3 *Spacing of longitudinal reinforcement*
The spacing of longitudinal prestressed or non-prestressed reinforcement in potential plastic hinge regions shall:
(a) For nominally ductile columns and columns containing potential limited ductile plastic regions, satisfy the provisions of 10.3.8.2 and 10.3.8.3;
(b) For columns containing potential ductile plastic regions satisfy the provisions of 10.4.6.2 and 10.4.6.3.
Where longitudinal reinforcing bars are also utilised as vertical shear reinforcement in beam-column joint cores, the distribution of bars shall be in accordance with 15.4.5.3.

19.4.4.4 *Transverse reinforcement in potential plastic regions*

Special transverse reinforcement in accordance with 10.4.7 shall be provided in the regions of pretensioned piles in which ductility is required. The centre-to-centre spacing of spirals shall be equal to or less than 0.25 times the pile width or diameter, or six times the diameter of the longitudinal strand or 200 mm, whichever is least. Shear strength provided by the concrete shall be assumed to be zero.

19.4.4.5 *Shear strength*

Shear strength requirements shall be in accordance with 9.4 for beams and 10.4 for columns.

19.4.5 *Prestressed moment resisting frames*

19.4.5.1 *Beam tendons at beam-column joints*

Except as provided by 19.4.5.2, and for structures designed in accordance with 19.4.6, the beam prestressing tendons which pass through joint cores shall be spaced at the face of the columns so that at least one tendon is centred at not more than 150 mm from the beam top and at least one at not more than 150 mm from the beam bottom.

19.4.5.2 *Partially prestressed beams*

For partially prestressed beams in which the non-prestressed reinforcement provides at least 80 % of the design moment for earthquake plus gravity load combinations, prestress may be provided by one or more tendons passing through the joint core and located within the middle third of the beam depth, at the face of the column. In such cases post-tensioned tendons may be ungrouted, provided anchorages are detailed to ensure that neither anchorage failure or cable de-tensioning can occur under seismic actions.

19.4.5.3 *Ducts for grouted tendons*

Ducts for post-tensioned grouted tendons through beam-column joints shall be corrugated, or shall provide equivalent bond characteristics. Corrugated ducts are not required for ungrouted tendons complying with 19.4.5.2 or for structures designed in accordance with Appendix B of Part 1.

19.4.5.4 *Jointing material*

Precast members may be connected at beam-column joints provided that the jointing material has sufficient strength to withstand the compressive and transverse forces to which it may be subjected. The interfaces shall be roughened or keyed to ensure good shear transfer and the retention of the jointing material after cracking.

19.4.5.5 *Joint reinforcement*

Design of joint reinforcement shall be in accordance with the provisions of 15.4.

Post-tensioning anchors shall only be located in exterior beam-column joint cores if it can be demonstrated that the joint can resist both the anchorage tensile bursting stresses and the diagonal tension from beam and column forces.

19.4.6 *Design of hybrid jointed frames*

Hybrid jointed frames shall be designed in accordance with Appendix B of Part 1.

APPENDIX A – STRUT-AND-TIE MODELS (Normative)

A1 Notation

a_1, a_2, a_3	dimensions of nodal zones, mm
a	shear span, equal to the distance between a load and a support in a structure, mm
A_c	the effective cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, mm^2
A_n	area of a face of a nodal zone or a section through a nodal zone, mm^2
A_{ps}	area of prestressed reinforcement in a tie, mm^2
A_{si}	area of surface reinforcement in the ith layer crossing a strut, mm^2
A_{st}	area of non-prestressed reinforcement in a tie, mm^2
A'_s	area of compression reinforcement in a strut, mm^2
b	thickness of concrete member forming a strut, mm
d	distance from extreme compression fibre to centroid of longitudinal tension reinforcement, mm
f'_c	specified compressive strength of concrete, MPa
f_{cu}	effective compressive strength of concrete in a strut or a nodal zone, MPa
f_{py}	specified yield strength of prestressing steel, or the 0.2 % proof stress, MPa
f_s	design steel tensile stress less than the lower characteristic yield strength for non-prestressed reinforcement, MPa
f'_s	stress in compression reinforcement, MPa
f_{se}	effective stress after losses in prestressed reinforcement, MPa
f_y	specified yield strength of non-prestressed reinforcement, MPa
F_n	nominal strength of a strut, tie, or nodal zone, N
F_{nn}	nominal strength of a face of a nodal zone, N
F_{ns}	nominal strength of a strut, N
F_{nt}	nominal strength of a tie, N
F^*	factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model at ultimate limit state, N
s_i	spacing of reinforcement in the ith layer adjacent to the surface of the member, mm
w_s	effective width of strut, mm
α_1	factor defined in 7.4.2.7
β_s	factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut
β_n	factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone
γ_i	angle between the axis of a strut and the bars in the ith layer of reinforcement crossing that strut
γ_1, γ_2	angle between strut and reinforcement
Δf_p	increase in stress in prestressing tendons due to factored loads, MPa
λ	correction factor related to the unit weight of concrete, see 7.7.4
ϕ	strength reduction factor

A2 Definitions

The following definitions are additional to those given in Section 1.

B-REGION. A portion of a member in which the plane sections assumption of flexure theory from 7.4.2.2 can be applied.

DISCONTINUITY. An abrupt change in geometry or loading.

D-REGION. The portion of a member within a distance equal to the member height h or depth d from a force discontinuity or a geometric discontinuity.

DEEP BEAM. See 9.3.10.1.

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

NODAL ZONE. The volume of concrete around a node that is assumed to transfer strut-and-tie forces through the node.

STRUT. A compression member in a strut-and-tie model. A strut represents the resultant of a parallel or a fan-shaped compression field.

BOTTLE-SHAPED STRUT. A strut that is wider at mid-length than at its ends.

STRUT-AND-TIE MODEL. A truss model of a structural member, or of a D-region in such a member, made up of struts and ties connected at nodes, capable of transferring the factored loads to the supports or to adjacent B-regions.

TIE. A tension member in a strut-and-tie model.

A3 Scope and limitations

A3.1 *General*

Strut-and-tie models are useful when designing regions of reinforced concrete structures where the theory of flexure based on a linear strain distribution does not apply. Examples are deep beams, regions of discontinuity or where high concentrated forces are applied, brackets, corbels and diaphragms or walls with openings. It is a relatively simple technique in which the designer is required to establish an admissible path for internal forces in equilibrium with factored external loads and reactions. Struts, consisting primarily of concrete, are assigned compression forces and ties consisting of reinforcing bars, are the tension members. Struts and ties are joined at nodes and the strut and tie model represents an idealised truss. Simplified stress trajectories, to be simulated by struts and ties, are shown in Figure A.1 for regions of discontinuity.

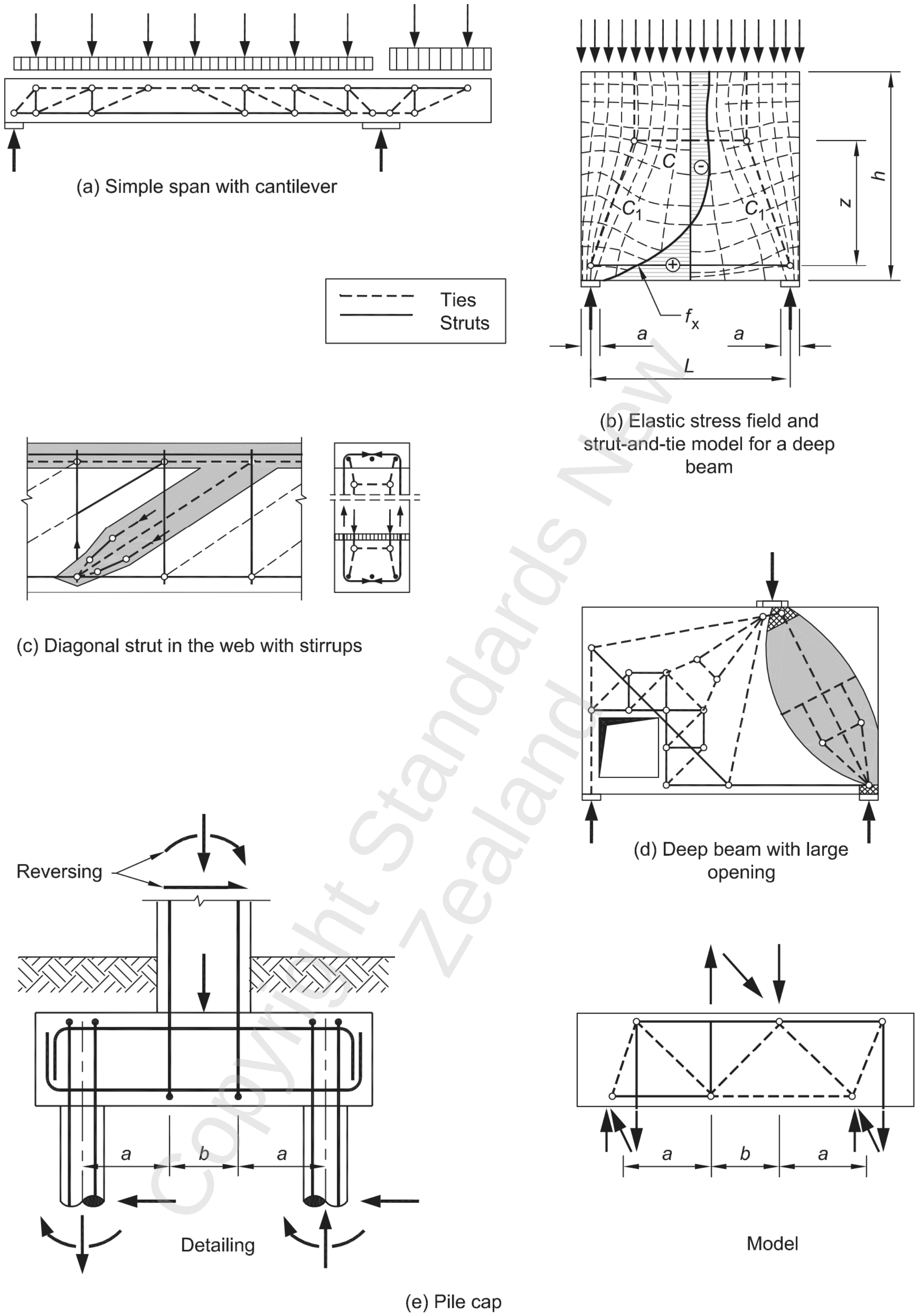


Figure A.1– Truss models with struts and ties simulating stress trajectories

A3.2 *Nodal zones*

Joints of strut-and-tie models are nodal zones where multi-directional stresses, satisfying equilibrium requirements for each node, need to be transferred.

A3.3 *Dimensions of nodal zones*

With rational approximations, the selected dimensions for nodal zones shall govern the relevant dimensions of adjacent struts and ties. Nodal dimensions shall be used to ascertain that the strength limitations of A7 are not exceeded.

A3

A3.4 *Serviceability limit state*

Recommended strengths for the ultimate limit state given in A5, and A7 are such that performance within the serviceability limit state may be considered to have been satisfied. However, control of possible secondary cracking should be considered in accordance with A5.3.

A4 Strut-and-tie model design procedure

A4.1 *Truss models*

Structural concrete members, or D-regions in such members, may be designed by modelling the member or region as an idealised truss. The truss model shall contain struts, ties, and nodes as defined in A2. The truss model shall be capable of transferring all factored loads to the supports or adjacent B-regions.

A4.2 *Equilibrium requirement*

The strut-and-tie model shall be in equilibrium with the factored applied loads and the reactions.

A4.3 *Geometry of truss*

In determining the geometry of the truss, the dimensions of the struts, ties, and nodal zones shall be taken into account.

A4.4 *Ties may cross struts*

Ties may cross struts. Struts shall cross or overlap only at nodes.

A4.5 *Minimum angle between strut and tie*

The angle between the axes of any strut and any tie entering a single node shall be equal to or greater than 25°. Where a single strut is used a larger angle shall be used.

A4.6 *Design basis*

Design of struts, ties, and nodal zones shall be based on:

$$\phi F_n \geq F^* \qquad \text{(Eq. A–1)}$$

where F^* is the force in a strut or tie, or the force acting on one face of a nodal zone, due to the factored loads; F_n is the nominal strength of the strut, tie, or nodal zone; and ϕ is the strength reduction factor specified in 2.3.2.2.

A5 Strength of struts

A5.1 *Strength of strut in compression*

The nominal compressive strength of a strut without longitudinal reinforcement shall be taken as the smaller value at the two ends of the strut of:

$$F_{ns} = f_{cu} A_c \qquad \text{(Eq. A–2)}$$

where

A_c is the cross-sectional area at one end of the strut, and

f_{cu} is the effective compressive strength of the concrete in the strut given in A5.2;

A5.2 Effective compressive strength of concrete strut

The effective compressive strength of the concrete in a strut shall be taken as:

$$f_{cu} = \beta_s \alpha_1 f'_c \qquad \text{(Eq. A–3)}$$

where

A3 α_1 is given by 7.4.2.7(c); and

β_s is given by:

(a) For a strut of uniform cross-sectional areas over its length β_s = 1.0;

(b) For struts located such that the width of the mid-section of the strut is larger than the width at the nodes (bottle-shaped struts):

 (i) With reinforcement satisfying A5.3 .... β_s = 0.75

 (ii) Without reinforcement satisfying A5.3 .... β_s = 0.60 λ

 where λ is

 1.0 for normal weight concrete,

 0.85 for sand lightweight concrete and

 0.75 for lightweight concrete.

 (iii) For struts in tension members, or the tension flanges of members .... β_s = 0.40

 (iv) For all other cases .... β_s = 0.60

A5.3 *Reinforcement for transverse tension*

If the value of β_s specified in A5.2(b)(i) is used, the axis of the strut shall be crossed by reinforcement proportioned to resist the transverse tensile force resulting from the compression force spreading in the strut. It may be assumed that the compressive force in the strut spreads at a slope of 2.5 longitudinal to one transverse to the axis of the strut.

A5.3.1 *Minimum reinforcement*

For f'_c not greater than 40 MPa, the requirement of A5.3 may be satisfied by the axis of the strut being crossed by layers of reinforcement that satisfy:

$$\sum \frac{A_{si}}{bs_i} f_y \sin\gamma_i \geq 1.5\,\text{MPa} \quad \text{(Eq. A–4)}$$

where A_{si} is the total area of reinforcement at spacing s_i in a layer of reinforcement with bars at an angle γ_i to the axis of the strut.

A5.3.2 *Placement of reinforcement*

The reinforcement required in A5.3 shall be placed in either:

(a) Two orthogonal directions at angles γ_1 and γ_2 to the axis of the strut; or

(b) In one direction at an angle γ_1 to the axis of the strut.

If the reinforcement is in only one direction, γ_1 shall be equal to, or greater than 40°.

A5.4 *Increased strength of strut due to confining reinforcement*

If documented by tests and analyses, an increased effective compressive strength of a strut due to confining reinforcement may be used.

A5.5 *Increased strength of strut due to compression reinforcement*

The use of compression reinforcement may increase the strength of a strut. Compression reinforcement shall be properly anchored, parallel to the axis of the strut, located within the strut, and enclosed in ties or spiral satisfying 10.4.7.4 and 10.4.7.5 as appropriate. In such cases, the strength of a longitudinally reinforced strut is:

$$F_{ns} = f_{cu}A_c + A'_s f'_s \quad \text{(Eq. A–5)}$$

A6 Strength of ties

A6.1 Nominal strength of tie

The nominal strength of a tie shall be taken as:

$$F_{nt} = A_{st}f_y + A_{ps}(f_{se} + \Delta f_p) \quad \text{(Eq. A–6)}$$

where (f_{se} + Δf_p) shall not exceed f_{py}, and A_{ps} is zero for non-prestressed members. Un-stressed pretension strand should have an overall limiting value of $\Delta f_p \leq 500$ MPa.

A6.2 Axis and width of tie

The centroid of the reinforcement in a tie shall coincide with the axis of the tie in the strut-and-tie model.

The assumed tie width shall account for the distribution of reinforcement, the available cover to the surface of the reinforcement and the dimensions of the nodes at the ends of the tie.

A6.3 Anchoring of tie reinforcement

A6.3.1 *By mechanical devices*

Tie reinforcement shall be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks, or straight bar development as required by A6.3.2 to A6.3.5.

A6.3.2 *Force developed in nodal zone*

Nodal zones shall develop the difference between the tie force on one side of the node and the tie force on the other side.

A6.3.3 ***Point of application of tie force for one tie***

At nodal zones anchoring one tie, the tie force shall be developed at the point where the centroid of the reinforcement in a tie leaves the nodal zone and enters the span.

A6.3.4 ***Point of application of tie force for two or more ties***

At nodal zones anchoring two or more ties, the tie force in each direction shall be developed at the point where the centroid of the reinforcement in the tie leaves the nodal zone.

A6.3.5 ***Anchoring transverse reinforcement***

The transverse reinforcement required by A5.3 shall be anchored in accordance with 8.6.3.

A6.4 *Tie force where bar development limited*

Where the size of a nodal zone is not large enough to allow the yield strength of the reinforcement to be developed, the amount of reinforcement provided should be based on reduced design tensile stresses, f_s, which can be developed in the given nodal zone.

A7 Strength of nodal zones

A7.1 *Nominal compression strength*

The nominal compression strength of a nodal zone shall be:

$$F_{nn} = f_{cu} A_n \qquad \text{(Eq. A–7)}$$

where f_{cu} is the effective compressive strength of the concrete in the nodal zone as given in A7.2 and A_n is (a) or (b):

(a) The area of the face of the nodal zone that F^* acts on, taken perpendicular to the line of action of F^*; or

(b) The area of a section through the nodal zone, taken perpendicular to the line of action of the resultant force on the section.

A7.2 *Compressive stress on face of nodal zone*

Unless confining reinforcement is provided within the nodal zone and its effect is supported by test and analysis, the calculated effective compressive stress on a face of a nodal zone due to the strut-and-tie forces shall not exceed the value given by:

$$f_{cu} = \alpha_1 \beta_n f'_c \qquad \text{(Eq. A–8)}$$

where $\alpha 1$ is given by 7.4.2.1 (c) and the value of β_n is given in (a) to (c) below as appropriate.

(a) In nodal zones bounded by struts of bearing areas, or both β_n = 1.0; or

(b) In nodal zones anchoring one tie β_n = 0.80; or

(c) In nodal zones anchoring two or more ties β_n = 0.60.

A7.3 *Nodal zones for three-dimensional strut-and-tie models*

In a three-dimensional strut-and-tie model, the area of each face of a nodal zone shall be equal to or greater than that given in A7.1, and the shape of each face of the nodal zones shall be similar to the shape of the projection of the end of the struts onto the corresponding faces of the nodal zones.

A8 Considerations of seismic actions

A8.1 *General*

The strut and tie method may be used to design an element, or part of an element, for energy dissipation under seismic conditions provided.

(a) Capacity design is applied to ensure the energy dissipation is confined to the selected ductile struts and/or ties;

(b) Struts, which may also be subjected to tension on other phases of the earthquake, are confined by longitudinal reinforcement and ties as required in 10.4.7.

(c) Any tie reinforcement in a yielding element is anchored within the extended nodal zone to sustain the overstrength capacity of the reinforcement.

(d) All other ties are proportioned to sustain the maximum action that may be induced when overstrength actions are sustained in the selected struts or ties.

(e) At nodes, see A8.3(e) below.

(f) Where the strut and tie system is such that under reversed loading yielding in an element occurs only in tension, the element shall be designed as a nominally ductile element.

A8.2 *Diaphragms modelled by strut and tie*

Strut-and-tie models are appropriate for the design and detailing of diaphragms that may have irregularly arranged penetrations. In satisfying the requirements of Section 13, attention needs to be paid to the mobilising of several internal load paths, each set being dependent on the direction of the reversible seismic forces. During seismic response it is preferable to inhibit yielding in diaphragms.

A8.3 *Openings in walls modelled by strut and tie*

Structural walls with irregular openings may be designed by the use of strut and tie models for ductile or limited ductile behaviour as set out below:

(a) An energy dissipating mechanism is selected and capacity design is used to ensure that yielding is confined to the selected struts or ties;

(b) Where seismic force reversals require ties in previous loading stages to act or struts, the reinforcement must be adequately restrained against buckling and the concrete adequately confined to sustain the required compressive strength;

(c) By means of a capacity design approach, the yielding of ties in which force reversal cannot occur, as in the case of stirrups in beams or columns, should be inhibited;

(d) Each nodal zone must be examined for the possible reversal of nodal forces. Concrete design compression stresses in elastic regions of such walls should be limited to those given in A5 based on the maximum possible number of ties that could develop at the node under consideration;

(e) At nodes, where as a result of ductility demands, the reinforcement in a tie member could have yielded, concrete design compression stresses should be limited to 50 % of the values given in A5. For example the particularly severe situation which arises when the nodal zone anchors tie in two or more directions, the limitation should be $f_{cu} = 0.5 \times 0.85 \times 0.60 = 0.26$. Figure A.2 illustrates a situation where these severe limitations on concrete compression strength are warranted.

Reinforcement which can be subjected to reversed cyclic inelastic strains should be provided with lateral support, using appropriate transverse reinforcement to prevent premature bar buckling.

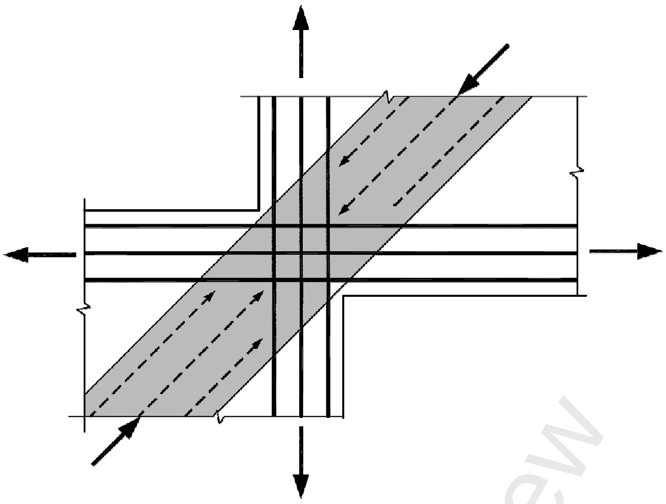


Figure A.2 – Typical nodal zone

APPENDIX B – SPECIAL PROVISIONS FOR THE SEISMIC DESIGN OF DUCTILE JOINTED PRECAST CONCRETE STRUCTURAL SYSTEMS (Normative)

B1 Notation

c	distance from the neutral axis to the extreme compression fibre, mm
d_{bl}	diameter of non-prestressed steel reinforcing bar, mm
E_{pt}	modulus of elasticity of prestressing tendons, MPa
$f_{pt,design}$	design upper limit for stress in prestressing tendons, MPa
$f_{pt,initial}$	initial stress in prestressing tendons, (after losses) MPa
f_{pty}	yield strength of prestressing tendons, MPa
f_y	yield strength of non-prestressed steel reinforcement, MPa
k_b	axial stiffness of one beam, N/mm
k_c	bending stiffness of one column at level of beam-column connection, N/mm
k_{pt}	axial stiffness of post-tensioned tendons, N/mm
L_{cant}	distance from the column face to the point of contraflexure of the beam, mm
L_p	plastic hinge length of equivalent monolithic beam including strain penetration, mm
ℓ_{sp}	strain penetration of non-prestressed steel reinforcing bar, mm
ℓ_{ub}	unbonded length of prestressing tendons, mm
ℓ'_{ub}	unbonded length of non-prestressed reinforcing bar, mm
M	total moment capacity, N mm
M_N	flexural strength contribution due to axial load, N mm
M_{pt}	flexural strength contribution due to post-tensioned tensions, N mm
M_s	flexural strength contribution due to non-prestressed steel reinforcement, or energy dissipating devices, N mm
n	total number of joint openings at beam-column interfaces along beam
S_p	structural performance factor
α_o	overstrength factor for non-prestressed steel reinforcement or energy dissipating device
Δ_{pt}	additional elongation at level of unbonded prestressing tendons due to gap opening, θ mm
Δ_s	elongation at level of non-prestressed steel reinforcement, mm
Δ_{sp}	elongation due to strain penetration of non-prestressed steel reinforcement, mm
ε_c	compressive strain in the concrete at the extreme fibre
$\varepsilon_{p,i}$	initial strain in unbonded post-tensioned prestressing
$\varepsilon_{pt}(\theta)$	additional strain in unbonded post-tensioned tendons due to gap opening θ
$\varepsilon_{pt,tot}$	total strain in unbonded post-tensioned tendons due to gap opening θ
$\varepsilon_s(\theta)$	strain in non-prestressed steel reinforcement due to gap opening θ
ε_u	ultimate strain of non-prestressed steel reinforcement
ε_y	yield strain of non-prestressed steel reinforcement
θ	rotation of gap opening
λ	moment contribution ratio between self-centering and energy dissipation
μ	structural ductility factor
ξ	viscous damping coefficient, %
ξ_{hybrid}	equivalent viscous damping of an hybrid connection/system
ξ_{lower}, ξ_{upper}	lower and upper bound values for equivalent viscous damping
ϕ_u	ultimate curvature of beam
ϕ_y	yield curvature of beam
Ω	factor indicating restraint effects to beam elongation

B2 Definitions

B2.1 *Jointed systems*

Jointed systems are structural systems in which the connections between the precast concrete elements are weaker than the elements themselves. Jointed systems do not emulate cast-in-place concrete construction. The connections of jointed systems can be of limited ductility or ductile.

B2.2 *Hybrid systems*

Hybrid systems are jointed structural systems in which the self-centering capability is provided by post-tensioning and/or axial compressive load, and energy dissipation is provided by yielding non-prestressed steel reinforcement or other special devices. Hybrid systems are ductile.

B2.3 *Equivalent monolithic systems*

Equivalent monolithic systems are structural systems in which the connections between the precast concrete elements are designed to emulate the performance of cast-in-place concrete construction. The connections can be of limited ductility or ductile.

B3 Scope and limitations

This Appendix applies to ductile jointed and hybrid precast concrete structural systems. The systems may be moment resisting frames, structural walls or dual systems, in which the precast concrete elements are joined together by post-tensioning techniques with or without the presence of non-prestressed steel reinforcement or other energy dissipating devices.

B4 General design approach

B4.1 *General*

Either a force-based or a displacement-based design approach shall be used for the seismic design of jointed and hybrid structural systems. Modifications to the inter-storey drift limits used in design shall be made in accordance with B4.2.

B4.2 *Drift limits*

Inter-storey drift limits as defined in NZS 1170.5 shall be adopted for jointed structures, except that drift limits corresponding to a damage control, or the serviceability limit state may be increased by up to 50 %, provided analytical calculations and/or experimental validation demonstrates a reduced level of damage, (both structural and non-structural), when compared to an equivalent monolithic structure. No increase in drift limit corresponding to the ultimate limit state shall be allowed where high inelastic demand and P-delta effects can govern the response.

B4.3 *Self-centering and energy dissipation capabilities of hybrid structures*

B4.3.1 *Combination of self-centering and energy dissipation*

An adequate combination of self-centering and energy dissipation contributions of a hybrid structural system shall be provided as specified in B4.3.2 and B4.3.3 in order to stay within maximum drift limits as well as avoiding residual deformations.

B4.3.2 *Condition for full self-centering*

The full self-centering of a general jointed connection shall be achieved by selecting, in the design phase, an appropriate moment contribution ratio λ as follows:

$$\lambda = \frac{M_{pt} + M_N}{M_s} \geq \alpha_o \qquad \text{(Eq. B–1)}$$

where M_{pt}, M_N, and M_s are the flexural strength contributions of the post-tensioned tendons, the axial load where present, and the non-prestressed steel reinforcement or energy dissipating devices calculated with respect to the centroid of the concrete compression resultant of the section, respectively, and α_o (≥ 1.15) is the overstrength factor for the non-prestressed steel reinforcement or the energy dissipating devices.

B4.3.3 *Evaluation of energy dissipating capacity*

The energy dissipation capacity provided by the flag-shape hysteresis rule typical of hybrid systems, shall be evaluated in terms of the equivalent viscous damping percentage, ξ, by interpolation between a pure dissipative system (that is equivalent monolithic system with elasto-plastic or near elasto-plastic behaviour) and a pure self-centering system (that is an unbonded post-tensioned system with a non-linear elastic behaviour).

B4.3.4 *Structural performance factor, S_p*

Values of the S_p factor, used to evaluate the input seismic loads according to a force-based design approach, shall be consistent with the values adopted for ductile structures from 2.6.2.2.

B5 Behaviour of connections

B5.1 *Inelastic behaviour of connections*

In jointed structural systems the inelastic demand shall be concentrated within the critical connections between the precast concrete elements as a result of the opening and closing of a crack at the interface between elements.

B5.2 *Behaviour of unbonded post-tensioned tendons*

The unbonded post-tensioned tendons in the precast concrete elements which cross the interfaces between elements shall be designed to remain in the elastic range during the design earthquake.

B5.3 *Hybrid systems*

In hybrid systems in addition to unbonded post-tensioned tendons there shall be present at the critical connections non-prestressed steel reinforcement or other means of energy dissipation.

B5.4 *Shear transfer at critical connections*

B5.4.1 *Means of shear transfer at connections*

Vertical shear forces shall be transferred at the critical connections by shear keys, concrete corbels, metallic corbels or other means for which there is experimental or analytical evidence of satisfactory performance.

B5.4.2 *Shear transfer by friction induced by tendons*

Friction induced by post-tensioned tendons shall not be used in design to transfer shear at critical connections due to gravity loads. Transfer of shear force due to the seismic loads may rely on friction at the interface induced by the tendons provided the tendons remain in the elastic range at the design level of inter-storey drift of the structure.

B5.4.3 *Minimum shear capacity of connections*

The minimum shear capacity provided in accordance with B5.4.1 shall be in any case at least equal to the design shear force due to the factored gravity loads.

B5.4.4 *Torsion transfer at critical connections*

Torsion forces in the beam, i.e. due to the weight of a floor system orthogonal to the primary frame, shall be transferred at the critical connections by means of appropriate shear keys, concrete or metallic corbels/brackets, dowel action of the non-prestressed reinforcement or other means for which there is experimental or analytical evidence of satisfactory performance.

B6 Design of moment resisting frames

B6.1 *General*

Design of beams, columns and beam-column joints shall satisfy the requirements of 19.4.3, 19.4.4 and 19.4.5, respectively. In particular, capacity design principles apply to achieve a desired beam sway, weak-beam strong-column inelastic mechanism.

B6.2 *Anchorage, location and longitudinal profile of the post-tensioned tendons*

B6.2.1 *Post-tensioning materials*

Strands or bars may be used for post-tensioned tendons provided that there is compliance with the limits on strain and stress included in this Appendix.

B6.2.2 *Loading cycles for anchorages*

Anchorages shall withstand, without failure, a minimum of 50 cycles of a loading for which the load in each cycle is varied between 40 % and 80 % of the minimum specified strength of the prestressing tendons.

B6.2.3 *Profile of post-tensioned tendons*

The profile of the post-tensioned tendons may be either straight or draped to follow the bending moment diagram.

B6.2.4 *Length of unbonded post-tensioned tendons*

No limit on the length of the unbonded post-tensioned tendons is required. The unbonded length shall be clearly defined and controlled as per design requirements.

B6.2.5 *Location of tendons at joint*

The location of the tendons in beams in the critical section where the gap may open and close can vary according to the design requirements. A concentric location in the beam is however preferred for seismic resisting systems in a high seismic region for an easier control of the additional elongation under lateral loading.

B6.3 *Prestressing force in beams*

B6.3.1 *Lower and upper bound for initial prestress*

The initial prestress shall be limited by a lower bound, in order to guarantee sufficient moment contribution M_{pt} to provide full self-centering capacity in accordance with B4.3.2 as well as by an upper bound in order for the tendons to remain in the elastic range for a target inter-storey drift level, while still providing self-centering properties.

B6.3.2 *Upper bound for initial prestress*

The condition for the upper bound limit for the initial prestress in force shall be expressed as:

$$f_{pt,initial} \leq 0.9\, f_{pty} - E_{pt}\varepsilon_{pt} \qquad \text{(Eq. B–2)}$$

where

$f_{pt,initial}$ is the initial prestress, after losses
E_{pt} is the modulus of elasticity of the tendons
ε_{pt} is the additional strain in the tendons due to the lateral drift/displacement, calculated as per B6.4.6; and
f_{pty} is the yield strength of prestressed tendons

B6.4 *Evaluation of flexural strength at target inter-storey drift levels*

B6.4.1 *Evaluation of nominal flexural strength*

The evaluation of the nominal flexural strength at a target inter-storey drift value shall be according to Sections 7, 9 and 10 with account taken of the special conditions of equilibrium and member compatibility in accordance with B6.4.2 to B6.4.11.

B6.4.2 *Strain compatibility not applicable*

Due to the presence of unbonded tendons compatibility between strain in the tendons and strain in the concrete does not apply at any given section.

B6.4.3 *Member compatibility and equilibrium*

Member compatibility and section equilibrium conditions shall be satisfied.

B6.4.4 *Evaluation of strain in non-prestressed steel reinforcement*

The evaluation of the strain in the non-prestressed steel reinforcement or additional energy dissipation devices shall rely on section compatibility considerations only if fully bonded conditions are present. If partial debonding exists then B6.4.7 or a similar approach properly validated by experimental tests shall be followed.

B6.4.5 *Evaluation of additional elongation of the unbonded tendons or bonded non-prestressed steel reinforcement*

The gap opening (rotation θ) mechanism shall be used to evaluate the additional elongation of the tendons, Δ_{pt}, and of the non-prestressed steel reinforcement or energy dissipation devices, Δ_s, which are assumed to be directly proportional to the distance from the neutral axis.

B6.4.6 *Strain in unbonded post-tensioned tendons*

The strain level in the unbonded post-tensioned tendons $\varepsilon_{pt}(\theta)$ due to the gap opening, θ, shall be calculated as:

$$\varepsilon_{pt}(\theta) = \frac{n\Delta_{pt}}{\ell_{ub}} \quad \text{(Eq. B–3)}$$

where n is the total number of joint openings at beam-column interfaces along the beam involving the tendons, and ℓ_{ub} is the unbonded length in the tendons.

The total strain in the unbonded post-tensioned tendons $\varepsilon_{pt,tot}$ at a given gap rotation θ shall be given by the sum of the initial strain due to the prestress and the additional strain due to the gap opening given in B6.4.6. That is:

$$\varepsilon_{pt,tot} = \varepsilon_{p,i} + \varepsilon_{pt}\,(\theta) \quad \text{(Eq. B–4)}$$

B6.4.7 *Strain in unbonded non-prestressed longitudinal steel reinforcement*

When energy dissipation is assigned to longitudinal non-prestressed reinforcement, a defined debonded length, ℓ'_{ub}, can be deliberately adopted in the beam adjacent to the interface in order to avoid premature fracture of the reinforcement. In these conditions, section compatibility does not apply and the strain in the steel $\varepsilon_s\,(\theta)$ due to the gap opening θ shall be evaluated as:

$$\varepsilon_s(\theta) = \frac{(\Delta_s - 2\Delta_{sp})}{\ell'_{ub}} \quad \text{(Eq. B–5)}$$

where Δ_{sp} is the contribution to the gap opening due to the strain penetration of the non-prestressed steel reinforcement assumed to occur at both ends of the small unbonded region, ℓ'_{ub}.

After simplifications an approximate formula that may be used is:

$$\varepsilon_s = \frac{(\Delta_s + 2/3\ell_{sp}\varepsilon_y)}{(\ell'_{ub} + 2\ell_{sp})} \quad \text{(Eq. B–6)}$$

where

ℓ_{sp} is the strain penetration taken as 0.022 $f_y d_{bl}$

f_y is the yield strength of reinforcement

d_{bl} is the diameter of the reinforcing bar

B6.4.8 *Maximum strain in non-prestressed steel reinforcement*

At design level inter-storey drift, the strain in the non-prestressed steel reinforcement or in the dissipation devices should not exceed 90 % of the ultimate deformation capacity; that is $\varepsilon_s\,(\theta) \le 0.9\ \varepsilon_u$.

B6.4.9 *Neutral axis position*

Evaluation of the neutral axis depth c as well as of the concrete compression strain ε_c corresponding to a given level of inter-storey drift or gap opening rotation may be obtained by an iterative procedure assuming member compatibility conditions as per B6.4.10.

B6.4.10 *Concrete compression strain*

The compressive strain in the concrete at the extreme fibre, ε_c, may be evaluated using the following expression, which satisfies member compatibility conditions:

$$\varepsilon_c = \left[\frac{(\theta L_{cant})}{\left(L_{cant} - \frac{L_p}{2} \right) L_p} + \phi_y \right] c \quad \text{(Eq. B–7)}$$

where

c is the neutral axis depth

L_p is the plastic hinge length of an equivalent monolithic connection (including strain penetration component)

L_{cant} is the distance between the column interface and the point of contraflexure (length of the beam cantilever), and

ϕ_y is the yield curvature of the section in an equivalent monolithic connection

B6.4.11 *Evaluation of neutral axis position*

For use with B6.4.10, simplified design charts or tables with the position of the neutral axis at different limit states may be adopted, if based on analytical procedures validated through experimental results.

B6.5 *Cyclic moment behaviour and energy dissipation*

B6.5.1 *Hysteresis behaviour*

The cyclic moment-rotation behaviour of a generic hybrid connection shall be described by a flag-shape hysteretic rule given by the combination of a non-linear elastic rule with an energy dissipating rule (elasto-plastic; Ramberg-Osgood, or other stiffness degrading rule), representing the moment contributions of the post-tensioned tendons, M_{pt}, and of the mild steel or energy dissipation devices, M_s.

B6.5.2 *Flag-shaped hysteresis rule*

The properties of the flag-shape hysteresis rule depend on the ratio between the moment contributions, which shall be evaluated about the concrete compression force resultant.

B6.5.3 *Equivalent viscous damping*

The equivalent viscous damping of an hybrid connection/system, ξ_{hybrid}, depends on the ratio of the moment contributions (M_{pt}/M_s) and may be directly evaluated from the resultant flag-shape hysteresis rule or as interpolation between lower and upper bound values given by an unbonded connection ($\xi_{lower} = 5$ %) and a monolithic frame system ξ_{upper} (Equation B–8).

$$\xi = 5 + 30\left(1 - \frac{1}{\sqrt{\mu}}\right)\% \quad \text{(Eq. B–8)}$$

where μ is the structural ductility factor.

B6.5.4 *Contact damping*

In addition to the hysteretic damping evaluated according to B6.5.3, contact (radiation) damping can also be taken into account, provided experimental evidence of the dynamic rocking behaviour of the connection/system is available.

B6.6 *Design of column-to-foundation connection*

B6.6.1 *Performance*

The column-to-foundation connection shall be designed according to the target performance, in order to provide satisfactory self-centering properties and energy dissipation capabilities to the whole frame system.

B6.6.2 *Design approach similar to beam-column connection design*

A design approach similar to that derived for the beam-column connections in B6 shall be followed with modifications based on case-by-case considerations.

B6.6.3 *Contribution of column axial load to self centering*

The self-centering contribution of the axial load in the column, due to the unfactored gravity axial load shall be taken into account in the definition of the global hysteresis behaviour and self-centering capacity.

B6.6.4 *Prevention of sliding*

Sliding at the column-to-foundation connection shall be prevented by using appropriate shear keys or by relying on dowel action of the non-prestressed steel reinforcement passing through the critical section and on the shear-friction capacity provided by the vertical axial force due to unfactored gravity loads.

B7 Design of structural wall systems

B7.1 *General*

The design of jointed structural wall systems shall follow the general approach indicated in the previous sections with modifications as in B7.2 to B7.7.

B7.2 *Energy dissipation devices*

Energy dissipation devices shall be internal or external, placed either at the base section or between coupled panels and relying on the relative vertical movement during the rocking motion of the wall.

B7.3 *Axial loads*

The contribution of the axial load in terms of strength and stiffness shall be taken into account using unfactored gravity loads.

B7.4 *Ratio between self-centering and energy dissipation contributions*

The ratio between the self-centering and energy dissipation contributions shall account for axial force due to gravity load as in B7.3.

B7.5 *Effects of seismic actions on axial force*

The effects of seismic actions on the axial force shall be considered whenever they lead to less desirable behaviour due to a reduction of energy dissipation or an increase in P-delta effects in each direction of seismic action.

B7.6 *Overstrength of energy dissipation device*

Overstrength of the energy dissipation device shall be accounted for in order to ensure a full self-centering capability.

B7.7 *Displacement incompatibility due to rocking*

Displacement incompatibility issues due to the rocking motion of the wall (uplifting) and involving the connection details with the diaphragm, shall be addressed as in B8.

B8 System displacement compatibility issues

B8.1 *Gravity load carrying systems*

Structural wall or frame systems, which are primarily assigned to carry gravity loads, shall be able to accommodate the global system displacements without reduction of their vertical load-bearing capacity. If yielding of secondary elements is expected due to the deformation induced by the primary systems, allowance should be made in the amount of post-tensioning to overcome the inelastic behaviour and guarantee full re-centering capacity.

B8.2 *Non-structural elements*

The expected damage to non-structural elements shall be evaluated from the displacements of the structural system.

B8.3 *Diaphragms*

The effects on diaphragm action due to the interaction between floor systems and the lateral load resisting systems (B8.5) shall be taken into account. Both the horizontal and vertical relative displacement incompatibility shall be estimated and properly accommodated.

B8.4 *Beam elongation*

B8.4.1 *Effects*

When frame systems are adopted for lateral load resisting systems, the effects of beam elongation (increase of distance between column centrelines) shall be taken into account in terms of: damage to and interaction with the floor system, increase of column curvature and of flexural and shear demand, increase of beam moment capacity due to the beam axial force, and increase of residual local deformations.

B8.4.2 *Seating of precast floor units*

Seating details for precast concrete floors shall take into account the expected beam elongation effects.

B8.4.3 *Post-tensioning force*

The effects of beam-elongation on the increase or decrease of the strain in the post-tensioning reinforcement shall be evaluated.

B8.4.4 *Estimation of post-tensioning strain due to beam elongation*

A simplified estimation of the strain increase in the post-tensioning steel, ε_{pt}, due to beam-elongation effects within a frame system can be obtained multiplying Equation B–4 by a factor $\left(1-\frac{1}{\Omega}\right)$ as follows:

$$\varepsilon_{pt} = \varepsilon_{in} + \frac{n\Delta_{pt}}{\ell_{ub}}\left(1-\frac{1}{\Omega}\right) \quad \text{(Eq. B–9)}$$

where

Ω is an indicator of the restraint effects given in a two bays, three columns, one storey sub-assembly, by:

$$\Omega = \frac{k_b}{k_c + 2k_{pt}} + 1 \quad \text{(Eq. B–10)}$$

where

k_b is the axial stiffness of one beam

k_c is the bending stiffness of one column at the level of the beam-to-column connection, and

k_{pt} is the axial stiffness of the post-tensioned tendons spanning the entire subassembly

B8.5 *Floor-to-lateral-load resisting system incompatibility*

B8.5.1 *Relative vertical displacements incompatibility*

The expected relative vertical displacements and connection forces between lateral resisting systems and diaphragms shall be evaluated.

B8.5.2 *Connection details*

Special connection details able to accommodate the relative displacements and sustain the increased forces as estimated in B8.5.1 shall be adopted.

B8.5.3 *Location of connections*

In order to minimize additional relative displacements and forces, the connections shall be placed in regions of relatively limited vertical displacement incompatibility and dimensioned to accommodate the expected deformations without affecting their capacity to transfer the inertia forces to the lateral load resisting systems.

B8.5.4 *Design strength for the collectors*

The design of the floor-to-lateral resisting systems collectors shall account for the expected overstrength coming from a possible diaphragm inelastic response as well as for the increased floor acceleration values due to high mode effects.

B8.5.5 *Inelastic behaviour and energy dissipation of collectors*

The floor-to-lateral resisting systems connectors may be assigned an inelastic behaviour with energy dissipation, if adequate evidences of satisfactory global performance from analytical studies on the overall system (floor-lateral resisting system interaction) are provided.

NOTES

(There is no Appendix C so as to avoid confusion with commentary clauses.)

NOTES

APPENDIX D – METHODS FOR THE EVALUATION OF ACTIONS IN DUCTILE AND LIMITED DUCTILE MULTI-STOREY FRAMES AND WALLS (Normative)

D1 Notation

D1.1 Standard symbols

A_g gross area of column section, mm^2
$E_{o,i}$ lateral force at level i when overstrength actions are sustained, N
f'_c specified compressive strength of concrete, MPa
h_c column depth, a section dimension, mm
I_e effective moment of inertia of a section, mm^4
k relative flexural stiffness, mm^3
L length of member between centrelines of supports, mm
L_n clear length of column between beam faces, mm
M_n nominal flexural strength, N mm
N^*_o capacity design axial load on a column, N
N_{oe} axial load in a column due to shear induced in a beam by end moments when overstrength moments act in the beam, N
R_m moment reduction factor for column under low axial compression or in axial tension
R_v axial load reduction factor
T_1 the computed period of the structure in its first mode of translational vibration, s
V^*_{col} capacity design column shear force, N
V_E shear force in a column found from an equivalent static of first mode analysis, N
V_{oe} shear force at the face of a column induced by end moments in a beam when overstrength moments act in the beam, N
β modification factor for dynamic magnification factor
ϕ strength reduction factor
ϕ_o overstrength factor for a joint zone or the base of a column
$\phi_{ol,i}$ overstrength factor for lateral force at a level in a frame
$\phi_{o,fy}$ overstrength factor depending on reinforcement grade
ϕ'_o average overstrength factors for beam-column joint zones located above and below the column being considered
ω dynamic magnification factor for bending moments
ω_{max} maximum value of ω which acts in mid-height region of a multi-storey frame
ΣM_{ob} beam input overstrength moment into a beam and column joint zone at intersection of centre-lines at intersection of centre-lines when overstrength moments act in primary plastic regions, N mm
ΣM_n sum of bending moments in beams sustained at the intersection of the beam and column centrelines when nominal moments act in the beams at the column faces, N mm
$M_{oc,bottom}$ Overstrength moment in a column at the bottom of the first storey, N mm
$M_{oc,top}$ Overstrength moment in a column at the top of the first storey, N mm

D2 General

This Appendix specifies methods of determining design actions for structural members and parts of members, which contain primary plastic hinge regions for situations where capacity design is required by 2.6.5 for:

(a) Ductile and limited ductile moment resisting frame structures;
(b) Ductile and limited ductile walls;
(c) Ductile and limited ductile dual wall frame structures.

As the design methods in this appendix are based on capacity design a strength reduction factor of 1 shall be used (see 2.6.5 and 2.3.2.2) for all calculations of member capacities .

D3 Columns multi-storey ductile frames

D3.1 *General*

Where not exempted 2.6.7.2 columns by in multi-storey ductile frames, shall be designed by either Method A or Method B as identified in the following clauses. Both methods give the structure a high level of protection against the formation of a storey column sway mechanism and consequently are suitable for capacity design.

Method A gives a high level of protection against the formation of plastic regions in columns between 3 h_c above the first storey and 3 h_c below the top storey.

Method B gives a lower level of protection against the formation of localised plastic regions. However, it still gives a high level of protection against the formation of a storey column sway mechanism.

Outline of Methods

In both methods:

(a) A suitable ductile failure mechanism shall be identified and the locations of associated primary plastic regions (hinges) shall be defined. Acceptable ductile failure mechanisms are identified in 2.6.7.2.

(b) The magnitudes of the bending moments and shear forces acting in the beams at the faces of the columns shall be found when overstrength moments act in the primary plastic regions.

(c) The magnitude of the total moment applied to a beam-column joint at the intersection point of the beam and column centrelines shall be calculated. This moment is referred to as the beam input overstrength moment.

(d) The reinforcement in the columns in any storey need not exceed the amount required for a column designed as nominally ductile, with the strength reduction factor (ϕ) equal to 0.85 for actions derived from an analysis based on the assumption that the structure is nominally ductile and the S_p factor is 0.9. The requirements of 2.6.6.1 (b) shall apply to the structure.

(e) Different detailing provisions apply to Method A and Method B, and shall be applied as appropriate.

In Method A within the zone between 3 h_c above the first storey and 3 h_c below the top storey:

(i) The position of lap splices in longitudinal reinforcement is not limited, see 10.4.6.8.2 (a) but outside of this zone this relaxation shall not be applied.

(ii) The quantity of confinement reinforcement may be reduced as specified in 10.4.7.4.3 and 10.4.7.5.3. Outside this zone this reduction shall not be applied.

In Method B the relaxation in the location of lap splices (in 10.4.7.4.3 (a)) and the quantity of confinement (10.4.7.4.3) shall not be applied.

The two methods do not apply to:

(a) Frames of two storeys or less in which column sidesway mechanisms are intended to form the primary energy dissipating mechanism as defined in 2.6.7.2;

(b) Frames in which column displacements are predominantly controlled by structural walls, as in wall frame (dual) structures;

(c) Columns that act as props, which are not required or intended to contribute to lateral force resistance.

A2 **Method A** is intended for use in the design of regular or near regular ductile moment resisting frames, in which most columns exhibit a point of inflection in each storey, except those near the base of the building, when subjected to first mode or equivalent static analysis actions.

Method B is a general approach, which may be used on a wider range of ductile moment resisting frame structures than Method A. There are no restrictions in terms of regularity. With this Method the designer has the freedom to locate potential plastic regions in some of the columns.

D3.2 Design moments and shears in columns by Method A

D3.2.1 *General*

Three steps are required to determine design moments in the columns at each beam-column intersection as set out below:

(a) The beam input overstrength moment at each beam-column joint shall be distributed into the columns above and below the intersection of the beam and column centre-lines as set out in D3.2.2.

(b) The column moments from step (a) shall be multiplied by the dynamic magnification factor, ω, and the modification factor, β, as set out in D3.2.3.

(c) The critical capacity design moments for the columns at the beam faces shall be calculated as set out in (i) and (ii) below;

   (i) The bending moments found from (a) and (b) above, which are the intersection point of the beam and column centrelines, shall be projected to the column at the face of the beam as detailed in D3.2.4.

   (ii) Where the axial load associated with capacity design conditions in a column is small, or it is subjected to tension in the critical load case, the design moment may be reduced as set out in D3.2.5.

D3.2.2 *Distribution of beam input moment into columns*

Divide the frame being designed into two regions on the basis of an equivalent static or first mode analysis. Region 1 is located between the mid height of the top storey in the frame and the level in which the lowest point of inflection occurs in each column. Region 2 is located between the line connecting the lowest points of inflection in the columns and the mid height of the storey containing the primary plastic hinge (generally at the base of the columns). In both regions the actions acting on each beam-column joint zone are considered individually. A2

Region 1

In this region, the moments from the equivalent static or first mode analysis in each column immediately above and below the centroid of the joint zone being considered are multiplied by ϕ_o, such that the sum of the two moments is equal to the sum of the over-strength moments which may be sustained by the beam at the centroid of the beam-column joint zone.

Region 2

In this region, the moments sustained in the column immediately above and below the centre of the joint zone being considered are multiplied by the factor $\phi_{o,b}$, which is given by:

$$\phi_{o,b} = M_{o,base} / M_{E,base}$$

Where $M_{o,base}$ is the overstrength moment at the critical section of the primary plastic hinge in the column being considered and $M_{E,base}$ is the corresponding moment induced by the equivalent static or first mode actions.

Both regions

The required design strengths (M^*) of the critical section in the potential primary plastic hinge in the column (generally at the base of the column), and in the section immediately below the uppermost level, are found from the critical combination of ultimate strength actions including either wind or earthquake actions as specified in 2.3.2 and 2.6.7.5(d). The overstrength in the column primary plastic hinge is determined from the section details used to satisfy the design strength requirements, with the material strengths and moment amplification specified in 2.6.5. The overstrength moments at the critical sections in the beams are found as defined in 2.6.5 and 9.4.1.6.2.

D3.2.3 *Dynamic magnification and modification factors*

The moments in the columns at the level of the beam centre-line at each joint zone shall be multiplied by an appropriate dynamic magnification factor, ω, which is defined in (i) below, and an appropriate modification factor β which is defined in (ii) below. Limits on the product of $\omega\beta$ are given in (iii) while (iv) defines the appropriate values of dynamic magnification factor for columns, which are part of two or more frames.

(a) The maximum value of the dynamic magnification factor, ω_{max} is given by:

$\omega_{max} = 0.6T_1 + 0.85$ ....(Eq. D–1)

but not less than 1.3 or more than 1.8.

The value of dynamic magnification factor, ω, varies over the height of the building. At the base of the columns and at the top of the upper storey the value of ω shall be taken as 1.0. Between 30 % of the height of the frame above the base and the third highest level in the frame, ω shall be taken as ω_{max}. The value of ω at the second level is the larger of 1.3 or that obtained by linear interpolation between the value at the base of the column, and ω_{max} at 30 % of the height of the building. For the second to top level the value is the larger of 1.3 or that obtained by linear interpolation between ω_{max} at the third highest level and 1.0 at the highest level.

(b) The capacity design moments in the columns at a beam-column joint zone, found above, may be reduced by the modification factor, β, which is given by Equation D-2. The maximum value of β is 1.0 and this value shall be used at the base of the columns and in the top storey of the building. Else where the value of β is given by;

A2

$$\beta = \left(1.4 - \frac{\sum M_o'}{2.5\phi_{o.fy}\sum M_n'}\right) \quad \text{(Eq. D–2)}$$

Where $\phi_{o,fy}$ is defined in 2.6.5.6;

A2

and $\sum M_o'$ and $\sum M_n'$ are the sums of the beam overstrength and nominal strength moments respectively, acting at the column faces of the beam-column joint being considered.

ΣM_n is the corresponding sum of the moments when the beams are sustaining their nominal strength moments.

(c) The following limits apply to the product of dynamic magnification factor and modification factor, $\omega\beta$. All levels except the base of the columns and the top storey of the building the product of the dynamic magnification factor and the modification factor, $\omega\beta$, shall be equal to or greater than 1.3.

In the top storey and at the base of the columns the minimum value of $\omega\beta$ shall be equal to or greater than 1.2.

(d) Where a column is part of more than one frame bi-axial actions are induced in the column and the capacity design actions shall be found by considering the actions arising from all the beams framing into it at the level being considered. The dynamic magnification and modification factors for the moments in the column for the first frame shall be as defined in (a) and (b) above. The corresponding dynamic magnification and modification factors ($\omega\beta$) for the simultaneous actions from the second or subsequent frames shall be taken as 1.0. Where the enclosed angle between two frames is less than 45° the dynamic magnification for the two frames shall be given the same dynamic magnification and modification factors.

D3.2.4 *Critical design moments in columns*

The critical design moments for a column are at the level of the top and bottom faces of the beams. These critical values shall be found by calculating to moment at a beam face from the value found at the level of the beam centre-line together with the assumption that the shear force is equal to 60 % of the capacity design shear force in the column, which is defined in D3.2.6.

D3.2.5 *Reduction in design moments for cases of small axial compression*

Where a column is subjected to small axial compression or net axial tension, in the critical load case, some reduction in the column design moments may be made. The minimum reduced column design moment at the face of the column may be found by multiplying the design moment found in D3.2.4 by the reduction factor, R_m given in Table D.1. The axial load level required in this table is defined in D3.4.

Table D.1 – Moment reduction factor R_m

A2

	Tension							Compression			
$\frac{N_o^*}{f_c' A_g}$	≤ –0.150	–0.125	–0.100	–0.075	–0.050	–0.025	0.00	0.025	0.050	0.075	≥ 0.10
ω											
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.1	0.85	0.86	0.88	0.89	0.91	0.92	0.94	0.95	0.97	0.98	1.00
1.2	0.72	0.75	0.78	0.81	0.83	0.86	0.89	0.92	0.94	0.97	1.00
1.3	0.62	0.65	0.69	0.73	0.77	0.81	0.85	0.88	0.92	0.96	1.00
1.4	0.52	0.57	0.62	0.67	0.71	0.76	0.81	0.86	0.90	0.95	1.00
1.5	0.44	0.50	0.56	0.61	0.67	0.72	0.76	0.83	0.89	0.94	1.00
1.6	0.37	0.44	0.50	0.56	0.62	0.69	0.75	0.81	0.88	0.94	1.00
1.7	0.31	0.38	0.45	0.52	0.59	0.66	0.73	0.79	0.86	0.93	1.00
1.8	0.30	0.33	0.41	0.48	0.56	0.63	0.70	0.78	0.85	0.93	1.00

NOTE – ω is the local value of the dynamic magnification factor applicable to the design of the column section at that level.

D3.2.6 *Design shears in columns*

The design shear force in a column for seismic actions along an axis, V^*_{col}, shall be taken as the appropriate value given in (a) or (b) below, but in no case shall it be less than 1.6 times the shear force induced by the seismic design forces.

(a) In first storey columns the capacity design shear forces shall be equal to or greater than:

$$V^*_{col} = 1.15\,(M_{oc,bottom} + M_{oc,top}) / L_n \quad \text{(Eq. D-3)}$$

where $M_{oc,bottom}$ and $M_{oc,top}$ are the overstrength bending moments at the bottom and top of the column in the first storey and L_n is the clear height of the column in the storey. In calculating $M_{oc,bottom}$ allowance shall be made for the increase in strength arising from confinement of the plastic hinge region by any foundation beam or pad as required in 2.6.5.5(b).

(b) In columns above the first storey and excluding the top storey, V^*_{col}, shall be given by:

$$V^*_{col} = 1.3\,\phi_o'\; V_E \quad \text{(Eq. D–4)}$$

Where V_E is the shear in the column being considered found from an equivalent static or first mode analysis for seismic actions and ϕ_o' is the average overstrength factor for the beam-column intersections for each end of the individual column in the storey being considered.

(c) In the top storey, where the column is expected to form a plastic region before the beam, Equation D-3 shall be used to find V^*_{col}. Where this condition is not met Equation D–4 shall be used

(d) In columns, which intersect with beams on two or more axes, the simultaneous action of the shear forces applied by the beams on each axis shall be considered in the design for shear in the column.

D3.2.7 *Design of columns*

The columns shall be designed to sustain simultaneously the critical combinations of capacity design axial forces as set out in D3.4, design bending moments, as set out in D3.2.4 and D3.2.5 and design shear forces as set out in D3.2.6.

D3.3 Design moments and shears in columns by Method B

D3.3.1 *General*

This method of assessing the capacity design actions in the columns involves the following steps for each level of the frame:

(a) Establish the location of the assumed points of inflection in the columns for Method B, as set out in D3.3.2

(b) Determine the beam input overstrength moment into each beam-column on each joint in the level being considered, as set out in D3.1;

(c) Scale the lateral force acting at a level from an equivalent static or a first mode analysis so that it is consistent with overstrength actions being sustained in the beams in the level being considered. Distribute this lateral force to each of the beam-column joints in the level, as set out in D3.3.3 and D3.3.8;

(d) Select the points of inflection in the storeys above and below the level being considered. From these determine the resultant shears in each column due to the beam input overstrength moment and the lateral forces acting on each of the beam-column joint zones, as set out in D3.3.4;

(e) Multiply the column shears found in (d) by appropriate dynamic magnification and modification factors as set out in D3.3.5;

A3 (f) Find the design moments at the critical sections of the columns, as set out in D3.3.6;

(g) Determine the critical design axial load acting in each storey of the column as set out in D3.4

(h) Proportion the column to sustain the critical combinations of moment, shear and axial load, as set out in D3.5.

D3.3.2 ***Location of points of inflection in columns***

In **Method B** the locations of the points of inflection in the columns are assumed.

(a) Where an equivalent static or first mode analysis indicates that a point of inflection occurs in a storey, the points of inflection assumed for Method B, may be located at any convenient location within the mid half of the storey. Generally the most convenient location is at the mid-height of the storey.

(b) Where an equivalent static or first mode analysis indicates that the points of infection are not within the storey height, calculations shall demonstrate that the columns have sufficient ductility to accommodate the inelastic deformation associated with the chosen location of points of the inflection. In this case it may be necessary to assume some other location for the point of inflection than at the mid-height of the storey.

D3.3.3 ***Lateral seismic forces at a level***

(a) A lateral force corresponding to overstrength actions, $E_{o,i,}$ acting at the level being considered in a frame, shall be found by scaling values from an equivalent static or first mode analysis by the factor, $\phi_{ol,i}$. The numerical value of $\phi_{ol,i}$ is equal to the ratio of the sum of the beam input overstrength moments in the level to the corresponding sum of bending moments from the equivalent static or first mode analysis.

Where the chosen ductile failure mechanism includes a primary plastic hinge forming in a column the sum of the beam input moments at a joint, ΣM_{ob}, may be taken as the sum of the moments at the intersection of the beam and column centre-lines applied to the joint zone by the columns, when overstrength moments act in primary plastic hinge regions.

(b) At all levels except the top, the lateral force is distributed to the individual beam-column joint zones in the level being considered. There is considerable freedom as to how this lateral force, $E_{o,i,}$ is distributed to each joint zone, though the distribution must be such that the requirements of D3.3.8 are satisfied.

(c) The shear force induced in the columns above and below the joint being considered, due to the portion of the lateral force $E_{o,i}$ applied to that joint, is calculated from statics assuming;

(i) The component of lateral force acts as a point load on the column at the height of the beam centre-line;

(ii) The column is supported by shear forces acting at the level of the selected points of inflection in the storeys above and below the level being considered;

(iii) No bending moments are transferred from the columns to the beams

D3.3.4 ***Column shear due to beam overstrength moments***

Each beam-column joint at each level shall be considered in turn to determine the shear forces induced in the columns above and below the level being considered due to the beam input overstrength moment. These shear forces shall be calculated from statics assuming:

(a) The beam input overstrength moment acts at the intersection point of the beam, and column centre-lines;

(b) No lateral force acts on the joint zone;

(c) The columns are supported by lateral shear forces acting at the assumed points of inflection in the storeys above and below the level being considered.

D3.3.5 ***Resultant column shears***

The resultant shear force in each column shall be taken as the sum of the shear forces due to the component of the overstrength lateral force acting the joint zone being considered, as set out in D3.3.3, and the corresponding shear force due to the input beam overstrength moment, as set out in D3.3.4. The sum of these two values shall be multiplied by the appropriate dynamic magnification and modification factors ($\omega\beta$) as set out in (a), (b) and (c) below.

(a) The dynamic magnification factor, ω, shall be taken as not less than;

 (i) 1.3 for all storeys except the top two;

 (ii) 1.15 for the second to highest storey;

 (iii) 1.0 for the highest storey.

(b) The modification factor β except at the base or in the top storey is given by:

$$\beta = \left(1.25 - \frac{\sum M_o'}{4.0\phi_{o.fy}\sum M_n'}\right) \quad \text{.......(Eq. D–5)}$$ A2

The maximum value of β is 1.0 and this value, shall be used at the base of the columns and in the top storey of the building.

and $\sum M_o'$ and $\sum M_n'$ are the sums of the beam overstrength and nominal strength moments respectively, acting at the column faces of all the beam-column joints of the frame in the level being considered. A2

$\phi_{o,fy}$ is defined in 2.6.5.6;

$\sum M_n'$ is the sum of the bending moments in the beams at the column faces in the level being considered when nominal moments act at the critical sections of all the potential plastic regions. A2

(c) The following limits apply to the $\beta\omega$ values:

In all storeys except the top two $\beta\omega \geq 1.2$

In the second to top storey $\beta\omega \geq 1.1$

In the top storey $\beta\omega \geq 1.0$.

(d) Where a column is part of more than one frame bi-axial actions are induced in the column and the capacity design actions shall be found by considering the actions arising from all the beams framing into it at the level being considered. The dynamic magnification and modification factors for the shear force in the column from the first frame shall be as defined in (a) and (b) above. The corresponding dynamic magnification and modification factors ($\omega\beta$) for the simultaneous actions from the second or subsequent frames shall be taken as 1.0. Where the enclosed angle between two frames is less than 45° the same dynamic magnification and modification factors shall be used for the two frames.

D3.3.6 ***Capacity design column moments***

The capacity design moments in the columns at the face of the beams shall be taken as the product of the shear in the column, found in step, D3.3.5 times the distance between the point of inflection and the face of the beam. The calculated shear in a column is in general found twice, as in the calculations with Method B the shears are found for the columns above and below each level. The critical shear in any storey shall be taken as the maximum of these two values.

D3.3.7 ***Design shear strength for columns***

For all columns, except those in the first storey, the capacity design shear strength shall be equal to or greater than a shear force equal to 1.15 times value found by dividing the sum of the nominal column moment strengths in the critical sections at the top and bottom of the storey by the distance between these critical sections.

In the first storey columns the minimum capacity design shear strength shall be equal to or greater than by:

$$V^*_{col} = 1.15\,(M_{oc,bottom} + M_{oc,top}) / L_n \quad \text{(Eq. D–6)}$$

where $M_{oc,bottom}$ and $M_{oc,top}$ are the overstrength bending moments at the top and bottom of the column in the first storey and L_n is the clear height of the column in the storey. In calculating $M_{oc,bottom}$ allowance shall be made for the increase in strength arising from confinement of the plastic hinge region by any foundation beam or pad as required in 2.6.5.5 (b).

D3.3.8 *Limit on distribution of column shear forces*

The distribution of the lateral force, $E_{o,i}$, to the joint zones in step D3.3.3 (ii) shall be such that the primary plastic hinge regions maintain their chosen locations when the structure is pushed laterally into the inelastic range.

D3.4 Capacity design axial forces for Methods A and B

The design axial forces in the columns shall be based on the assumption that the structure sustains dead load and long-term live load and that overstrength actions are sustained in all the primary plastic regions in the structure.

The component of axial force in a column, N_{oe}, which is due to the shear induced in the beams from the end moments (ΣV_{oe}) when overstrength moments act, may be reduced such that:

$$N_{oe} = R_v\, \Sigma V_{oe} \quad \text{(Eq. D–7)}$$

where R_v is a coefficient given by the expression:

$$R_v = 1.0 - 0.015\, n \geq 0.70 \quad \text{(Eq. D–8)}$$

n is the number of storeys above the level being considered,
ΣV_{oe} is the sum of the component of the shears in the beams due to the end moments, which are sustained when overstrength actions act in the beams.

D3.5 Design of columns

In all cases all sections of a column shall be designed to satisfy the minimum requirements of both the ultimate limit state and of capacity design actions (as set out in this Appendix). Where a column is incorporated in more than one frame it shall be designed to sustain the simultaneous actions transferred to it by the beams in all the frames connected to the column.

D4 Ductile and limited ductile walls

D4.1 General

A ductile failure mechanism, which is consistent with 2.6.5.2 and 2.6.8.1, shall be identified.

D4.2 Design moment envelope

The envelope for bending moments obtained from an equivalent static or modal analysis shall be;

(a) Multiplied by the ratio of the flexural overstrength moment sustained in the primary plastic region to the corresponding design action at this section
(b) Modified to allow for higher mode effects.

A recommended envelope for uniform walls with a near uniform distribution of seismic mass over the height of the wall is given in the commentary.

Longitudinal reinforcement in the wall shall, except at the top of the wall, be extended for a distance of the wall length (ℓ_w) plus a development length for the bar beyond the envelope.

D4.3 Design shear force envelope

The design shear force envelope for a structural wall shall be obtained from an equivalent static analysis by:

(a) Multiplying the shear force at each level by the ratio of the flexural overstrength to the design flexural action due to seismic forces at the critical section of the primary plastic region;

(b) Modifying the shear force envelope to allow for higher mode effects.

A suitable shear force envelope is given in the Commentary for the case of a uniform wall with a near uniform distribution of seismic weights at each level.

D4.4 Walls which are not uniform

Where walls are not uniform a rational method of design shall be used based on the concepts in this APPENDIX and 2.6.5.

D5 Wall-frame structures – Ductile and limited ductile

D5.1 General

Buildings in which the lateral resistance is provided by both walls and frames (dual structures) may be designed as a dual system as set out in Chapter 6 of Reference D.3 provided that:

The structure when analysed by the equivalent static method or by the first mode response in a modal analysis, each wall does not have more than one point of inflection above the point of maximum moment in the wall, and at some point within the lower third of the building the walls in the structure, the sum of the shear forces in all walls exceeds 30 % of the storey shear force.

In such structures the walls can be used to prevent the formation of column sway and mixed column beam sway modes from developing. This gives more freedom to the location of plastic regions in the beams and columns of the frame and it reduces the magnitude of dynamic amplification factors for columns.

Structures falling outside these criteria shall be the subject of rational design based on the concepts in contained in this appendix and 2.6.5.

NOTES

A3

APPENDIX E – SHRINKAGE AND CREEP
(Normative)

E1 Notation

A_g gross area, mm^2
E_c mean value of the modulus of elasticity of the concrete at 28 days
f'_c compressive strength of concrete, MPa
k_1 shrinkage strain coefficient
k_2 a factor dependent on α_2, the time after loading, and the member thickness
k_3 a factor dependent on the age of the concrete at the time of loading
k_4 relative humidity factor
k_5 a modification factor for high strength concrete
k_6 a modification factor for aggregate type
t_h the hypothetical thickness of a member, mm
u_e the exposed perimeter of a member cross section plus half the perimeter of any enclosed voids contained therein, mm
α_1 a factor associated with shrinkage related to the member thickness, defined as $0.8 + 1.2e^{-0.005t_h}$
α_2 a factor associated with creep related to the member thickness, defined as $1.0 + 1.12e^{-0.008\,t_h}$
ε_{cs} the design shrinkage strain of concrete
ε_{csd} the drying shrinkage strain
$\varepsilon_{csd.b}$ the basic drying shrinkage strain
ε^*_{cse} the chemical (autogenous) shrinkage strain
σ_o constant sustained stress
τ age of the concrete
φ_{cc} design creep coefficient
$\varphi_{cc.b}$ the basic creep coefficient of concrete

E2 Shrinkage

E2.1 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, in accordance with AS 1012.13, modified for long-term value as outlined in CE2.1 (in Part 2); or

(c) By calculation in accordance with E2.2.

E2.2 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}), as follows:

$$\varepsilon_{cs} = \varepsilon_{cse} + \varepsilon_{csd} \qquad \text{(Eq. E–1)}$$

The autogenous shrinkage strain shall be taken as:

$$\varepsilon_{cse} = \varepsilon^*_{cse} \times (1.0 - e^{-0.1t}) \qquad \text{(Eq. E–2)}$$

where t is the time (in days) after setting and ε^*_{cse} is the final autogenous shrinkage strain given by:

$$\varepsilon^*_{cse} = (0.06f'_c - 1.0) \times 50 \times 10^{-6} \qquad \text{(Eq. E–3)}$$

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

$$\varepsilon_{csd} = k_1 k_4 \varepsilon_{csd.b} \qquad \text{(Eq. E–4)}$$

A3 and k_1 is obtained from Figure E.1 and k_4 is obtained from Table E.1. Refer to Table CE.1 (in Part 2) for average relative humidity values for various locations around New Zealand. For internal environments a relative humidity of RH = 50 % may be adopted.

In the derivation of k_1, the hypothetical thickness of the member, t_h, shall be calculated as follows:

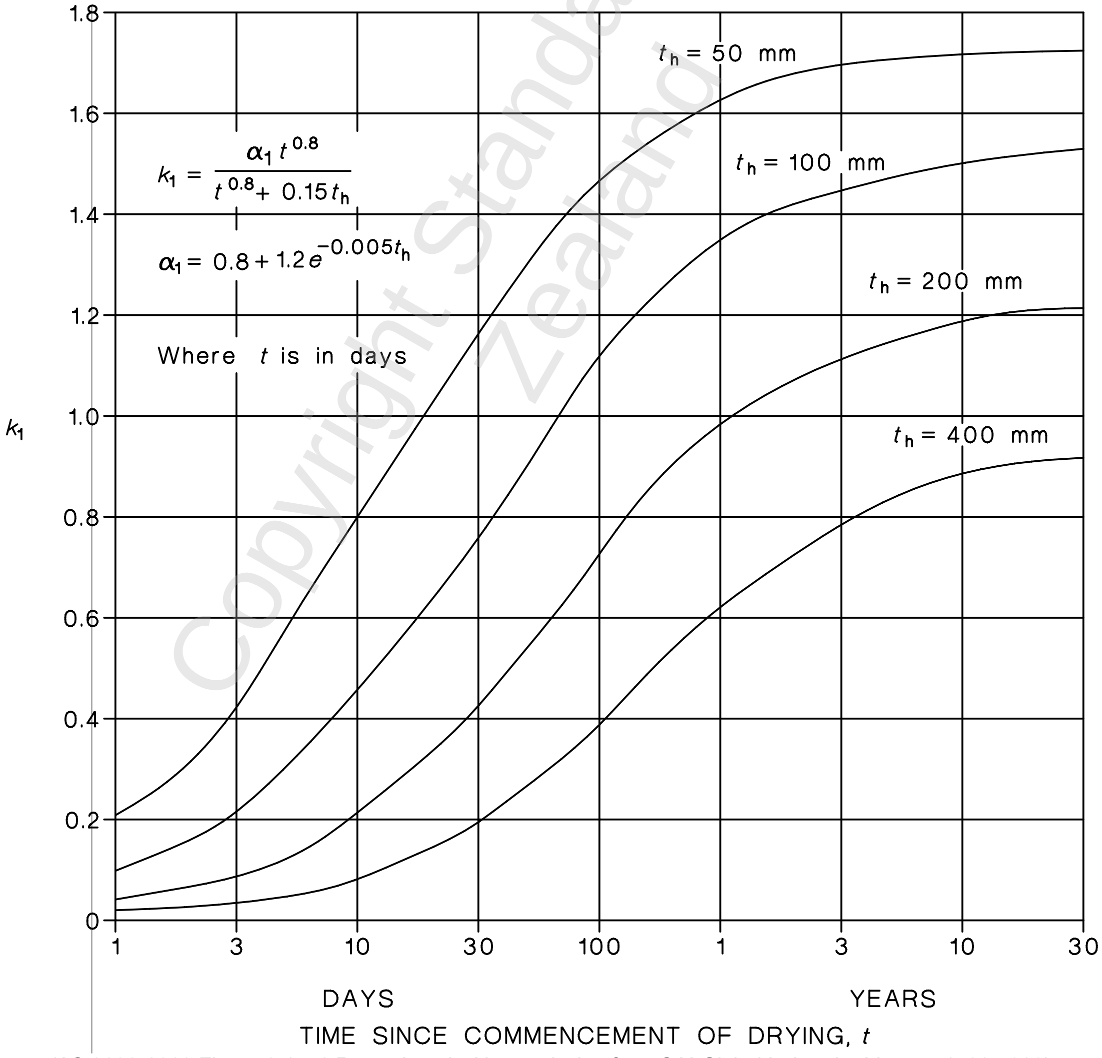
$$t_h = \frac{2A_g}{u_e} \qquad \text{(Eq. E–5)}$$

where u_e is the exposed perimeter of a member cross section plus half the perimeter of any enclosed voids contained therein.

The basic drying shrinkage strain ($\varepsilon_{csd.b}$) depends on the quality of the local aggregates, average values for which, for various aggregate types and locations around New Zealand, are given in Table E.2. Alternatively, values for $\varepsilon_{csd,b}$ can be determined as outlined in E2.1(a).

Consideration shall be given to the fact that ε_{cs} has a range of ± 30 %, and this should be allowed for in specifying shrinkage values to be achieved in concrete production.

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.



(AS 3600-2009 Figure 3.1.7.2 Reproduced with permission from SAI Global Ltd under Licence 1409-c080)

Figure E.1 – Shrinkage strain coefficient (k_1) for various values of t_h

A3

Table E.1 – Relative humidity factor k_4

Relative humidity (%)	40	50	60	70	80	90
Relative humidity factor (k_4)	0.74	0.68	0.61	0.50	0.39	0.21

Table E.2 – Basic drying shrinkage strain ($\mathcal{E}_{csd.b}$) for various aggregate types and locations around New Zealand

Location	Aggregate type	Basic drying shrinkage strain (microstrain)
Auckland Hunua, Whangarei, Hamilton	Northern greywacke	720
Hastings, Palmerston North, Masterton, Wellington, Blenheim, Kaikoura	Central greywacke	1100
Christchurch, Timaru, Oamaru	Southern greywacke	700
Auckland	Basalt*	660
Kaitaia, Tauranga	Other andesite / Basalt gabro	960
New Plymouth, Taranaki	Taranaki andesite	730
Waiau	Limestone	390
Nelson	Greywacke – siltstone	1100
Westport Queenstown, Wanaka Invercargill	Granite – greywacke Schist – greywacke Igneous – greywacke	590
Dunedin	Phonolite	520
* Use of Auckland basalt aggregate is declining. In general for shrinkage design for the Auckland locality it is recommended that the value for Auckland greywacke be adopted instead.		

E3 Creep

E3.1 General

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

$$\mathcal{E}_{cc} = \varphi_{cc}\, \sigma_0 / E_c \quad \text{(Eq. E–6)}$$

where
E_c = mean value of the modulus of elasticity of the concrete at 28 days
φ_{cc} = design creep coefficient at time (t) determined in accordance with E3.3.

E3.2 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 0.4 f'_c and shall be:

(a) Determined from measurements on similar local concrete;
(b) Determined by tests in accordance with AS 1012.16; or
(c) Taken as the value given in Table E.3.

Table E.3 – 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

E3.3 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 so 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:

A3 $\varphi_{cc} = k_2\, k_3\, k_4\, k_5\, k_6\, \varphi_{cc.b}$ ....(Eq. E–7)

where:

k_2 is obtained from Figure E.2 with t_h calculated in accordance with Equation E–5, and k_4 is obtained from Table E.1.

k_3 depends on the age of the concrete (τ) at the time of loading (in days) and is given by the following:

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

k_5 is a modification factor for high strength concrete, which shall be taken as:

k_5 = 1.0 when $f'_c \leq 50$ MPa; or

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

and the factor $\alpha_3 = 0.7/(k_4\alpha_2)$; and α_2 is defined in Figure E.2

k_6 is a modification factor for aggregate type, which may be taken from Table E.4.

Table E.4 – Modification factor for aggregate type (k_6)

Aggregate Type	Auckland basalt	Northern greywacke	Central greywacke	Southern greywacke	Other andesite/ basalt gabro	Taranaki andesite	Dunedin phonolite
Factor k_6	1.1	1.0	0.8 - 1.4	0.9	1.2	0.9	0.8

NOTES –

(1) Localities applicable to each aggregate type are as given in Table E.2.

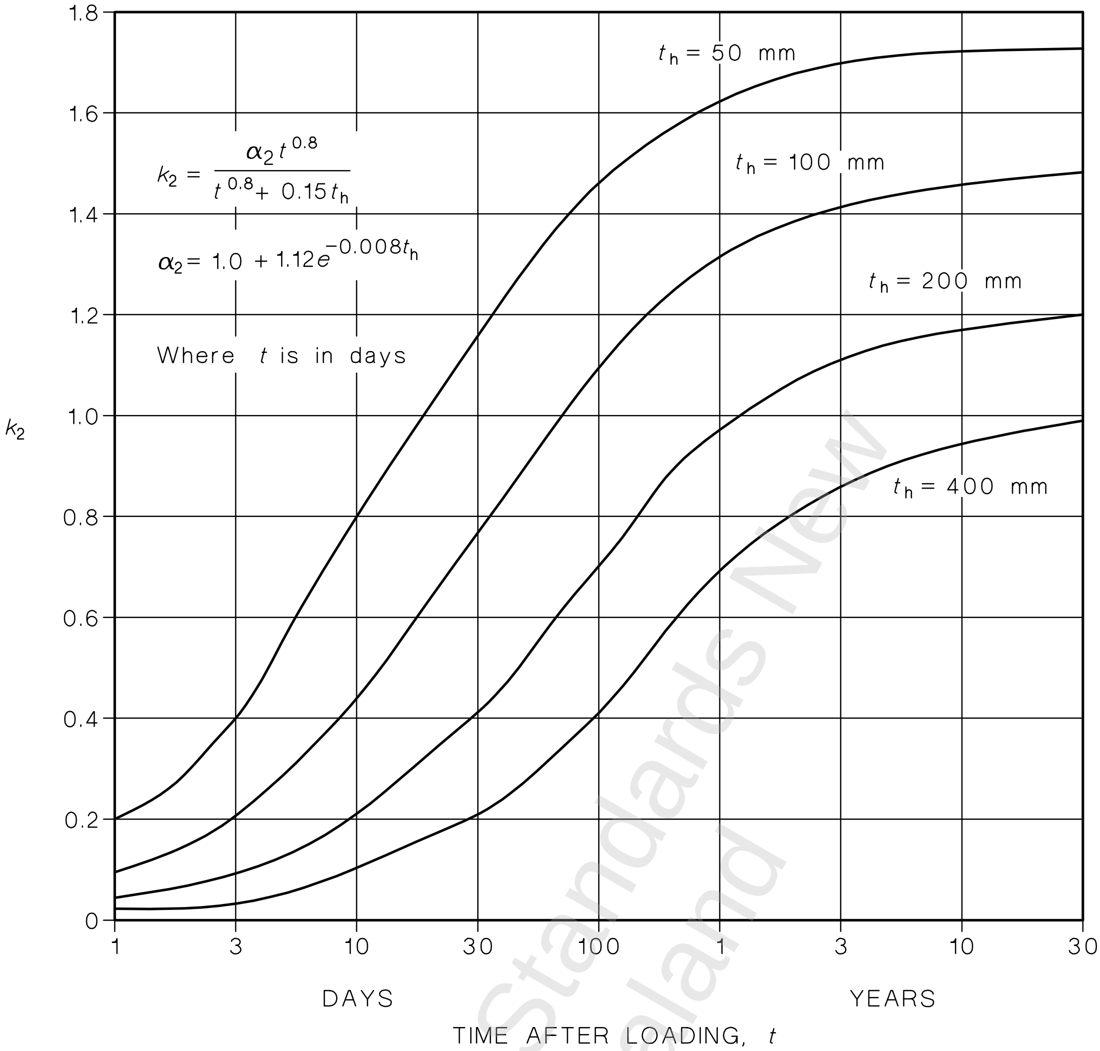
(2) k_6 factors have been developed from limited data and consequently high variability in this factor is likely.

Consideration shall be given to the fact that φ_{cc} has a range of approximately ± 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 0.5 f'_c.

A3



(AS 3600-2009 Figure 3.1.8.3(A) Reproduced with permission from SAI Global Ltd under Licence 1409-c080)

Figure E.2 – Coefficient k_2

NOTES

INDEX

Index updated for A3

A

B

D

E

F

G

H

I

J

L

M

N

O

P

R

S

T

Y

NZS 3101.2:2006

Incorporating Amendment No. 1, 2, and 3

New Zealand Standard

Concrete structures standard

Part 2: Commentary on the design of concrete structures

ISBN (Print) 978-1-77673-089-8
ISBN (PDF) 978-1-77673-090-2

NOTES

Contents

Table

Figure

REFERENCED DOCUMENTS

In this Commentary reference is made to the following:

NEW ZEALAND STANDARDS

NZS 1170:- - - -	Structural design actions
Part 5:2005	Earthquake actions – New Zealand
NZS 3104:2003	Specification for concrete production
NZS 3106:1986	Code of practice for concrete structures for the storage of liquids
NZS 3109: 1997	Specification for concrete construction
NZS 3404:- - - -	Steel structures standard
Part 1:1997	Steel structures standard
A3 Part 1:2009	Steel structures standard - Materials, fabrication, and construction
NZS 3122:1995	Specification for Portland and blended cements (General and special purpose)
NZS 3124:1987	Specification for concrete construction for minor works
NZS 3604:1999	Timber Framed Buildings
A3 NZS 4203:1992	General structural design and design loadings for buildings (withdrawn)
NZS 4229:1999	Concrete masonry buildings not requiring specific engineering design

A3

JOINT AUSTRALIA/NEW ZEALAND STANDARDS

AS/NZS 1170:----	Structural design actions
Part 0:2002	General principles
A3 Part 1:2002	Permanent, imposed and other actions
Part 2:2003	Wind actions

A3

AS/NZS 1554.3:2004	Structural steel welding – Welding of reinforcing steel
AS/NZS 3678:2011	Structural steel – Hot-rolled plates, floorplates and slabs
AS/NZS 4548:- - - -	Guide to long-life coatings for concrete and masonry (in 5 parts)
AS/NZS 4671:2001	Steel reinforcing materials
A3 AS/NZS 4672:- - - -	Steel prestressing materials
Part 1:2007	General requirements
AS/NZS 4676:2000	Structural design requirements for utility service poles

AMERICAN STANDARDS

American Concrete Institute

ACI 313-97	Standard practice for design and construction of concrete silos and stacking tubes for storing granular materials
ACI 318-02	Building code requirements for reinforced concrete
A3 ACI 355.2-04	Qualification of post-installed mechanical anchors in concrete and commentary
ACI 360-97	Design of slabs on grade
ACI TI.1-01	Acceptance Criteria for Moment Frames Based on Structural Testing

American Society for Testing and Materials

A3

ASTM C1202-97	Test method for electrical indication of concrete's ability to resist chloride ion penetration

AUSTRALIAN STANDARDS

A3

AS 1012:- - - -	Methods of testing concrete
Part 10:2000	Determination of indirect tensile strength of concrete cylinders (“Brazil” or splitting test)
Part 11:2000	Determination of the modulus of rupture
Part 13:2015	Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
Part 16:1996	Determination of creep of concrete cylinders in compression
AS 1418:- - - -	Cranes, hoists and winches (in 18 parts)
AS 1478:- - - -	Chemical admixtures for concrete, mortar and grout (in 2 parts)
AS 1530:- - - -	Methods for fire tests on building materials, components and structures
Part 4:1997	Fire-resistance tests of elements of building construction
AS 1544:- - - -	Methods for impact tests on metals
Part 2:2003	Charpy V-notch
AS 3582:- - - -	Supplementary cementitious materials for use with portland and blended cement (in 3 parts)
AS 3600:2001	Concrete structures
AS 3774:1996	Loads on bulk solids containers
AS 4072:- - - -	Components for the protection of openings in fire-resistant separating elements
Part 1:1992	Service penetrations and control joints

BRITISH STANDARDS

A3

BS 2573:- - - -	Rules for the design of cranes
BS 6349:- - - -	Maritime structures (in 7 parts)
BS 8204-2:2003	Screeds, bases and in-situ floorings. Concrete wearing surfaces. Code of practice

NORWEGIAN STANDARD

NS 3473:1992	Design of concrete structures

OTHER PUBLICATIONS

Bridge Manual (SP/M/022) Third edition. New Zealand Transport Agency, 2013. A3
Building Industry Authority New Zealand Building Code Handbook and Approved Documents 1992.
New Zealand Railways Corporation Code. Part 4: Code supplements, Bridges and Structures.

NEW ZEALAND LEGISLATION
Building Act 2004

NOTES

NEW ZEALAND STANDARD

CONCRETE STRUCTURES STANDARD

Part 2 – Commentary on the Design of Concrete Structures

C1 GENERAL

C1.1 Scope

C1.1.1 *Relationship to NZ Building Code*

Part 1 of this Standard is intended to be called up as a verification method for compliance with the New Zealand Building Code in Approved Documents B1: Structure – General, B2: Durability and C4: Structural Stability in Fire.

General design loadings applied to buildings are specified in AS/NZS 1170 and NZS 1170.5.

Suitable documents for determining design loadings and performance requirements for special purpose structures in reinforced or prestressed concrete include the following:

(1) For the design of water retaining structures e.g. reservoirs:
- NZS 3106 Code of practice for concrete structures for the storage of liquids

(2) For the design of bins, or silos for storage of bulk materials:
- ACI 313 Standard practice for design and construction of concrete silos and stacking tubes for storing granular materials
- AS 3774 Loads on bulk solids containers

(3) For the design of cranes:
- AS 1418 Cranes, hoists and winches (in 18 parts)
- BS 2573 Rules for design of cranes (in 2 parts)

(4) For the design of ground bearing slabs:
- ACI 360 Design of slabs on grade

(5) For the design of bridges:
- New Zealand Transport Agency's Bridge Manual A3
- NZ Rail Corporation Code. Part 4: Code supplements, Bridges and Structures, section 2; Design (for rail bridges)

(6) For the design of wharfs, and other marine structures (for ship mooring etc.):
- BS 6349 Maritime structures (in 7 parts)

(7) For concrete poles
- AS/NZS 4676 Structural design requirements for utility service poles – added to p.9. A3

Some special purpose structures may be subject to unusual loading conditions or require standards of material performance that are not appropriately covered by this Standard. These designs are outside the scope of this Standard as a verification method demonstrating compliance with the New Zealand Building Code (NZBC) and must be treated as an alternative solution. When considering the approval of an alternative solution, a Territorial Authority may accept those aspects which comply with NZS 3101, AS/NZS 1170 and NZS 1170.5 and any other referenced loading standard that is called up in the NZBC approved documents, demonstrating compliance with the NZBC although the design as a whole will continue to be regarded as an alternative solution.

A3

C1.1.1.1 *Minimum requirements*

When designing structures it is essential that load paths are identified and detailed to ensure that design actions can be sustained. In particular it is essential to track load paths through junctions between different elements and to ensure they have adequate strength and ductility to sustain the range of structural actions associated with the design limit states.

The Standard sets out the minimum requirements. However, not every detail that may arise in a design can be covered in the Standard. Where this situation arises the design should be based on first principles and the fundamental concepts on which the design of concrete structures is based.

C1.1.4 ***Interpretation***

This Commentary is intended to be read in conjunction with NZS 3101:Part 1. It not only explains the provisions of the Standard, but in certain cases it summarises the technical background that led to the formulation of a particular clause, and suggests approaches, particularly in Appendices, which satisfy the intent of the Standard. A list of references is provided at the end of each commentary section to assist designers in areas where design procedures have not been fully formulated and give additional background to the code clauses.

Clause numbering of the commentary is identical to that of the Standard except that clauses are prefixed with the letter 'C'. A cross-reference such as "5.5.1.3" refers to that clause in the Standard, while "C5.5.1.3" refers to the corresponding commentary clause.

C1.3 Design

In the preparation of this Standard, the Committee made an assumption as to the level of knowledge and competence expected of users of this Standard. This assumption is that the user (termed the design engineer) is either a professional engineer, experienced in the design of concrete structures, or, if not, is under the supervision of such a person.

In many places this Standard requires properties to be assessed or verified by test, but the Standard does not specify the details of such testing. In these cases the design documentation will need to provide sufficient detail for the testing requirements to be followed together with requirements for the documentation and reporting of the tests and include details of the acceptance criteria.

C1.4 Construction

The aim of the Committee has been to make the monitoring requirements of this Standard consistent with those of the building control system established under the Building Act 2004.

The communication of structural design to the constructor, and the Territorial Authority at the time the building consent application is made, rests primarily on the plans and specifications of the design engineer as the owner's agent.

Within the Building Act 2004 (section 7) the term "Plans and Specifications" is deemed to include "proposed procedures for inspection during construction". It is appropriate, therefore, that a framework for construction review be established for works being constructed to this Standard.

Adequate review, in the context of this clause, means such construction monitoring which, in the opinion of the design engineer and subject to the approval of the Territorial Authority, is necessary to provide acceptable reliability that the construction has been carried out in accordance with the design intent. It also includes various detailed inspections performed as required of specialised work. Such specialised work may include operations involving bending (including re-bending) or welding of reinforcement on site. Although these are permitted, subject to meeting the requirements specified in this Standard and in NZS 3109, sufficient supervision needs to be provided on site to ensure they are performed correctly.

The suitably qualified person as required by this clause should ideally be the design engineer but can be any person who is competent to undertake the review. Because of the nature of the design process and the importance of the design being communicated effectively, the review of the construction phase is essential to ensure:

(a) The design is being correctly interpreted;
(b) The construction techniques being used are appropriate, and do not reduce the effectiveness of the design;
(c) The work is completed generally in accordance with the plans and specifications.

The extent of involvement of the reviewer in a particular application will depend on:

(a) The size and importance of the construction;
(b) The complexity of the construction;
(c) The criticality of particular structural elements(s) within the construction, and the consequences of non-compliance;
(d) The material(s) of the construction (including inherent variability and particular manufacturing and field control requirements for that material);
(e) The relevant experience of the constructor(s);
(f) The status of the quality assurance programme adopted by the constructor(s).

The design engineer should nominate the level of construction monitoring considered to be appropriate to the work described in the plans and specifications included with the building consent application. Typically, this nomination will be expressed in terms of the construction monitoring levels specified in Appendix 1 of Reference 1.1.

In many projects the design engineer may apply different levels of construction monitoring to different parts of the work, as appropriate to the standards of expertise, or quality assurance, held by the constructor. In this context, a constructor is deemed to include any contractors, subcontractors and/or suppliers involved in the construction.

It may be appropriate for the suitably qualified person to provide the territorial authority with written confirmation that the construction review has been completed prior to the code compliance certificate being issued. In such circumstances the Territorial Authority may require documentary evidence of the agreement between the owner and the suitably qualified person (in the form of a signed undertaking from the owner) before the Building Consent is issued.

C1.5 Definitions

For consistent application of the Standard, it is necessary that terms be defined where they have particular meanings in the Standard. The definitions given are for use in the application of this Standard only and do not always correspond to ordinary usage.

Reinforced concrete is defined to include prestressed concrete. Although the behaviour of a prestressed member with unbonded tendons may vary from that of members with continuously bonded tendons, bonded and unbonded prestressed concrete are combined with conventionally reinforced concrete under the generic term “reinforced concrete”.

By definition, plain concrete is concrete that contains less than the minimum reinforcement required by this Standard.

A number of definitions for loads are given as the Standard contains requirements that must be met at various load levels. Loads and forces are as specified or defined separately for each of the serviceability and ultimate limit states by AS/NZS 1170 and NZS 1170.5 or other referenced loading standard. These documents also contain short-term, long-term and ultimate load factors for determining design loads from the appropriate serviceability and ultimate limit state load combinations.

A3 An objective that the structure possesses sufficient robustness to provide protection against collapse in an earthquake that exceeds the design ultimate limit state seismic action is largely achieved. It should be noted that material strain limits are associated with a loss of lateral strength. Collapse does not occur at this limit due to the beneficial effect of redistribution of actions, the gradual loss of lateral load capacity, and with loss of gravity load carrying capacity occurring at deformations greater than those causing loss of lateral strength.

To achieve the level of robustness described above, the design of the supports for stairs, ramps, cladding and other items whose failure could create a falling hazard on an egress route or a public area, is to be capable of sustaining the peak displacements and deformations associated with levels of earthquake shaking well in excess of those associated with ultimate limit state shaking, referred to in the Standard as a maximum considered earthquake level earthquake.

REFERENCES

1.1 "The Briefing and Engagement of Consultants", IPENZ/ACENZ Wellington, New Zealand, January 2004, 35 pp.

C2 DESIGN PROCEDURES, LOADS AND ACTIONS

C2.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 2 of the Standard:

f_{cmax}	the additional compressive stress due to live load plus impact, MPa, see C2.5.2.2
f_{cmin}	minimum compressive stress level in the concrete due to dead load, creep, shrinkage, temperature etc., MPa, see C2.5.2.2
f_{cr}	concrete stress range between maximum and minimum compressive stress, MPa, see C2.5.2.2
f_{smin}	algebraic minimum stress level in reinforcement, MPa, see C2.5.2.2
f_{smax}	f_{smin} plus the additional tension stress due to live load plus impact, MPa, see C2.5.2.2
f_{sr}	reinforcing steel stress range between maximum and minimum stresses, MPa, see C2.5.2.2
h_d	height of rolled on deformation on a deformed bar, mm
ℓ_y	ductile detailing length, mm
N^*	design axial load for ultimate limit state, N
N_n	nominal axial load strength, N
p	ratio of tension reinforcement = A_s/bd
p'	ratio of compression reinforcement = A'_s/bd
r	base radius of rolled-on transverse deformation on reinforcing bar, mm, see C2.5.2.2
S_p	the structural performance factor defined in NZS 1170.5
V_n	nominal shear strength of section, N
μ_p	ductility factor for a part

A3

A2

A3

C2.2 Design requirements

C2.2.1 *Design considerations*

The aim of structural design is to provide a structure which is durable, serviceable and has adequate strength while serving its intended function and which also satisfies other relevant requirements such as robustness, ease of construction and economy.

A structure is durable if it withstands expected wear and deterioration throughout its intended life without the need for undue maintenance.

A structure is serviceable and has adequate strength if the probability of loss of serviceability and the probability of structural failure are both acceptably low throughout its intended life.

C2.2.3 *Design for robustness, durability and fire resistance*

A3

Design displacements in NZS 1170.5 are values that are expected to be exceeded several times during ultimate limit state, serviceability limit state or maximum considered earthquake level earthquakes. These values are appropriate for assessing structural damage in structural elements (refer to NZS 1170.5 C4.4). However, in the design of stairs, ramps, panels and brittle elements (such as hollow-core units) it is the peak deformation or peak displacement that is critical. The peak value is equal to the design value divided by S_p.

C2.3 Design for strength and stability at the ultimate limit state

C2.3.2 *Design for strength*

C2.3.2.1 *General*

The basic requirement for the ultimate limit state may be expressed as follows[2.1]:

Design action ≤ Design strength

$S^* \leq \phi S_n$ ....(Eq. C2–1)

In the ultimate limit state procedure, the margin of structural safety is provided in the following two-ways:

(a) The design action, S^*, is determined from the governing ultimate limit state combination, given in AS/NZS 1170.0 for buildings and for highway bridges in the Transit New Zealand Bridge Design Manual [2.2]. Thus, for example, the ultimate design moment M^* on a building is the bending moment induced by 1.2 times the dead load (permanent action) with the additional moment induced by 1.5 times the live load (imposed action) being added to this value if it increases the magnitude of the resultant moment. It is written in equation form as:

$S^* = 1.2\ G \ \& \ 1.5\ Q$ ....(Eq. C2–2)

where S^* is the action, the design bending moment, M^*, in this case.

(b) The "design strength", ϕS_n, of a structural element is computed by multiplying the nominal strength S_n, by a strength, reduction factor, ϕ, which in general is less than 1.0. The nominal strength is computed by the standard procedures assuming that the member will have the exact dimensions and design (lower characteristic) material properties used in the computation [2.1].

For this Standard, notations with the superscript "*" such as M^*, N^*, and V^* refer to the required design actions, these being the critical actions due to the specified combinations of loads and forces for the ultimate limit state. The design strength values are equal to the strength reduction factor times the nominal strength, such as ϕM_n, ϕN_n, and ϕV_n.

C2.3.2.2 *Strength reduction factors, ultimate limit state*

The design strength of a member, as used in this Standard, is the nominal strength calculated in accordance with the provisions and assumptions stipulated in the Standard multiplied by a strength reduction factor ϕ, as detailed in 2.3.2.2 for the ultimate limit state and 2.6.3.2 for serviceability limit state load combinations involving seismic forces. The rules for computing the nominal strength of a member are based on chosen limits of stress, strain, cracking or crushing, and conform to research data for each type of structural action and to established structural engineering practice.

The basis for the selected values of strength reduction factor are detailed in the study by MacGregor [2.1], which ascertained that for the values of ϕ similar to those in 2.3.2.2 and load factors corresponding to AS/NZS 1170, target values of the safety index, β, of 3.0 for dead and live load, 2.5 for dead and live and wind forces and 2.0 for dead and live and earthquake forces applied. These values for the safety index are within the range implicit in AS/NZS 1170.

The strength reduction factor accounts for uncertainties in design computations and relative importance of various types of members. It provides for the possibility that small adverse variations in material strengths, workmanship, and dimensions, while individually within acceptable tolerances and limits may combine to result in understrength [2.1, 2.3, 2.4]. It should be noted that the variation in strength of reinforcement is small compared with concrete. Consequently where strength depends primarily on reinforcement a high strength reduction factor is used while where it is strongly influenced by concrete a smaller strength reduction factor, is appropriate.

A3 For members subject to flexure, or flexure with axial tension or small levels of axial compression, failure is initiated by yielding of the tension reinforcement and it takes place in a ductile manner. Hence, for beams and slabs subjected to pure flexure or flexure with axial tension or small compressive axial loads, a strength reduction factor of 0.85 is appropriate. The exception to this is for singly reinforced walls. These are thin elements with high height–to-thickness ratios. The stability of these walls under earthquake actions is uncertain for the following reasons:

(a) The longitudinal reinforcement is spread along the wall, midway between the wall faces, consequently there is limited flexural restraint available for out-of-plane actions; A3

(b) To reach their ultimate strength, high reinforcement strains have to be sustained by reinforcement located close to the tension end of the wall. The high tensile strains in this reinforcement, combined with the loss of stiffness due to the wide cracks, reduce the buckling resistance of both the wall and the reinforcement when the earthquake forces reverse and the zone is subjected to compression.

To allow for the uncertain performance of singly reinforced walls in earthquakes the strength reduction factor is reduced to 0.70, which results in these walls sustaining only limited inelastic deformation in the maximum considered earthquake.

The shear strength of a member is a function of both the tensile strength of concrete and the strength of shear reinforcement, and consequently the appropriate strength reduction factor is 0.75. Where the strength depends entirely on the strength of concrete, lower strength reduction factors are used. Thus for bearing on unconfined concrete the value is 0.75. No strength reduction factor is given for tension in plain (unreinforced) concrete as reliance on such strength to maintain equilibrium in a structure is not generally appropriate. The value of 0.6 stated previously in the Standard is inappropriate due to the likelihood of the effective tensile strength being significantly reduced from the nominal value as a consequence of tensile stresses that can be induced by self-strain actions, such as differential shrinkage and temperature (see C2.3.2.3).

All columns are required to contain confinement reinforcement which complies with one of the following clauses: 10.3.10.5, 10.3.10.6, 10.4.7.4 or 10.4.7.5. Structural testing has shown that the confinement leads to increased effective concrete strength in the confined core. Providing the column strength is calculated in accordance with 7.4.2 an overstrength of at least 15 % can be sustained, and considerably more in some cases. Tests have shown that plastic hinges in such members have high section ductility. Consequently ϕ for such members is set at 0.85 in recognition of this reserve strength[2.5].

For details associated with post-tension anchors see 16.3.2 and 19.3.12.

In capacity design, in relation to seismic design, where regions of members are designed to sustain overstrength actions in potential plastic hinge regions, the maximum likely actions that may be induced are considered. An example of this is in the design for shear, where the maximum shear force is calculated for the overstrength bending moments in the potential plastic hinges together with the gravity loading. For such extreme loading cases a margin of strength between the overstrength and the nominal strength is considered adequate. This corresponds to using a strength reduction factor of 1.0.

C2.3.2.3 *Tensile strength of concrete* A3

In general, designers should avoid relying on the tensile strength of concrete where the resultant tensile forces are required to satisfy equilibrium in the ultimate limit state. Exceptions to this occur in a number of situations which are specifically noted in the Standard. Examples of where this occurs are found in clauses related to design for shear, torsion, calculation of deflections, and stiffness of members used in the analysis of structures for seismic actions.

The action of bond between reinforcement and concrete induces hoop tension in the concrete surrounding reinforcing bars, which can lead to local cracking. However, this cracking does not influence member strengths. The local cracking reduces bond locally, which results in a redistribution of bond stress. Development lengths allow for this anticipated redistribution of actions.

It should be noted that tensile stresses can be induced in concrete by self-strain actions, which are frequently not considered in design. Due to the brittle nature of concrete, self-strain actions can greatly reduce the tensile stress that results in cracking of concrete. Examples of self-strain actions include those induced by temperature change, differential temperature, differential creep and shrinkage, and heat of hydration of concrete. Tensile stresses also arise due to Poisson expansion of concrete where section shapes change, such as at diaphragms in bridges.

C2.4 Design for serviceability

C2.4.1 *General*

While deflection and control of cracking are the predominant serviceability criteria, other criteria should be examined where required. Where necessary the effects of potential vibration from wind forces, machinery, vehicular pedestrian traffic movements on the structure, should be assessed to ensure the structure is serviceable for the occupants and potential contents.

In ductile structures, in some situations involving seismic load combinations, the serviceability design action can exceed the corresponding ultimate limit state action. In these cases either the structure needs to be designed to sustain the serviceability strength actions, or allowance for the inelastic deformation needs to be made in determining the crack widths, deflections etc. The design strength for serviceability
A3 may be taken as 1.1 times the nominal strength, see 2.6.3.2.

C2.4.2 *Deflection*

C2.4.2.2 *Bridges*

In many existing reinforced and prestressed concrete bridges, sag has developed in the spans due to the effect of long-term creep under the permanent loads and actions acting on the bridges. This is deleterious for both the ride quality of the bridge deck surface and for drainage of the bridge deck and has generally required “shape correction” of the bridge decks by application of an overlay. Applying an overlay, in turn, adds additional dead load to the structures, reducing their capacity for live load.

From an aesthetics perspective, where the bridge design vertical profile is a level grade, the development of sag in the spans is also highly undesirable, creating a sense of instability, whereas, conversely, a small amount of upward hog creates a sense of stability.

The vertical profile of the bridge spans between support points over the design life of the bridge should not deviate from the optimum vertical profile for the roading alignment by more than 50 % of the dead load deflection above this profile, nor deviate below this profile.

Options for achieving and maintaining the required vertical profile within the specified limits over the short and long-term include the following:

- Building in an appropriate amount of initial camber
- In prestressed concrete design, designing the prestress to fully offset the deflection due to the dead load. Care is required with this approach to take into account the reduction in eccentricity of the prestress over the long-term due to creep in the concrete. (see Appendix CE)

C2.4.3 *Minimum thickness*

A3 **C2.4.3.1** *Slabs and beams for buildings*

(a) *One-way spans*

Deflection calculations for slabs show that if the bending moment due to the long-term live load combined with the dead load exceeds the value given by Equation 2–2 the deflection can exceed 1/200 of the span. The k_1 value makes some allowance for the beneficial influence of redistribution of bending moments due to the loss of stiffness that occurs in the negative moment zones with flexural cracking.

(b) & (c) *Two-way construction (non-prestressed) for buildings*

Deflections of two-way systems of construction of the types considered in Section 12 need not be calculated if the minimum overall thickness requirements of this section are satisfied. Table 2.2 and Equations 2–3 and 2–4 provide an overall thickness consistent with that found from experience to give satisfactory control of deflections for flat slabs, flat plates and conventional two-way slabs supported on stiff beams. Table 2.2 and the equations provide for the cases ranging from slabs without beams through to slabs on stiff beams, and enable adjustment of the thickness for different design yield strengths of the reinforcement.

The degree of cracking has been observed to be less in two-way slabs than in beams and one-way slabs, with a consequent smaller effect of steel stress or strain on the stiffness of the element. This conclusion was reached and the form of the expression involving yield strength in Equations 2–3 and 2–4 was chosen after study of the results of the extensive tests on floor slabs described in the references listed in the Commentary for Section 12.

(d) *Composite precast and in situ concrete construction for buildings* A3

In terms of this Standard, composite members refer to members comprised of precast and *in situ* concrete, or a combination of concrete elements, precast or cast-*in situ*.

Composite structural steel-concrete members are covered in NZS 3404.

A3

C2.4.3.2 *Bridge structure members*

Deflection of bridges is not usually critical. The superstructure will often have a built-in camber to account for deflection under dead load. In some cases it may be important for aesthetic or other reasons to limit deflections under long-term loadings or live loads. The minimum thicknesses specified in Table 2.3 are based on AASHTO[2.6] requirements. They are introduced primarily to guard against excessive traffic-induced vibrations giving concern to pedestrians or occupants of stationary vehicles. These requirements may be waived if special consideration is given to design for vibration[2.2].

C2.4.4 ***Crack control***

C2.4.4.1 *Cracking due to flexure and axial load in reinforced concrete members in buildings*

This commentary gives some background to the different methods of crack control given in (a), (b) and (c) in 2.4.4.1: A3

(a) The criterion indicates that cracking is unlikely to occur and potential crack widths do not need to be assessed where the maximum tensile stress in the section is less than $0.4\sqrt{f'_c}$. This stress limit is less than the value used for deflection calculations due to the reduction in modulus of rupture with size and tensile stresses induced in the concrete arising from the restraint against shrinkage provided by the reinforcement;

(b) The criteria given in 2.4.4.3, 2.4.4.4, 2.4.4.5 and 2.4.4.7 for the maximum spacing of reinforcement have been developed to control crack widths to an average of 0.2 mm and a maximum that corresponds approximately to an upper characteristic value (about 19 out of 20 cracks have a width less than this value) of 0.4 mm[2.7]. However, it should be noted that these design criteria were developed from tests of flexural members in laboratories. There was no record of shrinkage of the concrete at the time tests were made and no allowance was made for the effects of shrinkage inducing compression strains in the reinforcement prior to cracking. Compression induced in reinforcement by shrinkage or creep of concrete has the potential to increase crack widths. For most buildings the criteria should be adequate. Where high shrinkage of concrete is anticipated, or where control of cracking is a major issue, potential crack widths should be assessed by the criteria given in 2.4.4.6;

(c) Equation 2–7, which is given in 2.4.4.6, but with $f_{s,ch}$ replaced by f_s, was used to develop the reinforcement spacing limits in (b). Further information on the background to Equation 2–7 is given in C2.4.4.6. Where crack control is a significant issue, or where shrinkage is high, crack widths should be assessed using 2.4.4.6, which makes an allowance for the increased stress change in the reinforcement associated with shrinkage in concrete.

Recommended limits on crack width for buildings are given in C2.4.4.6.

A3 **C2.4.4.2** *Crack widths in bridge structures*

The external concrete surfaces of bridges can be visible to the public. Unsightly cracking may need to be avoided for aesthetic reasons and to avoid raising alarm with the public over the load carrying capacity of bridges.

Bridges are subject to variable live loading and associated dynamic loading. This loading generally makes up a significant proportion of the total load on the bridge. To limit the degree of cracking, and the extent to which cracks that form are worked over the life of the bridge, leading to deterioration, crack width limitations are imposed.

A3

With structural elements buried below ground, the vibrational effects of live loading tend to be damped out by the soil-structure interaction. Cracking below ground also is not visible and so not cause for public alarm. For these reasons, crack width limitations have not been applied to below ground structural elements.

C2.4.4.4 *Spacing of reinforcement for crack control on the extreme tension face*

This is intended to be a simple conservative means of ensuring crack widths are acceptable in the majority of situations. The resultant maximum crack widths may be of the order of 0.35 mm to 0.5 mm where these criteria are satisfied. The stress in the reinforcement may be taken as $0.6f_y$, in place of the value determined by elastic transformed section theory.

The criteria, which are based on Reference 2.7, do not apply to situations where the cracks are induced as a result of restraint, which may be due to shrinkage from heat of hydration, drying shrinkage, differential temperature or thermal restraint. See C2.4.4.8 for control of cracks in these cases.

A3 **C2.4.4.5** *Crack control on the sides of beams and slabs*

This requirement applies to relatively deep beams. It has been found [2.8, 2.9] that where this intermediate reinforcement is omitted wide cracks can develop in the serviceability limit state in the mid-region of the tension zone. The width of these cracks can be several times the corresponding crack width adjacent to the main flexural tension reinforcement. The formation of wide cracks in a beam web can result in a significant reduction in shear strength[2.10].

A3 The criteria for the spacing of skin reinforcement are consistent with 2.4.4.6 and the area of reinforcement is sufficient to transmit a tension force across the crack equal to $0.25\sqrt{f'_c}$ to the area of concrete surrounding each bar of skin reinforcement.

C2.4.4.6 *Assessment of surface crack widths*

The principle factors influencing crack spacing are:

- The bond performance of the reinforcement;
- The quantity and arrangement of reinforcement contained in the effective area of concrete surrounding this reinforcement;
- The average tensile strain at the level of the member being considered; and
- The distance between the point where the crack width is being assessed and the centre of the nearest reinforcing bar.

A3 The value given by Equation 2–7 was derived from tests on reinforced concrete beams in a laboratory. It was developed to predict a crack width such that close to 95 % of all the cracks will be less than the value given by the equation, with the average crack being approximately 45 % of the predicted value. However, in developing this equation the stress change in the reinforcement was assumed to be the stress sustained when the load was applied, that is it was assumed to be equal to f_s. The influence of shrinkage of the concrete inducing compression in the reinforcement before the crack formed was not considered. In the laboratory only a limited amount of shrinkage would have developed before the tests were made. Consequently where significant shrinkage occurs with time, both the stress change and resulting crack widths will be underestimated unless allowance is made for the compression stresses induced in the concrete before the concrete cracks[2.11]. To allow for the influence of shrinkage on crack width the stress change is taken as the stress sustained in the reinforcement after the crack has formed, f_s, plus half the compression stress induced by final shrinkage, $f_{s,c}$. Taking half of the induced compression by final shrinkage makes a nominal allowance for the shrinkage that would have occurred in the test beams used to develop the equations, and the reduced effect of shrinkage that develops after initial cracking has occurred on further increase in crack widths. A3

The approach given in 2.4.4.6 is based on Reference 2.7. This approach is similar to a method proposed by Beeby[2.12]. Appendix CE gives a method of assessing the compression stress induced in reinforcement due to shrinkage in concrete.

Table C2.1 gives recommended maximum calculated crack widths, which are considered to be acceptable in different situations for buildings.

Table C2.1 – Recommended maximum surface width of cracks at the serviceability limit state for buildings

Material	Load category				
	I **Immediately after the transfer of prestress before time dependent losses**	**II** **Permanent loads plus variable loads of long duration; or permanent loads plus frequently repetitive loads**	**III** **Specified serviceability limit state loads for buildings where Load Category II does not apply**	**IV** **Permanent loads plus infrequent combinations of transient loads,**	**Exposure classification (refer Table 3.1)**
Reinforced concrete Prestressed concrete	- 0.3 mm	0.4 mm 0.2 mm	0.4 mm 0.3 mm	0.5 mm 0.4 mm	A1
Reinforced concrete Prestressed concrete	- 0.2 mm	0.3 mm 0.1 mm	0.3 mm 0.2 mm	0.4 mm 0.3 mm	A2, B1, B2
Reinforced concrete Prestressed concrete	- Zero	0.2 mm Zero	0.2 mm 0.1 mm	0.3 mm 0.2 mm	C, U

NOTE –
For members incorporating a combination of significant quantities of reinforcing and prestressing steel, the allowable crack widths shall be chosen from Table C2.1 on the basis of the location and proportion of the prestressing steel. Where the prestressing tendons are not in the anticipated cracked zone, or where principal deformed reinforcement is located between any tendons and the tensile concrete surface, the allowable crack widths for reinforced concrete may be applied.

C2.4.4.7 *Crack control in flanges of beams*

Consideration should also be given to adding reinforcement outside this width to control cracking.

C2.4.4.8 *Control of thermal and shrinkage cracking*

Differential temperature conditions can arise for example in chimneys, or pipes containing hot fluids, or on roofs and bridge decks which may be heated by solar radiation. The deformation induced by such differential temperatures can lead to excessive cracking, particularly at the supports of beams.

The setting of concrete liberates heat. Problems can arise in thick members when the concrete contracts as it cools, and this can cause wide cracks to form, and it can lead to significant flexural type deformation. The sequence of pouring concrete can reduce the potential cracking and insulating an exposed concrete surface can reduce differential thermal strains from initiating cracking. References 2.13, 2.14, and 2.15 contain useful information on assessing potential cracking together with methods that can be adopted to control cracking due to the heat of hydration, differential temperature and shrinkage cracking.

Rapid evaporation from freshly cast concrete can lead to plastic shrinkage, which can have adverse effects on strength and durability of the concrete. This can be controlled by limiting the rate at which evaporation can occur from freshly placed concrete [2.15, 2.16].

C2.5 Other design requirements

C2.5.1 *General*

Account shall be taken during design of any particular performance requirements of a structure. Consideration should also be given to the consequences of unforeseen events, which given due regard to both the risk of occurrence and the function of the structure, may require explicit consideration in design.

C2.5.2 *Fatigue (serviceability limit state)*

C2.5.2.1 *General*

Members in some structures, for example deck slabs of bridges, may be subject to large fluctuations of stress under repeated cycles of live loading.

C2.5.2.2 *Permissible stress range*

The limitations on the range of stress of 150 MPa under live load, irrespective of the grade of reinforcing used, are based on AASHTO standards [2.6] and were considered necessary to avoid the possibility of premature fatigue failure in the reinforcing bars. The range of stress of 150 MPa is allowed for straight reinforcing steel. The effect of the 150 MPa range is usually to limit crack widths to approximately 0.25 mm.

This stress range is further reduced in the CEB-FIP Code where the stress occurs in a bar bend (as a function of d_b) and where corrosion can be expected [2.17] and further general information on fatigue may be
A2 obtained from References 2.18 and from "Comite Euro-internationale du Beton, "Fatigue of Concrete Structures", Bulletin D' Information No. 188, June 1988.

The allowed relaxation of the requirements of this clause, if a special study is made, is in recognition of views expressed [2.19] that the specified requirements are conservative. The requirements of a special study may be deemed to be satisfied if the following revised AASHTO procedures [2.6] are followed:

Concrete

The stress range, f_{cr} , between the maximum compressive stress (f_{cmax}) and the minimum compressive stress (f_{cmin}) in the concrete at the serviceability limit state, at points of contraflexure and at sections where stress reversals occur, shall not exceed $0.5f'_c$ where:

f_{cr} $= f_{cmax} - f_{cmin}$

f_{cmin} is the minimum compressive stress level in the concrete due to dead load, creep, shrinkage, temperature, etc. (MPa)

f_{cmax} $= f_{cmin}$ plus the additional compressive stress due to live load plus impact (MPa)

Reinforcement

The stress range, f_{sr}, between the maximum tension stress (f_{smax}) and the minimum stress (f_{smin}) in straight reinforcement at serviceability limit state, shall not exceed:

f_{sr} $= f_{smax} - f_{smin} = [\,145 - 0.33\, f_{smin} + 55\, (r/h_d)\,]$

f_{smin} is the algebraic minimum stress level due to dead load, creep, shrinkage, temperature etc. (MPa) (tension positive, compression negative)

f_{smax} $= f_{smin}$ plus the additional tension stress due to live load plus impact (MPa)

r / h_d is the ratio of base radius to height of rolled-on transverse deformation; when the actual value is not known use 0.3.

Bends in primary reinforcement and welding shall be avoided in regions of high stress range. The suitability of mechanical connections for splices should be checked where repetitive stress fluctuations occur.

Fatigue shall be checked for normal serviceability limit state live loads only. Overloads are specifically excluded from the requirements of this clause.

C2.5.3 *Structural diaphragms* A3

Floor and roof systems in buildings are required to act as diaphragms to tie structural elements together and to transmit lateral forces to and between the lateral force-resisting elements.

Lateral forces arise from, but are not limited to:

(a) Wind;
(b) Soil pressures;
(c) Water pressure;
(d) Seismic forces.

Consideration should be made of self-strain forces within the structure (including the diaphragms) that arise from the natural difference in the deflection profile of walls and moment resisting frames and actions associated with creep, shrinkage and temperature change. Detailed design requirements are given in Section 13.

C2.6 Additional design requirements for earthquake effects

C2.6.1 *General*

C2.6.1.1 *Deformation capacity*

A key feature of this Standard is that structures are designed so that material strains in potential plastic regions do not exceed permissible values at the ultimate limit state. In practice the level of detailing that is used in potential plastic regions needs to be matched to the predicted material strain level.

The term “material strain” is used as a generic term for curvature, shear deformation or axial strains etc., while the term potential plastic region refers to a region where inelastic deformation occurs due to yielding of reinforcement or crushing of concrete. In most cases “material strain” refers to curvature and “potential plastic region” refers to a potential plastic hinge.

C2.6.1.2 *Classification of structures*

The classification of the structure is related to the ability of a structure as a whole to sustain cyclic deformation without loss of strength. It does not refer to the ability of individual potential plastic hinge regions within a structure to sustain inelastic deformation.

A2

A2

A2 A3 **C2.6.1.3.1** *Classification nomenclature*

NZS 1170.5 uses the term "material strain" to identify the elastic plus the inelastic deformation in a plastic region. However, for a reinforced concrete plastic hinge there is no simple way of establishing the strain levels in concrete or reinforcement. Consequently, in this Standard a nominal curvature is used in place of strains in the individual materials.

The classification is based on the ability of a potential plastic region to sustain deformation due to seismic ground motion. As the level of detailing increases, so the capacity of the inelastic zones to sustain deformation increases, and this enables the structure to sustain increased inelastic displacement without loss of strength.

In previous editions of this Standard it has been implicitly implied that material strains in plastic hinge regions are proportional to the structural ductility factor. However, the structural ductility factor by itself is a poor guide to deformation demand in plastic regions. Consequently, in this Standard the level of detailing that is required is related to an assessment of the local deformation demand in the critical inelastic zones.

It should be noted that a structure of limited ductility can be composed of a mix of members containing ductile and limited ductile potential plastic regions. Nominally ductile structures may generally contain a mixture of limited ductile plastic regions (LDPR) and nominally ductile plastic regions (NDPR). Ductile structures should only contain ductile plastic regions, as the accuracy with which deformation demands can be predicted decreases with an increase in the structural ductility factor.

Where a limiting material strain limit is to be established by special study, note that the maximum permissible strain for the ultimate limit state must be sustained under a few cycles of displacement with a high level of confidence. To give the structure the robustness envisaged in NZS 1170.5, the average maximum permissible material strain should be capable of sustaining in excess of 1.5 times the limiting material strain nominated for the ultimate limit state given in Table 2.4. This limit corresponds to the anticipated deformation demand for a maximum considered earthquake.

A2 **C2.6.1.3.2** *Material strain limits in plastic regions*

This clause sets out how to find plastic hinge rotations and shear deformation in diagonally reinforced coupling beams associated with ultimate limit state earthquake actions. These values are divided by the effective plastic hinge length, ℓ_p, or effective length of plastic region, L_n, to give the material strain as a curvature or a shear deformation, as appropriate. The values of ℓ_p and L_n are given in 2.6.1.3.3.

A3

A2 Where a limiting material strain limit is established by a special study, the design limit should be consistent with the objectives given in NZS 1170.5 (see NZS 1170.5 clauses 2.1 and C2.1).

A3 Deformation in plastic hinges is made up of elastic and inelastic components. With time history analyses, the sum of these two components can be found directly from the results of the analyses. However, with equivalent static or modal response spectrum analyses, the inelastic component can be found from the structural displacements associated with inelastic deformation. For all ductile, limited ductile, and nominally ductile plastic regions in group (i) the elastic component of curvature in plastic hinges may be taken as $\dfrac{2f_y}{E_s h}$, where f_y is not taken greater than 425 MPa solely for the purpose of calculating this curvature. In nominally ductile plastic regions for members in group (ii) the value of K_d is limited to 1.0 where there is inadequate longitudinal reinforcement in the compression zone or inadequate shear

A3

stirrups to provide some buckling restraint. In group (ii) members the value of K_d is further reduced to give protection against a premature shear failure, see 2.6.1.3.4(c). This expression $\frac{2f_y}{E_s h}$ is a simplification of limiting elastic curvatures developed by Priestley and Kowalsky[2.21, 2.22]. This curvature can be used for columns even when the elastic limit of the section is reached due to inelastic deformation of the concrete, rather than tensile yielding of the reinforcement.

A method of assessing the elastic component of deflection for a diagonally reinforced coupling beam is described in C6.8, and Reference 2.28 gives additional background information. The shear deformation in a diagonally reinforced coupling beam, corresponding to the deflected shape profiles defined in NZS 1170.5 is shown in Figure C2.1A.

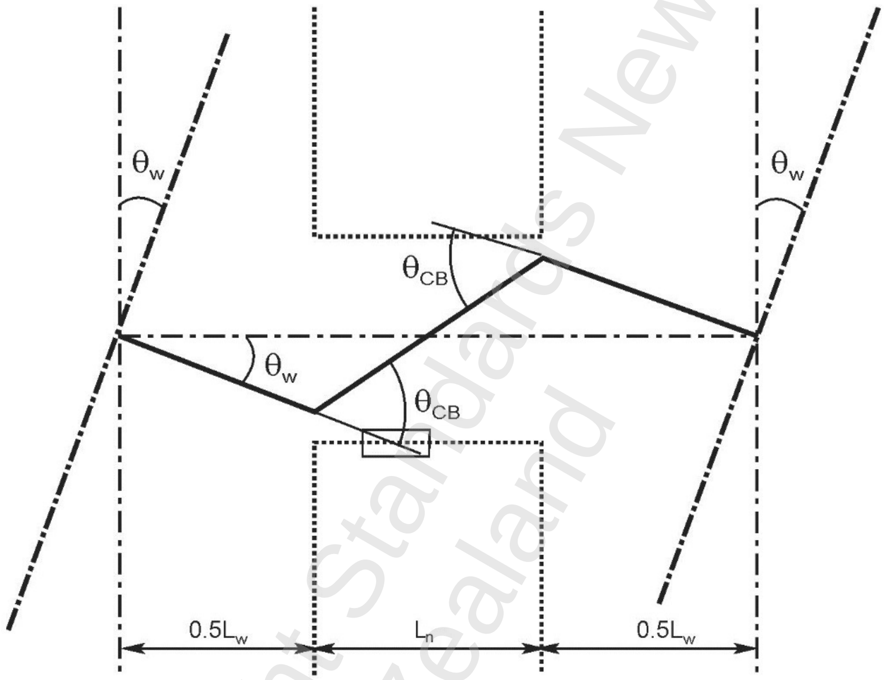


Figure C2.1A – Deformation of coupling beams

In NZS 1170.5 the difference in the deflected envelope values for elastic response between two adjacent levels is taken as a measure of the elastic inter-storey drift (as the peak lateral deformations at adjacent levels occur at different times the difference in envelope values is less than the maximum inter-storey drift). For the equivalent static method using NZS 1170.5, the elastic envelope is derived from clauses 5.2 and 6.2.3, and for the modal response spectrum analysis the corresponding envelope comes from NZS 1170.5 clause 5.2.2. The nominal elastic inter-storey drift is increased to allow for inelastic deformation arising from a number of different actions (NZS 1170.5 clause 7.2). In NZS 1170.5, clause 7.3, the difference in the lateral deflection envelope between adjacent levels is multiplied by the drift modification factor to give the total design inter-storey drift (elastic plus inelastic). The drift modification factor allows for the observation that analyses of the same structure by equivalent static or the modal response spectrum method generally give smaller resultant storey drifts than those predicted when the same structure is analysed by the inelastic time history method. The inelastic component of drift is equal to the total inter-storey from clause 7.2 of NZS 1170.5 times the drift modification factor minus the elastic component of inter-storey drift (which is not multiplied by the drift modification factor).

Unidirectional plastic hinges sustain appreciably greater inelastic rotation than reversing plastic hinges. Fortunately, they are able to sustain greater curvature before strength degradation occurs[2.20]. A method of calculating the rotation demand in unidirectional plastic hinges is given in 2.6.1.3.2 (b)(ii). References 2.20 and 2.38 give some background to the behaviour of this form of plastic hinge.

Diagonally reinforced coupling beams, where the diagonal reinforcement does not extend over the entire clear span, have been found to perform poorly, with the yielding limited to a short length of the reinforcement close to the bend in the bars[2.39, 2.40].

C2.6.1.3.3 *Effective plastic region lengths*

The effective plastic hinge length, ℓ_p, is the length assumed for the purposes of calculating section curvature. Inherent assumptions in this calculation are that plane sections remain plane and that the curvature is uniform over the length of the plastic region. These assumptions are not valid, consequently the calculated curvature should be treated as an index of the material strain levels rather than an actual measure of these strains. The situation is illustrated in Figure C2.1.

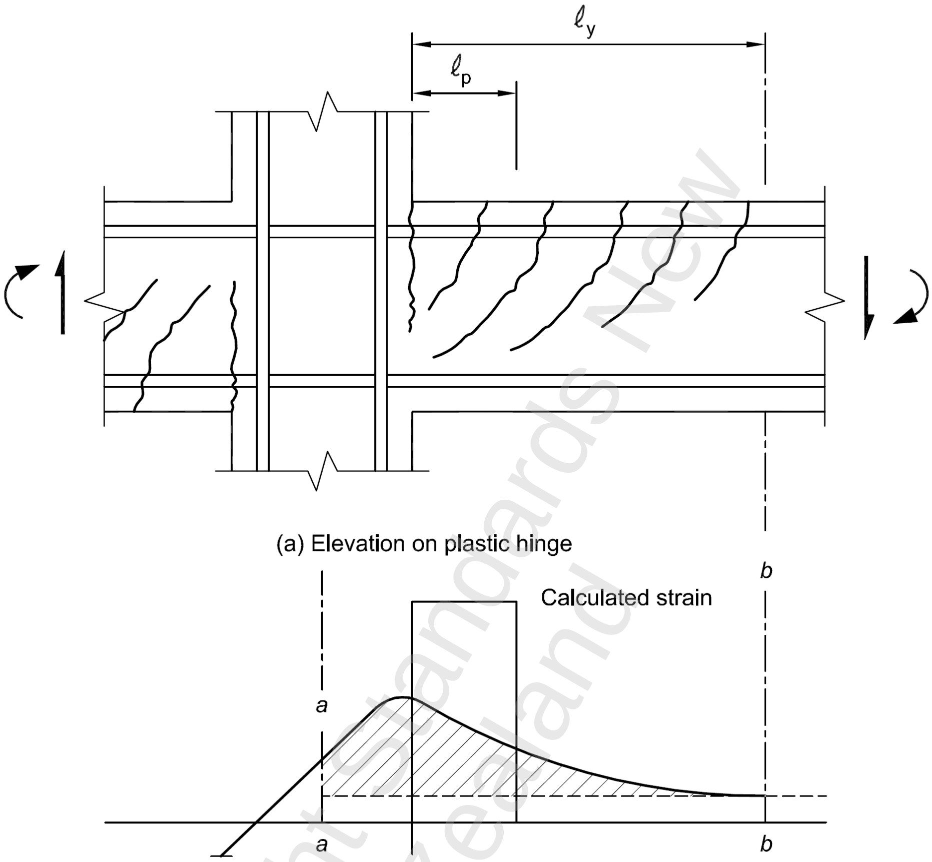


Figure C2.1 – Effective plastic hinge length

A2 As shown in Figure C2.1 the effective plastic hinge length ℓ_p is generally less than half of the length, ℓ_y, over which the reinforcement yields, or the concrete sustains appreciable inelastic deformation. Ductile detailing is required over this length, which is referred to as "the ductile detailing length, ℓ_y." It should be noted that actual reinforcement strains can be very different from values calculated using the effective plastic hinge length, ℓ_p. This difference arises as appreciable yielding extension may occur in beam-column joint zones, or anchorage for bars, and in addition elongation of plastic hinge regions can greatly increase reinforcement strains. There is no simple way of assessing these actions, consequently, they are ignored in curvature calculations.

A3 For members where the ratio of effective depth to overall depth, *d/h*, is less than 0.75 the effective plastic hinge length is increased by the k_p factor in Equations 2–9(a) and 2–9(b). This adjustment makes an allowance for greater proportional contribution to flexural rotation that occurs due to the close location of the reinforcement to the compression face of the member, and to the increase in rotation that arises for the extension of reinforcement in its development length.

Where unidirectional positive moment plastic hinges form in the span of a beam, inelastic deformation can develop on both sides of the critical section. In addition, as the shear is low in such zones, the plastic deformation spreads over a relatively long length compared to the effective plastic hinge lengths. Consequently, provided the positive moment reinforcement is uniform in the region of these plastic hinges, limiting material strains are reached in the negative moment plastic regions well before the positive moment hinges are critical.

C2.6.1.3.4 *Material strain limits*

The material strain limits are set limits for deformation due to seismic actions and they are not intended to apply to non-seismic cases. A3

The deformation limits for potential plastic regions defined in (c) depend on the level of detailing, nominally ductile, limited ductile or ductile detailing, and on the type of actions, namely reversing or unidirectional actions. Most plastic hinges are designed to sustain reversing actions. In these the same region sustains both positive and negative inelastic rotations. In this situation the plastic rotation in the plastic regions is closely related to the magnitude of the drift of the structure.

Unidirectional plastic hinges may form where gravity loads are relatively high compared to the maximum seismic actions that can be sustained when plastic hinges develop. In beams where this type of deformation occurs negative moment plastic hinges form at or close to the columns and positive moment plastic regions form in the mid-span region of the beam. Each inelastic displacement of the structure induces negative and positive rotations in their respective plastic regions. In beams each inelastic excursion of the structure causes the beam to deflect downwards as the earthquake progresses. The maximum inelastic component of rotation in a unidirectional plastic hinge is sustained at the end of the earthquake ground motion. There is no simple relationship between the plastic hinge rotations and the maximum drift of the structure. While unidirectional plastic hinges sustain greater rotation demands than reversing plastic hinges this is partly compensated by the greater rotational capacity of this form of plastic hinge[2.38].

There are two groups of nominally ductile plastic regions. In group (i) the members containing the nominally ductile plastic hinge have:

(a) Longitudinal reinforcement in both the tension and compression sides of the beam, and the reinforcement in the compression side of the member is restrained against buckling regardless of whether the bars are required to act in compression or not; and
(b) The ultimate strength of the member is limited by flexure and not by shear.

Group (ii) nominally ductile plastic regions are those regions that do not satisfy the requirements for group (i). These members have little ability to sustain inelastic deformation.

The material strain limits for the different forms of potential plastic region and type of inelastic action are given in Table 2.4 together with the notes in (a) to (c) in the clause. These limits are given in terms of a nominal curvature at first yield. This is a simplification of an expression developed by Priestley and Kowalsky[2.21, 2.22]. A number of minor changes have been made from the values in these papers to allow for the difference in test results, which are based on measured material properties, to design values that are based on lower characteristic values. Some background to material strain limits is given in Reference 2.37.

Paragraph (d) gives the limiting deformation for diagonally reinforced coupling beams as a shear strain. The deformation of a diagonally reinforced coupling beam may be modelled as a shear stiffness element based on first yield of the diagonal reinforcement in compression and tension or more simply by modifying the flexural stiffness to represent both flexural and shear deformation (see C6.9.1.3).

Paragraph (e) gives limits for deformation of squat shear walls. These values have been derived from structural tests given in Reference 2.41. In these members there is an interaction between flexure and shear and the structural actions need to be considered together. Further information is given in C7.7.11.

C2.6.1.4 *Stiffness of members for seismic analysis*

The stiffness of reinforced concrete members depends on many factors, such as the quantity of reinforcement and its grade, the level of axial load and the extent of cracking in the member. Further details on these factors are given in C6.9.1.

Where elastic analysis methods (equivalent static, modal response spectrum and elastic time history) form the basis for determining design actions the stiffness of members which are designed to sustain inelastic deformation should be based on a member stiffness which is equal to the stiffness at first yield of the longitudinal reinforcement. For walls the stiffness should only be reduced to allow for flexural cracking in

A3 storeys where flexural cracking is anticipated under ultimate limit state actions. Further background to the stiffness of structural walls is given in References 2.23 and 2.24.

C2.6.2 *Seismic actions*

C2.6.2.1 *General*

The seismic design actions in a structure are found by analysing an analytical model of the proposed structure. The method of analysis is by one of the methods given in NZS 1170.5, namely the equivalent static method, the modal response spectrum method or the time history method. With the first two approaches the analytical model is assumed to behave elastically, while with the time history approach inelastic behaviour may also be modelled. The structural performance factor makes some allowance for aspects of behaviour that are not included in analyses (see NZS 1170.5). The structural ductility factor allows for the overall ability of the structure to sustain repeated inelastic displacements without significant loss of strength. These two factors enable the response of the analytical model to be modified to allow for non-linear behaviour so that the required design strengths and displacements can be predicted from the analytical model.

Where the seismic actions are defined in a referenced loading standard, which is not NZS 1170.5, the A3 structural performance factor may need to be increased above the minimum values given in 2.6.2.1 to comply with the referenced loading standard.

C2.6.2.2 *Structural performance factor*

The structural performance factor allows for a number of beneficial effects not considered in analysis (see NZS 1170.5). The structural ductility factor for a specific type of structure has been derived from analytical studies and sub-assembly tests. However, allowance has to be made for the difference between the loading sequence applied in standard tests and that imposed in earthquakes. Generally in tests several cycles of displacement to the maximum ductility displacement are applied. However, the peak displacement is attained only once in an earthquake, but several cycles to a displacement level just below the peak value can be sustained. The damage that results in failure is accumulated for all the inelastic displacements. With the higher levels of ductility the motion is more damped. Consequently the number of high peak displacements sustained in a given earthquake motion decreases as the structural ductility is increased. Hence the S_p factor, which allows for this and other effects, reduces as the ductility increases[2.25].

A3 The S_p factor may be reduced to 0.7 for a nominally ductile structure provided that:

(a) If it is subjected to a major earthquake it would develop a ductile failure mechanism in preference to a brittle failure mechanism;

(b) It is detailed in such a way that all the beams that may develop plastic regions meet the detailing requirements of limited ductile beams as set out in 2.6.2.2.2 (b); and

(c) All the walls, but excluding singly reinforced walls, are detailed to meet the detailing requirements in the ductile detailing length for limited ductile walls.

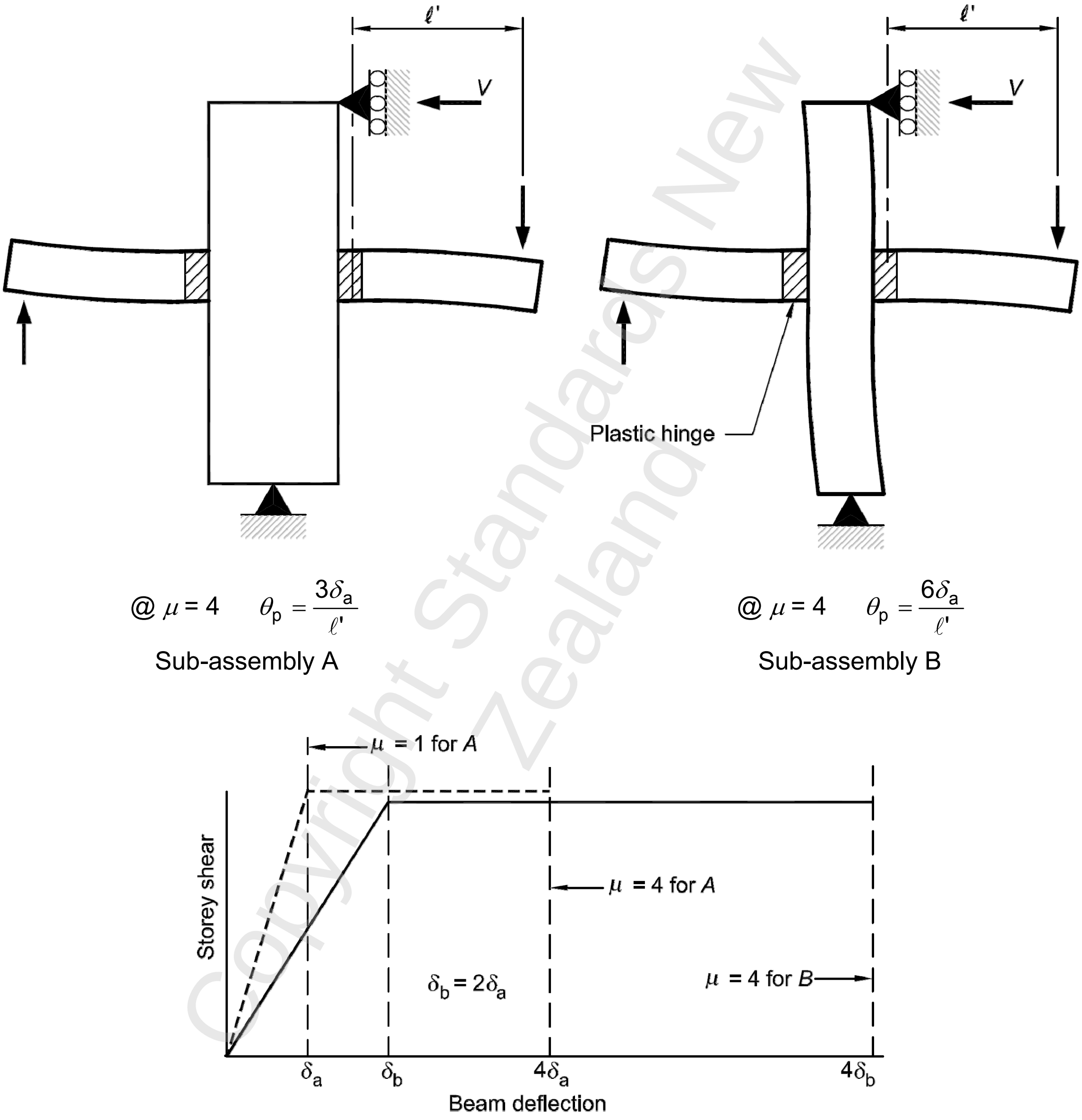
Increasing the nominal design actions in (b) and (d) in 2.6.2.2.2 allows for the increase in inelastic demand due to the reduction in the S_p factor and the difference in yield forces associated with the upper and lower characteristic yield strengths.

C2.6.2.3 *Structural ductility factor*

The structural ductility factor, μ, has an important influence on the required design strength and to a lesser extent the deformations in potential plastic regions. It is one of the factors determining the level of detailing required in a structure, or part of a structure.

It should be noted that the structural ductility factor gives a measure of the ability of the structure as a whole to sustain inelastic displacements in terms of the limiting elastic displacement found from the analytical structure. It does not give a reliable indication of the inelastic deformation that must be sustained in a plastic hinge region, as illustrated in Figure C2.2. In this Figure two beam-column sub-assemblies are shown, in which the beams are identical. However, the column for sub-assembly A is stiff while for B it is flexible. When loads are applied to the beams, plastic hinges form at a displacement of δ_a

in structure A and δ_b in structure B. Due to the lower stiffness of the column in structure B the displacement δ_b is greater than the corresponding displacement δ_a in structure A. For the purpose of this illustration it is assumed that δ_b is equal to twice δ_a at a ductility of 1. Hence at a displacement ductility of 4 the total rotation in the plastic hinges is equal to $(\theta_e + 3\delta_a/\ell')$ for sub-assembly A and $\theta_e + 6\delta_a/\ell'$ for sub-assembly B, where θ_e is the elastic component of rotation in the plastic hinge length. This illustrates the need to base detailing requirements on material strains (curvature) rather than on the structural ductility factor. The structural ductility factor enters into the relationship in that as it increases the reliability with which the material strains can be predicted decreases. A3



A3

Figure C2.2 – Material strains and structural ductility factor

It should be noted that Table 2.5 sets out the maximum structural ductility factors that may be used. The actual limit on the design value of μ in a particular structure may be controlled by the limiting material strain that can be sustained in critical inelastic regions.

At first yield the section curvature increases with the yield stress as does the deflection of the structure. For any building the maximum permissible structural ductility factor is limited to the ratio of maximum permissible inter-storey displacement divided by the displacement at ductility of 1, which is close to the displacement at first yield. Hence for any structure where the inter-storey drift or displacement is close to the limiting value the available structural ductility factor decreases as the yield stress of reinforcement increases.

In designing ductile moment resisting frames it is generally the stiffness requirements, which determine the member sizes. With high-grade reinforcement in the beams either larger section sizes are required than those with a lower grade of reinforcement or the structural ductility factor needs to be reduced. With the latter case the increase in structural actions, and consequently increase in design strengths, tends to offset the reduction in quantity of reinforcement due to its higher yield strength [2.26]. A similar situation also arises with slender walls[2.27]. Consequently changing from Grade 300 to Grade 500 reinforcement may not result in an appreciable saving in the quantity of longitudinal reinforcement due to the reduced structural ductility factor and the increased design actions.

A3 For the design of ductile structures, in which the lateral force resistance is provided by a mixture of walls and moment resisting frames or other lateral force-resisting elements, the structural ductility factor should be limited so that the maximum permissible value for any lateral force-resisting element, given in Table 2.5, is not exceeded.

For all ductile and limited ductile structures capacity design is required to ensure that inelastic deformation is confined to the specific regions, which have been proportioned to sustain such deformation. The steps involved in capacity design are set out in 2.6.5.

C2.6.3 *Serviceability limit state*

C2.6.3.1 *General*

An elastic analysis can only be used to determine deflections and inter-storey drifts for the serviceability limit state if the strength of the structure is sufficient to prevent significant inelastic deformation from occurring.

Generally the ultimate limit state controls the design strength and an elastic analysis may be used to determine serviceability limit state deformations. For example, for a structure with a structural ductility factor, μ, of 5, the R factor for a 500-year return earthquake is 1.0 (Table 3.5, NZS 1170.5) and the divisor, (μ), used to find design actions from the elastic response spectrum is 5.0 (NZS 1170.5, 5.2) for structures which have a fundamental period of 0.7 s or more. For this limit state the strength reduction factor is 0.85 for flexure and axial load. Hence the required nominal strength for seismic actions corresponds to (0.20/0.85) 0.235 times the actions derived from the elastic response spectrum. With the serviceability limit state and a 25-year return period the R factor is 0.25. The corresponding strength reduction factor (see 2.6.3.2) is 1.1. Consequently the design seismic actions correspond to (0.25/1.1) 0.23 times the values calculated from the elastic response spectrum. However, the ultimate limit state may not control the minimum required strength for all structures with a structural ductility factor of 5 or less, as redistribution of actions is permitted in the ultimate limit state but not in the serviceability limit state. Furthermore for a certain selection of buildings the return period for the serviceability earthquake is appreciably greater than 25 years (SLS2 earthquake in NZS 1170.5, 2.14(b)).

For structures designed with a structural ductility factor of 3 or less the ultimate limit state seismic actions are appreciably greater than the corresponding serviceability limit state actions. Consequently for nominally ductile structure and structures of limited ductility, it can be assumed that there is sufficient strength to prevent appreciable non-linear behaviour from occurring and elastic-based methods of analysis may be used.

Where the structural ductility factor used for the ultimate limit state exceeds 3.0 the serviceability limit state actions may be greater than the corresponding ultimate limit state actions. In such cases it may be necessary to either increase the strength of the structure or to allow for inelastic deformation in assessing displacements and deflections. Where the latter option is followed it is necessary to consider the added deflection, which may occur due to the natural redistribution of structural actions associated with inelastic deformation (shake down). In cases where unidirectional plastic hinges form in the serviceability limit state allowance should be made for the accumulated inelastic deformation arising from the complete earthquake record and not just the limit corresponding to the maximum displacement[2.20].

C2.6.3.2 *Strength reduction factor*

Where it is necessary to check for strength for serviceability limit state actions it is considered adequate to use mean material properties rather than values based on lower characteristic strengths. In recognition of this the permissible strength reduction factor is increased to 1.1.

C2.6.4 *Ultimate limit state*

Inter-storey drift limits are specified in NZS 1170.5. These limits are set to control damage in a design level ultimate limit state earthquake and as protection against excessive P-delta actions.

C2.6.5 *Capacity design*

C2.6.5.1 *General*

Capacity design is required for ductile and limited ductile structures and for nominally ductile structures where inelastic deformation may be concentrated in members over a small portion of the height of the structure. Capacity design ensures that in the event of a major earthquake:

(a) Non-linear deformation is confined to selected potential plastic regions, which have been detailed to sustain these deformations;

(b) Non-ductile failure modes are suppressed.

For a general background on capacity design refer to Reference 2.28.

C2.6.5.2 *Identification of ductile mechanism*

Permissible ductile mechanisms are identified for moment resisting frames, for walls and for wall frames in 2.6.7 or 2.6.8. From the selected mechanism the potential primary plastic regions are identified. In the capacity design process these zones are designed to have both the required strength and the ductility to sustain the inelastic deformation, which may be imposed on them. The remainder of the structure is detailed to ensure that the inelastic deformation is confined to the potential primary plastic regions and that non-ductile failure mechanisms are suppressed.

Secondary plastic regions may also arise due to actions which are not considered in the analysis. These include actions induced by elongation of plastic regions and higher mode effects that arise when the dynamic characteristics of a structure change with the formation of plastic regions.

C2.6.5.3 *Detailing of potential plastic regions*

The critical design actions in the limit states determine the required flexural strength in potential plastic regions. The critical plastic hinge rotations in these zones are identified from the deformed shapes for the ultimate limit state, which are defined in NZS 1170.5. From the plastic hinge rotations and the effective plastic region lengths the material strain levels (usually curvature) can be identified. This enables the potential plastic region to be identified as a nominally ductile plastic region, limited ductile plastic region or ductile plastic region in 2.6.1.3, together with the required level of detailing.

C2.6.5.4 *Overstrength actions*

The load combinations for the serviceability and ultimate limit state load combinations determine the critical design actions for the potential primary plastic regions. These are detailed so that the design strength is equal to or greater than the design action for the chosen failure mode. Each potential primary plastic region is examined and its maximum likely strength, known as overstrength, is assessed for the details as designed. To determine the overstrength actions the materials are assumed to have their upper characteristic strengths together with an allowance for strain hardening. Appropriate material properties for the assessment of overstrength actions are given in 2.6.5.5. Where axial loading exists care is required to ensure the critical axial load level is selected. This level in general needs to be determined from the overstrength actions acting in the structure together with the gravity loads rather than from an elastic-based analysis (see Appendix D).

C2.6.5.5 *Likely maximum material strengths*

As the magnitude of inelastic tensile strains increases so the effective width of tensile flange increases. For this reason, the effective width of tension flange in Tee beams and Tee or L-shaped walls is greater A2

A2 for flexural overstrength calculations than it is for nominal strength calculations (see 9.4.1.6.1 and 9.4.1.6.2 for beams, and 11.3.1.3 and 11.4.1.3 for walls). Over-strength calculations should be based on the appropriate section, which allows for the effective widths of flange associated with high tensile strains in the reinforcement, with upper characteristic reinforcement yield strengths increased to allow for expected hardening and with likely concrete strengths. The concrete strength in these calculations is taken as $\left[f_c' + 15\right]$ MPa, as the design strength corresponds to a 28 day value, which is generally significantly less than the average strength, and in addition there is some gain in strength after 28 days.

The overstrength factors given in 2.6.5.5 for reinforcement, $\phi_{o,fy}$, allow for the difference in the yield stress levels between the design and upper characteristic values, together with an allowance for strain hardening. A manufacturer may demonstrate that other values are appropriate for their reinforcement by using the analysis method that is described in Reference 2.29.

C2.6.5.6 *Ends of columns*

Columns, which are bounded by rigid members such as a foundation pad, see Figure C2.3, often have strength considerably in excess of that indicated from standard flexural theory. The concrete in the compression zone of the column close to the pad is partially confined by the rigid member in addition to the confinement from the reinforcement. As a result of this confinement spalling is delayed in this zone and the compressive strength of the concrete is increased. The resultant load carrying capacity increases
A2 with the magnitude of the axial load that acts. Test results have been analysed to develop Equation 2–10, which relates the likely overstrength of such a column to its nominal strength. It should be noted the nominal strength is found from standard theory using the design material strengths.

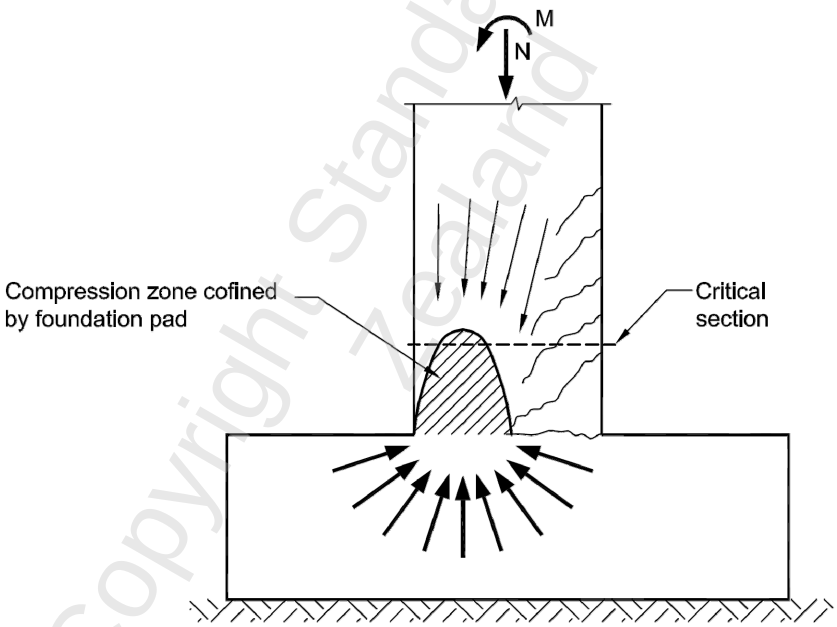


Figure C2.3 – Strength enhancement at base of column

C2.6.5.7 *Capacity design for regions outside potential plastic regions*

In detailing regions of a structure outside potential plastic regions, care is required in assessing the critical actions induced by gravity loading and overstrength actions. For example in a beam to determine the safe cut off position for flexural reinforcement allowance should be made for the case where overstrength actions may act in one plastic hinge in a beam but actions below the overstrength value act in a second plastic hinge.

The formation of plastic hinges in a structure changes its dynamic characteristics. In many cases the distribution of structural actions changes markedly from those found by elastic analysis. For example, the bending moments acting in a column, when plastic hinges have formed in a beam, can be considerably higher than would be anticipated by scaling the design actions from an elastic analysis. This change in distribution of structural actions is allowed for by either using a dynamic magnification factor or by specifying the distribution of actions over the length of a member. Further details on allowing for dynamic magnification effects are given in Appendix D.

C2.6.5.8 *Concurrency and capacity design*

NZS 1170.5 allows the design strengths for ductile and limited ductile structures to be found assuming that the seismic forces act along a single axis, provided this is orientated to give the most critical design action at the section being considered. However, seismic shaking occurs simultaneously along two axes at right angles. Hence a member such as a column, which forms part of two intersecting frames, is subjected to bending moments, shears and axial load simultaneously from the beams in both frames. The bending moments applied to the column from the beams in one axis reduce the capacity of the column to sustain bending moments induced from the beams on a second axis. Consequently to prevent a plastic hinge forming in the column it needs to be designed to sustain the maximum likely bi-axial actions that can be transmitted to it by the beams from the two frames.

It is considered unlikely that the dynamic amplification of actions will occur simultaneously along both axes of a column. For this reason it is considered sufficient to design the column to sustain the overstrength actions along one axis amplified by the dynamic magnification factor (or equivalent) together with the overstrength actions from the second axis (ω =1.0). The latter values are not amplified by a dynamic magnification factor. Further details on concurrency and appropriate values of dynamic magnification factor are given in Appendix D.

The critical axial load acting on the column is determined from the gravity load and overstrength actions transmitted to the column by the beams. Two alternative methods of determining the critical actions in columns of moment resisting frame structures are given in Appendix D. Any additional axial forces induced by vertical ground motion are neglected, as the duration of any vertical pulse in a column is too short for significant yielding to occur.

C2.6.5.9 *Transfer diaphragms* A3

Floor and roof systems in buildings are required to act as diaphragms to tie structural elements together and to transmit inertial forces to the lateral force-resisting elements. In addition self-strain forces arise from the natural difference in the deflection profile of walls and moment-resisting frames, elongation of plastic regions and actions associated with creep, shrinkage and temperature change. Detailed design requirements are given in Section 13.

C2.6.5.10 *Elongation in plastic regions*

Most practical methods of structural analysis do not model elongation in reinforced concrete members. It is important for designers to be aware of the significance of elongation, particularly where seismic actions are being considered, so that allowance can be made for any adverse effects.

Elongation occurs in reinforced concrete when flexural cracking occurs. This is due to the tensile strains being greater than the compression strains. The net effect of this is that the length of the member, measured along its mid-depth, increases. The elongation has major effects when plastic hinges form, as the tensile strains in plastic hinge regions can be very much greater than compression strains, which can result in elongation of a few percentages of the member depth.

Some background to the mechanisms that cause elongation to occur are described in C7.8.

C2.6.6 ***Additional requirements for nominally ductile structures***

C2.6.6.1 *Limitations for nominally ductile structures*

A stated objective in NZS 1170.5 is that structures should be capable of sustaining the action of the maximum considered earthquake (2,500-year return) with a margin of safety against collapse. Earthquake response spectra specified by NZS 1170.5 for nominally elastic structures require large seismic design forces to be used. However, some appreciable ductility may be required to sustain the maximum considered earthquake. This has been illustrated in a recent earthquake[2.30]. Reinforced concrete components designed in accordance with the general requirement of this Standard are considered to possess some inherent, albeit limited, capacity for ductility. Therefore, in terms of accommodating ductility

A3

A3

demands, in the design of well-conditioned structures, the application of the additional seismic requirements of all relevant Sections of this Standard is not necessary. However, for this exemption to be used two conditions need to be considered. This requirement necessitates the clear identification of plastic mechanisms that could be mobilised should larger than anticipated ductility demands arise.

(a) When the selected structural system is such that in terms of regularity and the relative strengths of members as built, the system would qualify to be designed as a ductile one or one with limited ductility, the exemption from the additional seismic requirements applies. Typical examples are nominally ductile multi-storey frames in which, under the action of exceptionally large earthquake forces, the formation of a "soft storey" is not expected. For moment resisting frames the criterion in

A2 2.6.6.1(a) may be assumed to be satisfied where at each elevated level, excluding the two highest,

$$\sum M_{n,col}\frac{L_{col}}{L'_{col}} \geq 1.15\sum M_{n,beam}\frac{L_{beam}}{L'_{beam}}$$

where $\sum M_{n,col}$ and $\sum M_{n,beam}$ are the sums of the nominal strengths of the columns and beams at the faces of the beam-column joint zones in the level being considered, and L_{beam} and L_{col} are the centre to centre spans of the beam and column value respectively for each M_n, and L'_{beam} and L'_{col} are the corresponding clear spans.

(b) When the configuration of the structural system is such that a plastic mechanism, should it be required, is inadmissible in terms of the requirements of this Standard for ductile structures or those of limited ductility, attention must be given to local ductility demands. These may be significant. With the identification of members that may be subjected to inelastic deformations clearly in excess of those envisaged for nominally elastic structures, the relevant additional design requirements for seismic effects for members of ductile or limited ductile structures must be applied.

Examples are multi-storey frames in which, because in the absence of the application of capacity design, the possibility of plastic hinge formation at both ends of all columns in a storey is not excluded. The end regions of columns in such frames should be detailed as required in accordance with procedures for limited ductile plastic regions (LDPR), or for ductile plastic regions (DPR). In frames with more than three storeys and where, because of their dominant strength, plastic hinges in beams could not develop, material strains in potential plastic regions should be identified and detailed appropriately (2.6.1.3). Ductility in such frames, if it arises, may be expected to develop only in one of the storeys. Depending on the estimated local ductility demand, the special detailing of the affected elements is necessary. The same principles apply to piers formed in between openings in walls, and also to walls with irregular openings.

C2.6.7 *Additional requirements for ductile and limited ductile moment resisting frames*

C2.6.7.4 *Alternative design methods for columns in multi-storey frames*

Appendix D gives two methods of assessing critical bending moments, axial forces and shear forces in columns in ductile and limited ductile moment resisting frames.

C2.6.7.5 *Design actions in columns*

A2 This clause requires columns to be designed to resist the axial load level calculated from the capacity actions and the overstrength bending moments applied to the column by the beams framing into it. Where a column forms part of more than one moment resisting frame the column should be designed to resist the bi-axial actions that are induced in it (see 2.6.5.8).

Capacity design requirements in 2.6.5.8 determine the required strength of a column in the region between the primary plastic hinge and bottom of the highest storey. Part (d) of this clause extends the requirement to consider bi-axial actions in determining the minimum design strength to the primary plastic hinge region and also to the region immediately below the uppermost level.

C2.6.8 *Ductile walls and dual systems*

C2.6.8.1 *Inelastic deformation of structural walls*

The important role of structural walls in the seismic resistance of buildings is discussed in Reference 2.28.

The principal aim of a designer should be to ensure that energy could be dissipated by flexural yielding of the wall in the plastic hinge region, which is generally located at the base of the wall. A3

C2.6.8.2 *Shear strength of structural walls*

Recommendations are made in Appendix D for methods of allowing for the influence of flexural overstrength and dynamic magnification on shear forces induced in a wall.

C2.6.8.3 *Coupled walls*

The desired energy dissipation in coupled structural walls is for plastic deformation to be restricted to the coupling beams and the base of the walls. To achieve this objective it is important to allow for the strength of the coupling beams. This strength is increased by the restraint provided to their elongation, which arises from the foundation beam, the flexural stiffness of the walls and from elements, such as floors, that tie the walls together. Detailed design provisions are given in 11.4.9 with further background information in the commentary. Additional information on the design of coupling beams may be found in Reference 2.28. A3

C2.6.8.4 *Ductile dual structures*

With the development of inelastic deformation in dual structures some redistribution of structural actions may occur from weaker elements to stronger elements. Allowance for this redistribution may be made in the design. Procedures for capacity design of such structures is discussed in Reference 2.28.

C2.6.9 *Structures incorporating mechanical energy dissipating devices*

An alternative approach from the conventional seismic design procedures on which this Standard is based is that of “base isolation”. Earthquake generated forces are reduced by supporting the structure on a flexible mounting, usually in the form of elastomeric rubber bearings, which will isolate the structure from the greatest disturbing motions at the likely predominant earthquake ground motion frequencies[2.31, 2.32]. Damping, in the form of hysteretic energy dissipating devices, is introduced to prevent a quasi-resonant build-up of vibration. This approach is finding application more frequently. Potential advantages over the conventional design approach that relies on ductility appear to include simpler component design procedures; use of non-ductile forms or components; construction economies, and greater protection against earthquake induced damage, both structural and non-structural. The greatest potential advantages are for stiff structures fixed rigidly to the ground, such as low-rise buildings or nuclear power plants. Because these structures are commonly constructed in reinforced concrete, these provisions have been included in this Standard although the principles may be applicable to other materials. Bridges often already incorporate elastomeric rubber bearings, and the greatest benefits for such structures may derive from the potential for more economic seismic resistant structural forms[2.33, 2.34].

The design and detailing of structures designed for base isolation and incorporating mechanical energy dissipating devices should satisfy the criteria set out in the following paragraphs.

Moderate earthquakes

For a moderate earthquake, such as may be expected two or three times during the life of a structure, energy dissipation is to be confined to the devices, and there is to be no damage to structural members.

“Design” earthquake

For a “design” NZS 1170.5 earthquake the designer may adjust the strength levels in the structural members to achieve an optimum solution between construction economies and anticipated frequency of earthquake induced damage. However, the Standard requires that the degree of protection against yielding of the structural members be at least as great as that implied for the conventional seismic design approach without dissipators. (In many cases this could be achieved with substantial construction cost savings. That is, the lower structural member strength requirements more than compensate for the extra costs of the devices.) It is recommended that the extent to which the degree of protection is increased above that minimum, to reduce the anticipated frequency of earthquake induced damage, should be resolved with regard to the client’s wishes.

Extreme earthquake

For an extreme earthquake there is to be a suitable hierarchy of yielding of structural and foundation members that will preclude brittle failures and collapse. This may be achieved by appropriate margins of strength between non-ductile and ductile members and with attention to detail.

Although the design criteria outlined above encompasses three earthquake levels, the design practice need be based only on the "design" earthquake. In the course of that design, the implications of yield levels on response to the "moderate" earthquake would have to be considered, as would also the implications of strength margins and detailing for an "extreme" earthquake. In general, the lower ductility demand on the structure means that the simplified detailing procedures of limited ductility design would be satisfactory.

Because applications of these devices to structures designed for seismic resistance are still being developed, numerical integration inelastic time history analyses should generally be undertaken for design purposes. Such analyses should consider acceleration records appropriate for the site, in particular taking account of any possibility of long period motions. As experience is accumulated, there is potential for development of standardised design procedures for common applications.

C2.6.10 *Secondary structural elements*

C2.6.10.1 *Definitions*

Secondary elements include primary gravity load resisting elements such as frames which are in parallel with stiff structural walls and do not therefore participate greatly in resistance of lateral forces. Caution must, however, be exercised in assumptions made as to the significance of participation. Frames in parallel with slender walls should be designed and detailed as fully participating primary members. Although the contribution of secondary elements to lateral force resistance may eventually be neglected, it is best to include them in the analysis of the total structural system subjected to lateral design forces. This will indicate the degree of participation in the generation of displacement-induced forces. For convenience of reference and specification of requirements, secondary elements have been subdivided into groups, that is, Group 1 and Group 2 elements.

C2.6.10.2 *Group 1 secondary elements*

To avoid any form of deformation induced loading in Group 1 elements, separations must be meticulously detailed. Similarly close attention must be given to details of supports, and to their positioning. Reference 2.35 discusses separation, while Reference 2.36 discusses such aspects as the conflict between these separation requirements and the requirements of sound attenuation, fire protection and the like. The design force is specified as an equivalent static force. Since these forces are already scaled to account for amplification of accelerations within the structure, no additional scaling of deflections and element actions is required. Often Group 1 elements are geometrically complex, and where appropriate the yield line method, for instance, of Section 12 would be appropriate to their analysis.

Ductile behaviour remains the prime objective of adequate detailing and must be sought by the detailer. The details, however, need not be elaborate to allow such behaviour. Wall panels, for instance, may be reinforced with a single layer of reinforcement without any confinement, and still provide adequate ductility.

The following values for the ductility factor for a part, μ_p, are suggested for partitions, prefabricated panels and parapets:

(a) connected so that instability is prevented if strength degrades or integrity is impaired.................... $\mu_p = 3$

(b) other (e.g. vertical cantilevers)

   (i) doubly reinforced .... $\mu_p = 2$

   (ii) singly reinforced...$\mu_p = 1.25$

C2.6.10.3 *Group 2 secondary elements*

In the consideration of Group 2 elements:

(a) The additional seismic requirements of the relevant sections of the Standard need not be complied with when the elastic deformation-induced actions on the element are derived from elastic analysis using deformations corresponding with the ultimate limit state;

(b) Where ductile action is relied on to produce adequate inelastic deformation capacity, all additional seismic detailing requirements of relevant sections must be met;

(c) NZS 1170.5 sets out the requirements to be met in regard to inertia forces and to design lateral deformations, and the commentary to that standard provides guidance on methods of calculation;

(d) The deformation calculated in accordance with NZS 1170.5 may be exceeded in some structures and in localised areas. Furthermore the pattern of deformation will usually vary significantly from the first mode pattern assumed in calculation. These variations should be taken into account in assessing member actions when they might have a marked effect on element performance.

(e) In certain cases elastic response may not be desirable, as forces may become excessive and even lead to inferior performance of the primary structure. Therefore inelastic action is permissible. However, elements must be designed for at least the elastic fraction of the total deformation of the primary elements, to prevent excessive damage in moderate earthquakes. Normally elastic actions will be selected. In most instances achievement of this will not prove to be unduly onerous. In many cases design of Group 2 elements for strength will be controlled by 1.2 G &1.5 Q.

(f) Inelastic action may be assumed only when detailing allows adequate ductility. A2

Limited capacity design might be appropriate for shear force determination and in determining whether or not adjacent members yield, but it is not considered necessary to amplify column moments for higher mode effects to prevent yielding of columns because column hinging is not of particular significance.

C2.6.10.4 *Stairs, ramps and panels* A3

Stairs and ramps are required to remain functional after they have been subjected to levels of earthquake shaking well in excess of those associated with ultimate limit state shaking. See Commentary C8.8 of NZS 1170.5 for more information. The requirement is intended to ensure continuity of access across stairs and ramps. The clause does not mean that stairs and ramps should remain undamaged after maximum considered earthquake level shaking.

Cladding panels are required to remain attached to the supporting structure when subjected to levels of earthquake shaking well in excess of those associated with ultimate limit state shaking, including maximum considered level of earthquake shaking.

REFERENCES

2.1 MacGregor J.G., "Load and Resistance Factors for Concrete Design", ACI Journal, July – August 1983, pp. 279-287.

2.2 Transit New Zealand , "Bridge Manual", Wellington, New Zealand, 2002.

2.3 Winter, G., "Safety and Serviceability Provisions in the ACI Building Code", ACI–CEB–FIP–PCI Symposium, ACI Special Publication, 1979.

2.4 Cornell, C.A., "A Probability Based Structural Code", Proceedings ACI, Vol. 66, No. 12, December 1969, pp. 974-985.

2.5 Prestland, R.A., "Seismic Performance of Retrofitted Reinforced Concrete Bridge Piers", Civil Engineering, University of Canterbury, Research Report 2001-3

2.6 AASHTO, "Standard Specifications for Highway Bridges, 17th Edition," American Association of State Highway and Transportation Officials, Washington, 2002

2.7 Frosch, R. J., "Another Look at Cracking and Crack Control in Reinforced Concrete", Structural Journal, American Concrete Institute, Vol. 96, No. 3, May 1999, pp. 437-442.

2.8 Frantz, G.C. and Breen, J.E., "Design Proposal for Side Face Crack Control Reinforcement for Large Reinforced Concrete Beams", Concrete International: Design & Construction, Vol. 2, No. 10, Oct. 1980, pp. 29-34.

2.9 Frantz, G.C. and Breen, J.E., "Cracking in the Side Faces of Deep Reinforced Concrete Beams", American Concrete Institute Journal, Sept.-Oct. 1980, pp. 307-313.

2.10 Collins, M.P. and Kuchma, D., "How Safe are our Large Lightly Reinforced Concrete Beams, Slabs and Footings?", ACI Structural Journal, Vol. 96, No. 4, July 1999, pp. 482-490.

2.11 Sritharan, S. and Fenwick, R.C., "Creep and Shrinkage Effects in Prestressed Beams", Magazine of Concrete Research, Vol. 47, No. 170, Mar. 1995, pp. 45-55.

2.12 Beeby, A.W., "The Prediction of Crack Widths in Hardened Concrete", The Structural Engineer, Vol. 57A, No. 1, Jan. 1979, pp. 9–17

A3 2.13 Bamforth, P.B. "Early-Age Thermal Crack Control in Concrete", Ciria Report C660, 2007.

2.14 Bryant, A.H., Wood, J. and Fenwick, R.C., "Creep and Shrinkage in Concrete Bridges", RRU Bulletin No. 70, National Roads Board, Wellington, 1984, p. 103.

2.15 ACI Committee 224, "Control of Cracking in Concrete Structures", ACI 224R-90, Manual of Concrete Practice, Part 3, 1993 (probably available in later editions).

2.16 Mindess, S., Young, F.J. and Darwin, D., "Concrete", Prentice Hall, second edition, 2002.

A2 2.17 Federation international du beton (FIB), "Structural Concrete - the text book on Behaviour, Design and Performance", Vol. 2, FIP, Lausanne, 2000.

2.18 ACI Committee 215, "Considerations for Design of Concrete Structures Subjected to Fatigue Loading Manual of Concrete practice, Pt. 1, 1992 and later editions.

2.19 Hawkins, N.M., "Fatigue Design Considerations for Reinforcement in Concrete Bridge Decks", Journal American Concrete Institute, Proceedings, Vol. 73, No. 2, February 1976, pp. 104-114.

2.20 Fenwick, R.C., Dely, R. and Davidson, B.J., "Ductility Demand for Uni-directional and Reversing Plastic Hinges in Ductile Moment Resisting Frames", Bulletin of NZ National Society for Earthquake Engineering, Mar. 1999, Vol. 32, No. 1, pp. 1-12.

2.21 Priestley, M.J.N. and Kowalsky, M.J., "Aspects of Drift and Ductility Capacity of Rectangular Cantilever Walls," Bulletin of NZSEE, No. 2, June 1998, pp. 73-85.

2.22 Priestley, M.J.N., "Brief Comments on Elastic Flexibility of Reinforced Concrete Frames and Significance to Seismic Design", Bulletin NZSEE, Vol. 31, No. 4, Dec. 1998, pp. 246-259

2.23 Fenwick, R. and Bull, D., "What is the Stiffness of Reinforced Concrete Walls", Journal of Structural Engineering Society of NZ, Vol. 13, No. 2, Sept. 2000, pp. 23-32, and discussion by Paulay, T. and Priestley, M.J.N., SESOC Journal, Vol. 15, No. 1, Apr. 2002, pp. 30-41.

2.24 Fenwick, R., Hunt, R. and Bull, D., "Stiffness of Structural Walls for Seismic Design", Journal of Structural Engineering Society of NZ, Vol. 14, No. 2, Sept. 2001, pp. 17-21.

2.25 Davidson, B.J., Bell, D.K. and Fenwick, R.C., "The Development of a Rational Procedure to Determine Structural Performance Factors", University of Auckland, School of Engineering Report, No. 609, May 2002, p. 36.

2.26 Park, R., "Use of the New Grade 500E Reinforcing Steel", SESOC Journal, Vol. 14, No. 1, April, 2001, pp. 29-31.

2.27 Paulay, T., "A Note on New Yield Stress", SESOC Journal, Vol.13, No. 2, Sept. 2000, pp. 43-44.

2.28 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced and Masonry Buildings", Published John Wiley & Sons, 1992, p. 744.

2.29 Bull, D. and Allington C., "Overstrength factor for Pacific Steel Micro-Alloy Grade 500 Reinforcement: April 2002" SESOC Journal No. 1, No. 16, April 2004, pp. 52-53.

2.30 Park, R., Billings, I.J., Clifton, G.C., Cousins, J., Filliatrault, A., Jennings, D.N., Jones, L.C.P., Perrin, N.D., Rooney, S.L., Sinclairt J., Spurr, D., Tanaka, H. and Walker, G., "The Hyogo-ken Nambu Earthquake of 17th January 1995," Bulletin of NZSEE, Vol. 28, No. 1 Mar. 1995, pp. 1-98.

2.31 Booth E. Ed., "Concrete Structures in Earthquake Regions: Design and Analysis", Longman Scientific and Technical, 1994, p. 380.

2.32 Skinner, R. I., Robinson, W. and McVerry, G., "An Introduction to Seismic Isolation", John Wiley and Sons Ltd., West Essex, England, 1993.

2.33 NZNSEE, "Papers Resulting from Deliberations of the Society's Discussion Group on the Seismic Design of Bridges", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol 13, No. 3, September 1980, pp. 226-307.

2.34 Priestley, M.J.N., Seible, F. and Calvi, G.M., "Seismic Design and Retrofit of Bridges", John Wiley and Sons, 1996, p 686.

2.35 Glogau, O.A., "Separation of Non-structural Components in Buildings", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 9, No. 3, September 1976, pp. 141-158.

2.36 Allardice, N.W., "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames", Section K: Parts, Portions and Secondary Elements", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 2, June 1977, pp. 102-105.

2.37 Fenwick, R. and Dhakal, P.D., "Material Strains and Relevance to Seismic Design" and "Material Strain Limits for Seismic Design of Concrete Structures", SESOC Journal, Vol. 20, No. 1, Apr. 2007, pp. 5-28. A3

2.38 Fenwick, R.C. and Megget, L.M., "Elongation and Load Deflection Characteristics of Reinforced Concrete Members Containing Plastic Hinges", Bulletin NZ National Society for Earthquake Engineering, Vol. 26, No.1., Mar. 1993, pp. 28-41.

2.39 Royal Commission, Canterbury Earthquakes, Vol. 2 – Performance of Christchurch CBD Buildings, 2012, Pages 168-175.

2.40 Restrepo-Posada, J.I., "Seismic Behaviour of Connections Between Precast Concrete Elements", PhD thesis, University of Canterbury Report, 93-3.

2.41 Paulay, T., Priestley, M.N.J. and Synge, A.J., "Ductility in Earthquake Resisting Squat Shear Walls", Journal ACI, Vol. 79, No. 4 July 1982, pp. 257-269.

NOTES

C3 DESIGN FOR DURABILITY

C3.2 Scope

C3.2.1 *Concrete*

The minimum concrete strength considered in this design Standard is 20 MPa. Compressive strengths of 17.5 MPa may be used under the non-specific design Standards NZS 3604, NZS 4229 and NZS 3124, which have their own durability requirements. A3

C3.2.3 *Design considerations*

This clause sets out the procedure for design for durability to protect the reinforcement which involves the determination of the exposure classification followed by consideration of concrete quality, cover and chemical content.

This Standard recognises corrosion of reinforcement to be the most common and obvious form of durability failure. This may manifest itself as any one of, or a combination of, surface staining; cracking along reinforcement close to a surface and associated spalling of a surface; or it may proceed undetected.

The following simplified explanation of the reinforcement corrosion process will assist users in understanding the basis of measures provided in this Standard to prevent this type of durability failure.

For simplicity, the process of corrosion can be divided into 2 phases – initiation and propagation. Generally the reinforcement is protected against corrosion by the alkalinity of the concrete surrounding it. The initiation phase is considered to be the period over which this protection is reduced to the level where active corrosion can commence. The propagation phase is considered to be the period from commencement of corrosion to the stage where corrosion products cause a failure in the surrounding concrete.

In the initiation phase, the protection afforded by the alkalinity of the concrete can be reduced by two processes – carbonation (neutralisation of the high pH by infiltration of atmospheric carbon dioxide, a slow, continuous process) and chloride ingress.

In the propagation stage, the reinforcement will corrode at a rate which depends on the type of cementitious binder, the availability of oxygen and moisture, the temperature of the concrete, the presence of reactive ions and residual alkalinity.

It is generally recognised that fine cracks in concrete do not significantly affect corrosion initiation of embedded reinforcement but larger crack widths may cause premature corrosion activity locally. Reference 3.1 considers that corrosion is not affected by crack widths less than 0.4 mm.

Chloride ions can be introduced into the concrete by way of admixtures, contaminated aggregates, salt depositions on reinforcement and formwork, or they can permeate into the hardened concrete during acid etching or from salt spray deposited on the member surface. Limitations are placed on the quantity of chlorides which can be introduced into the fresh concrete (see 3.14).

The procedure given in the Standard for durability design involves firstly classifying the severity of the environment to which the concrete surfaces are exposed. For that exposure classification, a minimum concrete quality is specified by strength, and a minimum cover is then required for the reinforcement to be protected against corrosion. The onset of corrosion is influenced by chloride concentration at the concrete/reinforcement interface. The basic principle is that where corrosion of the reinforcement, once initiated, is likely to be rapid, then higher levels of protection are required. More severe environments require increasingly better protection and this is reflected by the requirement for better quality concrete and larger covers.

Because strength can be easily specified and measured, the specified compressive strength, f'_c has been adopted as the compliance criterion in most cases. However it should be remembered that f'_c as represented by standard cured cylinders, is at best only an indirect measure of concrete quality in place from a durability viewpoint, reflecting only the quality of concrete after 28 days curing in a fog room at 21 °C. This amount of curing in practice is seldom achieved on the site.

Research has shown the importance of early, continuous curing and this is the basis for the curing requirements for concrete in the various exposure classifications (refer to 3.6). After initial curing, further improvement in concrete properties due to exposure to the weather is doubtful, being highly dependent on the orientation of the member and local climatic conditions. The curing specification requirements of NZS 3109 must be followed.

Appropriate covers for the given exposure classification depending on the chosen concrete quality are specified in 3.11.

C3.2.4 *Design for particular environmental conditions*

Requirements for chemical resistance, abrasion resistance and exposure to freezing and thawing are additional to the general requirements of 3.2.3. For example, a concrete floor to a blast freezer would have to satisfy both the requirements for abrasion resistance (refer to 3.9), and the requirements for freezing and thawing (refer to 3.10) in addition to the requirements given in 3.4 to 3.8.

C3.3 Design life

Durability is indirectly defined as the ability to withstand the expected wear and deterioration throughout the intended life of the structure without the need for undue maintenance. The expected wear and deterioration may include the influences of weathering, chemical attack and abrasion. It is a complex matter involving a large number of interrelated factors such as:

(a) Assessment of environmental loading;

(b) Attention to design details, including reinforcement layout, appropriate cover and provision for shedding of water from exposed surfaces. For example, the design geometry of structures in the marine tidal zone to prevent splash may have a much greater influence on durability than the concrete specification itself;

(c) Suitable mix design; and

(d) Correct construction practices including adequate fixing of reinforcement and the placing, compacting and curing of the concrete, all of which are important [3.2, 3.3, 3.4].

This Standard specifies minimum requirements for only some of these areas. Reference should also be made to NZS 3109 for basic requirements with regard to construction practices.

C3.3.1 *Specified intended life*

Major renovation may be considered as maintenance work necessary to maintain the serviceability of a structure to enable it to fulfil its functional requirements, which exceed 20 % of the replacement value of the structure. Normal maintenance may include some surface cracking and even some minor spalling.

The extent of maintenance which is required on a structure will depend on the environment to which it has been subjected and its vulnerability to deterioration. A proactive inspection/monitoring programme carried out by the owner will ensure that repairs take place to surface defects when they first appear, rather than leaving them until they become a major repair. Such a programme will ensure that the design life expectations are realised and preventive maintenance strategies may even extend the life beyond the intended life.

C3.4 Exposure classification

An important part of establishing life is the system of exposure classification. Most classifications focus on conditions leading to corrosion of reinforcement. Classifications for chemical resistance (XA) aim to protect attack on the concrete itself.

The applicable exposure class for a particular structure is clearly an issue for the designer to solve through the specified concrete quality and cover. The responsibility for ensuring that the design strength and cover are adequate to meet the durability requirements for the particular environment does not lie with the builder, contractor or the material supplier.

The Standard proposes a range of classifications, based primarily on experience, which depend on the type of structure. Exposure to tidal and splashing salt water is classified as C. The more moderate exposure of being permanently submerged in seawater is classified as B2. Despite the high content of sulphates and chlorides in seawater, an extra level of protection is provided by the formation of an impermeable surface layer of carbonates, and the lack of dissolved oxygen, particularly at depth. Structures occasionally subject to direct contact by the sea should be assessed by the designer as to the most appropriate classification of B2 or C.

Contact with liquids is a difficult area in which to provide firm classifications. Fresh water can cause significant leaching of the partly soluble concrete components, as can repeated exposure to condensation. Running water and frequent wet and dry cycles in water-retaining structures can also cause physical and chemical degradation. These problems become additive to those associated with reinforcement corrosion.

In potable water situations, the Langelier Index for determining the softness of water can be used. This index is an evaluation used in water engineering and considers the corrosive nature of water by examining the water pH in relation to the presence of calcium and other dissolved solids. The aggressive nature of the water increases as the index moves from zero to a negative value. A value of –1.5 would be viewed as being significantly aggressive to concrete. For non-potable water conditions special evaluations as to the relevance of the Langelier Index value would need to be made. An alternative reference for considering the potential corrosive nature of water is given in Reference 3.5.

Definitions of environmental conditions have been derived from the general concepts followed by AS 3600. The XA classification has been taken from European Standard EN 206 [3.6]. The classifications may be summarised as follows:

(a) **Exposure classifications A1 and A2** – relatively benign environments, such as in the interior of most buildings, or in inland locations, remote from the coast, where the provision of adequate cover will give satisfactory performance. Life is generally based on carbonation resistance.

(b) **Exposure classification B1** – moderately aggressive environment forming the coastal perimeter. The extent of the affected area varies significantly with factors such as onshore wind patterns, topography and vegetation. Reinforcement protection can be satisfactorily provided by a combination of appropriate concrete quality and associated cover. Life is generally based on carbonation resistance.

(c) **Exposure classification B2** – aggressive environment such as locations 100 m to 500 m from an open sea frontage where salt spray is carried by onshore winds. Typical environmental recommendations are given in Table 3.2. The designer must, in consideration of the local site conditions, determine the appropriate classification of exposure. Life is based on chloride resistance of the cover concrete.

The recommendations in Table 3.1 are based upon durability studies on metals carried out by BRANZ [3.7, 3.8].

The influence of wind patterns in relation to an open sea frontage are particularly important in considering specific site evaluations permitted in 3.4.2.4.

Site evaluations may be further enhanced with wind frequency data which is available from the National Institute of Water and Atmospheric Research Ltd. The data is collected from over 100 weather stations throughout New Zealand and is presented as a 10-year average wind rose analysis.

A tidal estuary situation would be one example where a special evaluation is advisable.

An exposed steel corrosion rate of 150 g/m^2/year was used to delineate the boundary between zones A2 and B1. A typical corrosion rate for a B2 zone was 180 g/m^2/year. These figures are based upon the general corrosion studies undertaken by BRANZ [3.7, 3.8, 3.9].

It should be noted that exposed steel is used to determine a corrosion risk only. These corrosion figures have no direct relationship to calculating corrosion rates of reinforcing steel protected within a concrete member.

(d) **Exposure classification C** – the most aggressive chloride based environment for which guidance on concrete quality and cover is given. This classification includes offshore environments as well as open sea frontages with rough seas and surf beaches where significant salt spray is carried by onshore winds. Typical environmental recommendations are given in Table 3.2. The designer must, in consideration of the local site conditions, determine the appropriate classification of exposure.

Site evaluations may be further enhanced with wind frequency data which is available from the National Institute of Water and Atmospheric Research Ltd.

(e) **Exposure Classification XA** – Concrete is susceptible to attack from a number of different chemicals but acid attack is the most common due to the alkaline nature of concrete. Acids can combine with the calcium compounds in the hydrated cement paste to form soluble materials that are readily leached from the concrete to increase porosity and permeability. The main factors determining the extent of attack are the type of acid, its concentration and pH.

(f) **Exposure classification U** – these are environments for which the Standard gives no guidance. They may be more severe than exposure classification C, or as benign as exposure classification A1. For these the designer has to quantify the severity of the exposure along the above lines and choose methods of protection appropriate to that exposure. Classification U also applies to elements where the design life is either less or greater than 50 years. In such a case, specific assessment of materials construction practices, environment and required performance etc, must be undertaken.

A marine exposure situation, where chloride ions enter the concrete by hydrostatic pressure, for instance in a tunnel which is immersed in sea water or saline ground, is classification U. In this situation chloride builds up on the inside, remote to the face in saline contact, so called ‘wicking action’. The chloride concentration on the inside will depend on the hydrostatic pressure, and the permeability of the concrete as well as the air flow influencing evaporation on the inside face.

The durability of steel fibre concrete is not specifically addressed in this Standard. Steel fibres at or near the surface will corrode and cause brown stains on the surface. However the flexural strength of members is not effected by such staining [3.10].

C3.4.3 *Chemical exposure classification*

The action of acids (as an aggressive substance) on the hardened concrete (as a reactive substance) is the conversion of the calcium compounds (calcium hydroxide, calcium silicate hydrate and calcium aluminate hydrate) of the hydrated Portland cement, to the calcium salts of the attacking acid. For example, the action of sulphuric acid gives calcium sulphate, which precipitates as gypsum.

The rate of reaction of different acids with concrete is determined not so much by the aggressiveness of the attacking acid, but more by the solubility of the resulting calcium salt. If the calcium salt is soluble, then the reaction rate will be determined largely by the rate at which the calcium salt is dissolved. Factors that influence the rate of attack are:

- The concentration and type of sulphate and the pH in soil and groundwater;
- The water table and the mobility of the groundwater;
- The compaction, cement type and content, type of aggregate, water/binder ratio and curing regime of the concrete;
- The form of construction;
- C_3A content of the cement as well as its C_3S to C_2S ratio;

- Frost: concrete below ground is unlikely to be affected by frost but the combination of sulphate and frost attack represents particularly severe conditions.

A precondition for chemical reactions to take place within the concrete at a rate, which has any importance in practice, is the presence of water in some form (liquid or gas) as a transport mechanism. The accessibility of the reactive substance in the concrete is therefore the rate-determining factor when an aggressive substance enters. For practical purposes, this is often translated into limiting values for w/c ratio. The rate-increasing factor of increasing temperature is mainly due to the effect on the transport rate (higher temperatures result in higher mobility of ions and molecules). Depending on the type of reaction, the accessibility will be determined by the permeability of still sound concrete or by the passivating layer of the reaction products. The most important chemical reactions that may lead to concrete deterioration are:

- The reaction of acids, ammonium salts, magnesium salts and soft water with hardened cement;
- The reaction of sulphates with the aluminates in the concrete;
- The reaction of alkalis with reactive aggregates in the concrete.

C3.4.3.1 *Chemical attack from natural soil and groundwater*

The Standard focuses on natural groundwater and soils with high acid or sulphate levels which can attack concrete in a rapid and destructive manner.

The Baumann-Gully soil acidity is a measure of the content of exchangeable hydrogen ions which the humus component of the soil is capable of releasing.

One of the factors which affects the rate of acid attack is the reserve acidity of the soil, i.e. the volume of concrete a given amount of soil can neutralise given a long enough period of time. The reserve acidity of the soil is a function of stagnant, medium or flowing soil classification which depends on the groundwater flow rate and the soil type (extremes are stagnant heavy soils such as clay with little or no groundwater movement, permeability less than 10^{-5} m^3/s; sandy and flowing permeable soil combined with a significant flow rate of groundwater)[3.11].

C3.4.3.2 *Other chemical attack*

Biological processes on the surface of concrete can result in both mechanical and chemical deterioration of the surface [3.2]. This can be particularly severe in moist warm surface conditions.

As well as occurring naturally, sulphuric acid and sulphates in acid solution are frequently present in industrial wastes. Hydrogen sulphide induced corrosion is one of the major deterioration mechanisms for concretes subjected to industrial and domestic sewerage. Where possible and only when the right conditions prevail, some of the sulphides escape into the sewer atmosphere in the form of hydrogen sulphide gas H_2S, which dissolves in condensed moisture on the concrete surfaces. It is then oxidised by sulphur oxidising bacteria to produce sulphuric acid H_2SO_4, which is extremely aggressive to the concrete matrix. The sulphide corrosion mechanism of concrete is twofold, where the first phase is direct acid attack, and the second phase constitutes sulphate attack causing further expansion deterioration mechanisms.

The emission of certain pollutants by industry is known to increase the risk of degradation of the concrete or corrosion of reinforcement. It is impossible to define within the Standard all industrial processes. Designers must consider the individual industrial processes applicable to the design to determine whether an industrial classification of B1 should be upgraded to classification U requiring special conditions to apply. Industrial plants burning sulphide containing fuels, or emitting acidic gases, may be considered a severe risk and subject to the "industrial'' classification. A limit of 3 km represents a reasonable estimate but engineering judgement should be used depending on the nature and scale of the industrial pollutants and the prevailing wind directions. Structures located in areas of geothermal activity should be regarded as having an industrial exposure classification.

In situations where the concrete is exposed to attack by concentrated sulphuric acid, such as in some structures associated with waste water reticulation and treatment, the normal provisions of specifying a low water to binder ratio, high binder content or supplementary cementitious materials (SCMs) might not be adequate in providing the intended service life. Use of a sacrificial concrete surface layer or A3

A3 incorporating calcareous aggregate will extend service life, as will the use of proprietary binder systems such as calcium aluminate cement instead of Portland cement in the surface layer. Under some conditions some form of physical barrier protection such as a chemical-resistant coating or lining might be necessary to protect the concrete adequately. Such material should be designed for the intended application and installed and maintained in accordance with the manufacturer's specifications. Reference 3.30 describes a New Zealand case study.

Thaumasite sulphate attack (TSA) is a special type of sulphate attack applicable to buried concrete structures. Concretes using limestone aggregates or a ground limestone binder may be susceptible to TSA. The process is accelerated at low temperatures [3.12].

Some SCM concretes provide enhanced acid resistance to Portland cement concretes.

The effect of various common substances on concrete floors is given in Reference 3.13.

C3.5 Requirements for aggressive soil and groundwater exposure classification XA

Requirements for concretes subjected to natural aggressive soil and ground water attack can be summarised as follows:

- A3 Use of low water to binder ratio concretes with SCMs in appropriate mixes reduces concrete surface permeability and porosity, and thereby increases acid resistance. Where low pH and high exchangeable soil acid conditions prevail, specifying a low water to binder ratio, high binder content, binder containing SCM, calcareous aggregates and/or a sacrificial layer may be inadequate. Under such conditions some form of physical barrier protection may be necessary. Such barrier materials should be designed for the intended application and installed and maintained in accordance with the manufacturer's specifications.
- Care is needed in aggregate selection. Calcareous aggregates being an acid neutralising buffer zone should be considered especially in sacrificial layers. Increasing concrete cover will also prolong the life of the concrete elements.
- Crack limitations need to be considered in design of structural and hydraulic concrete elements.

In some of the special conditions that may arise in this category, e.g. for low pH < 4.0, the use of special chemical resistant coatings over the structural concrete may provide a more favourable design solution. Refer to 3.12.2.

C3.6 Minimum concrete curing requirements

Adequate curing is critical to achieve the required durability performance. The cover depth and concrete envelope quality have a direct relationship to the corrosion risk, and hence if curing is compromised, this will effect durability more than compressive strength. Curing is a very important aspect of securing satisfactory durability performance and hence any comparative tests undertaken must realistically model practical curing regimes which are compatible with *in situ* or precast concrete production.

The requirements for concrete are a minimum strength and an initial curing period equal to or greater than three days for exposure classification A1, A2, or B1 and seven days for exposure classification B2 and C. The reduction of capillary channels and their interconnection within the concrete is primarily influenced by water cement ratio and subsequent curing period. For concretes in the C Zone the higher constituent content of cement paste hydration products provides greater subsequent chemical binding action when faced with chloride diffusion. The low w/c ratio also requires that there is free water available on the surface for curing, so as to prevent self desiccation of the concrete surface. The use of active curing procedures such as ponding or continuous sprinkling, or continuous application of a mist spray is recommended. Curing membranes or polythene sheet curing should not be used for the C zone.

Where special curing features, such as might be used in a precast concrete factory, then comparison with the prescriptive requirements for strength and cover may lead to combination adjustments of these values.

Accelerated curing generally has a detrimental effect on durability, this is more significant for SCM concretes. Thus seven days water curing is still recommended after the completion of the accelerated curing cycle.

C3.7 Additional requirements for concrete for exposure classification C

C3.7.1 *Supplementary cementitious materials*

Concretes containing supplementary cementitious materials are necessary to provide the performance required in the C zone.

NZS 3122 and AS 3582 provide methods of evaluating cementitious materials but provide no guarantee of durability performance. The specifier should verify the performance of specific materials with the supplier.

In considering the alternative binder types, attention must be given to any changes in construction techniques. For example, some concretes will require longer curing regimes than currently specified for GP cement concrete to achieve satisfactory durability performance and the effect of the curing temperature can be more critical. Evaluation of alternative cement types does require significant project lead-in time if test results are not available.

The use of these alternative cementitious materials requires evaluation which should include the aspects of concrete supply, placement and curing. While many of the evaluations represent an accelerated testing regime compared to the life performance of the concrete, the test procedures themselves are normally extended for a period of months before final results can be established. Consequently the use of materials, where the evaluation has not previously been satisfactorily completed, requires significant project lead-in time to allow for evaluation. Durability testing is typically carried out after 56 days curing, recognising the fact that some SCM concretes take longer than 28 days to reach optimum hydration.

The principal chemical process of aging for exposure zones A1 and A2 is carbonation. Performance criteria could be demonstrated by using such tests as:

- Absorption
- Sorptivity
- Accelerated carbonation testing

For exposure zones B1, B2 and C there is an increasing dominance of the influence of chloride ions penetrating the concrete. However the ingress of chlorides in the near surface zone in still influenced by absorption as this surface concrete undergoes wetting and drying cycles. Absorption and sorptivity testing is most appropriate for evaluating this surface zone.

Deeper into the concrete, chloride transport is dominated by diffusion driven by a chloride concentration gradient. Fick's second law of diffusion is universally used to model this ingress of chloride.

Whilst it is recognised that chloride ingress is driven by a number of factors including diffusion, the use of this one equation simplifies matters somewhat from reality.

There is no single internationally accepted test for chloride ingress [3.14] but the test methods listed below are recommended for comparative performance purposes.

- NT BUILD 443 Chloride Diffusion Test
- NT BUILD 492 Rapid Migration Test

Chloride ingress over time is complicated by the fact that the diffusion coefficient, a material property measure, reduces with time. This is more significant for concretes containing SCMs and in fact dominates the long-term chloride ingress and lessens the influence of the early-age chloride diffusion. The surface chloride concentration is also not constant and has been demonstrated to show an increase over at least the first 15 years of exposure[3.27]. For this reason comparative chloride diffusion measurements on young concretes are not necessarily an indicator of long-term performance. A3

A3 Thus as well as the short-term tests above, site exposure studies such as that carried out by BRANZ[3.15] and Opus[3.27] are important in evaluating the time reduction factor. By measuring the ingress of chlorides into concrete with time for different exposure situations, the reduction in chloride diffusion can be calculated and the time reduction factor determined for concretes incorporating different cementitious types.

The Rapid Chloride Test ASTM C1202 has been used extensively in both research and for quality control yet it has some shortcomings [3.16, 3.17]. These relate to its bias towards some cementitious types based on the chemical make up of the pore water inside the concrete test specimen. The test is therefore most appropriately applied to comparative testing for quality control purposes of one concrete containing the same cementitious binder type. The Rapid Migration Test NT BUILD 492 overcomes the shortcomings of the Rapid Chloride Test and uses the same basic equipment, so is regarded as a superior quality control test [3.18].

One of the most effective ways of controlling the quality of a concrete containing supplementary cementitious materials is to monitor concrete performance in the course of a contract relative to performance levels established in a pre-contract trial placement. This approach requires lead times of up to 3 months before a contract commences to allow the required testing to be carried out.

The suggested approach is as follows:

(a) Carry out a trial placement of concrete in the form of a slab or other appropriate shape to confirm workability and placeability of concrete.
(b) Carry out Chloride Diffusion Test NT BUILD 443, Rapid Migration Test NT BUILD 492 and compressive strength testing on cylinders removed from the trial concrete as supplied. Confirm that the Chloride Diffusion Test result indicates the concrete will provide the required durability. Monitor concrete quality in the course of the contract using a combination of compressive strength testing and the Rapid Migration Test NT BUILD 492 with acceptance based on the results achieved in the pre-contract trial. An appropriate margin for variability in these parameters will need to be established for each different contract. For example, 0.85 of the compressive strength achieved in the trial has been used as an acceptance level for some contracts.
(c) Carry out Chloride Diffusion Test NT BUILD 443, Rapid Migration Test NT BUILD 492 and compressive strength testing on concrete cores removed from the trial placement. If in the course of the contract there is dispute about the performance of concrete already in place these results can be used to establish acceptance levels for cores removed from the concrete in question. There is likely to be greater variability in the test results from cores compared to cylinders.

Refer also to Reference 3.19.

C3.7.2 *Water/binder ratio and binder content*

The strength and curing requirements for the C zone are based on the use of concrete containing a supplementary cementitious material with a minimum f'_c, a maximum water binder ratio of 0.45, and a minimum binder content of 350 kg/m^3. The achievement of these parameters together with providing concrete of a suitable workability for placement and compaction will require controlled use of chemical admixtures.

Concrete supplied for the C zone will be classed as ‘Special Concrete’ in accordance with NZS 3109 Amendment No. 1 August 2003 and NZS 3104. Consequently there may be special testing requirements over and above the routine compressive strength testing carried out by the concrete supplier. Any additional testing requirements would need to be discussed with the concrete supplier. If any special durability testing is required, the lead time may be significant.

C3.8 Requirements for concrete for exposure classification U

For more severe cases than C, the principal actions call for the use of low water binder ratios approaching 0.3. In such situations construction techniques are critical. Additional durability enhancement measures as are detailed in C3.12.2 should be considered.

C3.9 Finishing, strength and curing requirements for abrasion

Achieving adequate abrasion resistance for concrete floors depends primarily on the effective use of power trowels on the concrete as it sets, effective curing and then to a lesser extent on the cement content [3.20, 3.21].

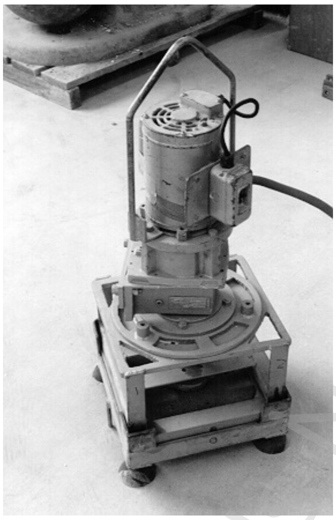


Figure C3.1 – Accelerated abrasion machine

Compressive strength is the most easily measured way of ensuring a minimum cement content. The UK Concrete Society Publication TR34 recommends a minimum cement content of 325 kg which will normally be achieved with a 40 MPa concrete.

The water cement ratio is of importance. It should not exceed 0.55. Reducing this to 0.5 is likely to enhance abrasion resistance but reducing this further is unlikely to enhance resistance. The water cement ratio for a particular strength can vary widely around New Zealand dependent on the aggregate and sand.

Coarse aggregate usually has no direct influence on the abrasion resistance, except in floors in very aggressive environments where the surface is likely to be worn away. Coarse or fine aggregates should not contain soft or friable materials.

Class AR1 and Special are likely to require the use of a dry shake finish.

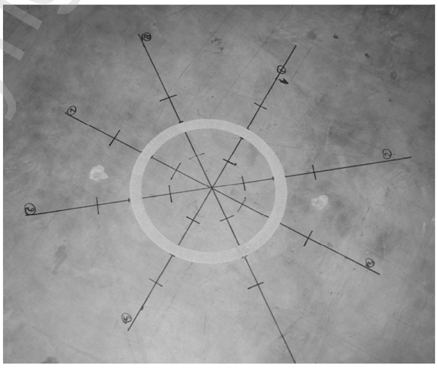


Figure C3.2 – Accelerated abrasion wear circle

A test method for assessing the abrasion resistance of concrete floors is described in BS 8204-2, Annex B, Determination of the abrasion resistance value. This test utilises a rolling wheel abrasion testing machine which leaves a 225 mm wide circle on the floor up to 1 mm deep. Thus in warehouses it might be appropriate to test under rack positions. The test should be seen as a way to determine if a construction process in conjunction with a concrete mix is suitable.

Table C3.1 – Relationship between class and test wear depth centre

Class	Maximum test wear depth (mm)
Special	0.05
AR1	0.10
AR2	0.20
AR3	0.40

Reference 3.20 outlines New Zealand research using the Accelerated Abrasion Machine.

This clause specifies additional requirements for abrasion exposure over and above the requirements for other exposure criteria. For example, concrete for a reinforced concrete external pavement subject only to light, pneumatic tyred traffic, but located in the coastal zone would have to comply with the requirements for B2 and those requirements would take precedence. On the other hand for an internal factory floor subject to medium to heavy pneumatic tyred traffic, the requirements for abrasion under this clause would take precedence.

Floors which receive repeat power trowelling can exhibit ‘craze cracking.’ In general this is cosmetic only and has no effect on abrasion resistance.

C3.10 Requirements for freezing and thawing

The role of air entrainment in providing resistance to freeze-thaw degradation is well established and this clause presents the usually accepted values. Those given represent an envelope of accepted practice. In general the larger the nominal aggregate size the lower the required amount of entrained air to give the desired protection.

Severity of exposure is also dependent on the presence of moisture on the surface prior to freezing.

If the surface is also subject to abrasion the upper values of air entrainment given may be too high to permit the desired abrasion resistance to be achieved; if so an intermediate value will have to be chosen.

A3 The calculation of the number of cycles of freezing needs to consider that when concrete is cooled below zero degrees immediate freezing of water in the cement paste will not occur. The size of pores in the concrete are important, for example for pore sizes of 10 nm water will not freeze until −5 °C [3.28]. Reference to the Meteorological Service of New Zealand can provide some indication of weather freeze-thaw patterns. Provided in Table C3.2 are average numbers of cycles per annum in which the temperature falls below −5 °C. Measurements at screen level are taken at 1.3 m above ground and are considered applicable to buildings. When the expected number of cycles exceeds 25 or 50 occasions, as appropriate, structural concrete should meet the provisions of (a) and (b) of 3.10. Increasing the concrete strength assists in reducing the pore size of the concrete and increasing the tensile strength.

When exposed aggregate building surfaces, vertical or horizontal, are used, it is recommended that durability provisions follow the structural ground slab provisions.

A3 **Table C3.2 – Examples of frost cycles below –5 °C (New Zealand Meteorological Service)**

	Frost cycles per annum	
	Screen	**Ground**
Christchurch	1.1	23.7
Dunedin	0.05	7.83
Invercargill	1.64	22.6
Queenstown	0.6	26.52

C3.11 Requirements for concrete cover to reinforcing steel and tendons

C3.11.2 *Cover of reinforcement for concrete placement*

Larger covers than those given in the Standard may need to be specified for other reasons; for example, the achievement of required surface finish, allowance for abrasion of surface, the use of bundled bars, the congestion due to a number of reinforcement layers, the configuration of narrow webs and large prestressing ducts and the influence of aggregate size.

Concrete which can achieve adequate compaction without vibration, such as self compacting concrete may allow the use of closer bar spacings.

C3.11.3 *Cover for corrosion protection*

C3.11.3.1 *General*

The protection of the reinforcement is provided by a combination of concrete quality and thickness of cover.

In 3.11.3.2 and 3.11.3.3, the covers quoted assume that placing tolerances specified in NZS 3109 are met. If there is doubt that these can be achieved on the project, then larger covers should be specified to allow for increased tolerance. In addition, covers will need to be increased where special concrete surface finishes reduce the nominal dimensioned cover.

C3.11.3.2 *Formed or free surfaces*

In general, covers increase as the severity of the exposure increases. Provision has been made to permit reduced covers in situations where concrete grades higher than the minimum specified for the exposure classifications are used.

In Table 3.6 and Table 3.7, a default minimum cover of 50 mm has been used for the C Zone. Also covers in all exposure classifications do not reduce for strengths above 60 MPa. It is considered that default minimum covers need to be maintained to allow some buffer to offset the risk of inadequate workmanship not achieving the design covers.

C3.11.3.3 *Casting against ground*

The increase in cover requirements relates to the casting of items directly against the ground i.e. not against formwork constructed to NZS 3109. Where blinding concrete or sand blinding treatment of a base course has been used to produce a surface similar in tolerance to formwork to NZS 3109, then the cover requirements may be determined by direct reference to Table 3.1 and Table 3.6 or Table 3.7.

C3.12 Chloride based life prediction models and durability enhancement measures

C3.12.1 *The use of life prediction models*

Concretes containing supplementary cementitious materials have the potential to provide enhanced marine durability when compared to GP cement concretes. Because of the complexity of marine durability predictions, there are a number of predictive models available to determine design life of the marine derived concrete structures, which have application to the B2 and C zones in particular. Table 3.6 or Table 3.7 may be more conservative than alternative solutions derived using a model. Models should incorporate factors of safety on the calculated design life to allow for uncertainty in input values.

A model is a powerful design tool where the designer is able to evaluate the effect of the various variables of the predicted design life. However, there is an increased risk in using a model, if the designer does not have sufficient knowledge on the appropriate input data, then the resulting output could be spurious.

There are several physical/mathematical models [3.22] which offer predictions of the service life of reinforced concrete structures subject to chloride environments. Certain of these models are concerned only with the so-called "initiation phase" (time to first rusting of the steel reinforcement). Others deal with the subsequent "propagation phase" (time to first cracking or spalling of the concrete). Reference 3.23

provides the European derived methodology for a performance based durability design based on a probability approach.

The various combinations of specified concrete strength and minimum cover requirements for Exposure Classification C as listed in Table 3.6 and Table 3.7 were determined with reference to a number of different model solutions taken together with marine exposure site data obtained from an on-going BRANZ research programme on chloride ingress. In the light of various uncertainties concerning the current state of knowledge in relation to the modelling of the propagation phase, the Committee adopted an approach based on securing a minimum time to first rusting of 40 years and 80 years for the Specified Intended Life of 50 and 100 years respectively.

A3 The initiation models are usually founded on some derivative of Fick's Law of diffusion. Experience has shown that the chloride profiles which develop within a body of concrete exposed to a chloride environment can generally be fitted using a Fick's Law type expression. The mathematics of obtaining semi-empirical solutions for different boundary conditions including diffusion coefficient and surface chloride concentration are provided in Reference 3.29. However, the resulting diffusion coefficients (often termed "effective" diffusion coefficients in recognition of the fact that chloride ingress can be driven by a variety of different mechanisms) do not follow classical diffusion theory with respect to constancy over time. Thus, with increasing periods of exposure, effective diffusivity values tend to decrease. Such reductions in effective diffusivity over time can be especially marked for concretes containing supplementary cementitious materials. Initiation models generally cater for this feature by way of a power-law type expression with a power index. In addition recent research[3.27] has demonstrated that the surface chloride concentrations slowly increase and may take many years to stabilise.

The following values are offered as guidance to designers with respect to the use of initiation models. Some models have their own guidelines [3.22]:

Surface chloride levels

Exposure Classification C	2.0 – 3.5 % on mass of cementitious materials (MS)
	3.0 – 5.5 % on mass of cementitious materials (GBS)
Exposure Classification B2	0.8 – 1.0 % on mass of cementitious materials (GP)
	0.8 – 1.2 % on mass of cementitious materials (SCMs)

Time-dependency of chloride diffusion coefficients for C zone (m) (time reduction indices)

65 % GBS	0.35 – 0.74
8 % MS	0.15 – 0.51
GP	0.08 – 0.34

The time dependency coefficients have a significant effect on life prediction which can swamp the influence of other factors.

Chloride threshold for black steel corrosion

0.3 – 0.5 % on mass of cementitious materials

The choice of corrosion threshold should take into account the likely background chloride level present in the concrete.

Designers who choose to implement life-performance models as an alternative to the Exposure Classification C (or B2) provisions of Table 3.6 and Table 3.7 are urged to maintain a sensible approach with respect to nominated minimum covers. For this reason the committee recommends that the covers using a model solution should not be more than 10 mm below the corresponding value in Table 3.6 or Table 3.7.

A3 Durability modelling should be conducted by appropriately qualified and experienced persons. Reliance on chloride diffusion coefficients obtained from short-period exposure studies may result in erroneous conclusions if time-dependent diffusivity linked to time dependent surface chloride concentration is not appropriately considered.

C3.12.2 *Other durability enhancing measures*

There are a number of additional measures which can be used to enhance the durability of the 'standard' situation. It is recommended that these be used to increase life or the certainty of life prediction, rather than to reduce cover.

Protective surface coatings may be taken into account in the assessment of the exposure classification [3.13]. However, care should be exercised when assessing the ability of a surface coating to protect the surface and to continue to do so during the life of the building. The choice of a suitable coating is outside the scope of the Standard, but the designer should be warned that an inadequate poorly maintained coating may lead to more rapid degradation than no coating. Maintenance of coatings in the C zone is often impractical.

For systems relying wholly or partly upon coating systems, the New Zealand exposure life of the coating will need to be considered as a separate study as well as the contribution achieved from the concrete cover. High levels of UV light combined with humidity and temperature factors in New Zealand lead to more rapid deterioration of some coatings compared, for example, with European experience. Some coatings are acceptable for providing resistance to CO_2 penetration but provide little protection for chloride ion ingress. Hence it is essential to select the correct coating system that is compatible with the exposure classification.

It is important to ensure that the coating permits water vapour transmission.

International research has indicated that the following performance criteria need to be adopted when considering the suitability of a coating:

(a) Water vapour transmission resistance less than 4 metres of air barrier;
(b) CO_2 diffusion resistance greater than 50 metres of air barrier;
(c) No chloride ion diffusion using, for example, the Taywood method after 1 year's immersion;
(d) UV performance needs to be based upon a minimum of 5000 hours of accelerated weathering.

While coating manufacturers data may be indicative of the various performance ratings, it is important for the specifier to have regard to independent certification of information. This may be available from the coating manufacturer. AS/NZS 4548 gives test methods for evaluating coatings for water transmission resistance, water vapour transmission resistance, carbon dioxide diffusion, chloride ion diffusion and crack bridging ability. It must be appreciated that test evaluations will often taken 12 months for completion.

The indicative performance criteria of (a) and (b) above are based on tests performed by Klopfer, BRE (UK), Taywood Engineering and Aston University (UK).

The type of reinforcement used will have the potential to affect the corrosion threshold. The recommended corrosion threshold for conventional reinforcement is 0.4 % on the mass of cementitious materials. Equivalent recommended figures for galvanised reinforcement and stainless steel (316 or better) are 1.0 % and 3.0 % respectively. Epoxy coated reinforcement has given mixed results as regards durability enhancement with significant failures. The issue relates to keeping the coating intact during the construction.

Once galvanised steel becomes corrosively active, it will corrode very quickly. The propagation phase will therefore be shorter than for conventional reinforcement. The threshold level for stainless steel is about the same as typical surface chloride levels in the splash zone. Hence generally there are no cover requirements for stainless steel.

Controlled permeability formwork (CPF) changes the characteristics of the near surface zone through a reduction in water/ cementitious ratio. Reference 3.24 gives a method for calculating the depth of the affected zone and the D_c value of this zone is found on average 45 % less than concrete cast against conventional forms.

Permanent GRC formwork can be treated in a similar way to CPF.

A3 Corrosion inhibitors have the effect of increasing the level of chloride that can be tolerated before corrosion commences. Calcium nitrite is the most widely used inhibitor and the one for which there is the most data. Reference 3.24 describes how the corrosion threshold level is affected.

Integral waterproofers have the effect of reducing the surface chloride level.

Cathodic protection is a repair option which may be used to extend the life of a deteriorating structure by slowing down/halting the rate of corrosion of the reinforcement. For exposed structures where it is considered that there is some risk of corrosion during the life, cathodic protection may be provided at the construction phase (cathodic prevention). This allows the corrosion state of the reinforcement to be A3 monitored throughout the life of the structure. Most types of cathodic protection and prevention need ongoing maintenance during the life of the structure.

C3.13 Protection of cast-in fixings and fastenings

The following points should be noted when specifying corrosion protection:

(a) Hot-dip galvanising of mild steel can engender embrittlement in cold-worked sections, reducing ductility;

(b) Stainless steel, where attached to mild steel by welding, can promote crevice corrosion, or galvanic corrosion in contact areas;

(c) A number of non-ferrous metals may corrode by contact with Portland cement[3.25];

(d) For large fixings where use of stainless steel may be prohibitive, heavily galvanised fixings may be a viable option.

C3.14 Restrictions on chemical content in concrete

C3.14.1 *Restriction on chloride ion for corrosion protection*

The protection of reinforcement by the provision of an adequate cover of dense concrete relies primarily on the protection afforded by the alkalinity of the concrete. This protection will prevent the initiation of corrosion until carbonation has advanced close to the steel surface, which usually takes decades. However, if chloride ions are present in sufficient quantity, corrosion can be initiated even in an alkaline environment. Moreover chloride ions accelerate the corrosion process so their presence should be minimised.

When considering the effect of chlorides on corrosion it is necessary to distinguish between ‘’free” chloride present in the pore water and chloride bound by the cement in the matrix. The “bound” chlorides do not take part directly in corrosion, whereas the “free” chlorides may rupture the passive protective film on the surface of the reinforcing bars. “Free” chloride ions increase the electrical conductivity of the pore water and the rate of dissolution of metallic ions. Nevertheless as the proportion of “free” to “bound”’ chlorides is subject to change, and “bound” chlorides may go into solution, it is considered desirable to place limits on the total chloride content rather than just the “free” chloride content. For this reason limits were placed on the acid soluble chlorides, as determined by standard test, which are closely related to total chlorides.

Limits on chloride ion content are quoted as mass per m^3 of concrete which is consistent with the test method. Concrete producers producing to NZS 3104 should be able to verify that their concrete is within the allowable limits for chloride content.

C3.14.2 *Restriction on sulphate content*

An upper limit of 5 % of sulphur trioxide (SO_3) by mass of cement has been set to minimize the expansive influence of sulphate on the concrete. This includes the sulphate in the cement as well as aggregates and water. Great care should be taken when rock waste from mining is used as an aggregate. Many mineral ores include sulphides that oxidise to sulphates.

C3.14.3 *Restriction on other salts*

Some admixtures used in place of chloride accelerators may give rise to increases in ionised salts that may be detrimental. Compliance evidence to AS 1478 for admixtures should be sought.

C3.15 Alkali silica reaction, ASR A3

Most of the South Island and lower half of the North Island do not have reactive aggregates. ASR is only a potential issue if reactive aggregates are being used.

CCANZ TR3 (Reference 3.26) recommends procedures for minimising the risk of damage caused by alkali silica reaction in concrete made from reactive aggregates. For Normal Concrete produced in accordance with NZS 3104 it specifies maximum concrete alkali content. Concrete that exceeds the alkali limit becomes Special Concrete. A range of precautions is specified for Special Concrete depending on the risk associated with the specific structure, materials available, environmental conditions during construction and in service, and the specified design life. A3

REFERENCES

3.1 Francois R. and Arliguie G., "Effect of Microcracking and Cracking on the Development of Corrosion in Reinforced Concrete" Magazine of Concrete research 1999, 51, No. 2, April, pp. 143-150.

3.2 fib "Structural Concrete – Textbook on Behaviour, Design and Performance" – Vol. 3. Ch.5 "Durability", CEB Bulletin.

3.3 Cement and Concrete Association of New Zealand. "The Guide to Concrete Construction" TM 35 1999.

3.4 Concrete Institute of Australia "Durable Concrete Structures" CIA Z 7 2001.

3.5 Biczok, I., "Concrete Corrosion Concrete Protection", Hungarian Academy of Sciences, Budapest, 1992, p. 545.

3.6 EN 206–1:2000 , "Concrete – Part 1: Specification, Performance, Production and Conformity".

3.7 Building Research Association of New Zealand, "Atmospheric Corrosion in New Zealand", Report No. 38, pp. 4-8.

3.8 Building Research Association of New Zealand, "Atmospheric Corrosion Rates Over Two Years Exposure at 156 Sites in New Zealand", Reprint No. 112, 1992, pp 1-13.

3.9 Building Research Association of New Zealand, "Atmospheric Corrosion in New Zealand", Reprint No. 105, 1991.

3.10 Nemegeer, Dirk., Vanbravant, Johan., Stang, Henrik., "Final Report on the Durability of Steel Fibre Reinforced Concrete" under the Industrial and Materials Technologies Programme(Brite-Euram III)

3.11 BRE "Concrete in Aggressive Ground Special" Digest 1 Parts 1-4.

3.12 "Thaumasite Form of Sulphate Attack" UK Concrete February 1999, pp. 37-39.

3.13 Cement & Concrete Association of New Zealand "Concrete Ground Floors and Pavements for Commercial and Industrial Use – Part 1" Appendix C TM 26, 1999.

3.14 Concrete Institute of Australia "Recommended Practice; Performance Criteria for Concrete in Marine Environments" CIA Z 13 2001.

3.15 Building Research Association of New Zealand, "Durability of Concrete under Marine Exposure in New Zealand" D H Chisholm & NP Lee SR 126 (2005).

3.16 Geiker, M., Thaulow, N. and Andersen, P.J., "Assessment of Rapid Chloride Ion Permeability Test of Concrete with and without Mineral Admixtures". In Durability of Building Materials J.M. Baker, P.J. Nixon, A.J., Majumdar and H. Davis eds. E & FN Spon, London, 1990, pp. 493-502.

3.17 Trinh Cao, H. and Meck, E.A., "Review of the ASTM C 1202 Standard Test and its Applicacability in the Assessment of Concrete's Resistance to Chloride Ion Penetration". Concrete in Australia, October 1996, pp. 23-26.

3.18 Hooton, R.D., Thomas M.D.A. and Stanish K., "Prediction of Chloride Penetration in Concrete", FHWA-RD-00-142, October 2001. Federal Highway Administration, US Department of Transportation, Vancouver.

3.19 Cement & Concrete Association of New Zealand. "Specifying Concrete for Performance" TM 10 2003.

3.20 Cement & Concrete Association of New Zealand "The Abrasion Resistance of Industrial Concrete Floor Slabs" TR 08, 1994.

3.21 Sadegzadeh, Massud., "Abrasion Resistance of Concrete", University of Aston, Birmingham 1985.

3.22 Building Research Association of New Zealand, "Evaluation of Service-life Prediction Models for Concrete Durability" BRANZ Report DC-0750 for Standards New Zealand.

3.23 fib Model Code for Service Life Design (fib MC – SLD).

3.24 New Zealand Concrete Society, "Concrete Durability by Design: Limitation of the Current Prescriptive Approach and Alternative Methods of Durability Design" Keynote Address NZ Concrete Society Conference 2002 – Dr Phillip Bamforth. Technical Paper TR 27.

3.25 Cement and Concrete Association of New Zealand "Non-ferrous Metals" IB 45 2004.

3.26 Cement and Concrete Association of New Zealand "Alkali Silica Reaction", TR 3 2004.

A3 3.27 Opus International Consultants, "Durability of Marine Concrete under New Zealand Conditions", August 2012.

3.28 Mindess, S., Young, J.F. and Darwin, D., "Concrete", 2nd edition, 2003.

3.29 Frederiksen, J.M., Mejlbro, L. and Nilsson, L.-O., “Fick’s 2nd Law: Complete Solutions for Chloride Ingress into Concrete with Focus on Time Dependent Diffusitivity and Boundary Conditions” Report TVBM-3146. Lund Institute of Technology, Division of Building Materials. Lund, Sweden, 2009. A3

3.30 Bruce, S. and Freitag, S., “Durability of Concrete in Wastewater Reticulation and Treatment Facilities a Case Study”, NZ Concrete Industry Conference, NZ Concrete Society Technical Report TR29, 2005.

NOTES

C4 DESIGN FOR FIRE RESISTANCE

C4.1 Notation

A2

When reinforcement is arranged in several layers as shown in Figure C4.1, and where it consists of either reinforcing or prestressing with the same characteristic strength f_y and f_p respectively, the axis distance of the group of bars, a_m , should be not less than the axis distance, a, provided in the tables in Section 4. The axis distance may be determined by the expression: A3 A2 A3

$$a_m = \frac{A_{s1}a_1 + A_{s2}a_2 + ... + A_{sn}a_n}{A_{s1} + A_{s2} + ... + A_{sn}} = \frac{\Sigma A_{si}a_i}{\Sigma A_{si}} \quad \text{(Eq. C4–1)}$$

where:

a_i is the axis distance of the i[th] bar

a_m is the axis distance of the group of bars

A_{si} is the area of flexural tension reinforcement of the i[th] bar, mm^2

f_p is the stress in the prestressing, MPa

f_y is the lower characteristic yield strength of longitudinal reinforcement, MPa

When reinforcing consists of steels with different characteristic strengths, A_{si} should be replaced by $A_{si}f_y$ or $A_{si}f_p$ in Equation C4–1.

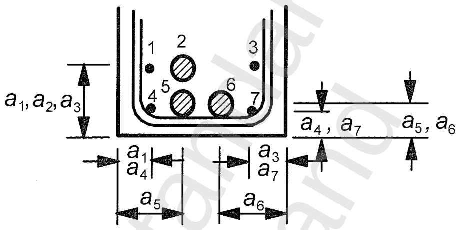


Figure C4.1 – Determination of axis distance

C4.2 Scope

This section is based on the previous version of this Standard, modified with reference to the Australian standard (AS 3600 current version and draft amendments) and Eurocode 2 [4.1]. Some changes also reflect the properties of New Zealand concretes where these have been adequately tested.

Terminology has been changed to conform with that of the New Zealand Building Code.

In building regulations, the specification of various fire resistance levels in relation to standard fire test conditions ensures that relatively higher or lower levels of fire resistance are achieved by various types of construction. This section gives rules whereby concrete members can be proportioned and detailed to satisfy regulatory requirements for particular fire resistance levels.

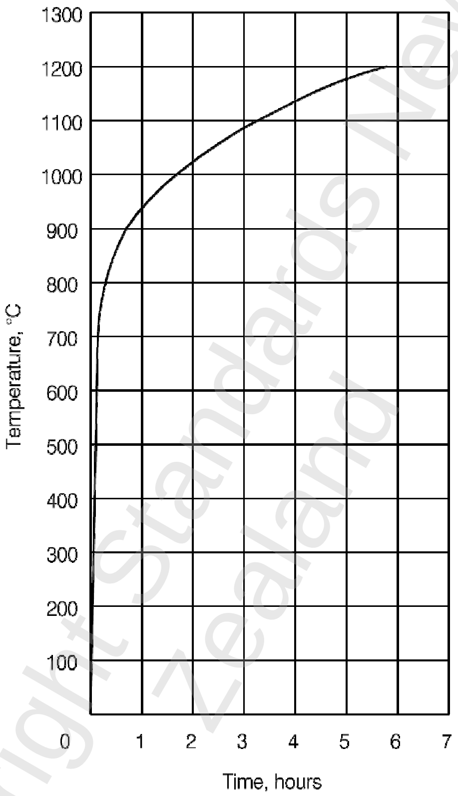
The term “fire resistance rating” refers to the level of fire resistance that will be required for the structural member by the building regulations. In the New Zealand Building Code, the fire resistance rating is expressed in minutes, in the order Structural Adequacy/Integrity/Insulation. Thus a FRR of 90/90/90 means that the structural members are required to have a resistance period for structural adequacy of 90 minutes, integrity of 90 minutes and insulation of 90 minutes, i.e. the minimum times that would need to be achieved if the members were tested for these criteria in accordance with AS 1530:Part 4.

AS 1530:Part 4 specifies conditions for the assessment of the fire resistance of a building component or member. A prototype specimen is tested in a furnace which is operated so that the furnace temperature-time relationship is as shown in Figure C4.2. The standard fire resistance test provides an internationally accepted basis for the assessment of the relative degree of fire resistance of different materials and building components. A3

It must be emphasised that the temperature versus time relationship corresponding to an actual fire is dependent on many factors, including quantity and type of fuel, fuel geometry, ventilation and other compartment characteristics and is likely to be significantly different from the standard time/temperature curve in a standard fire test. Furthermore, in an actual fire, unaffected portions of a building apply constraints on members which are almost impossible to simulate in a prototype test situation.

It is important to realise that AS 1530:Part 4 specifies not only a particular temperature-time relationship but also specifies the direction from which prototype test members are to be heated in the furnace, namely; floor and roof assemblies (slabs and beams) from below, walls from either sides but not both sides simultaneously and columns from all vertical sides. These requirements have important consequences on the manner in which subsequent Clauses in this section are framed and the application of these clauses to the design of members in actual buildings.

Provided that the implications of these limitations are taken into account when interpreting regulatory requirements, the use of the standard fire test will remain an important component of building regulations.



A3 **Figure C4.2 – Standard furnace temperature-time curve**

C4.3 Design performance criteria

C4.3.1 *General performance criteria*

A summary of international developments in design of structures for fire resistance is given in Reference 4.2.

C4.3.2 *General rules for the interpretation of tabular data and charts*

Most of the tabular data in this section have been taken from Reference 4.1 with a few exceptions where noted.

A change from the previous Standard is that the distance from the surface of the concrete to the reinforcing steel is now specified in terms of the "axis distance" rather than the "cover" referred to elsewhere in this Standard. Computational and experimental studies have shown that the temperature of a reinforcing bar is more accurately predicted in this way.

The tabular data are based on an assumption that the cover concrete remains in place for the duration of the fire, with no spalling. Spalling is an unpredictable phenomenon which is more likely to occur in fresh concrete or in other situations where evaporation of moisture within the heated concrete can lead to a rapid increase in pore pressure. Some spalling is also related to unstable behaviour of some aggregates at elevated temperatures, but this has not been found to be a problem with typical New Zealand concretes. High strength concrete is more prone to spalling because of its low porosity, which can be largely overcome by mixing into the concrete at least 2 kg/m^3 of monofilament propylene fibres which will melt under fire attack to increase the porosity [4.1].

C4.3.3 *Increase in axis distance for prestressing tendons*

Increased axis distances are specified in this section because high strength steels used for prestressing bars and tendons have more significant loss of strength at elevated temperatures than mild steels. See Reference 4.1 for more detail on this requirement.

C4.4 Fire resistance ratings for beams

Table 4.1 and Table 4.2 are from Reference 4.1.

C4.5 Fire resistance ratings for slabs

C4.5.1 *Insulation for slabs*

The minimum thicknesses in Table 4.3 are based on experimental testing of New Zealand concretes at BRANZ [4.3]. These values are slightly less than in the values in Reference 4.1 based on the measured performance of fire resistance of typical New Zealand concretes. The small increase in thickness for 30 minute FRR compared to the previous version is in response to the knowledge that temperatures in short duration fires are often higher than those in used in standard fire resistance testing Standards.

C4.5.2 *Structural adequacy for slabs*

Table 4.4 and Table 4.5 are from Reference 4.1.

Table 4.5 applies to “flat slabs”. A “flat slab” is a continuous two-way reinforced concrete slab of uniform thickness, supported only on columns with no beams. The increased thickness above the values in Table 4.3 is to prevent possible punching shear around the supporting columns during fire exposure. Slabs with column capitals or other local thickening near the columns should comply with Table 4.5 in any area of potential shear failure and with Table 4.3 elsewhere in the slab.

The values in Table 4.6 for “ribbed slabs” are from Reference 4.1. A “ribbed slab” is a concrete slab which has a top flange constructed integral with webs or ribs projecting below the flange, acting structurally with T-beam behaviour in one direction or two orthogonal directions.

C4.6 Fire resistance ratings for columns

The values in Table 4.7 are from Reference 4.1. These values have been determined for columns with an effective length no longer than 3.0 m and no significant eccentricity of loading. The table gives larger values of minimum dimension and axis distance for heavily loaded columns.

Clause 4.6.2 is based on axially loaded columns without significant bending moments. If the design is dominated by bending, the column should be considered to be a beam [4.4]. Reference 4.1 gives additional tables for combined bending and axial load in fire exposed reinforced concrete columns.

C4.7 Fire resistance ratings for walls

C4.7.1 *Insulation for walls*

The minimum thicknesses in Table 4.8 are based on experimental testing of New Zealand concretes at BRANZ, as described in C4.5.1.

C4.7.2 *Structural adequacy for walls*

The values in Table 4.9 are from Reference 4.1, which also gives tabulated data for the case where a wall is exposed to fire from both sides at the same time.

A3

C4.8 External walls or wall panels that could collapse inward or outward due to fire

C4.8.1 *General*

Clause 4.8 applies to external walls which could collapse inwards or outwards from a building as a result of a fire inside the building. This section is not restricted to those buildings close to a property boundary where it is necessary to prevent spread of fire to adjacent property, because it is also necessary to provide protection to fire fighters who could be killed or injured if walls fall inwards or outwards, in accordance with the New Zealand Building Code.

A3

The traditional approach to external walls in buildings with non-fire-rated roofs has been to ensure that the walls remain standing in place after a fire, even if the roof collapses. This Standard also permits a more recent approach which allows walls to be pulled inwards by the collapsing steel frame, ensuring that the walls remain attached to the steel frame and to each other, to avoid large gaps between the walls which would allow spread of fire to adjacent property. This approach is summarised in Reference 4.4.

A3

C4.8.2 *Forces on connections*

The process of design for fire conditions will depend on the design philosophy used for ambient conditions. It is impossible to predict the behaviour accurately, so the forces given in this section are rough estimates of the possible forces which could develop under various scenarios.

A detailed analysis must consider all likely forces, including the face load on the wall, the forces resulting from thermal bowing of the concrete panels, the forces resulting from deformation or collapse of a steel roof structure, and the self weight of the walls due to deformations away from the vertical position. If a detailed analysis is not carried out the standard requires that the wall and connections be able to resist a face load in either direction of 0.5 kPa.

A3

The previous loading Standard (NZS 4203) required freestanding external walls to be designed to resist a face load of 0.5 kPa in the “after fire” condition. The value of 0.5 kPa was derived from previous Standard requirements for a nominal level of wind or earthquake in the after fire condition. This requirement is not included in AS/NZS 1170.0 or AS/NZS 1170.1, but is retained through NZBC verification method B1/VM1 in order to provide a nominal level of force for design of walls and connections, and to ensure some degree of robustness for this type of building. A significant change from NZS 4203 is that the face load is now required to be applied during the fire, not just after the fire. This is because:

(a) The primary concern of the New Zealand Building Code is with collapse of walls and possible fire spread during a fire;

(b) Walls able to resist this load during a fire will, in most cases, be able to resist a similar load if they are still standing after the fire;

(c) It is considered acceptable for walls to be pulled inwards during the fire, hence not remain standing after the fire.

A3

A3

C4.8.3 *Design of connections*

C4.8.3(a) The reduction to 30 % of the yield strength in ambient conditions is based on an expected steel temperature of approximately 680 °C, which is the maximum temperature reached in the Eurocode “external” fire[4.7]. In a real fire in a typical industrial building, it is likely that higher temperatures will be reached in the early stages of the fire before the roof burns through, but 680 °C is an estimate of the likely temperature if the fire continues to burn for some time after the roof has collapsed.

C4.8.3(b) For steel other than normal mild steel, the connections can be designed using the mechanical properties of the steel at 680 °C.

A higher level of design stress can be used if the steel in the connection is protected using approved fire protection materials, in which case specific calculations of steel temperatures will be necessary.

C4.8.3(c) Proprietary anchors will have fire resistance ratings based on standard fire resistance tests in accordance with AS 1530:Part 4 or a similar national or international standard. The minimum rating of 60 minutes for unsprinklered buildings is an estimate of the worst likely fire severity in a typical industrial building with a non fire-rated roof structure. The reduction to 30 minutes for sprinklered buildings reflects the much lower probability of a severe fire in such buildings. These values have been prescribed because it is impossible to accurately predict the severity of a fire in a single storey building with non-fire-rated roof construction.

These ratings apply even when there is no specific requirement by NZBC C/AS1 for the external wall to be fire rated, in order to protect fire-fighters and others outside the building who could be injured if the wall unexpectedly collapsed outward.

C4.8.4 *Fixing inserts*

There are a number of proprietary adhesive anchors which have been tested under fire conditions. Manufacturers of such systems specify design loads depending on the required fire resistance rating[4.8]. Fire-rated adhesive connections rely on synthetic organic resins with or without inorganic fillers or active ingredients such as cement. Epoxy grouted inserts without an approved fire resistance rating must not be used for connections which are required to carry loads during a fire, because most epoxy resins lose strength at temperatures over about 60 °C.

C4.10 Fire resistance rating by calculation

This clause allows fire resistance to be assessed by a recognised method of calculation, such as given in Reference 4.1.

In Reference 4.1, the design fire exposure allows for standard or realistic fire design curves to be used. *Simple calculation methods* are given for predicting the behaviour of single members based on simple assumptions. *Advanced calculation methods* provide the principles for computer analyses based on fundamental physical behaviour, for both thermal analysis and mechanical behaviour. These analyses need to take into account factors such as transient temperature gradients, variation of thermal properties with temperature, axial and flexural restraint, thermally induced forces, and thermally induced deformations, throughout the duration of the expected fire. The effects of creep are not explicitly included in the advanced calculation methods, but the stress-strain relationships have been modified to include creep in an indirect way.

Reference 4.1 includes comprehensive expressions for thermal and mechanical properties at elevated temperatures, and stress-strain relationships at elevated temperatures. This is very useful for any analytical modelling of fire behaviour of structures. The tabulated listings in the Eurocodes (many used in this Standard) are far more extensive than most other codes, the particular benefit to designers being that the tables include the improved fire resistance for members which are loaded below their design capacity at the time of a fire.

Simple hand methods of calculation and discussion regarding advanced calculation methods for fire resistance are given in Reference 4.*2*.

REFERENCES

4.1 Eurocode 2: "Design of Concrete Structures". ENV 1992, Part 1-2: "General Rules – Structural Fire Design," European Committee for Standardisation, Brussels, 2002.

4.2 Buchanan, A.H., "Structural Design for Fire Safety". John Wiley & Sons. 2001.

4.3 Wade, C.A., 1992, "Fire resistance of New Zealand Concretes"; Part 2. BRANZ Study Report No. 40, Building Research Association of New Zealand.

4.4 Bennetts, I.R. and Poh, W., "Steel Portal Frame Buildings – Support of External Concrete Wall Panels". Onesteel Fire Design Note No.1. August 2000. http://www.onesteel.com/images/db_images/productspecs/tech_stportals0601.PDF

4.5 Lim, L., "Stability of Precast Concrete Tilt Panels in Fire". Fire Engineering Research Report 00/08. University of Canterbury, 2000.

4.6 Lim, L. and Buchanan, A.H., (2003). "Stability of Precast Concrete Tilt Panels in Fire". SESOC Journal, Vol.16, No. 2, pp. 44-54. Structural Engineering Society New Zealand. 2003.

4.7 Eurocode 1: "Actions on Structures". ENV 1991, Part 1-2: "General Actions – Actions on Structures Exposed to Fire". European Committee for Standardisation, Brussels, 2002.

4.8 "Evaluation of Anchorages in Concrete Concerning Resistance to Fire". EOTA Report TR 020, May 2004. European Organisation for Technical Approvals, Brussels. http://www.eota.be/html/tr/TR020.pdf

C5 DESIGN PROPERTIES OF MATERIALS

C5.1 Notation

The following symbol which appears in this section of the commentary is additional to the symbols used in Section 5 of the Standard. A3

K_{fl} factor relating the modulus of rupture to the direct tensile strength of concrete.

C5.2 Properties of concrete

C5.2.1 *Specified compressive strength*

Although there has been considerable research undertaken recently into the specification and performance of high strength concrete, there is insufficient data and experience in New Zealand applications to justify the use of a design compressive strength greater than 100 MPa.

The design value of compressive strength adopted may be dictated by considerations of serviceability and durability rather than strength alone in certain situations.

The minimum permitted concrete compressive strength has been reduced to 20 MPa for consistency with Section 3 of this Standard. A3

C5.2.2 *Applicable density range*

The formula in 5.2.3 for E_c is valid down to ρ = 1400 kg/m^3.

C5.2.3 *Modulus of elasticity*

The modulus of elasticity of concrete can be represented with acceptable accuracy by the formulae stated. However, it must be recognised that E_c can vary considerably and is sensitive to aggregate type. References 5.9 and 5.12 illustrate the typical variation in elastic modulus values of concretes made with New Zealand aggregates. A3 In determining the distribution of design actions and deflections an elastic modulus corresponding to (f'_c + 10) MPa may be used. The 10 MPa is added so that the concrete strength and thus stiffness is representative of likely average strength values rather than a value corresponding to a lower characteristic strength. A3 Where strains or deflections are critical, designers should consider an appropriate range of values for the modulus of elasticity.

The formula provided for the modulus of elasticity has been reverted to that used in an earlier edition of this standard, which recent research has shown to more realistically predict experimentally observed stiffness values[5.1].

C5.2.4 *Direct tensile strength concrete*

The average direct tensile strength of concrete is more variable than the compressive strength with these values typically in the range of 6 % to 12 % of the compressive strength, with the percentage decreasing with increasing compressive strength. A3

The values of tensile strength given in the clause have been calibrated to closely match the values specified in the fib Model Code 2010[5.2] while being defined by a simpler formula. Similarly, the λ factor accounting for lightweight concrete has been adopted from the Model Code 2010.

If an estimate is required of the average direct tensile strength of concrete, this may be obtained by multiplying the lower characteristic value obtained from Equation 5–2 by a factor of 1.43. Similarly, an estimate of the upper characteristic direct tensile strength may be obtained by multiplying the lower characteristic value by a factor of 1.86.

The tensile strengths given by Equation 5–2 should only be taken as a guide to likely values as a number of factors that can have major influence on the strengths are not considered. Factors that can have a marked influence on the ratio of tensile strength to compression strength include:

A3

(a) The tensile strength, particularly the flexural tensile strength (modulus of rupture), decreases as the size of concrete subjected to tension increases. This change occurs:
   (i) As there is a greater chance of a weak section in larger specimens
   (ii) With thicker members differential shrinkage between the surface and inside regions can induce tensile stresses in the surface layers
   (iii) The non-linearity of the stress-strain relationship of concrete in tension[5.3];
(b) The proportion and type of course aggregate in the concrete can have a marked influence on tensile strength;
(c) The orientation of the concrete relative to the direction in which it is cast can influence tensile strength. This arises due to water gain, which gives the concrete directional properties in both tension and compression[5.4, 5.5]. The tensile strength is generally 10 % to 30 % lower when the stress acts in the direction of casting (that is in the vertical direction if the orientation of the member has not been changed from the casting position) compared with the direction at right angles to this direction. However, this difference varies with the type of aggregate, the admixtures that are used, the form of compaction and the aggregate type;
(d) Water gain in the upper layers of concrete as cast reduces the tensile and compressive strengths of concrete.

Tests should be conducted where tensile strengths are important for the integrity of the structure. It should be noted that tensile strengths are subjected to appreciable scatter and hence multiple tests are required to establish a reliable lower characteristic value.

The direct tensile strength of concrete is difficult to measure due to the complexity of simultaneously holding the specimen and applying a concentric load. For this reason the tensile strength is generally assessed indirectly, through tests such as the Brazilian splitting test (split cylinder) and the modulus of rupture test. In both cases, relatively small specimens are tested and the tensile strength is based on a linear elastic analysis of the specimen. However, concrete does not behave as a linear elastic material and in both tests allowance has to be made for this non-linear behaviour in assessing direct tensile strengths.

An idealised form of the stress and strain relationship for concrete in tension is shown in Figure C5.1.

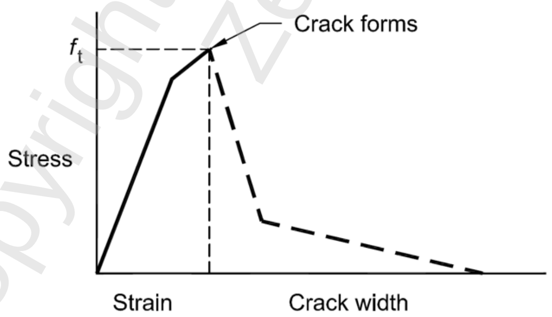


Figure C5.1 – Idealised stress-strain relationship for concrete

The stress-strain relationship for concrete in tension is generally a linear relationship (up to 85 % of the peak strength). Cracking occurs at the maximum stress and the tensile resistance decreases rapidly as the crack width increases[5.3]. Generally, the tensile resistance is exhausted at a crack width of about 0.2 mm. The tensile resistance across the crack arises from the crystals formed by hydration of the cement spanning the crack.

The direct tensile strength of concrete can be estimated from the results of either of the following simple test methods:

A. 1.0 times the indirect tensile strength from the Brazilian test, as given in AS 1012.10.

B. The value determined statistically from modulus of rupture tests carried out in accordance with AS 1012.11, with appropriate allowance made for the relationship between section size, flexural strength and tensile strength.

The relationship between the direct tensile strength, member depth and the modulus of rupture has been adopted from the fib Model Code 2010[5.2] and should be taken as: A3

$$f_t = K_{fl} f_r \quad \text{(Eq. 5–4)}$$

where

$$K_{fl} = \frac{0.06h^{0.7}}{1 + 0.06h^{0.7}} \quad \text{(Eq. 5–5)}$$

Where h is the member depth, mm.

C5.2.5 *Modulus of rupture for calculation of deflections*

The value of the modulus of rupture, f_r, used for calculating deflections is taken as $0.6\lambda\sqrt{f'_c}$, where λ is a factor which is 1.0 for normal concrete and less than 1.0 for lightweight concrete. This value is based on an empirical analysis of deflection measurements made on reinforced concrete beams. Consequently it is appropriate to use this value in assessing deflections of reinforced concrete members with typical dimensions.

C5.2.6 *Void*

A3

C5.2.9 *Coefficient of thermal expansion*

The value of 12 × 10^{-6}/°C should be satisfactory for most structural calculations. However, the actual coefficient varies over a wide range depending on the aggregate type, volume of cement-paste and the degree of saturation of the concrete. The coefficient of thermal expansion of self-compacting concrete is normally 10 % to 15 % higher than for conventionally placed concrete.

C5.2.10 and C5.2.11 *Shrinkage & creep*

Creep and shrinkage values for New Zealand concretes are given in Appendix E. Appendix CE gives background information together with simplified methods and worked examples on assessing actions associated with creep and shrinkage. A3

It should be noted that shrinkage and creep values cannot be accurately determined. The actual values depend on the composition of the concrete, the effective thickness of the elements making up the member and the environmental conditions such as temperature, wind speed, humidity and duration of wet curing. Increasing the damp curing period reduces shrinkage and increases strength. This is particularly significant in thinner members and in members with a low porosity. The amount of creep that occurs reduces with increasing age of concrete (maturity) at the time the load is applied.[5.1, 5.2, 5.6, 5.7, 5.8, 5.12].

The shrinkage and creep values given in Appendix E have been based on values in Australian codes, which have been adjusted for New Zealand concretes. These adjustments were based on test results given in References[5.6, 5.7, 5.8, 5.9 and 5.12].

C5.3 Properties of reinforcement

C5.3.1 *Use of plain and deformed reinforcement*

In general, plain round bars are preferable for ties and stirrups because the small radius bends which are required have undesirable metallurgical and mechanical effects on deformed bars[5.10]. Also, in most situations ties and stirrups do not rely on high bond strengths along their straight legs for their action. However, there are some cases, such as deep wall beams and lapped splices in tie legs, where it may be necessary for stirrups and ties to develop high bond values along their straight portions. In such cases it is acceptable to use deformed bars, provided that the diameters satisfy 8.4.2. A3

A common practice in the precast concrete industry is to use non-tensioned strand off-cuts as secondary reinforcement, or crack control reinforcement in precast, or pretensioned units. Reinforcement of this type should not be welded or heated.

C5.3.2 *Reinforcement grades*

When the long-term quality of the reinforcement cannot be demonstrated by B3, B4, and B6 of AS/NZS 4671, compliance with AS/NZS 4671 shall be demonstrated by B7.

In terms of AS/NZS 4671 Clause B7, a "batch" shall be interpreted as any bundle of reinforcement to be used. Each grade of bar, round or deformed profile and bar size shall be treated as a discrete test unit to be individually reviewed.

Verification that all products in the test unit are from the same cast is to be by the manufacturer's or processor's or supplier's certificate.

From the 15 test pieces per test unit of no more than a 100 tonnes (or part of), the test results shall be used to determine compliance with AS/NZS 4671. Up to 60 test pieces per 100 tonnes may be required by Clause B7, AS/NZS 4671, depending on the lack of compliance of the first 15 test pieces.

A3 Should the test unit not conform to AS/NZS 4671 then the material of the test unit shall not be used in structural elements being designed to NZS 3101.

It is important to note that any process involving heat e.g. welding, galvanising and hot bending can adversely affect the mechanical properties of quench and tempered reinforcing bar by modification of the microstructure.

Threading of quench and tempered bar removes some to all of the hardened outer layer resulting in a disproportionate loss of strength.

A3

Ductile reinforcement, Grade 300E or Grade 500E, should be used in all structural elements, which may be subjected to:

(a) Yielding due to seismic forces or displacements;
(b) Appreciable moment redistribution under any loading combination;
(c) Redistribution of structural actions due to stage by stage construction or by creep redistribution of actions;
(d) Opening of cracks due to shrinkage, thermal and creep movements in the concrete, or due to settlement of the foundations.

Where significant ductility is required then Grade 300E reinforcement is recommended. Grade 300E A3 reinforcement typically has greater ductility and toughness, compared with Grade 500E reinforcement (regardless of the method of manufacture).

A3

C5.3.3 *Strength*

The maximum lower characteristic yield strength of reinforcing steel covered by AS/NZS 4671 is 500 MPa. Before using steels with greater yield strengths than this, the designer should ascertain their properties to ensure that they are suitable for the intended application. The behaviour under actions including but not limited to bending, fatigue, exposure to high and/or low temperature, strain age embrittlement and strength variations shall be considered.

C5.3.4 *Modulus of elasticity*

The value E_s = 200,000 MPa for non-prestressed steel represents a realistic average value obtained from many tests.

C5.4 Properties of tendons

C5.4.3 Stress–strain curves

Reference 5.11 is a resource for evaluating stress strain characteristics for prestressing strands. A3

Some typical sizes of prestressing wires, strands, and bars are shown in Table C5.2.

Table C5.2 – Tensile strength of commonly used wire, strand and bar

Tendon material type and Standard	Nominal diameter (mm)	Area (mm^2)	Minimum breaking load (kN)	Nominal tensile strength (f_{pu}) (MPa)
Stress-relieved wire AS/NZS 4672.1	5.0	19.6	32.7	1670
	5.0	19.6	34.7	1770
	7.0	38.5	64.3	1670
7-wire ordinary strand, AS/NZS 4672.1	9.3	51.6	88.8	1720
	12.4	92.9	184	1720
	12.9	100	186	1860
	15.2	140	250	1790
Hot rolled bars, AS/NZS 4672.1	26	562	579	1030
	32	840	865	1030
	36	995	1025	1030

C5.5 Properties of steel fibre reinforced concrete

The design properties of steel fibre reinforced concrete are dependent on the post-cracking toughness of the composite material. The properties of the fibre, such as its aspect ratio, ultimate tensile strength and end anchorage have a significant influence on the performance of the fibre reinforced concrete. Different fibre properties will result in different fibre dose rates to meet specific design properties. Designs must be based on the test data supplied by the fibre manufacturer, or confirmed by tests. The design method of Appendix A to Section 5 may be used.

REFERENCES

A3 5.1 Al-Omaishi, N, M K Tadros, and S J Seguirant. "Elasticity Modulus, Shrinkage, and Creep of High-Strength Concrete as Adopted by AASHTO." PCI Journal 54 (3): Summer 2006 pp. 44–63.

5.2 fib Bulletin 65: Model Code 2010: Final Draft, Volume 1, International Federation for Structural Concrete (fib), Lausanne, Switzerland. 357p.

5.3 Gopalartnam, V.S. and Shah, S.P.,"Softening Response of Plain Concrete in Direct Tension", Journal of American Concrete Institute, Vol. 82, May - June 1985, pp. 310 323.

A3 5.4 Hughes, B.P. and Ash, J.E., "The Effect of Water Gain on the Behaviour of Concrete in Tension, Torsion and Compression", The Concrete Society, London, Technical paper PCS 54, Nov. 1969, pp. 17.

5.5 Fenwick, R.C., and Sue, C.F.F, "The Influence of Water Gain upon the Tensile Strength of Concrete", Magazine of Concrete Research, Vol.34, No.120, Sept. 1982, pp. 139 145.

5.6 AS 5100.5 –2004. "Bridge Design-Concrete", Standards Australia, Sydney

5.7 Bryant, A.H., Wood, J.A. and Fenwick, R.C., "Creep and Shrinkage in Concrete Bridges", Road Research Unit Bulletin No. 70, National Roads Board, New Zealand, 1984.

5.8 ACI Committee 209R-92, "Predicting Creep, Shrinkage and Thermal Effects in Concrete Structures, Manual of Concrete practice, Vol.1, 1993 and later editions.

5.9 Mackechnie, J. R., "Hardened Properties of Concrete Containing New Zealand Aggregates", SESOC Journal, Vol. 16, No.2, Sept. 2003, pp. 20-29.

5.10 Erasmus, L.A. and Pussegoda, N., "Safe Bend Radii for Deformed Reinforcing Bar to Avoid Failure by Strain Age Embrittlement", New Zealand Engineering, Vol. 33, No. 8, August 1978, pp. 170-177.

5. 11 Ravi, K.D. Mathers, K.T., "Stress-strain modelling of 270 ksi Low Relaxation Prestress Strands," PCI Journal, Vol. 37, No. 2, Mar. 1992, pp. 100-105.

A3 5.12 MacKechnie, J. and Fenwick, R., "Assessing dimensional stability of New Zealand concrete", SESOC Journal, Vol. 22, No. 2, Sept. 2008.

APPENDIX A TO C5

DESIGN PROPERTIES OF MATERIALS

C5A TEST AND DESIGN METHODS FOR STEEL FIBRE REINFORCED CONCRETE SUBJECTED TO MONOTONIC LOADING

C5.A1 Notation

The following symbols which appear in this Appendix, are additional to those used in Section C5 of the commentary.

A_c	cross section of concrete, mm^2
A_{ct}	area of concrete within tensile zone, mm^2
A_s	area of tension reinforcement, mm^2
b	width of the specimen, mm
b_f	width of the flanges, mm
b_w	minimum width of the web, mm
$CMOD_i$	crack mouth opening displacement for any increment ‘i’, mm
$CMOD_L$	crack mouth opening displacement at the end of the elastic limit, mm
$CMOD_1$	crack mouth opening displacement of 0.5 mm, mm
$CMOD_2$	crack mouth opening displacement of 1.5 mm, mm
$CMOD_3$	crack mouth opening displacement of 2.5 mm, mm
$CMOD_4$	crack mouth opening displacement of 3.5 mm, mm
d	effective depth, mm
e	eccentricity, mm
E_c	characteristic modulus of elasticity of concrete, MPa
E_{fcm}	mean secant modulus of elasticity of steel fibre reinforced concrete, MPa
e_v	the eccentricity of the prestressing force, mm
f'_c	concrete characteristic cylinder compressive strength of plain concrete at 28 days, MPa
F_c	compressive force in the concrete in the direction of the longitudinal axis, N
$f_{fct,ax}$	concrete axial tensile strength, MPa
$f_{fct,ef}$	the tensile strength of the concrete effective at the time when the cracks may first be expected to occur, MPa
$f_{fct,fl}$	concrete flexural tensile strength, MPa
$f_{fctk,ax}$	concrete characteristic axial tensile strength, MPa
$f_{fctk,fl}$	concrete characteristic flexural tensile strength, MPa
$f_{fctk,L}$	concrete characteristic value of limit of proportionality, MPa
$f_{fctm,ax}$	concrete mean axial tensile strength, MPa
$f_{fctm,fl}$	mean flexural tensile strength, MPa
$f_{fctm,L}$	mean value of LOP, MPa
$FL_{0.5}$	the value of $f_{R,1}$ reduced to the nearest multiple of 0.5 MPa, MPa
$FL_{3.5}$	the value of $f_{R,4}$ reduced to the nearest multiple of 0.5 MPa, MPa
$f_{R,1}$	residual flexural tensile strength, MPa
$f_{R,4}$	residual flexural tensile strength, MPa
$F_{R,i}$	load recorded at $CMOD_i$ or $\delta_{R,i}$, N
$f_{R,i}$	residual flexural tensile strength, MPa
$f_{Rk,4}$	the characteristic residual tensile strength of SFRC at $CMOD_4$, MPa
$f_{Rm,1}$	the mean residual flexural tensile strength of the SFRC at the moment when a crack is expected to occur, MPa
F_s	tensile force in the longitudinal reinforcement, N
f_{ywd}	design yield strength of the shear reinforcement, MPa

h	height of beam
h_f	height of the flanges, mm
h_{sp}	distance between tip of the notch and top of cross section, mm
k	coefficient which allows for the effect of non-uniform self-equilibrating stresses
k_1	Equation C5A–14
k_a	factor allowing for the influence of aggregate size on shear strength
k_c	coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking
k_d	factor allowing for the influence of member depth on shear strength
k_f	factor for taking into account the contribution of the flanges in a T-section
k_p	coefficient which takes account of the prestressing effect
k_x	factor dependent on the number of specimens
k_{xknown}	factor for known number of specimens when coefficient of variation is known
$k_{xunknown}$	factor dependent on the number of specimens when the coefficient of variation of the population is unknown
L	span of the specimen, mm
LOP	limit of proportionality
M_1	applied stress Figure C5.A7
M_2	assumed stress distribution, Figure C5.A7
N	number of specimens
N_{Sd}	prestressing force, N
p_w	Equation C5A-10
s	spacing of stirrups, mm
SFR	Steel Fibre Reinforced
SFRC	Steel Fibre Reinforced Concrete
s_p	standard deviation of stress, MPa
V_b	the shear resistance of the member without shear reinforcement, N
V_{fd}	contribution of the steel fibre shear reinforcement, N
$V_{rd,3}$	design shear resistance of a section of a beam with shear reinforcement and containing steel fibres, N
V_{wd}	contribution of the shear reinforcement due to stirrups and/or inclined bars, N
z	the internal lever arm, mm
α	the angle of the shear reinforcement in relation to the longitudinal axis; the ratio of prestressing
δ_R	deflection at flexural tensile stress, f_R, mm
$\delta_{R,1}$	deflection at flexural tensile stress, $f_{R,1}$, mm
$\delta_{R,4}$	deflection at flexural tensile stress, $f_{R,4}$, mm
ε	strain
θ	the angle of the concrete struts in relation to the longitudinal axis
σ	stress, MPa
σ_f	real stress in a cracked section
$\sigma_{f,1}$	0.45 $f_{R,1}$
$\sigma_{f,4}$	0.37 $f_{R,4}$
σ_s	the maximum stress permitted in the reinforcement immediately after formation of the crack, MPa
σ_2 , σ_3	stress across crack Figure C5.A3
τ_{fd}	design value of the increase in shear strength due to steel fibres
υ	Equation C5A–18

C5.A2 Introduction

This methodology was adopted from RILEM TC 162-TDF[5A.1] courtesy of RILEM. These recommendations were proposed and approved to be added to the relevant European Codes for design of concrete structures.

The design of steel fibre reinforced concrete according to the $\sigma - \varepsilon$ method is based on the same fundamentals as the design of normal reinforced concrete. The proposed method is valid for steel fibre concrete with compressive strengths of up to 50 MPa. Steel fibres can also be used in high strength concrete, i.e. concrete with $f'_c \geq 50$ MPa. However, care should be taken that the steel fibres do not break in a brittle way before being pulled out.

It must be emphasised that these calculation guidelines are intended for cases in which the steel fibres are used for structural purposes and not for example for slabs on grade. They also do not apply for applications such as increased resistance to plastic shrinkage, increased resistance to abrasion or impact, etc.

The anchorage capacity of steel fibres maybe lost in areas where significant cracking is expected. C5.A4.1.1(f) limits the maximum crack width for the use of fibres to 3.5 mm. Cracking of this magnitude can be expected in regions where plastic rotations are expected. For this reason steel fibres should not be relied upon in plastic hinge regions of primary or secondary lateral load elements unless supported by test data. The method described in this section applies to members subject to monotonic loading. The performance of fibre reinforced members subjected to fatigue or cyclic based cases should be determined by special study. Significant cracks can also be expected to form in regions where relative rotation between members is concentrated. For example the topping of a flooring system often cracks above a supporting beam. Under lateral loads the primary lateral load resisting beams will elongate due to geometric considerations or due to the formation of plastic hinges in the beams. The elongation of the beam may results in cracks in the diaphragm above the beams supporting the floor. These cracks are likely to exceed 3.5 mm and therefore fibres in these locations should not be relied upon unless supported by special studies.

Design procedures in this Appendix are for steel-fibre, reinforced concrete, only. They are based on the RILEM recommendations[5A.1]. Design procedures for some high performance synthetic fibres have been established by the fibre suppliers, but to date, no independent generic design rules have been proposed for these materials. Structural applications of synthetic fibres are currently outside the scope of this Standard.

Designers using synthetic fibres should follow the suppliers design guidelines, and confirm the results by special studies.

The use of synthetic fibres to control plastic shrinkage cracking, or to prevent explosive spalling of damp cover concrete during severe fires, is outside the scope of this Standard: their efficacy in these applications is well documented, but is proprietary information.

C5.A3 Material properties

C5.A3.1 *Compressive strength*

The compressive strength of steel fibre reinforced concrete (SFR concrete) should be determined by means of standard tests on concrete cylinders. The addition of steel fibres to concrete does not change the properties of the concrete unless the fibre content is high enough to make compaction difficult, in which case most properties (f'_c, E_c) will reduce.

The design principles are based on the characteristic 28-day strength, defined as that value of strength below which no more than 5 % of the population of all possible strength determinations of the volume of the concrete under consideration, are expected to fall. Hardened SFR-concrete is classified in respect to its compressive strength by SFR-concrete strength classes which relate to the cylinder strength f'_c (Table C5.A1). Those strength classes are the same as for plain concrete.

C5.A3.2 *Flexural tensile strength*

When only the compressive strength f'_c has been determined, the estimated mean and characteristic flexural tensile strength of steel fibre reinforced concrete may be derived from the following equations:

$f_{fctm,ax} = 0.3(f'_c)^{2/3}$ (MPa) .... (Eq. C5A–1)

$f_{fctk,ax} = 0.7 f_{fctm,ax}$ (MPa) .... (Eq. C5A–2)

$f_{fct,ax} = 0.6 f_{fct,fl}$ (MPa) .... (Eq. C5A–3)

$f_{fctk,fl} = 0.7 f_{fctm,fl}$ (MPa) .... (Eq. C5A–4)

The corresponding mean and characteristic values for the different steel fibre reinforced concrete strength classes are given in Table C5.A1.

Table C5.A1 – Steel fibre reinforced concrete strength classes: characteristic compressive strength f'_c (cylinders), mean $f_{fctm,fl}$ and characteristic $f_{fctk,fl}$ flexural tensile strength mean secant modulus of elasticity E_{fcm} in MPa

f'_c	20	25	30	35	40	45	50
$f_{ctm,fl}$	3.7	4.3	4.8	5.3	5.8	6.3	6.8
$f_{fctk,fl}$	2.6	3.0	3.4	3.7	4.1	4.4	4.8
E_{fcm}	25,000	26,500	27,900	29,200	30,300	31,500	32,600

If bending tests are performed, the following method[5.A1] can be used to determine the characteristic value of the limit of proportionality (LOP) (cf. bending test) [5A.2]:

$f_{fctk,L} = f_{fctm,L} - k_x s_p$ (MPa) .... (Eq. C5A–5)

with

$f_{fctk,L}$ is the characteristic value of LOP (MPa)
$f_{fctm,L}$ is the mean value of LOP (MPa)
s_p is the standard deviation (MPa)

$$s_p = \sqrt{\frac{\sum (f_{fctm,L} - f_{fct,L})^2}{(n-1)}}$$ .... (Eq. C5A–6)

n is the number of specimens
k_x is the factor dependent on the number of specimens; some values are given in Table C5.A2.

The maximum value of Equations C5A–4 and C5A–5 can be taken as the flexural tensile strength of the SFR concrete. In Table C5.A2, $k_{xunknown}$ means the coefficient of variation of the population is unknown; instead of the standard deviation of the population, the standard deviation of the spot check will be used.

Table C5.A2 – k_x as a function of the number of specimens

n	1	2	3	4	5	6	8	10	20	30	∞
k_{xknown}	2.31	2.01	1.89	1.83	1.80	1.77	1.74	1.72	1.68	1.67	1.64
$k_{xunknown}$	-	-	3.37	2.63	2.33	2.18	2.00	1.92	1.76	1.73	1.64

C5.A3.3 *Residual flexural tensile strength*

The residual flexural tensile strength $f_{R,i}$, which is an important parameter characterising the post-cracking behaviour of steel fibre reinforced concrete, is determined by the CMOD (crack mouth opening displacement) – or deflection controlled bending test [5A.2].

The residual flexural tensile strength $f_{R,1}$, $f_{R,4}$ respectively, are defined at the following crack mouth opening displacement ($CMOD_i$) or mid-span deflections ($\delta_{R,i}$):

$CMOD_1 = 0.5$ mm – $\delta_{R,1} = 0.46$ mm

$CMOD_4$ = 3.5 mm – $\delta_{R,4}$ = 3.00 mm

and can be determined by means of the following expression:

$$f_{R,i} = \frac{3F_{R,i}L}{2bh_{sp}^2} \text{ MPa} \quad \text{(Eq. C5A–7)}$$

where

b is the width of the specimen (mm)

h_{sp} is the distance between tip of the notch and top of cross section (mm)

L is the span of the specimen (mm)

$F_{R,i}$ is the load recorded at $CMOD_i$ or $\delta_{R,i}$ (N) (see Figure C5.A1)

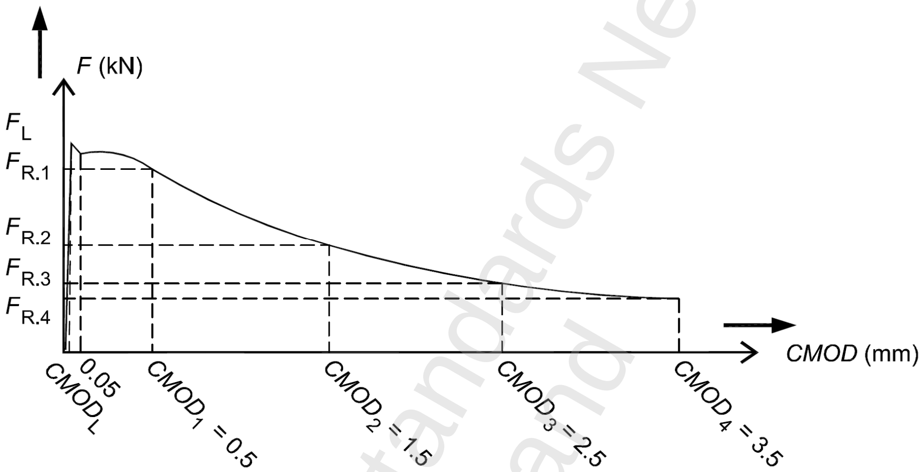


Figure C5.A1 – Load – CMOD diagram

The relationship between “characteristic” and “mean” residual flexural tensile strength is given in C5.A3.2 Equation C5A–5.

Hardened SFR-concrete is classified by using two parameters that are determined by the residual flexural strength $f_{R,1}$ and $f_{R,4}$. The first parameter $FL_{0.5}$ is given by the value of $f_{R,1}$ reduced to the nearest multiple of 0.5 MPa, and can vary between 1 MPa and 6 MPa. The second parameter $FL_{3.5}$ is given by the value of $f_{R.4}$ reduced to the nearest multiple of 0.5 MPa, and can vary between 0 MPa and 4 MPa. These two parameters denote the minimum guaranteed characteristic residual strengths at $CMOD$ values of 0.5 and 3.5 mm, respectively. The residual strength class is represented as FL $FL_{0.5}/FL_{3.5}$, with the corresponding values of the two parameters. For example, a SFRC with a characteristic cylinder compressive strength of 30 MPa, and $f_{R,1}$ = 2.2 MPa and $f_{R,4}$ = 1.5 MPa would have $FL_{0.5}$ = 2.0 MPa and $FL_{3.5}$ = 1.5 MPa and be classified as Grade 30 MPa FL2.0/1.5.

C5.A4 Design at ultimate limit states

C5.A4.1 *Ultimate limit states for bending and axial force*

C5.A4.1.1 *General*

The design method was originally developed without size-dependent safety factors. A comparison of the predictions of the design method and of the experimental results of structural elements of various sizes revealed a severe overestimation of the carrying capacity by the design method. In order to compensate for this effect, size-dependent safety factors have been introduced. It should be outlined that the origin of this apparent size-effect is not yet fully understood. Further investigation is required in order to identify if it is due to a discrepancy of material properties between different batches, to a size-effect intrinsic to the method, or a combination of both.

In assessing the ultimate resistance of a cross section, the assumptions given below are used:

(a) Plane sections remain-plane (Bernoulli);

(b) The stresses in the steel fibre reinforced concrete in tension as well as in compression are derived from the stress-strain diagram shown in Figure C5.A2;

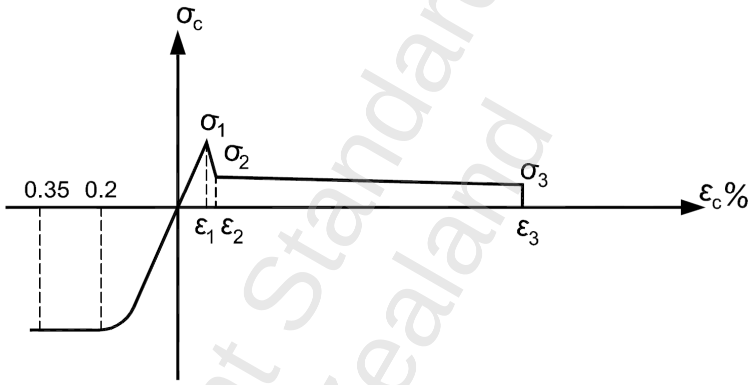
(c) The stresses in the reinforcement (bars) are derived from an idealised bi-linear stress-strain diagram;

(d) For cross sections subjected to pure axial compression, the compressive strain in the SFR-concrete is limited to – 0.20 %. For cross sections not fully in compression, the limiting compressive strain is taken as – 0.35 %. In intermediate situations, the strain diagram is defined by assuming that the strain is – 0.20 % at a level three-sevenths of the height of the compressed zone, measured from the most compressed face;

(e) For steel fibre reinforced concrete which is additionally reinforced with bars, the strain is limited to 2.50 % at the position of the reinforcement in Figure C5.A4;

(f) To ensure enough anchorage capacity for the steel fibres, the maximum deformation in the ultimate limit state is restricted to 3.5 mm. If crack widths larger than 3.5 mm are used, the residual flexural tensile strength corresponding to that crack width and measured during the bending test has to be used to calculate σ_3. It is recommended that this value, which replaced $f_{R,4}$, should not be lower than 1 MPa;

(g) In exposure Class C, where severe cracking is expected, the contribution of the steel fibres near the surface has to be reduced. For this reason the steel fibres should not be taken into account in a layer near the surface.



$\sigma_1 = 0.7\ f_{\text{fctm, fl}}\ (1.6 - \text{d})$ (d in m) (MPa)

$\sigma_2 = 0.45\ f_{R,1}\ k_h$ (MPa)

$\sigma_3 = 0.37\ f_{R,4}\ k_h$ (MPa)

$$E_c = \left(3320\sqrt{f'_c} + 6900\right)\left(\frac{\rho}{2300}\right)^{1.5}$$

k_h is the size factor (as shown in Figure C5.A3)

$$= 1.0 - 0.6\frac{h - 125}{475}$$

where

$125 \leq h \leq 600$ (h in mm)

$\varepsilon_1 = \sigma_1 / E_c$

$\varepsilon_2 = \varepsilon_1 + 0.01\ \%$

$\varepsilon_3 = 2.5\ \%$

Figure C5.A2 – Stress-strain diagram

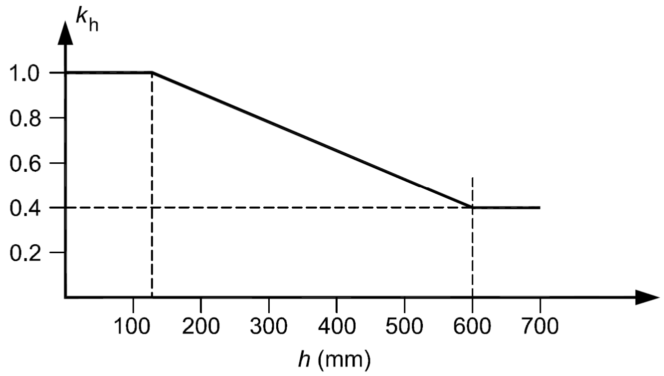


Figure C5.A3 – Size factor k_h

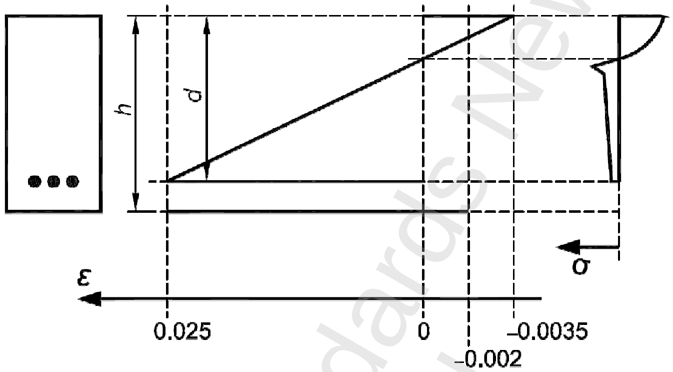


Figure C5.A4 – Stress and strain distribution

C5.A4.2 *Shear*

The calculation for shear shown here applies to beams and plates containing traditional flexural reinforcement (bar and mesh). It also applies to prestressed elements and columns in which axial compression forces are present. The approach proposed is the best possible until further evidence becomes available.

When no longitudinal reinforcement or compression zone is available, no generally accepted calculation method for taking into account the effect of the steel fibres for resisting shear can be formulated.

Where a member is subjected to shear stresses, the minimum area of longitudinal reinforcement provided shall comply with section 9 for beams and section 10 for columns.

Bent-up bars shall not be used as shear reinforcement in beams except in combination with steel fibres and/or stirrups. In this case at least 50 % of the necessary shear reinforcement shall be provided by steel fibres and/or stirrups.

For shear design of members with constant depth, the member is assumed to consist of compressive and tensile zones of which the centres are separated by a distance equal to the internal level arm z (Figure C5.A5). The shear zone has a depth equal to z and width b_w. The internal level arm is calculated perpendicular to the longitudinal reinforcement by ignoring the effect of any bent-up longitudinal reinforcement.

The parameters given in Figure C5.A5 are:

α the angle of the shear reinforcement in relation to the longitudinal axis (45° ≤ α ≤ 90°)
θ the angle of the concrete struts in relation to the longitudinal axis
F_s tensile force in the longitudinal reinforcement (N)
F_c compressive force in the concrete in the direction of the longitudinal axis (N)
b_w minimum width of the web (mm)
d effective depth (mm)

s spacing of stirrups (mm)

z the internal lever arm corresponding to the maximum bending moment in the element under consideration (mm) in a member with constant depth

An example of the standard method, i.e.: $\theta = 45°$, will be used for the shear analysis.

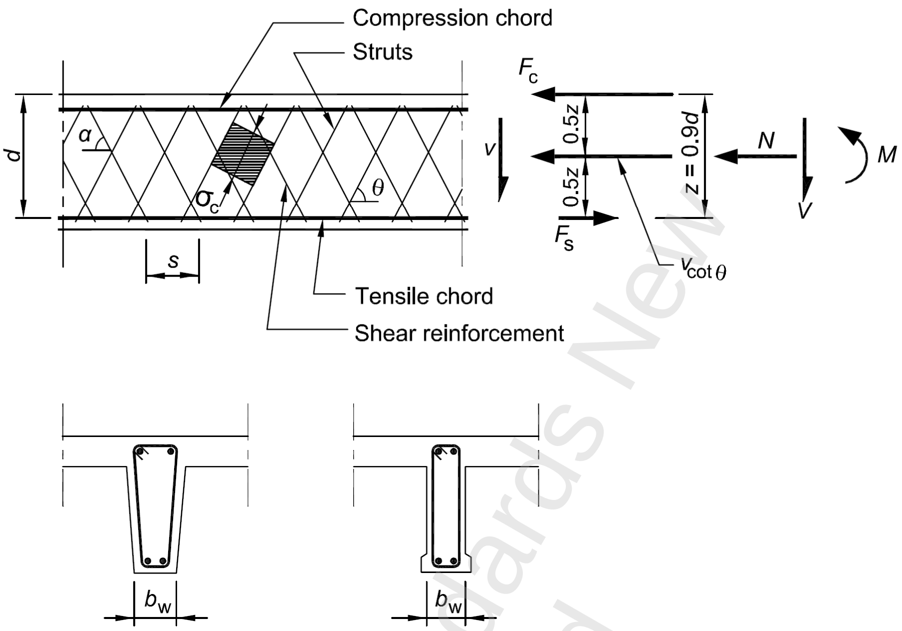


Figure C5.A5 – Strut and tie model

C5.A4.2.1 *Standard method*

The design shear resistance of a section of a beam with shear reinforcement and containing steel fibres is given by the equation:

$$V_{rd,3} = V_b + V_{fd} + V_{wd} \qquad \text{(Eq. C5A–8)}$$

with

V_b the shear resistance of the member without shear reinforcement, given by:

$$V_b = k_a k_d (0.07 + 10 p_w)\sqrt{f_c'}\, b_w d \qquad \text{(Eq. C5A–9)}$$

where

V_b shall not be more than $0.2\sqrt{f_c'}\, b_w d$ nor need be less than $0.08\sqrt{f_c'}\, b_w d$, and k_a and k_d are given by 9.3.9.3.4.

where

$$p_w = \frac{A_s}{b_w d} \qquad \text{(Eq. C5A-10)}$$

where

A_s is the area of tension reinforcement bars extending equal to or greater than "d + anchorage length" beyond the section considered, Figure C5.A6, mm^2

b_w is the minimum width of the section over the effective depth d (mm).

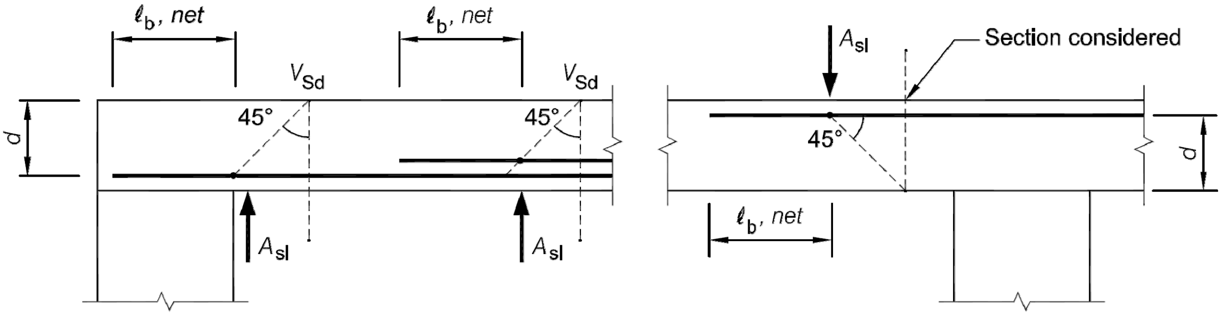


Figure C5.A6 – Section for determining P_w

V_{fd} is the contribution of the steel fibre shear reinforcement, given by:

$V_{fd} = 0.7\, k_f\, k_1 \tau_{fd}\, b_w d$ (N) ....(Eq. C5A–11)

where

k_f is the factor for taking into account the contribution of the flanges in a T-section:

$k_f = 1 + n\left(\dfrac{h_f}{b_w}\right)\left(\dfrac{h_f}{d}\right)$ and $k_f \le 1.5$ ....(Eq. C5A–12)

with

h_f is the height of the flanges (mm)
b_f is the width of the flanges (mm)
b_w is the width of the web (mm)

$n = \dfrac{b_f - b_w}{h_f} \le 3$ and $n \le \dfrac{3b_w}{h_f}$ ....(Eq. C5A–13)

$k_1 = 1 + \sqrt{\dfrac{200}{d}}$ (d in mm) and $k_1 \le 2$....................(Eq. C5A–14)

τ_{fd} is the design value of the increase in shear strength due to steel fibres

$\tau_{fd} =$ $0.12\, f_{Rk,4}$ ....(Eq. C5A–15)

V_{wd} is the contribution of the shear reinforcement due to stirrups and/or inclined bars, given by:

$V_{wd} = \dfrac{A_{sw}}{s} d f_{ywd}(1 + \cot\alpha)\sin\alpha$ (N) ....(Eq. C5A–16)

where

s is the spacing between the shear reinforcement measured along the longitudinal axis (mm)
α is the angle of the shear reinforcement with the longitudinal axis
f_{ywd} is the design yield strength of the shear reinforcement (MPa)

When checking against crushing at the compression struts, $V_{Rd,2}$ is given by the equation:

$$V_{Rd,2} = \frac{1}{2}\upsilon f_{cd} b_w 0.9d(1+\cot\alpha) \text{ (N)} \qquad \text{(Eq. C5A–17)}$$

with

$$\upsilon = 0.7 - \frac{f_c'}{200} \geq 0.5 \quad (f_c' \text{ in} < \text{MPa}) \qquad \text{(Eq. C5A–18)}$$

For vertical stirrups, or for vertical stirrups combined with inclined shear reinforcement, cot α is taken as zero.

C5.A5 Design at serviceability limit states

C5.A5.1 *General*

When an uncracked section is used, the full steel fibre reinforced concrete section is assumed to be active and both concrete and steel are assumed to be elastic in tension as well as in compression.

When a cracked section is used, the steel fibre reinforced concrete is assumed to be elastic in compression, and capable of sustaining a tensile stress equal to 0.45 $f_{R,1}$.

C5A5.2 *Minimum reinforcement*

The following formula is proposed for calculating the minimum amount of longitudinal reinforcement bars, $A_{s,}$ in order to obtain controlled crack formation:

$$A_s = (k_c k k_p f_{fct,ef} - 0.45 f_{Rm,1})\frac{A_{ct}}{\sigma_s}(\text{mm}^2) \qquad \text{(Eq. C5A–19)}$$

where

$f_{Rm,1}$ is the average residual flexural tensile strength of the steel fibre reinforced concrete at the moment when a crack is expected to occur (MPa),

A_s is the area of reinforcement bar within the tensile zone (mm^2). If A_s is smaller than zero only steel fibres are necessary to control cracking.

A_{ct} is the area of concrete within tensile zone (mm^2). The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack.

σ_s is the maximum stress permitted in the reinforcement immediately after formation of the crack (MPa). This may be taken equal to the yield strength of the reinforcement (f_{yk}). However, a lower value is likely to be needed to satisfy the crack width limits.

$f_{fct,ef}$ is the tensile strength of the concrete effective at the time when the cracks may first be expected to occur (MPa). In some cases, depending on the ambient conditions, this may be within 3 - 5 days from casting. Values of $f_{fct,ef}$ may be obtained from Equation C5A–1 by taking f_c' as the strength at the time cracking is expected to occur. When the time of cracking cannot be established with confidence as being less than 28 days, it is recommended that a minimum tensile strength of 3 MPa be adopted.

k_c is a coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking. The relevant stress distribution is that resulting from the combination of effects of loading and restrained imposed deformations.
k_c = 1 for pure tension ($e = M/N = 0$)
k_c = 0.4 for bending without normal compressive force ($e = \infty$).
In the range between $e = 0$ and $e = \infty$:

where $e/h < 0.4$

$$k_c = \frac{1+\dfrac{e}{0.4h}}{1+\dfrac{6e}{h}} \quad \text{(Eq. C5A–20)}$$

where $e/h \geq 0.4$

$$k_c = \frac{1+\dfrac{0.4h}{e}}{2.5\left(1+\dfrac{h}{6e}\right)} \quad \text{(Eq. C5A–21)}$$

k is a coefficient which allows for the effect of non-uniform self-equilibrating stresses. The value can be taken as 0.8 as a first approximation.

k_p is a coefficient which takes account of the prestressing effect:

$$k_p = 1 - \frac{\alpha}{k_c}\left(1 - k_c + 2.4\frac{e_v}{h} - 6\frac{e_v k_c}{h}\right) \quad \text{(Eq. C5A–22)}$$

where

α is the ratio of prestressing $\dfrac{\sigma_{cp}}{kf_{fct,ef}}$ .... (Eq. C5A-23)

$$\sigma_{cp} = \frac{N_{Sd}}{A_c}\text{kPa}$$

N_{Sd} is the prestressing force (N)

A_c is the cross section of concrete (mm^2)

e_v is the eccentricity of the prestressing force (mm)

if $e_v = 0$

$$k_p = 1 - \frac{\alpha}{k_c}(1 - k_c) \quad \text{(Eq. C5A-24)}$$

for pure bending ($k_c = -0.4$), it follows that:

$$k_p = 1 - 1.5\alpha \quad \text{(Eq. C5A-25)}$$

C5.A6 Detailing provisions

The rules applicable to normal reinforcement (bar, mesh) and prestressing tendons can be found in NZS 3101. Only requirements applicable to "steel fibre reinforced concrete" will be discussed below.

C5.A6.1 *Shear reinforcement in beams*

A minimum shear reinforcement is not necessary for members with steel fibres. But the fibres must be guaranteed that the fibres have a significant influence on the shear resistance. Fibre type and fibre dosage must be sufficient so that a characteristic residual flexural tensile strength $f_{Rk,4}$ of 1.0 MPa is achieved.

C5.A7 Derivation of stresses in $\sigma - \varepsilon$ diagram test

The stresses σ_2 and σ_3 in the $\sigma - \varepsilon$ diagram are derived from the residual flexural tensile strength as explained below.

The residual flexural tensile strength $f_{R,1}$ and $f_{R,4}$ are calculated considering a linear elastic stress distribution in the section [5A.3]. (Figure C5.A7 (a)). However, in reality, the stress distribution will be different. To calculate a more realistic stress σ_f in the cracked part of the section, the following assumptions have been made (Figure C5.A7 (b)): the tensile stress σ_f in the cracked part of the steel fibre concrete section is constant.

The crack height is equal to ±0.66 h_{sp} at $F_{R,1}$, to ±0.90 h_{sp} at $F_{R,4}$ respectively.

Requiring $M_1 = M_2$, σ_f can then be expressed as:
$\sigma_{f,1} = 0.45\ f_{R,1}$
$\sigma_{f,4} = 0.37\ f_{R,4}$

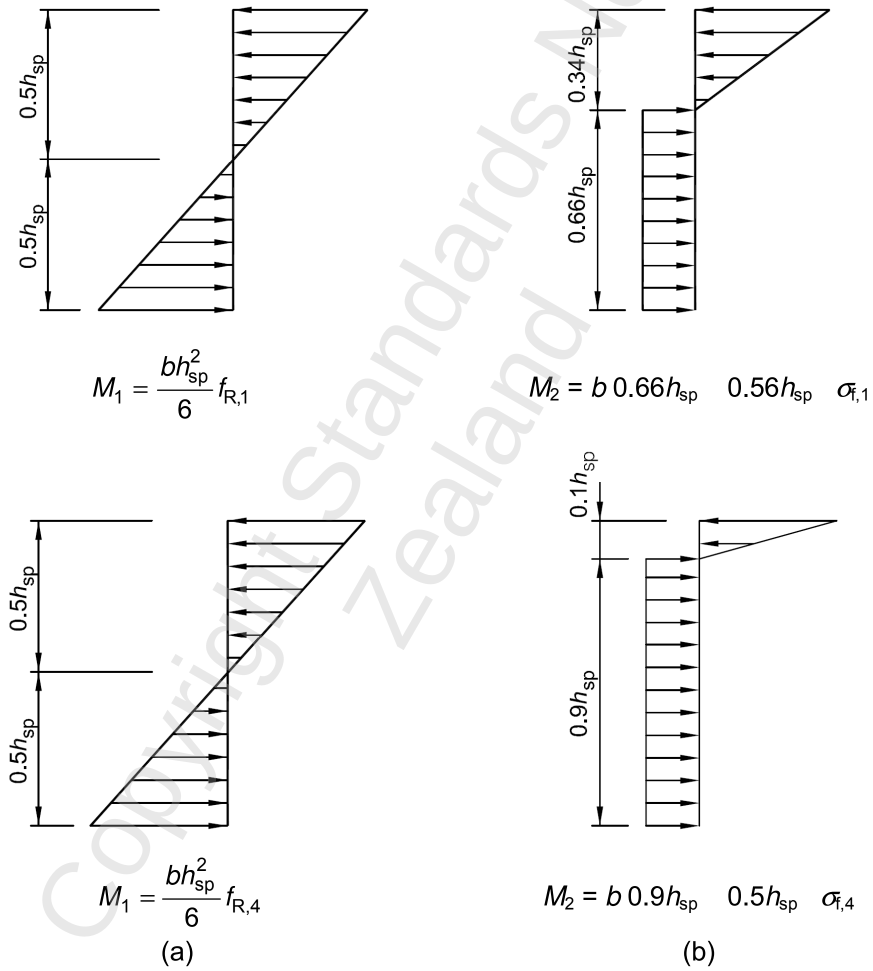


Figure C5.A7 – Stress distribution

REFERENCES

5A.1 RILEM TC 162-TDF: "Test and Design Methods for Steel Fibre Reinforced Concrete". P-8 Design Method. Materials and Structures, Vol. 36, October 2003, pp. 560-567.

5A.2 Vandewalle, L. et al., Recommendations of RILEM TC162-TDF: "Test and Design Methods for Steel Fibre Reinforced Concrete: Bending Steel" (Final Recommendation), Mater, Struct. 36 (2003), pp 560-567.

5A.3 ENV 1992-1: Eurocode 1: "Basis of Design and Actions on Structures" – Part 1: "Basis of Design", Annex D.3.2: Statistical Evaluation of Resistance/Materials Tests, 1994, pp. 78-80.

NOTES

C6 METHODS OF STRUCTURAL ANALYSIS

C6.1 Notation

The following symbols which appear in this section of the Commentary are additional to those used in Section 6 of the Standard:

a_s length of a support in the direction of the span, mm A3

A_g gross area of section, mm^2
A'_s area of compression reinforcement, mm^2
A_s area of non-prestressed flexural tension reinforcement, mm^2 A3
c_b depth of neutral axis at balanced conditions, mm

EI flexural rigidity of a member, N mm^2
f_y lower characteristic yield strength of non-prestressed reinforcement, MPa A3
h overall thickness or depth of member, mm
h_c depth of column, mm A3

L length of member between support centre lines (Fig C6.3), mm
L_1, L_2, L_3, L_4 Span length 1, span length 2 etc. in a slab (Fig C6.3)
L_d development length, mm
L_n clear span of member measured from face of supports, mm
L_o span length for determining static moment, mm
L'_o the smaller value of L_o for the adjoining spans, mm
L_t width of the design strip, mm
L_x clear span in short direction of rectangular slab, mm
L_y clear span in long direction of rectangular slab, mm
M bending moment, N mm
M_o total moment on span of design strip, and decompression moment in a prestressed member or a member under axial load (C6.8), N mm
M^*_v bending moment transferred from the slab to the support, N mm
M^*_x design bending moment at mid-span in X direction, N mm
M^*_y design bending moment at mid-span in y direction, N mm
N^* design axial load at ultimate limit state, N
p proportion of longitudinal flexural tension reinforcement in a beam A3
R a coefficient for modifying a beam or wall's effective second moment of area for the deformation due to development of reinforcement into the supporting member(s)
V shear force, N
v_c shear stress resisted by concrete, MPa

W_u uniformly distributed design load per unit dimension, factored for strength, N mm
α_c a coefficient to allow for deformation due to diagonal cracking (C6.9.1) A3
β_x, β_y factors for determining moments in two-way slabs
δ_x, δ_y numerical values (C6.7.3)
ϕ strength reduction factor
μ structural ductility factor

C6.2 General

C6.2.1 *Basis for structural analysis*

In the design of a structure, action effects such as bending moment, shear force and axial force must be determined at critical sections under the load combinations for both the ultimate and serviceability limit states. Various methods of structural analysis can be used in structural design, and 6.2.3 prescribes those that may be used in the design of concrete structures. Clause 6.2.1 indicates that any method of analysis, even the semi-empirical ones allowed in 6.2.1 must be used with understanding and in accordance with the basic principles of structural mechanics.

C6.2.2 *Interpretation of the results of analysis*

This clause emphasizes that the designer must consider carefully the design implications of all the simplifications and idealisations that are inevitably made in any structural analysis.

C6.2.3 *Methods of analysis*

This clause specifies the methods of analysis that may be used for the determination of action effects at the ultimate and serviceability limit states. The most frequently used methods of analysis, based on elastic concepts, are dealt with first, and the simplified approximate methods last. Plastic collapse methods of analysis for slabs, continuous beams and frames are included as well as more accurate non-linear methods of analysis based on computer simulation.

Where an element or part of an element has strain profiles that are not linear, such as anchorage zones of
A3 prestressed members, (see Section 16 and 19.3.13), deep beams (including pile caps, foundation beams) and floors acting as diaphragms, strut and tie models may be used.

C6.2.4 *Vertical loads on continuous beams, frames and floor systems*

The load arrangements specified in this clause should be sufficient for normal structures. A more extensive investigation should be undertaken for unusual structures.

C6.3 Linear elastic analysis

C6.3.1 *Application*

Concrete structures behave in a linear elastic manner only under small, short-term loads. With increasing load, cracks develop in the peak moment regions, and as the non-linear effects become increasingly important, the moment distribution departs more and more from the initial linear elastic distribution. Nevertheless, 6.3 allows the use of linear elastic methods to determine the moments, shears, etc. at both the serviceability and ultimate limit states.

While overall elastic behaviour is assumed in the structural analysis to determine moments in the structure as the basis for ultimate strength design, local inelastic action is at the same time assumed in undertaking the strength design of individual cross sections. Provided the structure is ductile, this design approach is justified by the lower bound theorem of plasticity.

C6.3.2 *Span lengths*

The centre-to-centre span is used in the analysis for equilibrium and static compatibility reasons. The
A3 finite size of supporting members is taken into account by 6.3.2 which defines the critical section for strength design.

C6.3.5 *Stiffness*

Clause 6.3.5 does not give specific values for stiffness to be used in elastic analysis for strength design. It only requires that reasonable assumptions be made to represent the limit state being considered, and that these assumptions be applied consistently throughout the structure. Within these limits, the designer is free to choose appropriate stiffness values.

One common assumption that is made for member stiffnesses, for determining design moment actions for both the ultimate and serviceability limit states, is to use 0.8 $E_c I_g$ for columns and 0.4.$E_c I_g$ for flexural members. The value of I_g is based on the gross section second moment of area (moment of inertia). Changing the stiffness values generally has only a small effect on the magnitudes of critical design moments. However, where deformations are to be calculated, be these plastic hinge rotations or deflections, it is important that realistic stiffness values are used.

Where deflections are to be assessed, or where a more accurate assessment of moments is required in the serviceability limit state, the stiffness values should be calculated as set out in 6.8. Where inelastic rotations are to be calculated in potential plastic zones, the EI value should be based on section properties that are consistent with the principles set out in 6.8.1 and either 6.8.2 or 6.8.3 or the value specified in the clause. A3

C6.3.6 *Secondary bending moments and shears resulting from prestress*

When prestress is applied to an indeterminate structure, the support restraints are likely to induce hyperstatic (parasitic) reactions, internal moments and other stress resultants. These 'secondary' effects must be taken into account in designing the structure for strength and serviceability. At service load, fully prestressed concrete structures are commonly uncracked and partially prestressed members are subject usually to only slight cracking. For serviceability design purposes, therefore the hyperstatic reactions and secondary moments and shears may be determined by elastic analysis of the uncracked structure (see 19.3.8 and 19.3.9). A3

C6.3.7 *Moment redistribution in reinforced concrete for ultimate limit state*

Where design actions for the ultimate limit state are obtained from an elastic analysis the values of moments, shears and reactions may be subsequently modified in recognition of inelastic behaviour. As the moments given in C6.7.2 and C6.7.3 already include approximations, their further reduction is not permitted.

In calculating inelastic rotations in potential plastic zones, it is essential to use realistic, or slightly conservative section properties in the analysis. Hence, unless a more fundamental analysis is made, it is recommended that calculations are based on the use of I_{cr} and E_c. A3

C6.3.7.1 *General requirements*

If the load on a statistically indeterminate structure is increased progressively from a low value to a relatively high value, the member behaviour changes from elastic to inelastic and there is a corresponding change in the relative magnitude of the moments at critical sections, i.e., a redistribution of internal moments occurs. If the structure is ductile, the moments change progressively from an initial linear elastic distribution to a fully plastic distribution, with plastic hinges forming in the peak moment regions eventually producing a mechanism. Equilibrium is always to be maintained. A3

C6.3.7.1.1 *Redistribution permitted*

Clause 6.3.7.2 gives criteria that can be used to determine the level of moment redistribution available in uniform beams where the proportion of longitudinal flexural reinforcement in the high positive and negative moment regions of a beam is of similar magnitude. Where these conditions are not satisfied, or where moment redistribution needs to be maximised, or the redistribution arises from settlement of supports, the analytical approach outlined in 6.3.7.1.1 should be used.

In the design for flexure for reinforced concrete beams and slabs an upper limit for level of reinforcement that may be used is based on the criterion that the neutral axis depth should not exceed 0.75 times the neutral axis depth at balance (0.75 c_b), see 9.3.8.1. This criterion has been set to allow for the variation in concrete strength and stiffness in different locations in members and to ensure these is always at least a small level of ductility available to cater for unexpected loading conditions, such as thermal loading, which might not have been specifically considered in design.

In carrying out redistribution of bending moments it is important not to remove the level of safety associated with the maximum reinforcement limit. The 0.75 c_b limit corresponds to a tensile strain in the reinforcement of $k\varepsilon_y$ where $k\varepsilon_y$ is 2.0 for Grade 300 reinforcement and 1.733 for Grade 500

A3 reinforcement. Hence to maintain this level of protection the tensile strain associated with elastic rotation is taken as $k\varepsilon_y$ in Equation 6–1.

A3 **C6.3.7.2** *Deemed-to-comply approach for reinforced concrete beams and one-way slabs for non-seismic cases and seismic load cases in nominally ductile plastic regions*

The extent to which moment redistribution can occur depends on the ductility, or potential for plastic deformation, in peak-moment regions and the span length of the member.

The ratio of the depth of the neutral axis to the effective depth of the member gives a measure of the inelastic deformation capacity of the plastic region in the beam. However, the ratio of the span of the member to the effective depth gives a measure of the level of inelastic deformation associated with moment redistribution. Consequently, both the ratios of c/d and L/d need to be considered in assessing the level of redistribution that can be safely assumed in design.

The equation used in the Standard previously for assessing moment redistribution failed to allow for the effect of length of span on the inelastic demand and it overpredicted the magnitude of moment redistribution that could be used safely in some cases.

For design purposes, moment redistribution is taken as a percentage increase or decrease in the elastically determined bending moment at a particular cross section, with an appropriate adjustment of the bending moment at all other sections so that the resulting bending moments and shear forces are in equilibrium with both the vertical and horizontal external loads and forces.

In a framed structure with beams constructed fully integral with their supports, for a single bending moment diagram, reduction of the support negative moments by up to 30 % will require an increase in the maximum positive moment in the mid-span region by the average of the reduction in the support negative moments. It will also result in a shift along the beam of the points of contraflexure in the redistributed moment bending moment diagram over the region 'X' as shown in Figure C6.1. In the negative moment region of the elastic analysis bending moment diagram, reduction of the support moment by 30 % will result in a greater than 30 % reduction of the moment over the rest of these regions, and a reversal of the sign of the moment between the elastic and redistributed moment diagrams points of contraflexure. It must be noted that the design is still to provide for at least 70 % of the elastic analysis bending moment throughout the elastic analysis negative moment regions to provide for the serviceability limit state.

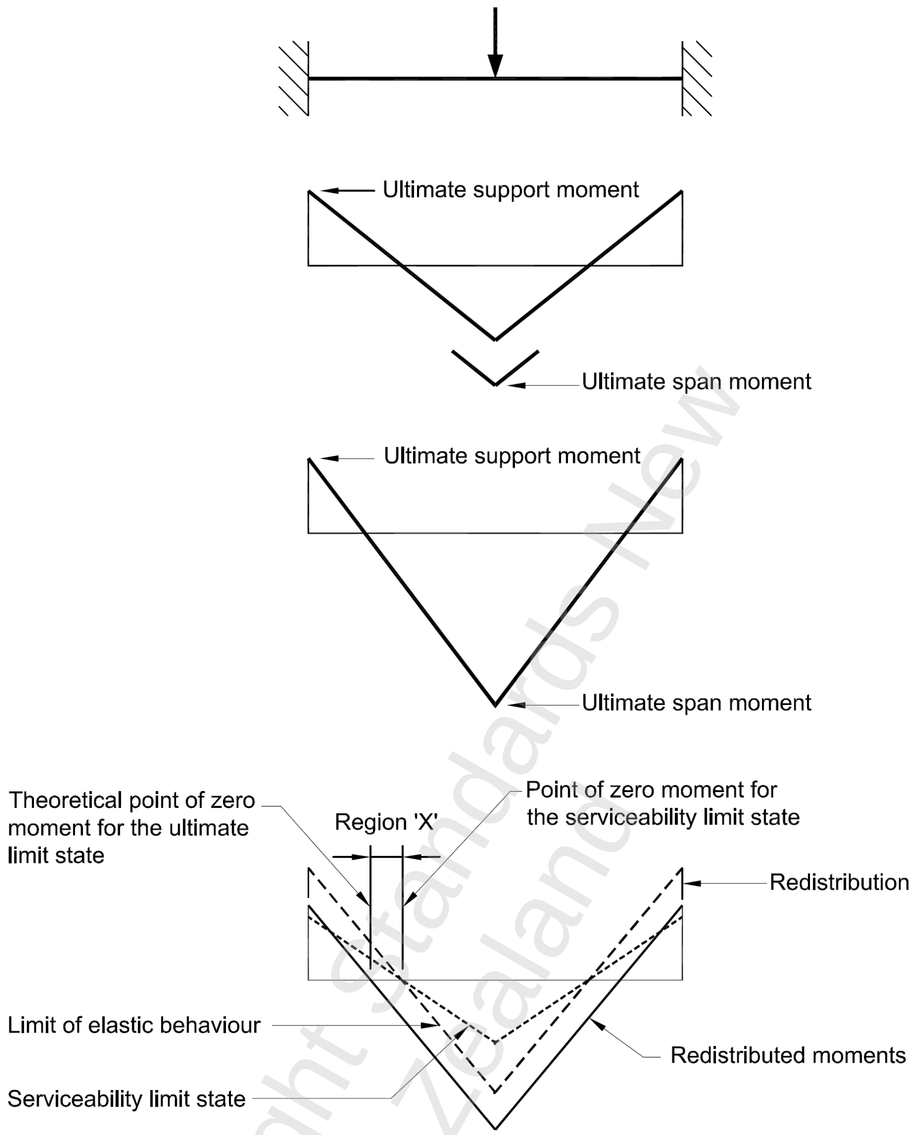


Figure C6.1 – Redistribution of moments

C6.3.8 *Idealised frame method of analysis*

This clause applies to the analysis of multi-storey 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 (d).

(a) *Idealised frames*

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

Each idealised 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 idealised frame in accordance with either 6.3 or 6.4 and the general requirements of 6.2.1 and 6.2.2.

For beams and slabs built integrally with supports the critical section for maximum negative bending moment may be taken at the face of the support.

(b) *Analysis for vertical loads*

The arrangement of vertical loads to be considered in the analysis of an idealised frame shall be in accordance with 6.2.4. 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:

(i) 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 that the floor is fixed at the support one span away, provided that the floor continues beyond that point.

(ii) 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 and held against sway.

A3 For the purposes of these analyses generally the longitudinal deformation in columns due to axial load can be neglected as elongation associated with flexural cracking tends to counteract this. However, where creep and shrinkage deformations may be high, or where settlement of foundations is anticipated, allowance should be made for this deformation.

In order to provide for live load acting on part of a span, the minimum shear force due to live load in any section of a member shall be taken as one-quarter of the maximum shear force due to live load in the member when subjected to uniformly distributed live load.

(c) *Analysis for horizontal loads*

A3 Floor slabs acting as horizontal diaphragms that distribute lateral forces among the frames is covered in Section 13.

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

(d) *Idealised frame method for structures incorporating two-way slab systems*

(i) *Application*

It is permissible to apply the idealised two-dimensional method of frame analysis to regular, 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 (d)(v) below; and

(E) Beam-and-slab systems, including thickened slab bands.

(ii) *Effective width*

The idealised 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 9.3.1.3.

(iii) *Distribution of bending moments between column and middle strips*

In the idealised 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 C6.1.

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 C6.1 – Distribution of bending moments to the column strip

Bending moment under consideration	Column strip moment factor
Negative moment at an interior support	0.60 to 1.00
Negative moment at an exterior support	0.75 to 1.00
Positive moment at all spans	0.50 to 0.70

(iv) *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 Section 7, as applicable.

In beam-and-slab construction, the spandrel beams shall be reinforced with at least the minimum torsional reinforcement required by Section 7.

(v) *Openings in slabs*

Slabs containing openings may be analysed in accordance with all of Section 12 without the need for further calculation provided that 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 that 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 Section 12.

C6.4 Non-linear structural analysis

C6.4.1 *General*

A rigorous analysis requires accurate mathematical modelling of the material properties as well as the structural behaviour. In practice, the analysis of structural behaviour at this level of complexity is undertaken using an appropriate computer facility and programme. At the present time, the use of this method will probably be restricted to exceptional structures. As more refined computer programmes and more powerful and cheaper computer facilities become available, use of rigorous methods of analysis can be expected to increase substantially. Provision is therefore made for non-linear analysis in this clause.

C6.4.2 *Non-linear material effects*

In concrete structures, the main sources of non-linear structural behaviour arise through non-linear material behaviour. This clause lists various sources of material non-linearity.

C6.4.3 *Non-linear geometric effects*

This clause draws attention to the fact that non-linear effects in concrete structures may also arise from geometric non-linearities, particularly when individual components are relatively slender.

C6.4.4 *Values of material properties*

Throughout this standard design strengths are based on lower characteristic material strengths, which are referred to as design strengths. However, in practice the use of lower characteristic material strengths and stiffness values will result in an over-estimate of deformation in the structure as a whole. Hence for the purposes of assessing deformation and distribution of actions in a structure the average material properties may be used. However, to retain the required safety index in the ultimate limit state all section strengths must be on design strengths.

C6.5 Plastic methods of analysis

As slabs usually contain relatively small proportions of reinforcement, the moment curvature graph for a typical slab segment has a long, almost flat plateau. In addition, one-way continuous and two-way slabs are statically indeterminate and are capable of undergoing significant redistribution of moments. Plastic methods of analysis therefore are eminently suitable for slabs. An important practical advantage is that the methods can be applied to slabs of irregular and complex shapes.

For more detailed information on the various methods of plastic analysis for slabs, see References 6.1, 6.2, 6.3, 6.4, 6.5, 6.6, 6.7, 6.8, 6.9, and 6.10.

C6.6 Analysis using strut-and-tie models

Although not formalised in most codes, the inappropriateness of the application of flexural theory based on linear strain distributions to squat members and regions of discontinuity has long been recognised. Details of analysis based on strut-and-tie models that idealise admissible internal load paths, are given in Appendix A. This approach is applicable to deep beams, corbels and brackets, diaphragms and walls particularly with openings, and regions of discontinuity in members.

C6.7 Simplified methods of flexural analysis

C6.7.1 *General*

The simplified methods of analysis contained in this clause are appropriate for hand calculation.

C6.7.2 *Simplified method for reinforced continuous beams and one-way slabs*

This clause provides a simple, approximate and conservative method for evaluating the moments and shears in certain continuous reinforced beams and one-way slabs.

If moment reversals occur during construction caused by temporary propping or similar actions, a separate analysis will be required.

Note that the moment values at different cross sections are not statically compatible and so should not be used for deflection calculations.

(a) *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 W_u is the uniformly distributed design load per unit length, factored for strength):

(i) At the first interior support:

(A) Two spans only .... $W_u L_n^2/9$

(B) More than two spans .... $W_u L_n^2/10$

(ii) At other interior supports .... $W_u L_n^2/11$

(iii) At interior faces of exterior supports for members built integrally with their supports:

(A) For beams where the support is a column .... $W_u L_n^2/16$

(B) For slabs and beams where the support is a beam .... $W_u L_n^2/24$

(b) *Positive design moment*

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

(A) In an end span .... $W_u L_n^2/11$

(B) In interior spans... $W_u L_n^2/16$

(c) *Transverse design shear force*

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

(i) In an end span:

(A) At the face of the interior support.. 1.15 $W_u L_n/2$

(B) At mid-span... $W_u L_n/7$

(C) At the face of the end support.. $W_u L_n/2$

(ii) In interior spans:

(A) At the face of supports .... $W_u L_n/2$

(B) At mid-span.. $W_u L_n/8$

C6.7.3 *Simplified method for reinforced two-way slabs supported on four sides*

The design bending moments for strength can be determined using the simple moment coefficients given for certain two-way slabs supported on four sides by beams or walls.

(a) *Design bending moments*

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

(i) The positive design bending moments at mid-span, M^*_x and M^*_y, on strips of unit width spanning, L_x and L_y, (where $L_y > L_x$) respectively shall be calculated from the following equations:

$$M^*_x = \beta_x W_u L_x^2 \qquad \text{(Eq. C6–1)}$$

$$M^*_y = \beta_y W_u L_x^2 \qquad \text{(Eq. C6–2)}$$

where W_u is the uniformly distributed design load per unit area factored for strength and β_x and β_y are given in Table C6.2.

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 be complied with by the minimum strength requirement for slabs.

(ii) The negative design bending moments at a continuous slab edge shall be taken as 1.33 times the mid-span values in the direction considered.

If the negative moment on one side of a common support is different from that on the other side, the unbalanced moment may be redistributed.

(iii) The negative design bending moment at a discontinuous edge, where there is a likelihood of restraint, may be taken as half the mid-span value in the direction considered.

(b) *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 12.5.6.7.

(c) *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 C6.2.

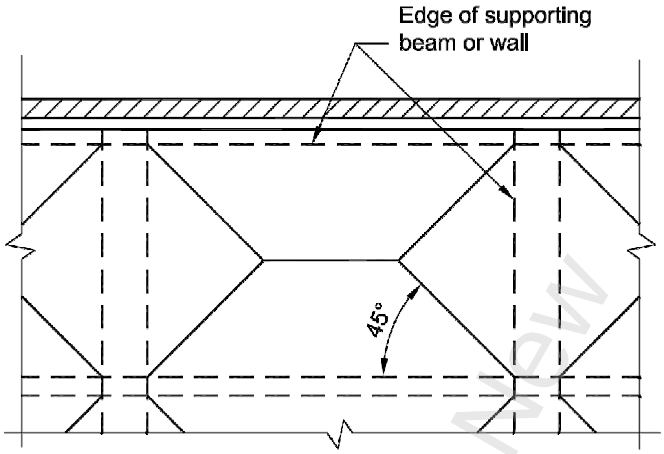


Figure C6.2 – Allocation of load

Table C6.2 – Positive bending moment coefficients for rectangular slabs supported on four sides

Edge condition	Short span coefficients (β_x) Values of L_y/L_x								Long span coefficients (β_y) for all 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

The positive moment coefficients specified in Table C6.2 are derived from yield-line theory and are obtained from the following equations:

$$\beta_y = \frac{2\left\{\sqrt{3+\left(\dfrac{\delta_x}{\delta_y}\right)^2} - \dfrac{\delta_x}{\delta_y}\right\}^2}{9\delta_y^2} \qquad \text{(Eq. C6–3)}$$

$$\beta_x = \frac{\left(\frac{L_x}{L_y}\right)\beta_y + 2\left[1 - 1\left(\frac{L_x}{L_y}\right)\right]}{3\delta_y^2} \qquad \text{(Eq. C6–4)}$$

where

δ_x = 2.0 if both short edges are discontinuous
= 2.5 if one short edge is discontinuous
= 3.1 if both short edges are continuous

δ_y = values as for δ_x but referred to the continuity of the long edges

The negative design moment at a continuous edge, or at a restrained discontinuous edge, is taken as a factored value of the positive moment.

These values should not be used for defection calculations (see 6.8).

C6.7.4 *Simplified method for reinforced two-way slab systems having multiple spans*

Two-way systems can be analysed for bending moments and shear forces either by the simplified method given here in C6.7.4 or by the idealised frame method described in C6.3.8.

Two-way slab systems are statically indeterminate to a large degree and can exhibit considerable variation in redistribution of moments from the uncracked state to final maximum capacity [6.11, 6.12]. Recent tests on edge panels[6.13, 6.14] have not only confirmed this but have indicated that when approaching maximum load capacity, the distribution of moments is controlled largely by the distribution of steel in the slab.

Thus in the analysis stage, there is no unique moment field which the designer needs to determine. Within wide limits, whatever moment the designer adopts should be acceptable for determining the flexural strength for the slab provided that equilibrium is satisfied.

Furthermore, the flexural strength of the slab is enhanced significantly by the development of very large in-plane forces (membrane action) as the slab approaches failure. In the case of slab-beam systems, this increased flexural strength is many times larger than the value calculated by ignoring the effect of in-plane forces. Recent tests[6.13] have shown that even in the case of a flat-plate floor, the in-plane forces significantly increase the flexural strength of the slab.

All these facts suggest that in the design process, any analysis involving a high degree of refinement is quite unnecessary and bears no relation to reality.

The designer is reminded that a more important consideration in the safety of a flat-slab system is the transfer of forces from the slab to the support by a combination of flexure, shear and torsion [6.15]. (See Section 12).

Definitions

Design strip, column strip and middle strip. The definitions embody all the rules necessary for laying out the various strips used for the design and detailing of two-way slab systems. Note that if the support grid is not rectangular throughout (i.e., one or more columns are offset), the transverse widths of the strips will vary along the affected spans. This will affect both the load and the stiffness of those spans.

For the purpose of this section, the following definitions apply:

(a) *Column strip*

That portion of the design strip extending transversely from the centreline of the supports:

(i) For an interior column strip, one-quarter of the distance to the centreline of each adjacent and parallel row of supports; or

(ii) For an edge column strip, to the edge of the slab and one-quarter of the distance to the centreline of the next interior and parallel row of supports;

but of total width not greater than $L/2$, as shown in Figure C6.3.

(b) *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 centreline of the supports:

(i) For an interior design strip, halfway to the centreline of each adjacent and parallel row of supports; or

(ii) For an edge design strip, to the edge of the slab and halfway to the centreline of the next interior and parallel row of supports (see Figure C6.3).

(c) *Middle strip*

The portion of the slab between two column strips or between a column strip and a parallel supporting wall (see Figure C6.3).

(d) *Span support*

The length of a support in the direction of the span (a_s) taken as:

(i) For beams or for flat slabs without either drop panels or column capitals, the distance from the centreline of the support to the face of the support; or

(ii) For flat slabs with drop panels or column capitals or both, the distance from the centreline 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 centreline of the support, which lies entirely within the surfaces of the slab and the support, as shown in Figure C6.4.

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

(e) *Transverse width*

The width of the design strip (L_t) measured perpendicular to the direction of bending being considered (see Figure C6.3)).

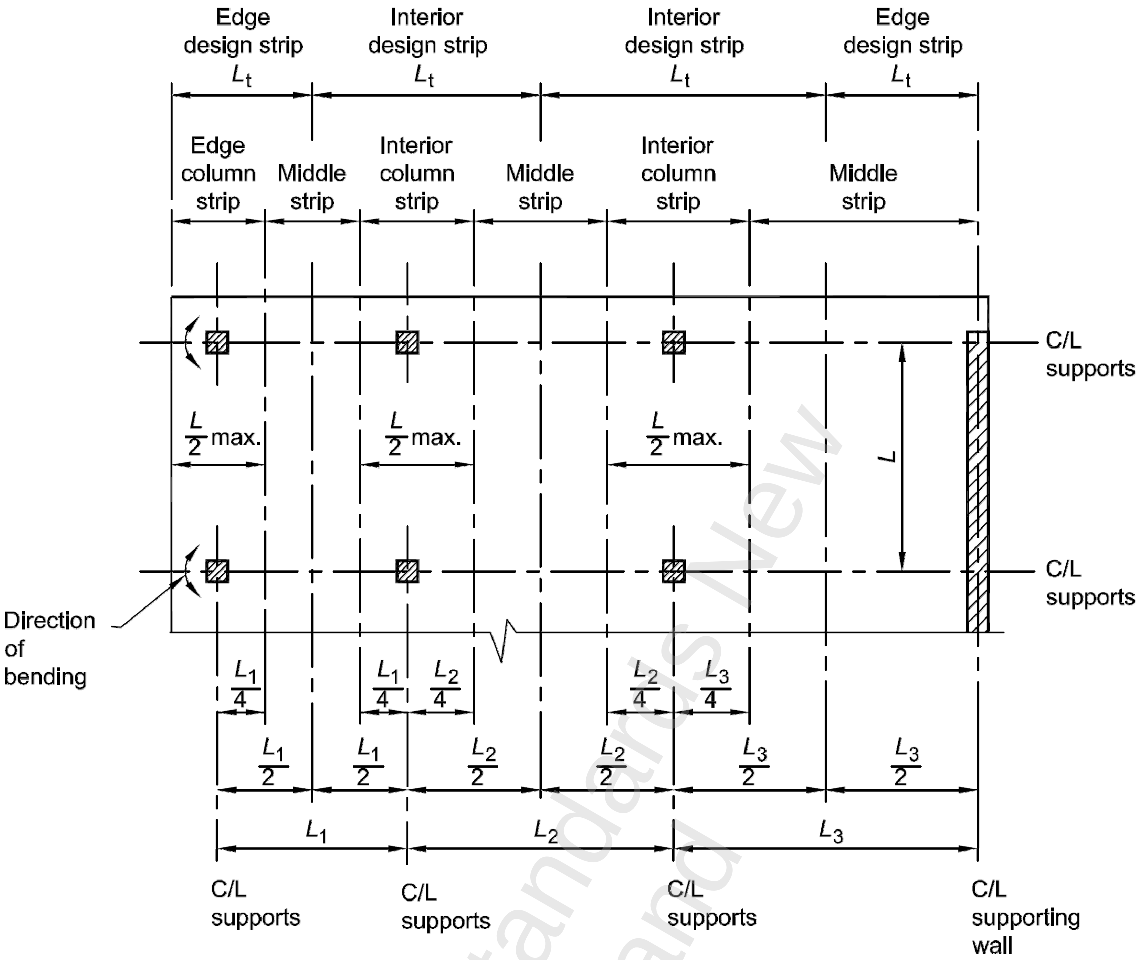


Figure C6.3 – Widths of strips for two-way slab systems

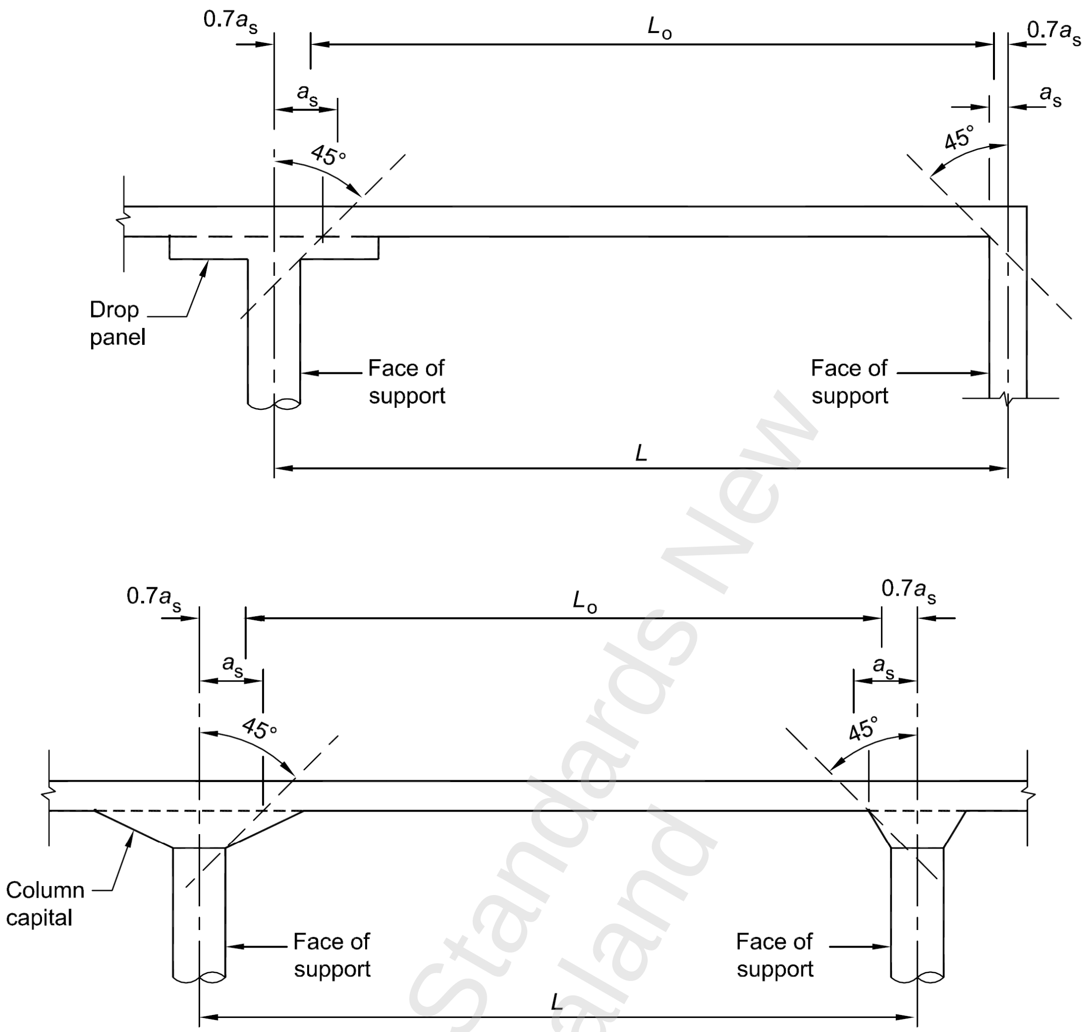


Figure C6.4 – Span support and span lengths for flat slabs

Total static moment for a span

The total static moment (M_o), for a span of the design strip shall be taken as equal to or greater than:

$$M_o = \frac{W_u L_t L_o^2}{8} \quad \text{(Eq. C6–5)}$$

where

W_u is the uniformly distributed design load per unit area, factored for strength

L_t is the width of the design strip

L_o is the L minus 0.7 times the sum of the values of a_s at each end of the span (see Figure C6.4)

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 Table C6.3 and Table C6.4.

These design moments may be modified by up to 10 % provided that 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 C6.3 – 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 C6.4 – Design moment factors for an interior span

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

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 C6.3.8 (d)(iii).

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:

$$M^*_v \geq 0.06\,[(1.25G + 0.75Q)L_t(L_o)^2 - 1.25GL_t(L'_o)^2 \qquad \text{(Eq. C6–6)}$$

where

L'_o is the smaller value of L_o for the adjoining spans

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

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 C6.7.3(c).

Openings in slabs

Only openings that comply with the requirements of C6.3.8(d)(v) shall be permitted in slabs analysed using the above simplified methods.

C6.8 Calculation of deflection of beams and slabs for serviceability limit state A3

C6.8.2 *Deflection calculation with a rational model*

Many factors influence the deflection of reinforced concrete and there are few realistic analytical models that can be used in practical design. Flexural cracking has a major influence on the stiffness of a section. However, the concrete between cracks can resist some of the tension force and by this means the average strain in the reinforcement is reduced. This is known as tension stiffening. As cracks develop tension stiffening reduces. With creep in the concrete the compression strains in concrete increase and tension stiffening reduces. This increases section curvatures and deflections. Shrinkage of the concrete can have two effects on deformation. First, it can result in initial compression strains being induced in the A3

A3 reinforcement, which when released by flexural cracking increases the change in strain in the reinforcement, the section curvature and the deflection. Second, shrinkage of the concrete increases the strain in the compression zone, which increases curvature and deflections. It should be noted that the sign of the curvature due to shrinkage depends on the sign of the gravity load bending moment acting in that location. Shrinkage acts to increase the strains on the compression side of the member. Hence shrinkage curvatures cannot be assessed in isolation from gravity load actions.

C6.8.3 *Calculation of deflection by empirical method*

A3 The empirical method of calculating deflections of reinforced concrete members was developed in the 1960s from tests on reinforced concrete beams with typical reinforcement contents used at the time. The reinforcement ratios in these tests did not include the low reinforcement quantities, such as those currently used in slabs. Recent research [6.26, 6.27, 6.28] has shown that this empirical method tends to underestimate long-term deflections of slabs where shrinkage tends to be high. It has been suggested that this underestimate occurs in part due to the flexural cracking moment being reduced by tensile stresses associated with shrinkage of the concrete. Where the long-term deflection of slabs with reinforcement proportion less than 0.7 % is critical, designers are advised to study the relatively recent literature on this topic[6.26, 6.27, 6.28].

(a) *Short-term deflection*

Equations 6–2 and 6–3, which are to be used to assess effective second moments of area for relatively uniform beams, or for regions of beams, were developed by Branson[6.16] from test results.

In calculating immediate deflection for uncracked members standard theory may be used assuming the second moment of area and elastic modulus are I_g and E_c. Where flexural cracking occurs an effective second moment of area, I_e, needs to include an allowance for tension stiffening between the flexural cracks. For simply supported beams or cantilever beams, where the sign of the moments does not change, Equation 6–2 can be used to assess an equivalent uniform second moment of area for the member. In this case M_a is the maximum moment in the span.

For continuous members Equation 6–2 can be used to assess equivalent second moments of area for the negative and positive moment regions of the member. In this case the M_a values are the maximum moments in their respective areas.

For non-uniform members, or where the variation in stiffness along a member is important, effective second moment of area values can be assessed at regular intervals along the member from Equation 6–3. These can be used to find the curvatures along the member and hence the deflected shape.

The short-term flexural deflection of members subjected to flexure and axial load may be assessed by finding the section properties at regular intervals along the member and determining the curvatures at each of these sections. These values can be used to find the deflected shape using standard flexural theory.

To find the short-term curvature at each section:

(i) Where the bending moment is less than the predicted flexural cracking moment, M_{cr}, the transformed section property may be used

(ii) Where the bending moment exceeds the predicted flexural cracking moment, the bending moment is divided into two parts. The first is the bending moment that is less than the decompression moment for the extreme tension fibre, M_o. The second is bending moment that exceeds M_o. The curvature for the first part is calculated using the transformed section properties, while the curvature for the second part can be assessed using the second moment of area, I_{se},, given by Equation 6–3, in which M_{cr} is replaced by (M_{cr} – M_o) and M_a by (M_a – M_o).

The component of long-term deflection given in 6.8.3(b) does not apply to members subjected to flexure and axial load. Some guidance for the assessment of long-term deflection for these members is given in the paragraph below and in Appendix CE.

It is noted that for additional load increments, such as live load, I_e must be calculated for total moment M_a, and the deflection increment computed from the total deflection, as indicated in References 6.19 and 6.20. For simplicity in the case of continuous beams, the standard procedure suggests a simple averaging of positive and negative moment values to determine I_e. In certain cases, a weighted average relative to the moments may be preferable, such as the methods suggested in Reference 6.20. Alternatively, where there is a significant difference between the negative and positive moment values, the individual values may be used for their respective regions.

Laboratory tests of beams subjected to short-term loading indicate that the measured deflection predicted using Equation C6–2 is typically within ±30 %. Greater discrepancy must be expected in practical construction[6.21].

Shrinkage and creep due to sustained loads cause additional deflections over and above those that occur when loads are first placed on the structure. The additional deflections are called "long-term deflections". Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, quantity of compression reinforcement, magnitude of the sustained load, and other factors.

Many factors influence the magnitude of the long-term deflection, which develops in a member. Clearly a member made from concrete, which has a high free shrinkage value and/or a high creep factor, will sustain greater long-term deflection than a member where the creep and shrinkage potentials of the concrete are low. Reinforcement acts to restrain creep and shrinkage movements and for this reason the longitudinal reinforcement in the compression zone has a significant influence on the magnitude of the long-term deflection. However, this is the only factor recognised by Equation 6–4 and other factors, which can have a major influence, are not accounted for directly. The empirical equation for this component of deflection does not take into account the sensitivity of the concrete to its specific creep and shrinkage characteristics. Thus thin members in conditions with a low relative humidity can be expected to sustain greater deflections than those implied by Equation 6–4. Consequently the reliability of the total deflection will be considerably less than that for the short-term loading. Designers should treat the values predicted by this method as an indication of the likely order of deflection rather than as an absolute value. It should be noted that typically the long-term deflection takes five years to fully develop, but this may take longer for thick members. The basis of Equations 6–2 and 6–3 is given in Reference 6.16. A3

(b) *Calculation of long-term deflection* A3

It should be noted that the deflection calculated in (b) is the additional deflection which develops over time. The maximum deflection is obtained by adding this value to the short-term deflection calculated in (a).

C6.8.4 *Calculation of deflection – prestressed concrete* A3

The short-term deflection of prestressed members, where flexural cracking is expected to occur, may be assessed as proposed for reinforced members subjected to axial load in C6.8.3. For the calculation of long-term deflection of prestressed members see Appendix CE.

C6.8.5 ***Shored composite construction***

Since few tests have been made to study the immediate and long-term deflections of composite members, the rules given in 6.8.4 were based on the judgement of ACI Committee 318 and on experience.

If any portion of a composite member is prestressed or if the member is prestressed after the components have been cast, the provisions of 6.8.4 (c) apply and deflections must be calculated accordingly. For non-prestressed composite members, deflections need to be calculated and compared with the limiting values in the referenced loading standard for the relevant serviceability limit state criteria only when the thickness of the member is less than the minimum thickness given in Table 2.1 or Table 2.2. In unshored construction the relevant thickness depends on whether the deflection before or after the attainment of effective composite action is being considered.

The modified effective modulus method can be adopted to analyse for staged construction. This has the advantage that this gives a reasonable estimate of the creep redistribution of actions that occurs when a structural form is modified after either loading or prestress has been applied. The method of analysis described in C6.8.4 is applied to each stage of construction, with the creep and shrinkage values for the different concretes being based on the values that are expected to develop in the stage being considered.

C6.9.1 *Linear elastic analysis*

A3 **C6.9.1.1 and C6.9.1.2** *Analyses to be based on anticipated levels of cracking, and ultimate limit state deflections to allow for post-elastic effects*

To obtain realistic predictions of periods of vibration and of lateral deflections in seismic load cases, allowance needs to be made for the effects of cracking on member stiffness. This is particularly important in modelling structural walls. Unless structural analysis indicates that flexural cracking is likely to occur in a structural member in the storey being considered, either gross section properties, or transformed section properties should be used, assuming concrete does not crack. For the purpose of assessment of flexural cracking the critical tensile strength f_t should be taken as $0.55\sqrt{1.2f'_c}$. The coefficient of 1.2 makes an allowance for the likely long-term gain in tensile strength with time.

The effective stiffness of reinforced concrete members is influenced by many factors. These include:

(a) The quantity of longitudinal reinforcement in the tension zone of the member and its grade;

(b) The tensile strength of the concrete and the extent of cracking, which affects the magnitude of tension stiffening;

(c) The initial conditions in the member before the structural actions are applied. For example, shrinkage and creep of the concrete can induce compression in the reinforcement. When the section cracks in flexure the compression is released and curvature is induced in the member;

(d) Deformation due to development of longitudinal reinforcement in support zones of adjacent structural elements.

The factors listed in (a) and (b) are the most significant and they have been included in the assessment of the effective stiffness values for beams shown in Figure C6.5 and Table C6.5. The values in the figure and table give a general guide to the typical magnitudes of effective stiffness and these values are generally sufficient for the purposes of analysis, provided the structural actions are of sufficient magnitude to cause flexural cracking to occur. To allow for the difference between design strength and average concrete strength the elastic modulus used with values in Table C6.5 may be based on a concrete strength corresponding to $1.2f'_c$.

For columns and walls elongation occurs with flexural cracking. Consequently there is little to be gained in reducing the effective area of the section for axial load calculations related to axial deformation. However, if section stresses are to be calculated allowance needs to be made for the existence of cracks.

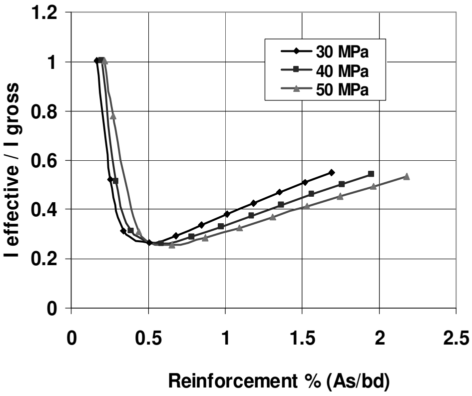
In most cases the area for shear deformation may be assumed to be 5/6 A_g. It is difficult to predict shear deformation, particularly in plastic regions, and reference should be made to specialist literature where it is judged essential to allow for this deformation. A method of allowing for shear deformation in coupling beams is described in C11.4.9.

In beams flexural cracking reduces the stiffness in the regions where high bending moments are sustained. To assess the effective stiffness of the beams, which are illustrated in Figure C6.5, section stiffness values were calculated at close centres along each beam using the empirical method proposed by Branson[6.16, 6.17], which is detailed in 6.8.3(a)(iii). The effective stiffness values were calculated for each beam when they were subjected to bending moments typical of those associated with sway of the structure. The bending moments at the ends of the beam were equal to the bending moment causing first yield of the flexural reinforcement.

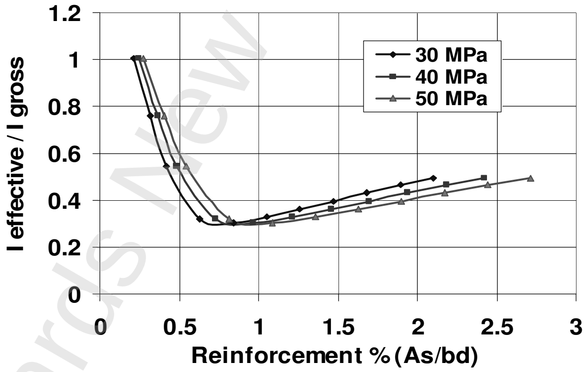
C6.5 shows in general terms how the effective stiffness at first yield of reinforcement of rectangular and T-beams varies with different concrete strengths and reinforcement proportions. With the T-beams the effective width of flange on each side of the web was assumed to be equal to half the beam depth while the thickness of the web was assumed to be close to 40 % of the beam depth.

Figure C6.5 illustrates the effect that the proportion of flexural reinforcement, $A_s/b_w d$, has on effective stiffness. With low reinforcement contents the stiffness is high. This occurs as flexural cracking does not extend far along each beam. With increasing reinforcement proportions the effective stiffness first decreases rapidly before increasing gradually with higher reinforcement contents. While concrete strength has relatively little influence on stiffness the reinforcement grade has a significant effect.

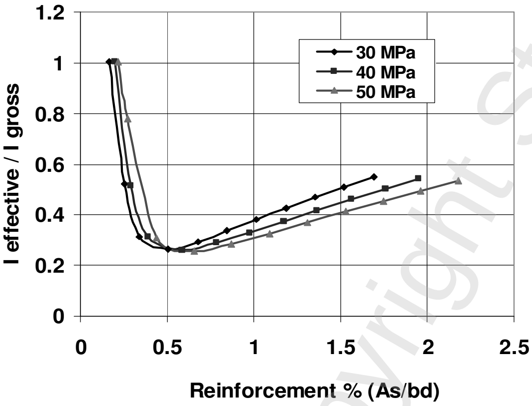
As identified in 7.8, elongation occurs in plastic hinges. This can result in structural elements being displaced and forces being induced in the member containing the plastic hinge as well as in the surrounding structure. It is important to allow for adverse effects that elongation may have as available software often does not model this action. In particular the deformation associated with elongation can significantly increase the relative movement of members such as precast floor units, stairs, ramps and panels, relative to the remainder of the structure. This movement has important implications in the design of support details. Design values of elongation are given in 7.8. Forces induced by elongation can have a significant influence on seismic performance. However, the Standard does not require the magnitude of elongation to be determined in the assessment of these forces. Design criteria are given in the appropriate clauses, which are based on the likely magnitudes of elongation that may be reached in a major earthquake and the ability of the surrounding structure to provide restraint to the imposed deformation. In the case of structural walls the maximum permitted axial load ratio $\left(N^*/A_g f'_c\right)$ has been limited to allow for the likely increase in axial load due to elongation of the wall. A3



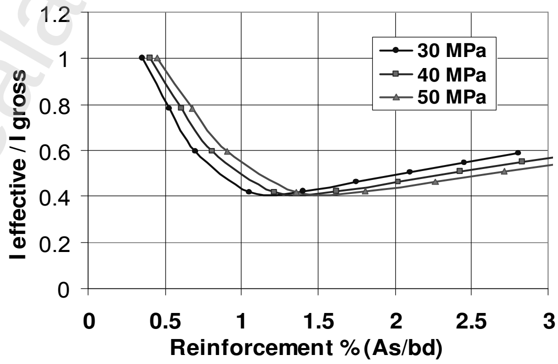
(a) Rectangular beam Grade 500



(b) Rectangular beam Grade 300



(c) T-beam Grade 500



(d) T-beam Grade 300

Figure C6.5 – Effective stiffness of beams

A3

Table C6.5 – Effective section properties as a proportion of gross section properties

Type of member	Ultimate limit state		Serviceability limit state		
	f_y = 300 MPa	f_y = 500 MPa	μ = 1.25	μ = 3	μ = 6
1 Beams					
(a) Rectangular ¶ (valid range)	$0.43\ I_g$ $0.5\% \le p \le 1.75\%$	$0.32\ I_g$ $0.3\% \le p \le 1.4\%$	I_g	$0.7\ I_g$	$0.50\ I_g$
(b) T and L beams ¶ (valid range)	$0.40\ I_g$ $0.75\% \le p \le 1.75\%$	$0.30\ I_g$ $0.45\% \le p \le 1.75\%$	I_g	$0.7\ I_g$	$0.50\ I_g$
2 Columns					
(a) $N^*/A_g f'_c > 0.5$	$0.80\ I_g\ (1.0\ I_g)$ ‡	$0.80\ I_g\ (1.0\ I_g)$ ‡	I_g	$1.0\ I_g$	As for the ultimate limit state values in brackets
(b) $N^*/A_g f'_c = 0.2$	$0.55\ I_g\ (0.66\ I_g)$ ‡	$0.50\ I_g\ (0.66\ I_g)$ ‡	I_g	$0.8\ I_g$	
(c) $N^*/A_g f'_c = 0.0$	$0.40\ I_g\ (0.45\ I_g)$ ‡	$0.30\ I_g\ (0.35\ I_g)$ ‡	I_g	$0.7\ I_g$	
3 Walls ¶					
(a) $N^*/A_g f'_c = 0.2$	$0.48\ I_g$	$0.42\ I_g$	I_g	$0.7\ I_g$	As for the ultimate limit state values
(b) $N^*/A_g f'_c = 0.1$	$0.40\ I_g$	$0.33\ I_g$	I_g	$0.6\ I_g$	
(c) $N^*/A_g f'_c = 0.0$	$0.32\ I_g$	$0.25\ I_g$	I_g	$0.5\ I_g$	
4 Coupling beams					
Diagonal and conventional reinforcement*	$0.40\ I_g \alpha_c$	$0.33\ I_g \alpha_c$	I_g	$0.5\ I_g$	As for the ultimate limit state values

NOTE –

¶ For additional flexibility, within joint zones and for conventionally reinforced coupling beams refer to the text.

* The value of α_c is given in Table C6.6B.

‡ The values in brackets apply to columns which have a high level of protection against plastic hinge formation in the ultimate limit state.

C6.9.1.3 *Walls, coupling beams and other deep members*

Diagonal tension cracking

Where diagonal cracking may have a significant effect on the elastic behaviour of a structure, allowance for the deformation should be made using a rational model.

Diagonal cracking in concrete members, where the shear stress exceeds $1.5v_c$, can be expected to increase the deformations by:

(a) Increasing the flexural tension force along the member due to tension lag, which increases the flexural deformation;

(b) Straining the shear reinforcement which induces shear deformation.

Structural walls where the ratio of h_w/L_w is equal to or less than 4.0

Allowance for the increase in lateral deflection due to diagonal cracking at first yield of longitudinal reinforcement may be made by multiplying the second moment of area used in flexural calculations (typical values are given in Table C6.5) by the coefficient α_w which are given in Table C6.6A.

A3

Table C6.6A – Factor α_w which allows for the reduction in stiffness due to diagonal cracking in structural walls

Ratio of h_w/L_w	0.5	0.75	1	1.5	2	3	4
α_w	0.25	0.32	0.45	0.6	0.7	0.75	0.8

The values for α_w have been found for typical members from an equation given in Reference 6.24, but simplified by removing parameters that do not have a major influence on the numerical values. Deflection calculations, found using modified second moments of area as described above, gave displacements at first yield that were comparable with measurements made in a limited number of structural tests reported in References 6.30 and 6.31.

Squat walls

Stiffness values and lateral strengths for walls with aspect ratios (h_w/L_w) less than 0.5 should be based on strut and tie analyses. The minimum angle between a diagonal strut and the horizontal should be equal to or greater than the larger of 25° or $\tan^{-1}\dfrac{h_w}{0.7L_w}$. In squat walls where h_w/L_w is less than 0.5, standard flexural theory can appreciably over-estimate the lateral strength due to the interaction of flexure and shear (see C7.8.11 and Figure C7.8).

Strains in the anchorage zone of longitudinal wall reinforcement in a foundation beam increases the lateral deflection of the wall. At first yield bond performance in the anchorage zone is generally good, giving relatively short development lengths. In addition it should be noted that the average development length is appreciably shorter than the design length given in the standard, as this needs to cover the scatter in test results. Allowing for these factors the equivalent extension of a bar in an anchorage zone at first yield is assessed as $\dfrac{0.11 f_y}{\sqrt{f'_c}}\varepsilon_y\, d_b$. Assuming that the distance from the longitudinal bar that first reaches its yield point is located at a distance of $0.7L_w$ from the zero strain axis, the lateral deflection due to anchorage of the reinforcement in the foundation beam at a height of h_w above the base at first yield, δ_h, is approximately $\delta_h = \dfrac{0.11 f_y\, \varepsilon_y\, d_b\, h_w}{0.7\, L_w \sqrt{f'_c}}$. This value should be added to the lateral deflection due to flexure and shear to give the total deflection.

Coupling beams

Coupling beams deform predominately in shear like mode. For these members there are two alternatives. First, an appropriate shear stiffness can be used in an analysis, in which case α_c in Table C6.5 should be taken as 1.0. Second, the alternative approach which is simpler, is to reduce the flexural stiffness to make an allowance for the added deformation caused by diagonal cracking and the deformation caused by the development of coupling beam reinforcement in the coupled walls. With the second option, α_c in Table C6.5 is taken from Table C6.6B. It should be noted that the recommended stiffness values for coupling beams were developed from test results[6.24] in which there was no restraint to elongation. In most cases some restraint arises from floors and foundation beams that tie the coupled walls together. This restraint induces axial load which increases the strength of the coupling beams. The influence of axial load induced by elongation on effective stiffness is uncertain (see 11.4.9 and C11.4.9).

A3

Table C6.6B – Factor α_c that allows for the deformation associated with diagonal cracking in coupling beams

$\frac{L_n}{h}$	**1**	**1.5**	**2.0**	**3.0**	**4**
Conventionally reinforced beams	0.16	0.24	0.31	0.42	0.5
Diagonally reinforced beams	0.32	0.48	0.62	0.84	1.0

Coupling beams deform predominately in a shear like mode. For these members there are two alternatives. First an appropriate shear stiffness can be used in an analysis, in which case α_c in Table C6.5 should be taken as 1.0. Second, the alternative approach, which is simpler, is to reduce the flexural stiffness to make an allowance for the added deformation caused by diagonal cracking and the deformation caused by the development of coupling beam reinforcement in the coupled walls.

Deformation due to development of reinforcement

Deformation within support joint zones is generally allowed for in analysis programs by first defining "rigid" joint zones and then applying a "rigid zone reduction factor" to account for deformations within the joints. This method is not easily applied to walls.

As an alternative for beams or walls, the deformation due to the development of reinforcement into the supporting member, or members, may be allowed for by multiplying the effective second moment of area, I_e, for the clear span, L_n by R, which is given by:

$$R = \frac{L_n}{(L_n + \alpha_{d1} + \alpha_{d2})} \qquad \text{.....(Eq. C6–8)}$$

Where the subscript 1 and 2 refers to each end of the member, and the value of α_d is the smaller of 0.4 L_d or $\frac{h_c}{3}$.

Where:

L_n = the clear span
L_d = the development length of the reinforcement anchored into the supporting member
h_c = the depth of the supporting column or foundation beam.

For cantilever members α_{d2} is zero.

C6.9.1.4 *Dual structures*

The design forces should be allocated to each element at each level of a dual structure in accordance with relative stiffness values, taking also torsional or concurrency effects into account. However, in cognisance of the inelastic response of ductile structures, some plastic redistribution from potentially weaker to potentially stronger elements may be considered. It is important that the primary energy dissipating elements and complete plastic mechanisms be clearly identified. Capacity design procedures modified to account for characteristic features of ductile frame and ductile wall behaviour, as well as for the interaction of these components in dual systems, should be used in respect of all elements. Such procedures are described in Reference 6.24.

Internal forces in diaphragms, required to rectify the inherent incompatibility of elastic deformations of frames and walls in dual systems, may be significant. Diaphragm actions in accordance with 13.4 require special attention, particularly when precast floor systems are used.

C6.9.1.5 *Redistribution of moments and shear forces*

Some additional considerations for moment redistribution when considering inelastic earthquake response are outlined in the following paragraphs. Redistribution of moments or shear forces is relevant only to those continuous members, such as beams, columns or walls, which form part of the plastic mechanism chosen to provide the required structural ductility capacity.

Beams, which contain potential plastic regions, are required to be designed to be ductile. The ductility capacity can be improved by moment redistribution. The purpose of moment redistribution in seismic design is to reduce the absolute maximum moment, usually in the negative moment region, and compensate for this by increasing the moments in the non-critical (usually positive moment) regions of a span. Thus a better distribution of strength along a span is attained and differences in the quantity of reinforcement used in the top and bottom of the beam may be reduced. This improves the ductility capacity of the plastic hinges (regions). Moreover, the critical moments at either side of a column, usually for different directions of lateral forces, can be equalised. This reduces the need to terminate beam bars in interior column-beam joints. Also the positive moment potential of beams at column faces can be more fully utilised. There is no limit on redistribution of bending moments provided equilibrium of forces is maintained and material strain limits are satisfied. Care is required to ensure that moment redistribution for the ultimate limit state does not violate strength and stiffness required for the serviceability limit state. A3 A3

In carrying out the redistribution it is important to ensure that the lateral storey shear force is not reduced. In regular moment resisting frames this can be achieved by keeping the sum of the beam terminal moments constant before and after redistribution. In irregular structures it is necessary to keep track of the column shear forces and make sure that in any level the sum of the shears is not reduced by redistribution.

In certain cases, particularly in gravity load dominated frames and those of limited ductility, the simultaneous formation of plastic hinges at both ends of some columns in a storey is permitted (see Appendix CD, Method B). A3

Irrespective of which redistribution method is employed the minimum nominal shear strength permitted in a column, at the ultimate state, should be equal to or greater than 1.7 times the shear force V_E, where V_E is the shear force induced in the member due to seismic actions.

From considerations of equilibrium for any span it is evident that if the support moments of a given bending moment pattern are changed then corresponding adjustments are required to be made in the mid-span region of the beam to maintain equilibrium for vertical loads and forces. A3

The full mechanism of a structure subjected to earthquake forces, consisting mainly of structural cantilever or coupled walls, comprises plastic hinges at the bases of all these walls. It may be advantageous to allocate more or less lateral design force to a structural wall than indicated by the elastic analysis. This process is acceptable provided equilibrium of lateral forces is maintained, material strain limits are not exceeded and damaging deformation does not occur in the serviceability limit state. A3

An extensive treatment of moment redistribution under seismic actions, together with examples, is given in References 6.24 and 6.25. However, it should be noted that with Amendment No. 3 the 30 % limit on moment redistribution is no longer applied.

REFERENCES

6.1 Johansen K.W., “Yield-Line Theory”, C & CA, London, 1962.

6.2 Johansen K.W., “Yield-Line Formulae for Slabs”, Cement and Concrete Association, London, 1972.

6.3 Jones L.L. and Wood R.H., “Yield Line Analysis of Slabs”, Thames and Hudson and Chatto & Windus, London, 1967.

6.4 Hillerborg A., A Plastic Theory for the Design of Reinforced Concrete Slabs”, Preliminary Publication, 6th Congress, IABSE, Lisbon, 1960.

6.5 Wood R.H. and Armer G.S.T., “The Theory of the Strip Method for Design of Slabs”, Proc. Int. Civil Engineers (London). Vol 41, Oct. 1968, pp. 285-311.

6.6 Kemp K.O., “A Strip Method of Slab Design with Concentrated Loads or Supports”, The Structural Engineer, Vol. 49, No. 12, Dec. 1971, pp. 543-8.

6.7 Hillerborg A., “Strip Method of Design”, A Viewpoint Publication. Cement and Concrete Association, London, 1975.

6.8 Rangan B.V., “Limit States Design of Slabs Using Lower-bound Approach”, Journal of the Structural Division, ASCE, Vol. 100, Feb. 1974, pp. 373-89.

6.9 Rangan B.V., "Limit States Design of Flat Plates and Slabs", IABSE Proceedings, Zurich, 1977,pp. 2–77.

6.10 Warner R.F., Rangan B.V, and Hall A.S., "Reinforced Concrete". Longman, Australia, 3rd Ed., 1988.

6.11 Neth V.W., de Paiva H.A.R. and Long A.E., "Behaviour of Models of a Reinforced Concrete Flat Plate Edge-column Connection", ACI Journal, Proceedings, Vol. 78, No. 4, July-August 1981, pp. 269–75.

6.12 Wiseinger F.P., "Design of Flat Plates with Irregular Column Layout", ACI Journal, Proceedings, Vol. 70, No. 2, Feb. 1973, pp. 117–23.

6.13 Rangan B.V. and Hall A.S., "Forces in the Vicinity of Edge Columns in Flat Plate Floors", Volume 1 – "Tests on R.C. Models", UNICIV Report No. R-203, The University of New South Wales, Kinsington, Jan. 1983, 24 p.

6.14 Rangan B.V. and Hall A.S., "Moment Redistribution in Flat Plate Floors', ACI Journal Proceedings, Vol. 81, No. 6, Nov.–Dec. 1984, pp. 601–8.

6.15 Wong, K.W., Hough, R., Gilbert, I.R. and Warner, R.F., 2003, "Stiffness Assumptions for Use in the Strength Design of Concrete Structures", Australian Journal of Structural Engineering, Institution of Engineers, Australia, Vol. 5, No. 2., pp 77-88.

6.16 Branson, Dan E., "Instantaneous and Time-dependent Deflections on Simple and Continuous Reinforced Concrete Beams", HPR Report, No. 7, Part 1, Alabama Highway Department, Bureau of Public Roads, August 1963 (1965), pp. 1-78.

6.17 ACI Committee 435, "Deflections of Reinforced Concrete Flexural Members", Journal American Concrete Institute, Proceedings, Vol 63, No. 6, June 1966, pp. 637-674. Also ACI Manual of Conbcrete Practice, Part 2, 1968, 99. 433-444.

6.18 Subcommittee 1, ACI Committee 435, "Allowable Deflections", Journal American Concrete Institute, Proceedings, Vol 65, No. 6, June 1968, pp.433-444.

6.19 Subcommittee 2, ACI Committee 209, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures, Designing for the Effects of Creep, Shrinkage and Temperature in Concrete Structures", pp. 51-93, SP-27. ACI Publication, American Concrete Institute, Detroit, 1971.

6.20 Branson, D.E., Discussions of "Proposed Revision of ACI 318-63: Building Code Requirements for Reinforced Concrete" by ACI Committee 318, Journal American Concrete Institute, Proceedings. Vol. 67, No. 9, September 1970, pp. 692-695.

6.21 ACI Committee 435, "Variability of Deflections of Simply Supported Reinforced Concrete Beams", American Concrete Institute Journal, Vol 69, NO. 1, January 1972.

6.22 Paulay, T., "Seismic Displacement Capacity of Ductile Reinforced Concrete Building Systems", Bulletin NZ Society for Earthquake Engineering, Vol. 36, No. 1, March 2003, pp 47-65.

6.23 "Papers Resulting from Deliberations of the Society's Discussion Group on Seismic Design of Reinforced Concrete Walls and Diaphragms", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2, June 1980, pp. 182–193.

6.24 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley & Sons, 1992, p. 767.

6.25 "Examples of Concrete Structural Design to NZS 3101(Red Book)" Cement and Concrete Association of New Zealand, Wellington, 1998.

A3 6.26 Gilbert, R.I., "Deflection Calculation of Reinforced Concrete Structures – Why We Sometimes Get It Wrong", ACI Structural Journal, Vol. 96, No. 6, Nov.-Dec. 1999, pp. 1027-1032.

6.27 Gilbert, R.I. and Guo, X.H., "Time Dependent Deflection and Deformation of Concrete Flat Slabs – An Experimental Study", ACI Structural Journal, Vol. 102, No. 3, May-June 2005, pp. 363-373.

6.28 Gribniak, V. and Kakaulshas, G., "Improving the Deflection Prediction Model from ACI318", ACI Structural Journal, Vol. 111, No. 6, Nov. Dec. 2014, pp. 1489-1494.

6.29 ACI, "Deflections of Reinforced Concrete Flexural Members", Reapproved 1989, ACI Manual of Concrete Practice, 1993 (probably available in later manuals).

6.30 Tran, T.A. and Wallace, J.W., "Cyclic Testing of Moderate Aspect Ratio Reinforced Concrete Structural Walls", ACI Structural Journal, Vol. 112, No. 6 Nov.-Dec. 2015, pp. 653-666.

6.31 Synge, A.J.,"Ductility of Squat Shear Wals", Dept. of Civil Engineering, University of Canterbury, Report 80-8, February 1980.

C7 FLEXURE, SHEAR, TORSION AND ELONGATION OF MEMBERS A3

C7.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 7 of the Standard.

A3

A_{oc} area enclosed by centreline of tube used to calculate torsional cracking, mm^2
A_{ps} area of prestressed reinforcement in flexural tension zone, mm^2 A3
A2
c_b neutral axis depth of a beam or slab at balanced conditions, mm A3
d_p distance from extreme compression fibre to centroid of prestressing reinforcement, mm
f_{se} effective stress in prestressed reinforcement after losses, MPa
f_{ps} stress in prestressing reinforcement at nominal flexural strength, MPa
f_s steel stress, MPa
f_{ys} an equivalent yield stress associated with the equivalent tube thickness (t_s), MPa A2
j ratio of internal level-arm of flexural forces to effective depth of the member
K_{gross} polar moment of inertia of section before torsional cracking, mm^4 A3
K_{cr} polar moment of inertia of section after torsional cracking, mm^4
K_1 constant in Equation C7–18
A3
q shear flow
T torque, (torsional moment), N mm
T_c torque resisted by concrete, N mm
A3
T_{cr} nominal torsional cracking moment, N mm
t_s an equivalent thin tube thickness, mm A2
v_t shear stress due to torsion prior to torsional cracking, MPa
α inclination of compression force relative to axis of a member
A3
ε_s steel strain
ε_y steel strain at first yield

C7.2 Scope

Section 7 covers basic aspects of the design of members for flexure, shear and torsion together with elongation in plastic hinges. A3

With regard to shear, design equations for the shear resisted by the concrete mechanisms and by the shear reinforcement are given in Section 9 for reinforced concrete beams and one-way slabs, Section 10 for reinforced concrete columns, Section 11 for walls, Section 12 for two-way slabs, Section 15 for beam-column joints and Section 19 for prestressed concrete. A3

C7.3 General principles

In these design provisions, except where specifically noted otherwise in a clause, it is assumed that the presence of shear does not affect the flexural strength, and that the presence of flexure does not affect the A3

A3 shear strength. However, the presence of axial load affects both the flexural strength and the shear strength.

C7.4 Flexural strength of members with shear and with or without axial load

C7.4.1 *Flexural strength requirement*

Equation 7–1 simply states that the design flexural strength, ϕM_n, of the section is required to be at least equal to the design bending moment on the section at the ultimate limit state, M^*.

C7.4.2 *General design assumptions for flexural strength*

C7.4.2.1 *Strength calculations at the ultimate limit state*

Normally the strength of the member will be based on the cracked cross section, including the concrete cover in compression outside the transverse reinforcement. Then the assumptions of 7.4.2 apply, which are the same as for members not designed for seismic forces. References 7.1, 7.2, 7.3 and others give theory and design aids. The use of an extreme fibre concrete compressive strain of 0.003, as specified in 7.4.2.3, will generally result in a satisfactory prediction for the flexural strength of a beam but may lead to a significant underestimate of the flexural strength of a column, particularly if high strength steel reinforcement is used. This is because, for beams with reinforcement only in the extreme fibres, the maximum moment may not show much variation over a large range of high strains, but for columns the strain level may have a significant effect on the stresses in the steel reinforcement arrayed around the column perimeter.

When considering the behaviour of the cross section at advanced strains in the inelastic range after the spalling of the cover concrete, the strength of the cross section should be based on the core of the member within the spiral or perimeter hoops. After spalling of the cover concrete, which typically occurs at extreme fibre compressive strains of 0.003 to 0.008, the increase in the strength and ductility of the confined concrete within the transverse reinforcement and the strain hardening of the longitudinal reinforcement will generally allow the core of the member to maintain a substantial moment and axial load capacity. Loss of concrete cover has a more significant effect on the moment capacity of members with small cross section than on members with large cross section at high curvatures. For columns with constant axial load and confined as required in 10.4.7.4.1 or 10.4.7.5.1 a significant increase in flexural strength of the cross section can occur after spalling of the cover concrete. References such as 7.4, 7.5 and 7.6 give information on the stress-strain behaviour of confined concrete at high strains which could be used in moment-curvature analysis to determine the flexural strength and curvature at far advanced strains.

C7.4.2.2 *Strain relationship to geometry*

Many tests have confirmed that the distribution of strain is essentially linear across a reinforced concrete cross section, even near the flexural strength [7.1].

Both the strain in the reinforcement and in the concrete are assumed to be directly proportional to the distance from the neutral axis. This assumption is of primary importance in design for determining the strain and corresponding stress in the reinforcement.

C7.4.2.3 *Maximum concrete strain*

The maximum concrete compressive strain at crushing of the concrete has been observed in tests to vary from 0.003 to much higher values in certain conditions [7.1]. However, the strain at the extreme compression fibre of the gross concrete cross section at which the ultimate (maximum) moments is developed is usually about 0.003 to 0.004 for members of normal proportions and materials.

A3 **C7.4.2.4** *Reinforcement stress-strain relationship*

For deformed bar reinforcement, it is generally sufficiently accurate to assume that when the stress in reinforcement is below the yield strength, f_y, the stress is proportional to strain. The increase in strength of the steel due to the effect of strain hardening of reinforcement is generally neglected for strength computations.

The modulus of elasticity shall be taken as specified in 5.2.3 or 5.4.2 as appropriate. A3

C7.4.2.5 *Concrete tensile strength*

The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength. The tensile strength of concrete in flexure at the ultimate limit state is neglected in strength design for flexure. For members with normal percentages of reinforcement, this assumption agrees well with tests[7.1]. A3

The strength of concrete in tension, however, is important in cracking and deflection considerations at the serviceability limit state and in the development of strength by bond of reinforcement and shear strength of the ultimate limit state.

C7.4.2.6 *Concrete stress-strain relationship*

This assumption recognises the non-linear stress distribution of concrete in compression at high stress. As maximum stress is approached, the stress-strain relationship for concrete is not a straight line but some form of a curve (stress is not proportional to strain). The general shape of the curve for unconfined concrete is a function of concrete strength and consists of a rising curve from zero to a maximum at a compressive strain between 0.0015 and 0.0035 followed by a descending curve to an ultimate strain (crushing of the concrete) of 0.004 or greater. As discussed under C7.4.2.3, this Standard sets the maximum useable compressive strain at 0.003 for design.

For normal strength concrete (f'_c less than about 55 MPa) the rising branch of the stress-strain curve is near linear up to about 0.5 f'_c, reaches maximum at a strain of about 0.002 and has a descending branch which becomes more steep as f'_c increases. For higher strength concrete the stress-strain behaviour becomes more brittle. For example when f'_c is in the range 80 MPa to 100 MPa the rising branch is near linear up to f'_c at a strain of about 0.003, and the descending branch is near vertical. High strength concrete has less post-peak deformability than normal strength concrete[7.7].

Research has shown that the important properties of the concrete stress distribution can be approximated closely by using any one of several different assumptions as to the form of stress distribution. This clause permits any particular stress distribution to be assumed in design if shown to result in predictions of flexural strength in reasonable agreement with the results of comprehensive tests. Many stress distributions have been proposed. The three most common are the parabola, trapezoid, and rectangle. Of importance is the reasonable prediction of the magnitude and location of the compression stress resultant.

C7.4.2.7 *Equivalent rectangular concrete stress distribution*

For practical design this clause allows the use of a rectangular compressive stress distribution (stress block) to replace the more exact concrete stress distribution. For the equivalent rectangular stress block, an average stress of $0.85f'_c$ is used when f'_c is less than 55 MPa; for higher concrete strength the average stress is taken to reduce linearly to a minimum of 0.75 f'_c at f'_c = 80 MPa and to remain constant at 0.75 f'_c when f'_c is higher than 80 MPa. The depth of the equivalent rectangular stress block is $a = \beta_1 c$. The value of β_1 is 0.85 for concrete with $f'_c \leq 30$ MPa; for higher strength concrete β_1 is taken to reduce linearly to a minimum of 0.65 at f'_c = 55 MPa and to remain constant at 0.65 when f'_c is higher than 55 MPa.

The recommended properties of the equivalent rectangular stress block for normal strength concrete are based on test results due to Hognestad et al [7.8], Rüsch [7.9] and others [7.1]. The Building Code of the American Concrete Institute (ACI 318) states that for concrete strengths greater than 55 MPa, research data [7.10, 7.11] supports the use of an equivalent rectangular stress block with average stress of 0.85 f'_c and β_1 = 0.65. However, tests at the University of Canterbury [7.12] on columns with concrete compressive strengths of about 100 MPa showed that this ACI recommendation can be non-conservative, and that to obtain good agreement between predicted and experimental flexural strengths of columns for concrete of that strength the average stress of the equivalent rectangular stress block had to be taken as less than 0.85 f'_c. Hence the values for the parameter α_1 given by Equation 7–2 were proposed [7.12]. The Norwegian concrete design standard NS 3473 has recommendations which agree with this trend for high strength concrete. The results of other recent research also agrees with this trend for high strength concrete.

The rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but does provide essentially the same results as those obtained in tests [7.13]. Flexural strength equations commonly used in design are based on the equivalent rectangular stress block.

C7.4.2.8 *Balanced conditions*

The balanced strain condition at the cross section of a member is used to determine the neutral axis depth, c_b, when the tension reinforcement just commences to yield as the concrete reaches its assumed ultimate strain of 0.003 at the extreme compression fibre.

C7.4.2.9 *Compression reinforcement*

For the additional strength due to compression reinforcement, see References such as 7. 1 and 7.13.

C7.5 Shear strength of members

C7.5.1 *General*

The equations for shear are presented in terms of shear force. However, critical shear force levels are calculated from limiting shear stress levels for two reasons. Firstly, the limiting shear stress levels are applicable to many different situations and hence this gives a rational basis for calculations. Secondly, the shear stress level gives the designer a feel for how critical shear is in any particular situation.

The nominal shear strength is taken as the nominal shear stress multiplied by the effective area of concrete, A_{cv}, that resists shear. This area is defined in the sections for the type of member that is being considered.

It is important to note that the implication that shear stresses are uniform over the shear area arises from a simplification made in the ACI 318 code in 1967. In fact shear stresses may be far from uniform and in carrying out strut and tie analysis it may be necessary to work from the actual distribution of shear stresses.

A3 C7.5.2 *Maximum nominal shear stress, v_{max}*

The maximum nominal shear stress is limited to prevent diagonal compression failure of the concrete. The presence of diagonal tension cracks in a web reduces the crushing strength of the concrete. A shear stress of 0.2 f'_c approximately corresponds to a diagonal compression stress of 0.45 f'_c, which is close to
A3 the level of diagonal compression which might be expected to result in compression failure. The diagonal compression strength continues to decrease as the crack widths widen.

A3 C7.5.3 *Nominal shear strength, V_n*

As stated by Equation 7–6 the total nominal shear strength of a section, V_n, is considered to be the sum of the nominal shear strength provided by the concrete mechanisms, V_c, and the nominal shear strength provided by the shear reinforcement, V_s. The nominal shear strength provided by the concrete mechanisms, V_c, is assumed to be the same for a member without shear reinforcement as for a member with shear reinforcement. The nominal shear strength, V_c, provided by the concrete mechanisms is taken
A3 to be equal to the shear force causing significant inclined cracking. These assumptions are discussed in the ACI-ASCE Committee 426 reports[7.14, 7.15] and other very different assumptions in References 7.16, 7.17, 7.18, 7.19, 7.20 and 7.21.

C7.5.4 *Nominal shear strength provided by the concrete, V_c*

The nominal shear strength provided by concrete in regions of members, which contain flexural cracks, depends upon shear transfer across cracks by aggregate interlock action, dowel action of longitudinal reinforcement and shear resisted by the compression zone. The shear resistance provided by aggregate interlock action decreases as crack widths increase. For this reason any action which increases crack widths, such as axial tension, reduces the shear strength provided by concrete, V_c.

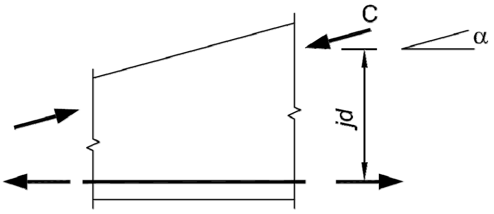


Figure C7.1 – Influence of inclination of compression force on shear strength

Inclination of the flexural compression force, C, in a member ($\tan \alpha$) may increase or decrease the shear strength, as illustrated in Figure C7.1, and as given by Equation C7–1. Where the internal lever-arm, jd, increases with increasing bending moment the shear strength is increased by the vertical component of the flexural compression force. Where the internal lever-arm decreases with increasing bending moment the shear strength of the concrete is deceased by the vertical component of the flexural compression force.

$$V_c = v_c A_{cv} + C \tan \alpha \quad \text{(Eq. C7–1)}$$

$$C = M^*/jd$$

where jd is the internal lever arm and α is the inclination of the compression force relative to the flexural tension force. A3

C7.5.5 *Nominal shear strength provided by the shear reinforcement*

Shear strength provided by shear reinforcement is calculated by a truss analogy. The diagonal compression forces in the web are assumed to be inclined at an angle of $\tan^{-1} j$ to the longitudinal axis of the member (j is the ratio of the internal lever-arm to the effective depth).

C7.5.7 *Location and anchorage of reinforcement*

With shear reinforcement a truss-like action occurs in a beam or slab, in which the longitudinal flexural tension reinforcement provides the tension chord, the compression zone acts as the compression chord, the stirrups act as tension members in the web and the concrete sustains diagonal compression forces.

It is essential that shear (and torsional) reinforcement be adequately anchored at both ends to be fully effective on either side of any potential inclined crack. This generally requires a hook or bend at the end of the reinforcement as provided by 7.5.7.1. Stirrups must enclose the flexural tension reinforcement to be effective in resisting shear. In shallow members conventional shear reinforcement may be ineffective as cover requirements may prevent the stirrups being anchored in the compression zone.

Lapped splices in stirrups may only be used where inelastic action due to seismic actions is not expected, deformed bars are used and shear stresses are not high. This situation arises in many bridge structures. The use of lapped stirrups can simplify construction particularly where haunched prestressed members are used (see C8.7.2.8). A3

C7.5.8 *Design yield strength of shear reinforcement*

Limiting the design yield strength of shear reinforcement to 500 MPa provides a control on diagonal crack width. A3

C7.5.9 *Alternative methods for determining shear strength*

The approach given in 7.5.1 to 7.7.8 is based on the method in the 2002 ACI Building Code (ACI 318), though this has been modified to allow for scale effects in members (see 9.3.9.) The method is largely empirical and it has been retained because of its simplicity and in this modified form has been shown to produce satisfactory shear strength predictions. More rational approaches have been developed in recent years, for example as in References 7.14, 7.15, 7.16, 7.17, 7.18. In Europe a "variable angle truss model" is used [7.17] and in Canada a modified compression theory model has been developed [7.15]. These alternative (more rational) approaches are more complex but can be used to advantage for some applications, such as prestressed concrete bridge girders. Where the "Strut and Tie" method is used V_c should be taken as zero except for the component of shear resistance provided by inclination of the compression force relative to the tension force (see Figure C7.1.)

C7.5.10 Minimum area of shear reinforcement

Members without shear reinforcement, or with a very low proportion of shear reinforcement, can fail in a brittle manner without warning. To prevent such failures due to unexpected loads, or due to loading situations not usually considered in design (such as actions induced by creep, shrinkage, thermal effects or accidental loading) the Standard requires a minimum area of transverse reinforcement given by 9.3.9.4.13 for beams and one-way slabs, 10.3.10.4.4 for columns and 11.3.11.3.8(b) for walls.

C7.6 Torsional strength of members with flexure and shear with and without axial loads

C7.6.1 *Members loaded in torsion*

A3 **C7.6.1.2** *Threshold torsional reinforcement*

In the design of reinforced concrete members to resist torsional loads, it is necessary to distinguish between two different types of torsion, one arising from equilibrium requirements and the other from the need to satisfy compatibility of deformation.

"Equilibrium torsion" is required to maintain equilibrium in the structure. This is generally the case in statically determinate structures. Where a member is subjected to "equilibrium torsion" it is necessary to provide adequate reinforcement to ensure that the member is capable of resisting the torsion required by statics. Figure C7.2 shows an example. For the cantilever canopy to be in equilibrium the beam must provide the corresponding torsional as well as flexural and shear strength.

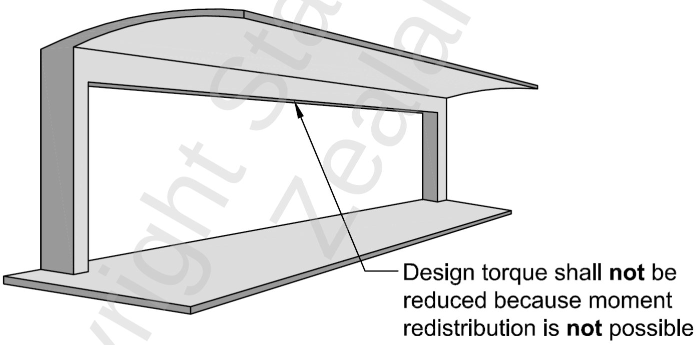


Figure C7.2 – An example where "Equilibrium torsion" is required to maintain the load

"Compatibility torsion" arises when twist is required to maintain compatibility of deformations in the structure. This kind of torsion occurs in statically indeterminate structures if the torsion can be eliminated by releasing relevant restraints. In such situations twist, torsion and torsional stiffness are interrelated. Figure C7.3 shows an example. The rotation of the ends of the floor beam introduces twist into the spandrels. The resulting bending moment at the ends of the floor beam and the corresponding torsion in the spandrels will depend on the relative values of flexural and torsional stiffness of these members.

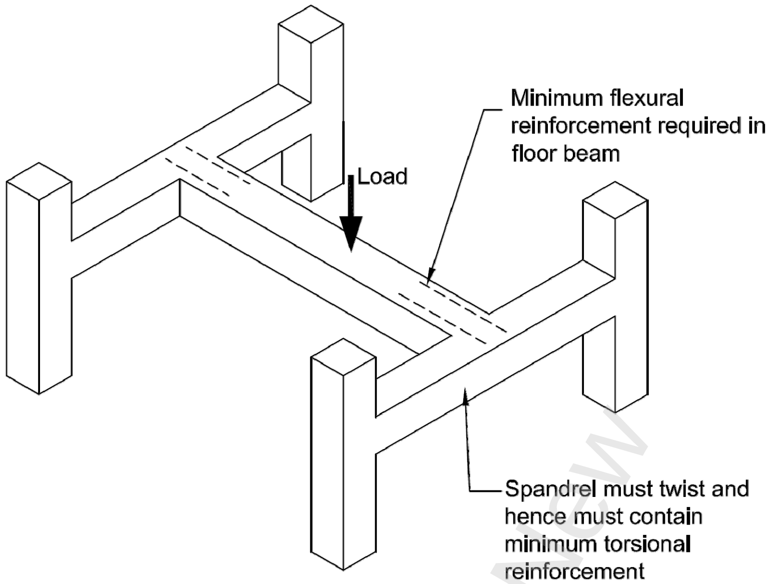


Figure C7.3 – A structure in which torsion arises because of compatibility requirements

Where torsional moments are induced in members due to twist associated with compatibility requirements, the torsional moments may be neglected provided the requirements of 7.6.2 are satisfied. This is acceptable because the torsional stiffness of a member decreases to a small proportion of its initial stiffness when torsional cracks form, which allows torsional moments to be redistributed. A3

A method of assessing the torsional stiffness for both equilibrium and compatibility-induced torsion after torsional cracks have formed is given in C7.6.1.3.

The provisions for design of torsion have been developed from recommendations from the European Concrete Committee[7.24] and ACI 318 Code[7.23] The torsional strength of prestressed concrete members is considered in References 7.15, 7.19, 7.26 and 7.27.

Prior to the formation of diagonal cracks, a reinforced concrete beam in torsion behaves essentially as an elastic beam. The reinforcement at this stage makes no contribution to torsional resistance. The dimensions of the cross section such as shown in Figure C7.4(a) and the properties of the concrete alone determine the response [7.1]. It is more convenient, and accurate enough for design purposes, to use the equivalent tube approach [7.27]. This approach replaces the actual cross section of the beam by an equivalent thin walled tube, as shown for example in Figure C7.4(b). This tube has the same external dimensions as the actual cross section and it has a wall thickness of: A2

$t_c = 0.75\, A_{co}/p_c$ .... (Eq. C7–2)

but not exceeding the actual wall thickness in hollow sections, where p_c is the external perimeter of the actual cross section and A_{co} is the area enclosed within this perimeter. A2

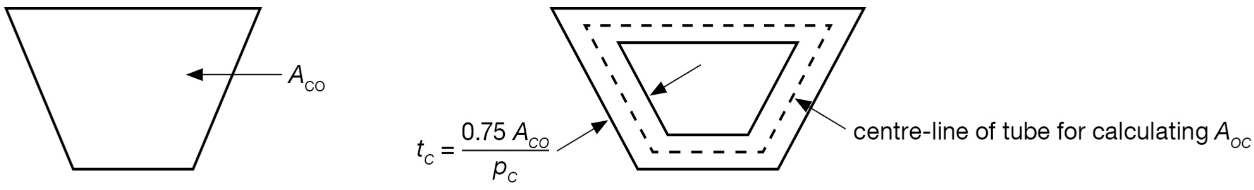
Using the well known relationship for the response of a thin-walled tube in torsion, the shear stress, v_t, produced by torsion, T, is given by:

$$v_t = \frac{T}{2A_{oc}t_c} \quad \text{............ (Eq. C7–3)}$$

where A_{oc} is the area enclosed by the mid-wall perimeter of the tube. This value can be calculated from the external dimensions of the section and the tube thickness or thicknesses. However, for solid sections a reasonable approximation is A_{oc} = 0.67 A_{co}, and for hollow sections this value is generally conservative. A2

Substituting into Equation C7–3, using the approximation for A_{oc} and assuming that diagonal cracking will occur when the principal tensile stress (equal to the shear stress) is equal $0.33\sqrt{f_c'}$, the torque required to crack the member, T_{cr}, is given by:

$$T_{cr} = 0.44\, A_{co}\, t_c \sqrt{f_c'} \qquad \text{(Eq. C7–4)}$$



(a) Actual cross section

(b) A_{oc} area enclosed by tube centreline

Figure C7.4 – Equivalent tube for calculating torsional cracking moment

C7.6.1.3 *Requirement for torsional reinforcement*

Where compatibility torsion arises it is necessary to evaluate both the flexural and torsional stiffness of the relevant members before and after torsional cracking occurs. The equivalent thin-walled tube may be used to determine the torsional rigidity $G_c K_{gross}$ of the uncracked section. Tube theory gives:

$$G_c\, K_{gross} = G_c \frac{4 (A_{oc})^2\, t_c}{p_{oc}} \qquad \text{(Eq. C7–5)}$$

where G_c is the shear modulus of the concrete, which can be taken as 0.4 E_c, A_{oc} is the area enclosed by the midline of the tube used for assessing torsional cracking shear stress, and p_{oc} is the perimeter of A_{oc} (in a solid rectangular section p_{oc} can be taken as $p_c - 4\,t_c$). The torsional rigidity of the cracked member is a small fraction only of the torsional rigidity of the uncracked section. The torsional rigidity in the cracked state is primarily determined by the deformation of the reinforcement; its value $G_c K_{cr}$ can be found from the stiffness of the equivalent thin-walled tube. This is:

$$G_c K_{cr} = \frac{E_s}{2} \frac{4 (A_o)^2}{p_o} \sqrt{\frac{A_t A_\ell}{s p_o}} \qquad \text{(Eq. C7–6)}$$

where E_s is the Young's modulus of the reinforcing steel. For typical beams it will be found that the torsional stiffness after cracking as given by Equation C7–6, is generally only a few percent of the torsional stiffness of the uncracked member, as given by Equation C7–5.

As torsional reinforcement is stressed only in the torsionally cracked state, it is appropriate to design this reinforcement for the moment it will sustain in its torsionally cracked state. For a statically indeterminate structure, these actions can be determined by performing an elastic based analysis using stiffness values appropriate for torsionally cracked (Eq. C7–6), and flexurally cracked members (see 6.8). For prestressed members, such an analysis may require the stiffness values in a member to be subdivided into flexurally and torsionally cracked and uncracked regions to obtain realistic actions. In the transition between uncracked and cracked section properties, the torsional stiffness reduces to a much greater extent than the flexural stiffness. This results in a redistribution of actions, with a major reduction in the magnitude of torsional moments [7.28].

Members designed to resist torsions, the magnitudes of which have been determined from the above stiffness values, will behave in a satisfactory manner. The analysis, however, may involve considerable work. If the torsion on the member arises only because the member must twist to maintain compatibility, the magnitude of the torsion will be almost directly proportional to the torsional stiffness. This is demonstrated in Figure C7.3 where the torsion in the spandrel is caused by the need for the spandrel to rotate with the end of the floor beam. Thus, decreasing the amount of torsional reinforcement decreases the stiffness, and as a result the applied torque is reduced. With the reduction in torque, the positive moments in the transverse beam increase.

In cases where torsion is not required for equilibrium, 7.6.1.4 allows the designer to provide a minimum amount of properly detailed torsional reinforcement and assume that the torsional stiffness of the member, and hence the torsion in the member, is zero. In reality, there will be some torsion in the member, and the presence of this torsion should be considered in detailing adjacent or supported members. For example, in the transverse beam spanning between the spandrel beams in Figure C7.3, torsion resisted by the spandrel beams induces negative moments in the transverse beam. To ensure satisfactory performance in the serviceability limit state, negative moment reinforcement should be provided near the support regions to control negative moment cracking. In many common situations, the actual torsional stiffness and torsional cracking moment cannot be accurately determined due to interaction with shear, flexure and axial forces, arising from restraint provided by the remainder of the structure. If a beam forms torsional cracks, it elongates by an amount equal to the average strain in the longitudinal torsional reinforcement, times the member length. With surrounding members, as occurs in many buildings and bridges, this elongation may be partially restrained. The resultant compression, due to this restraint, may increase the torsional strength in the restrained member, but reduce the torsional strength in the adjacent restraining members, which are subjected to tension due to their restraining action. Designers need to be conscious of the difficulty of accurately assessing torsional actions and take a conservative approach to the provision of flexural reinforcement in members where moments may be sustained by torsional resistance in supporting members. A3 A2 A3 A2

Under serviceability limit state conditions, the torsion based on the stiffness of uncracked sections may be large enough to cause cracking. If the resulting crack width is of concern, two thirds of the torque so computed should be used to determine the torsional reinforcement [7.15]. Effects of prestressing, if any, should be included.

A2

C7.6.1.6 *Torsional strength requirement* A3

By similarity to the requirements of 7.6.1.3 the design torsion, T^*, is related to the nominal torsional strength required, which is then used in all subsequent equations.

C7.6.1.7 *Torsional shear stress* A3

After diagonal cracks form, the torsional resisting mechanism of the member changes completely. The torsional shear stresses are now assumed to be provided solely by the diagonal compressive stresses in the diagonally cracked concrete. This diagonally compressed concrete is held in equilibrium by tensile stresses in the longitudinal and the transverse reinforcement. Because the transverse reinforcement cannot “hold” the concrete cover in equilibrium without generating high concrete tensile stresses, particularly at the corners of a section, at higher loads this concrete cover will spall off. Hence in the cracked state it is the dimensions of the reinforcing cage which governs the behaviour and not the exterior dimensions of the concrete.

In the cracked state the beam is again idealised as a thin-walled tube, but this time the mid-wall perimeter, p_o, is assumed to pass through the centres of the longitudinal bars in the corners of the closed stirrups, and the concrete wall thickness, t_o, is assumed to be:

$t_o = 0.75\ A_o/p_o$ ....(Eq. C7–7)

where A_o is the area enclosed by p_o, and t_o may not be taken greater than the actual wall thickness. A2

Figure C7.5(a) shows an example section in which $A_o = b_o h_o$ where b_o may be taken as 0.5 ($b'_o + b''_o$). Also $p_o = 2\ (h_o + b_o)$. The assumed thickness of the tube, t_o, is also indicated.

To check against crushing of the concrete due to the diagonal compression, the nominal torsional shear stress is calculated from:

$$v_{tn} = \frac{T_n}{2A_o t_o} \quad \text{(Eq. C7–8)}$$

A3 This torsional shear stress is limited to $0.2f'_c$, or 10 MPa, whichever is less. The value of 0.2 f'_c is a conservative approximation to the values for the maximum safe shear stress that will not lead to a failure in diagonal compression as predicted by the compression field theory [7.29].

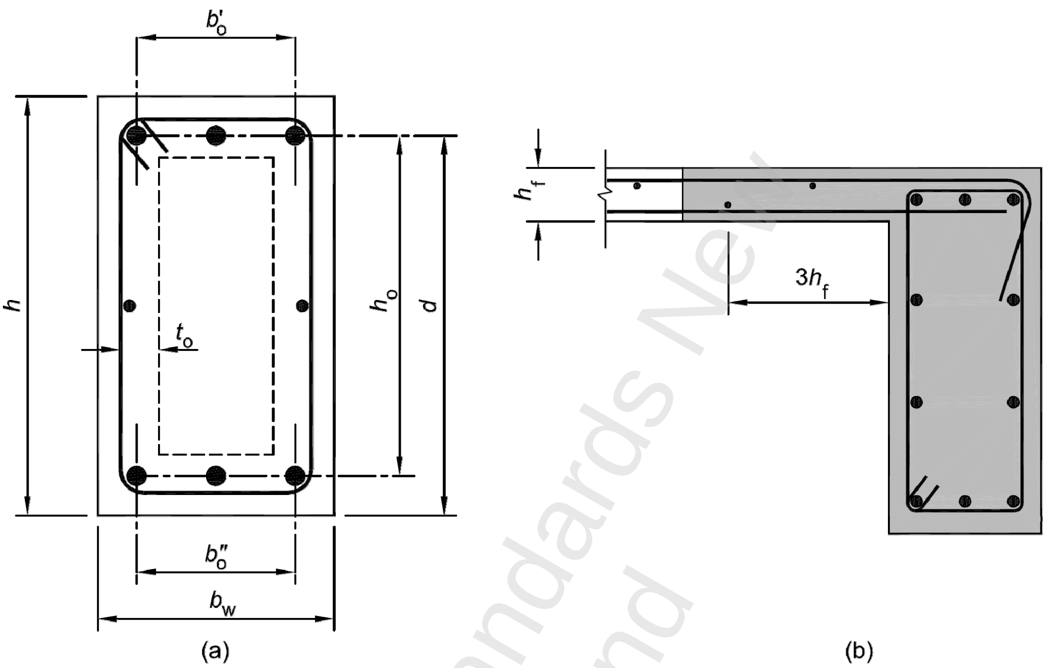


Figure C7.5 – Effective sections for torsional resistance

A3 **C7.6.1.8** *Torsion in flanged sections*

In determining the effective section for flanged beams, not more than three times the thickness of a flange, as shown in Figure C7.5 should be considered when computing A_o or A_{co}. The designer may neglect the contribution of flanges to torsional strength and stiffness.

A3 **C7.6.1.9** *Torsional and flexural shear together*

Equation 7–9 limits the maximum shear stress due to combined torsion and shear to $0.2f'_c$ or 10 MPa whichever is less.

A2 **C7.6.2** ***Reinforcement for compatibility torsion***

Where members may be subjected to twist due to the requirements of compatibility, but the torsional moments in the member are assumed to be negligible due to the loss of torsional stiffness, it is important to ensure that the member has sufficient ductility to enable the required level of twist to develop without causing a brittle failure. To achieve this, the clause requires longitudinal and transverse reinforcement to be added to the member to give it a nominal torsional strength equal to the smaller of:

(a) The torsional moment found in an analysis neglecting the reduction in stiffness due to torsional cracking; or

(b) The torsional moment equal to $T_{n.min}$ given by Equation 7–10.

A3 The minimum stirrup and longitudinal reinforcement required to satisfy Equation 7–10 corresponds to a torsional moment of approximately 67 % of the average torsional cracking moment. This reinforcement ensures that the member has adequate ductility to enable redistribution of the torsional actions, which occurs due to the loss of torsional stiffness associated with torsional cracking[7.19], but does not significantly decrease the flexural or shear strengths of the member.

Equation 7–10 gives an estimate of the torsional moment causing torsional cracking. Reinforcement to sustain this moment is based on a space truss where the concrete sustains diagonal compression forces inclined at an angle between 35° and 55° to the axis of the member. A2

For a member to have ductility in torsion, the torsional strength must be limited by yielding of the reinforcement rather than crushing of the concrete. For this case, the thin wall tube concept can be employed with an equivalent thin walled steel tube with a thickness of t_s and yield strength of f_{ys} given by:

$$t_s\, f_{ys} = \left(\frac{A_t\, f_{yt}}{s} \frac{A_\ell\, f_y}{p_o} \right)^{0.5} \qquad \text{(Eq. C7–9)}$$

which gives a nominal torsional strength of:

$$T_n = 2\, A_o\, t_s\, f_{ys} \qquad \text{(Eq. C7–10)}$$

Substitution for $t_s\, f_{ys}$ in Equation C7–10 gives Equation 7–11(a) and the limits given in Equation 7–11(b) limit the inclination of the diagonal compression forces to within the range of 35° to 55°.

As indicated in 7.6.2.2 and 7.6.2.3, any adequately anchored longitudinal reinforcement and any closed ties or stirrups, which are in the section to resist flexure or shear, may be included in the areas of transverse reinforcement, A_t, and longitudinal reinforcement, A_ℓ, required by Equation 7–11(a).

C7.6.3 *Torsional reinforcement details*

To ensure the effective response of the torsional reinforcement this clause requires that the closed stirrups be closely spaced and that reasonably sized longitudinal bars be placed in each corner of the closed stirrups [7.15, 7.30]. Because of the possibility of the cover concrete spalling, for reasons commented on in C7.6.1.7, closed stirrups for torsion are required to be anchored by either welding, or anchoring by 135° hooks, or by 90° hooks where a flange protects the concrete on the outside of the hook from spalling, as illustrated in Figure C7.5A. A3

Figure C7.5A illustrates acceptable ways of anchoring stirrups designed to resist torsion. A2

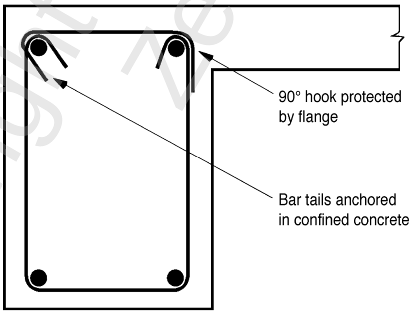


Figure C7.5A – Anchorage of stirrups for torsion

C7.6.4 *Design of reinforcement for torsion required for equilibrium*

C7.6.4.1 and C7.6.4.2 *Torsion design moment and torsion reinforcement*

Where a torsional moment required for equilibrium exceeds the limit given in 7.6.4.1, the required nominal torsional strength is taken as T^*/ϕ. A3

For torsion required for equilibrium, it is assumed that there is no interaction between torsion, flexure or shear, with or without axial load. Consequently, the requirements for transverse and longitudinal reinforcement to resist torsion are added to reinforcement areas required for the other actions in the load A2

A2 cases involving torsion. However, it should be noted that this is an approximation which does not hold in plastic hinge regions. In such zones, the longitudinal reinforcement increases the flexural strength and the torsional resistance decreases to a negligible value.

The equations for the area of closed stirrups and longitudinal reinforcement to resist torsion associated with equilibrium requirements, given in Equation 7–12, are based on the assumption that the diagonal compression struts, and hence diagonal tension cracks, form at an angle of close to 45° to the axis of the member. This assumption is made to be consistent with the assumption on which the shear design equations are based. It should be noted that this is in contrast to the more relaxed assumptions made where the torsional reinforcement is required for torsion induced by compatibility requirements alone.

Where a member is designed to resist torsion required for equilibrium, it is important to check that the deformation due to torsion in the serviceability limit state is acceptable, see C7.6.1.3.

C7.6.4.3 *Longitudinal torsional reinforcement reduction in compression zone*

Torsion induces longitudinal tension uniformly around the perimeter of the section. This distributed force is superimposed on any flexural forces in the section. Equation 7–12(b) gives the total longitudinal tension force, $A_\ell f_y$, required to resist torsion. The proportion of this force in the compression zone is b''/p_o, where b'' is the core dimension measured from centre-to-centre of the peripheral stirrup in the compression zone. Hence, from the above and Equation 7–12(b) the longitudinal tension force due to torsion in the compression zone, T_{tc}, is given by:

$$T_{tc} = v_{tn} t_o b'' \qquad \text{(Eq. C7–15)}$$

A3 The compression force due to flexure, C, in a beam where there is no axial load or prestress, may be taken as:

A2

$$C = \frac{M^*}{0.9d} \qquad \text{(Eq. C7–16)}$$

where the $0.9d$ represents the internal lever-arm, which is an approximation for an internal lever-arm in a cracked elastic section. Combining these two equations the area of longitudinal reinforcement required in the compression zone, $A_{\ell c}$, is given by:

$$A_{\ell c} = \left[\frac{v_{tn} t_o b'' - \dfrac{M^*}{0.9d}}{f_y} \right] \qquad \text{(Eq. C7–17)}$$

It should be noted that when $A_{\ell c}$ in the equation above is negative, it indicates that longitudinal reinforcement is not required for torsion. However bars must be placed in the corners of the stirrups to provide anchorage against the diagonal compression forces, which meet at the corners.

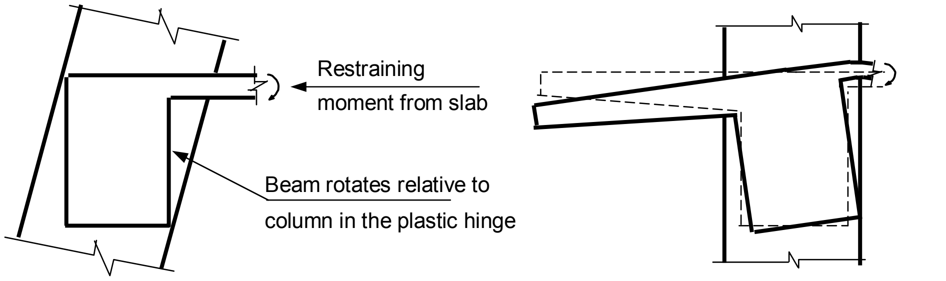
A3

C7.6.5 *Interaction between flexure and torsion*

The interaction between flexure and torsion has important implications for the seismic performance of the member. The yielding of flexural tension reinforcement in beams and columns can reduce the torsional resistance to essentially zero for the duration of yielding of the flexural tension reinforcement. As illustrated in Figure C7.6, this can result in the beam remaining in its original orientation where there is flexural restraint against rotation, but with twisting occurring between the beam and the column in the plastic region. Alternatively as shown in part (b) of the figure, if the cantilever moment exceeds the flexural restraint moment from the slab the beam will rotate relative to the column, with the relative rotation increasing each time the longitudinal beam reinforcement yields in tension.

The interaction between flexure and torsion also has important implications for shear cores in buildings. If the shear core yields in flexure a major loss in torsional resistance may result in twisting and failure of the shear core.

A3



(a) Restraint from slab holds beam in its initial orientation

(b) Moment from cantilever exceeds restraint moment from slab and beam rotates relative to column

Figure C7.6 – Rotation of beam relative to column when a plastic hinge forms in the beam adjacent to the column face

C7.7 Shear-friction

C7.7.1 *General*

With the exception of 7.7, virtually all provisions regarding shear are intended to prevent diagonal tension failures rather than direct shear transfer failures. The purpose of 7.7 is to provide design methods for conditions where shear transfer should be considered: an interface between concretes cast at different times, an interface between concrete and steel, reinforcement details for precast concrete structures, and other situations where it is considered appropriate to investigate shear transfer across a plane in structural concrete (see References 7.31 and 7.32). It should be noted that satisfying shear friction provisions does not ensure that diagonal tension failure will not occur. A3

C7.7.3 *Design approach*

Although uncracked concrete is relatively strong in direct shear there is always the possibility that a crack will form in an unfavourable location. The shear-friction concept assumes that such a crack will form, and that reinforcement must be provided across the crack to resist relative displacement along it. When shear acts along a crack, one crack face slips relative to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. At ultimate, the separation is sufficient to stress the reinforcement crossing the crack to its yield point. The reinforcement provides a clamping force $A_{vf}f_y$ across the crack faces. The applied shear is then resisted by friction between the crack faces, by resistance to the shearing off of protrusions on the crack faces, and by dowel action of the reinforcement crossing the crack. The successful application of 7.7 depends on proper selection of the location of an assumed crack [7.31, 7.32].

C7.7.4 *Shear-friction design method*

C7.7.4.1 *Shear-friction reinforcement perpendicular to shear plane*

The relationship between shear-transfer strength and the reinforcement crossing the shear plane can be expressed in various ways. Equations 7–13 and 7–14 of 7.7.4 are based on the shear-friction model. This gives a conservative prediction of shear-transfer strength. The equations include the effect of external force N^* acting normal to the shear plane. Other relationships that give a closer estimate of shear-transfer strength [7.33, 7.34, 7.35] can be used under the provisions of 7.7.3. For example, when the shear-friction reinforcement is perpendicular to the shear plane and there is no external force acting normal to the shear plane, the shear strength V_n is given by References 7.34 and 7.35.

$$V_n = 0.8A_{vf}f_y + A_cK_1 \quad \text{(Eq. C7–18)}$$

where A_c is the area of concrete section resisting shear transfer (mm^2) and K_1 = 2.75 MPa for normal density concrete, 1.38 MPa for all-lightweight concrete, and 1.72 MPa for sand lightweight concrete. These values of K_1 apply to both monolithically cast concrete and to concrete cast against hardened concrete with a rough surface as defined in 7.7.9.

In this equation, the first term represents the contribution of friction to shear-transfer resistance (0.8 representing the coefficient of friction). The second term represents the sum of the resistance to shearing of protrusions on the crack faces and the dowel action of the reinforcement.

When the shear-friction reinforcement is inclined to the shear plane, and there is no external force acting normal to the shear plane, such that the shear force produces tension in that reinforcement, the shear strength V_n is given by

$$V_n = A_{vf}f_y\,(0.8\sin\alpha_f + \cos\alpha_f) + A_cK_1\sin^2\alpha_f \quad \text{(Eq. C7–19)}$$

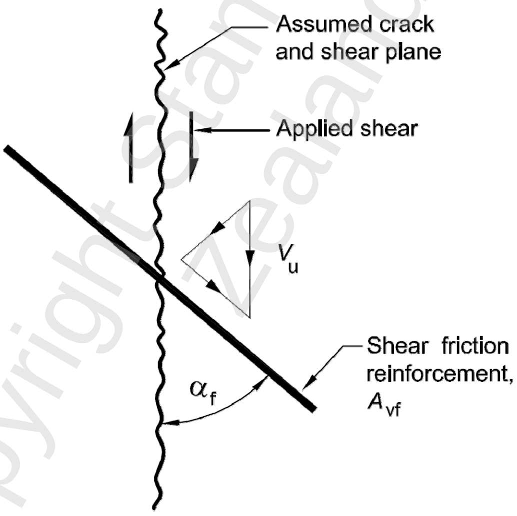
where α_f is the angle between the shear-friction reinforcement and the shear plane, (i.e. $0 < \alpha_f < 90°$).

When using the modified shear-friction method, the terms ($A_{vf}f_y/A_c$) or ($A_{vf}f_y \sin \alpha_f/A_c$) should not be less than 1.38 MPa for the design equations to be valid.

When using the shear-friction design method the required area of shear-transfer reinforcement A_{vf} is computed using Equations 7–13 and 7–14.

C7.7.4.2 *Shear-friction reinforcement inclined to shear plane*

When the shear-friction reinforcement is inclined to the shear plane, such that the component of the shear force parallel to the reinforcement tends to produce tension in the reinforcement, as shown in Figure C7.7, part of the shear is resisted by the component parallel to the shear plane of the tension force in the reinforcement.[7.35] Equation 7–14 should be used only when the shear force component parallel to the
A3 reinforcement produces tension in the reinforcement, as shown in Figure C7.7. When α_f is greater than 90°, the relative movement of the surfaces tends to compress the bar and Equation 7–14 is not valid.



A3

Figure C7.7 – Shear-friction reinforcement at an angle to assumed crack

With the exception of sliding shear resistance on potential plastic hinge regions the possible contribution of diagonal reinforcement acting in compression is discounted unless the diagonal bars are confined against buckling, as in coupling beams, or by stirrups in a beam (see 9.4.1.4). Sliding shear displacement opens up the crack, inducing tension in the diagonal bars. This is likely to offset the compression induced in a diagonal bar due to the shear displacement. For this reason the possible contribution of diagonal compression reinforcement is neglected except in the case of seismic plastic hinge regions where the magnitude of shear displacement is sufficient to overcome the tension force associated with the opening of the crack.

C7.7.4.3 *Coefficient of friction*

In the shear-friction method of calculation, it is assumed that all the shear resistance is due to the friction between the crack faces. It is, therefore, necessary to use artificially high values of the coefficient of friction in the shear-friction equations so that the calculated shear strength will be in reasonable agreement with test results. For concrete cast against hardened concrete not roughened in accordance with 7.7.9, shear resistance is primarily due to dowel action of the reinforcement. However, where laitance has not been removed and the surface not roughened the shear strength of the dowels alone should be calculated from Section 17, with allowance being made for any axial tension in the dowels. A3

The value of μ_f for concrete placed against as-rolled structural steel relates to the design of connections between precast concrete members, or between structural steel members and structural concrete members. The shear-transfer reinforcement may be either reinforcing bars or headed stud shear connectors; also, field welding to steel plates after casting of concrete is common. The design of shear connectors for composite action of concrete slabs and steel beams is not covered by these provisions, but should be in accordance with Reference 7.37. A3

C7.7.5 *Maximum shear stress for shear friction* A3

This upper limit on shear strength is specified because Equations 7–13 and 7–14 become non-conservative if V_n has a greater value.

C7.7.7 *Reinforcement for net tension across shear plane*

If a resultant tensile force acts across a shear plane, reinforcement to carry that tension should be provided in addition to that provided for shear transfer as indicated by Equations 7–13 and 7–14 . Tension may be caused by restraint of deformations due to temperature change, creep, and shrinkage. Such tensile forces have caused failures, particularly in beam bearings.

When moment acts on a shear plane, the flexural tension stresses and flexural compression stresses are in equilibrium. There is no change in the resultant compression $A_{vf}f_y$ acting across the shear plane and the shear-transfer strength is not changed. It is therefore not necessary to provide additional reinforcement to resist the flexural tension stresses, unless the required flexural tension reinforcement exceeds the amount of shear-transfer reinforcement provided in the flexural tension zone. This has been demonstrated experimentally[7.38].

It has also been demonstrated experimentally [7.32] that if a resultant compressive force acts across a shear plane, the shear-transfer strength is a function of the sum of the resultant compressive force and the force $A_{vf}f_y$ in the shear-friction reinforcement, as expressed in Equations 7–13 and 7–14. In design, advantage should be taken of the existence of a compressive force across the shear plane to reduce the amount of shear-friction reinforcement required, only if it is certain that the compressive force is permanent.

C7.7.8 *Shear-friction reinforcement*

If no moment acts across the shear plane, reinforcement should be uniformly distributed along the shear plane to minimize crack widths. If a moment acts across the shear plane, it is desirable to distribute the shear-transfer reinforcement primarily in the flexural tension zone.

Since the shear-friction reinforcement acts in tension, it should have a full tensile anchorage on both sides of the shear plane. Further, the shear-friction reinforcement anchorage should engage the primary reinforcement, otherwise a potential crack may pass between the shear-friction reinforcement and the body of the concrete. This requirement applies particularly to welded headed studs used with steel inserts for connections in precast and cast-in-place concrete. Anchorage may be developed by bond, by a welded mechanical anchorage, or by threaded dowels and screw inserts. Space limitations often require a welded mechanical anchorage. For anchorage of headed studs in concrete (see Reference 7.33).

C7.7.10 *Concrete placed against as-rolled structural steel*

When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, the steel shall be clean and free of paint.

C7.7.11 *Shear friction in walls* A3

Sliding shear occurs under inelastic cyclic loading when the compression forces in the wall are too small to force reinforcement, which has been strained beyond yielding in tension in a previous half cycle, to yield in compression and close the crack under load reversal. The problem of sliding shear generally occurs in squat walls, where the actions of flexure and shear can generate wide cracks at the base of the wall under repeated inelastic deformation.

A3

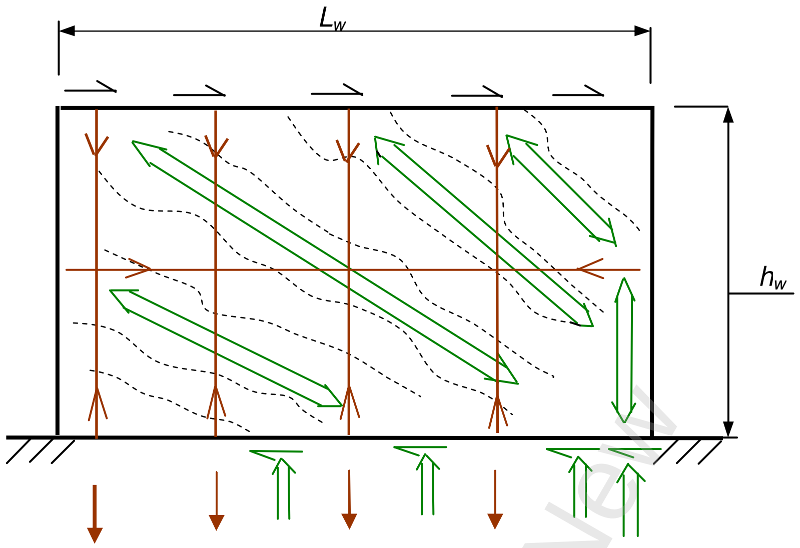


Figure C7.8 – Flexural and shear actions in a squat wall

The flexural and shear actions in a squat wall are illustrated in Figure C7.8. In squat walls, once diagonal cracking occurs the shear is resisted by a truss-like action, which requires vertical reinforcement to be provided in the mid-region of the wall. Potentially, if inelastic lateral deformation is sufficient the reinforcement outside the compression zone will yield in tension. With load reversal additional yielding may occur in the reinforcement in the mid-region of the wall. In addition, the flexural compression force may be insufficient to close the crack in the new compression zone, which was the previous tension zone. Under these reversed loading conditions a wide crack develops at the base of the wall, which destroys aggregate interlock action and results in a sliding type action[7.44, 7.45].

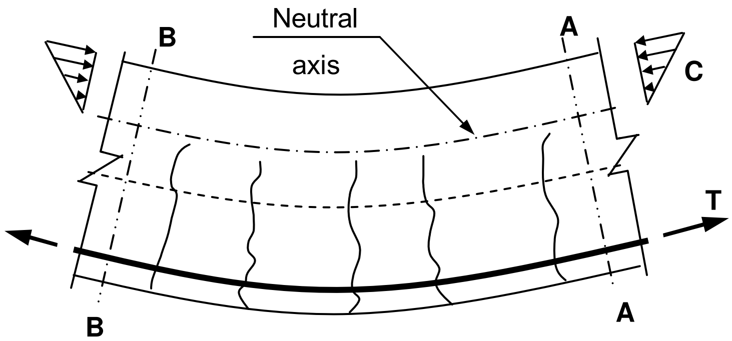
The dowel resistance of the reinforcement crossing the crack is small. Consequently, after a few inelastic cycles the sliding occurs at low load levels following load reversal. The resultant bending of the bars reduces the ability of the flexural compression force to yield the reinforcement in the compression zone and close the crack. Sliding at the crack results in a highly pinched form of lateral load versus deflection, and strength degradation[7.44, 7.45].

As illustrated in Figure C7.8 the truss-like action in a squat wall reduces the internal lever-arm between the positions of the resultant tension and compression forces at the base of the wall. Consequently, the flexural strength calculated from standard flexural theory can overestimate the actual strength.

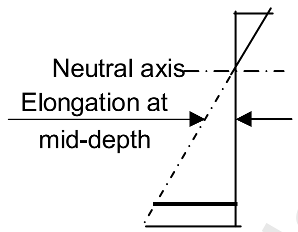
C7.8 Elongation

Elongation in structural members arises from the difference in compression and tension strains in reinforcement when members are subjected to bending as illustrated in Figure C7.9. Once a flexural crack is formed in an RC beam, elongation occurs as the tensile strains are greater than the corresponding compression strains. The magnitude of elongation is small and reversible prior to yield of longitudinal reinforcement, however, with the formation of a plastic hinge elongation increases significantly [7.39, 7.40, 7.41, 7.42, 7.43]. Where a unidirectional plastic hinge forms, standard flexural theory can be used to determine longitudinal strains on both sides of the plastic hinge and hence the elongation can be estimated. Tests have shown that Equation 7–15a predicts elongation with sufficient accuracy for design purposes[7.43].

A3



(a) Reinforced concrete beam subjected to flexure



(b) Displacement of A-A relative to B-B

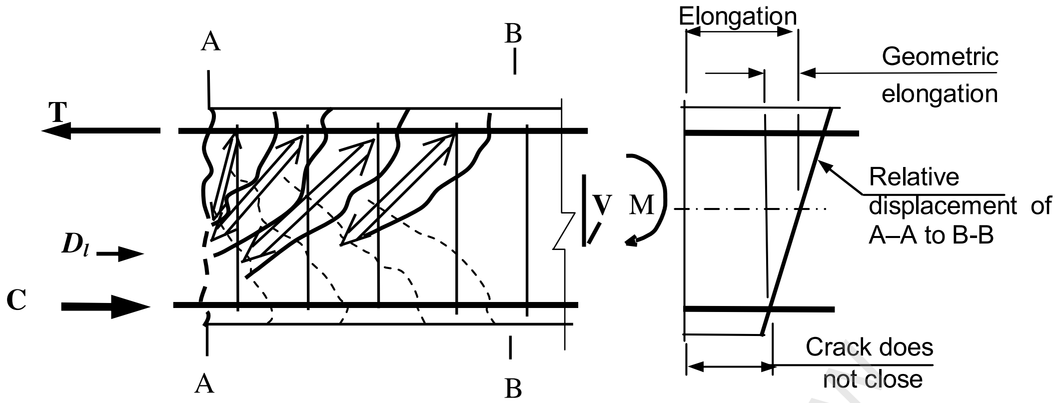
Figure C7.9 – Geometric elongation in a beam

In plastic hinge zones where reversing inelastic deformation is imposed, standard flexural theory no longer applies and the magnitude of elongation increases relative to that sustained by unidirectional plastic hinges. This increase arises from two actions[7.39,7.41,7.43] described below:

(a) Yielding of the longitudinal reinforcement results in wide flexural-shear cracks forming in the plastic hinge. Shear deformation results in some of the aggregate particles becoming dislocated at the cracks and these prevent the cracks formed from fully closing when a zone that was previously subjected to tension is being subjected to compression. This action results in the compression zone sustaining tensile strains;

(b) When diagonal cracks form in a plastic hinge the shear force is resisted almost entirely by tension in the stirrups and diagonal compression forces in the web, as illustrated in Figure C7.10. Considering a section through the plastic hinge it can be seen that the tension force, *T*, balances the flexural compression force, C, together with the longitudinal component of the diagonal compression forces, D_{ℓ}. It follows that in a beam without axial load the flexural tension force will always be greater than the flexural compression force. When reversing rotations are imposed on the plastic hinge, yielding occurs to a greater extent in tension than in compression. Consequently additional elongation occurs.

A3



(a) Crack pattern and forces in plastic hinge region

(b) Displacement over length of plastic region

Figure C7.10 – Elongation in reversing plastic region

There is no simple theoretical method of determining the magnitude of elongation that will arise in reversing plastic hinges as this depends on the number, magnitude and sequence of inelastic displacements imposed on plastic hinges[7.40]. Equations 7–15(a) to 7–15(c) have been developed from test results. Further information on elongation may be found in references [7.39, 7.40, 7.41, 7.42, 7.43].

Where unequal top and bottom reinforcement is used in a beam the elongation reduces when the smaller area of reinforcement is in compression and it increases when the larger area of reinforcement is in compression. The average elongation does not change significantly.

The application of axial force on the member reduces the extension in the compression zone. A limited number of tests indicate that an axial compression force of $0.08\,A_g f'_c$ causes the crack in the compression zone to close at ultimate limit state rotations. Hence where the axial load is equal to or greater than this value, only geometric elongation is assumed to occur and equation 7–15(a) can be used.

In structural walls with the exception of squat walls (see 7.7.11) and to a lesser extent in columns, tension reinforcement is spread along the section. This results in the area of compression reinforcement being appreciably smaller than the corresponding area of tension reinforcement. Consequently the compression reinforcement yields back to close the cracks in the compression zone. On this basis, only geometric elongation may be expected to arise.

When plastic hinges form, appreciable elongation can be induced. Restraint to this elongation by surrounding structural elements can induce compression in the member and tension in the surrounding elements. As most analysis packages do not model elongation, these actions are generally not predicted in structural analyses[7.39, 7.40, 7.41]. It is important designers are aware of elongation-induced actions so that appropriate allowance can be made for any adverse structural effects that may arise.

Figure C7.11 shows examples of floor plans in frame buildings where potential plastic hinge locations are labelled as U, R1 and R2 to indicate the extent that the floor restrains elongation in the plastic hinge. As explained by Fenwick et al.[7.42], type U plastic hinges are considered unrestrained by the floor slab and the full elongation as calculated by Eq. 7–15(b) applies. In type R1 (restrained) the prestressed units provide significant restraint to elongation and the elongation is approximately 50 % of that calculated by Eq. 7–15(b). Type R2 is an intermediate between type U and R1 and should be considered to attract the full elongation as calculated by Eq. 7–15(b) unless analysis can show otherwise.

A3

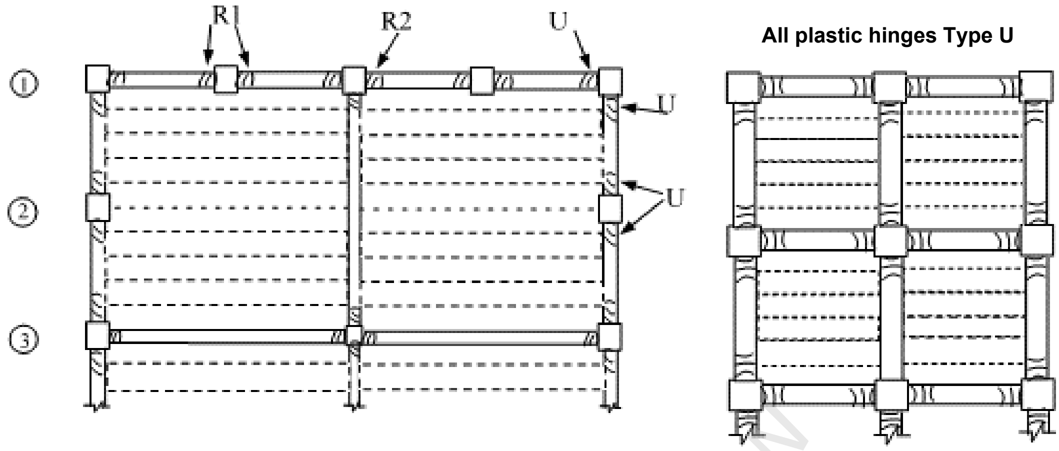


Figure C7.11 – Definition of unrestrained and restrained plastic hinges in frame buildings[7.42]

REFERENCES

7.1 Park, R. and Paulay, T., "Reinforced Concrete Structures", Wiley Interscience, New York, 1975, p. 769.

7.2 "New Zealand Reinforced Concrete Design Handbook", Cement and Concrete Association of New Zealand, Wellington 1992.

7.3 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced Concrete and Masonry Buildings", Wiley Interscience, New York, 1992, 774 pp.

7.4 Scott, B.D., Park R. and Priestley, M.J.N., "Stress-strain Behaviour of Concrete Confined by Overlapping Hoops at Low and High Strain Rates", ACI Journal, Proc. Vol. 79, No. 1, Jan.-Feb. 1982, pp. 13-27.

7.5 CEB-FIP "Model Code 1990", Published Thomas Telford, London, 1993, p. 473.

7.6 Mander, J.B., Priestley, M.J.N. and Park, R., "Theoretical Stress-strain Model for Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1804-1826.

7.7 ACI Committee 363, "State of the Art Report on High-Strength Concrete", ACI Journal, Proceedings Vol. 81, No. 4, July-August 1984, pp. 364-411.

7.8 Hognestad, E., Hanson, N.W. and McHenry, D., "Concrete Stress Distribution in Ultimate Strength Design", ACI Journal, Proceedings Vol. 52, No. 6, December 1955, pp. 455-479.

7.9 Rüsch, H., "Versuche zur Festigkeit der Biegedruckzone", Bulletin No. 120, Deutscher Ausschuss für Stahlbeton, Berlin, 1955, 94 pp.

7.10 Nedderman, H., "Flexural Stress Distribution in Extra High Strength Concrete", MS Thesis, University of Texas at Arlington, 1973.

7.11 Kaar, P.H., Hanson, N.W. and Capell, H.T., "Stress-strain Characteristics of High Strength Concrete". Douglas McHenry International Symposium on Concrete and Concrete Structures, ACI Publication SP55, 1978, pp. 161-185.

7.12 Li Bing., Park, R. and Tanaka, H., "Effect of Confinement on the Behaviour of High Strength Concrete Columns Under Seismic Loading", Pacific Conference on Earthquake Engineering, Proceedings Vol. 1, Auckland, November 1991, pp. 67-78.

7.13 Mattock, A.H., Kriz, L.B. and Hognestad, E., "Rectangular Concrete Stress Distribution in Ultimate Strength Design", ACI Journal, Proceedings Vol. 57, No. 8, Feb. 1961, pp. 875-928.

7.14 ACI-ASCE Committee 426, "The Shear Strength of Reinforced Concrete Members", Chapters I to 4, Proceedings, ASCE, Vol. 99, No. ST6, June 1973, pp. 1148-1157. Also ACI Manual of Concrete Practice, American Concrete Institute, Detroit. See also Reference 9.9.

7.15 ACI-ASCE Committee 426, "Suggested Revisions to Shear Provisions for Building Codes", ACI Committee Report ACI 426:1R-77, 1979, 82 pp.

7.16 Bresler, B. and MacGregor, J.G., "Review of Concrete Beams Failing in Shear", Proceedings, ASCE, V. 93, No. STI, Feb. 1967, pp. 343-372.

7.17 MacGregor, James G. and Hanson, John M., "Proposed Changes in Shear Provisions for Reinforced and Prestressed Concrete Beams", ACI Journal, Proceedings Vol. 66, No. 4, April 1969, pp. 276-288.

7.18 Collins, M.P. and Mitchell, D., "Design Proposal for Shear and Torsion", Journal, Prestressed Concrete Institute, Vol. 25, No. 5, Sept.-Oct. 1980, pp. 32-100.

7.19 Collins, M.P. and Mitchell, D., "Prestressed Concrete Structures", Prentice Hall, 1991, 784 pp.

7.20 Collins, M.P. and Mitchell, D., "Evaluating Existing Bridge Structures using the Modified Compression Field theory", ACI Publication Strength Evaluation of Existing Bridges, ACI SP-88 1985, pp. 109–141.

7.21 Eurocode 2, "Design of Concrete Structures, Part 1-1, "General – Common Rules for Buildings and Civil Engineering Structures"; Brussels, 1992.

7.22 AASHTO, "Standard Specifications for Highway Bridges, 17th Edition," American Association of State Highway and Transportation Officials, Washington, 2002.

A3 7.23 ACI 318:11, "Building Code Requirements for Structural Concrete", American Concrete Institute, 2011.

7.24 Comite European du Beton, Commission V, Manual de calcul, "Effort Tranchant – Torsion", CEB Bulletin, No. 92, June 1973, p. 278.

7.25 Comite European du Beton, Commission "Strength of Reinforced and Prestressed Concrete Beams CEB Approach", ACI Symposium 1976, Philadelphia, U.S.A..

7.26 Zia, P. and McGee, W.D., "Torsion Design of Prestressed Concrete", PCI Journal, Vol. 19, No. 2, March/April 1974, pp. 46-65, and No. 6, Nov./Dec. 1974, pp. 67-82.

7.27 Collins, M.P. and Mitchell, D., "Prestressed concrete basics", Canadian Prestressed Concrete Institute, Ottawa, 1987, 99. 353-383

7.28 Collins, Michael, P., "The Torque-twist Characteristics of Reinforced Concrete Beams", Inelasticity and Non-linearity in Structural Concrete, University of Waterloo Press, 1972, p. 211-232.

7.29 Collins, M.P. and Mitchell, D., "Prestressed Concrete Basics", Canadian Prestressed Concrete Institute, Ottawa, 1987. A3

7.30 Mitchell, D., and Collins, M.P., "Detailing for Torsion", ACI Journal, Proceedings, Vol. 73, No. 9, Sept. 1976, pp. 506-511.

7.31 Birkeland, P.W. and Birkeland, H.W., "Connections in Precast Concrete Construction," ACI Journal, Proceedings Vol. 63, No. 3, Mar. 1966, pp. 345-368.

7.32 Mattock, A.H., and Hawkins, N.M., "Shear Transfer in Reinforced Concrete – Recent Research," Journal of the Prestressed Concrete Institute, Vol 17 No. 2, Mar.-Apr. 1972, pp. 55-75.

7.33 "PCI Design Handbook – Precast and Prestressed Concrete, 4th Edition, Precast/Prestressed Concrete Institute, Chicago, 1992, 580 pp.

7.34 Mattock, A.H., Johal, L. and Chow, H.C., "Shear Transfer in Reinforced Concrete with Moment or Tension Acting Across the Shear Plane," Journal of the Prestressed Concrete Institute, Vol. 20 No. 4, July-Aug. 1975, pp. 76-93.

7.35 Mattock, A.H., "Shear Transfer in Concrete Having Reinforcement at an Angle to the Shear Plane," Shear in Reinforced Concrete, SP-42, American Concrete Institute, Farmington Hills, MI, 1974, pp. 17-42.

7.36 Mattock, A.H. Discussion of "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads," by PCI Committee on Precast Concrete Bearing Wall Buildings, Journal of the Prestressed Concrete Institute, Vol. 22, No. 3, May-June 1977, pp. 105-106.

7.37 "NZS 3404:1997, Steel Structures Standard, section 13", New Zealand Standards Association.

7.38 Mattock, A.H., Johal, L. and Chow, H.C., "Shear Transfer in Reinforced Concrete with Moment or Tension Acting Across the Shear Plane," Journal of the Prestressed Concrete Institute, Vol. 20 No. 4, July-Aug. 1975, pp. 76-93.

7.39 Peng, B., Dhakal, R.P., Fenwick, R., Carr, A. and Bull, D., "Elongation of Plastic Hinges in Ductile RC Members: Model Development", Journal of Advanced Concrete Technology, Vol. 9, No. 3, 2011, pp. 315-326. A3

7.40 Peng, B., Dhakal, R.P., Fenwick, R., Carr, A. and Bull, D., "Elongation of Plastic Hinges in Ductile RC Member: Model Verification", Journal of Advanced Concrete Technology, Vol. 9, No. 3, 2011, pp. 327-338.

7.41 Peng, B., Dhakal, R.P., Fenwick, R., Carr, A. and Bull, D., "Multi-Spring Hinge Element for Reinforced Concrete Frame Analysis", ASCE Journal of Structural Engineering, Vol. 139, No. 4, 2013, pp. 595-606.

7.42 Fenwick, R., Bull, D. and Gardiner, D., "Assessment of Hollow-Core Floors for Seismic Performance", Research Report 2101-2, Civil and Natural Resources Engineering, University of Canterbury, 2010, p. 151.

7.43 Fenwick, R.C. and Megget, L.M., "Elongation and Load Deflection Characteristics of Reinforced Concrete Members Containing Plastic Hinges", Bulletin NZ National Society for Earthquake Engineering, Vol. 26, No. 1., Mar. 1993, pp. 28-41.

7.44 Paulay, T., Priestley, M.J.N. and Synge, A. J., "Ductility in Earthquake Resisting Squat Shear Walls", ACI Journal, Vol. 79, No. 4, July-August 1982, pp. 257-269.

7.45 Synge, A.J., "Ductility of Squat Shear Walls", Department of Civil Engineering, University of Canterbury, Report 80-8, February, 1980, pp. 142.

NOTES

C8 STRESS DEVELOPMENT, DETAILING AND SPLICING OF REINFORCEMENT AND TENDONS

C8.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 8 of the Standard:

A_s area of non-prestressed tension reinforcement, mm^2 A3

c_b minimum top cover to reinforcing bar, mm
c_p minimum clear spacing between reinforcing bars in a horizontal layer, mm
c_s minimum side cover to reinforcing bar, mm
h_c column depth parallel to the longitudinal beam bars being considered, mm
h_b beam depth, mm
L_s splice length, mm
N_s restraining force developed by a circular tie against one vertical column bar, N
R tensile strength of circular tie or spiral, N

C8.2 Scope

The provisions of Section 8 apply to detailing of reinforcement and tendons including the design of anchorage, development and splices. Provisions also cover minimum bend radii, minimum reinforcement in walls and shrinkage and temperature reinforcement in slabs. Requirements are also given for detailing spiral, circular and rectangular hoop reinforcement in columns and for stirrups and ties in flexural members.

The provisions for development include deformed and plain bars and wire, bundled bars, welded and smooth wire fabric and prestressing strand.

They also cover standard hooks in tension, mechanical anchorage and anchorage of transverse reinforcement. A comprehensive set of requirements is given to govern development of flexural reinforcement. Provisions for splices deal with lap splices, welded splices and mechanical connections.

C8.3 Spacing of reinforcement

C8.3.1 *Clear distance between parallel bars*

The spacing limits of this clause have been developed from successful practice over many years, remaining essentially unchanged through many codes. The minimum limits were established to permit concrete to flow readily into spaces between bars and forms without developing honeycomb, and to prevent the concentration of bars on a line that might result in shear or shrinkage cracking. The spacing between two bar bundles in slabs must conform with 8.3.1

C8.3.4 *Bundled bars*

Bond research [8.1] showed that bar cut-offs for beams and splices for columns should be staggered. Bundled bars should be tied, wired or otherwise fastened together to ensure that they remain in position.

The limitation that bars larger than 32 mm be not bundled in beams or girders of buildings has been taken from the ACI 318 Code which applies primarily to buildings. The 1974 American Association of State Highway and Transportation Officials (AASHTO) design criteria [8.2] for reinforced concrete bridges permitted two-bar bundles of bars up to 57 mm in bridge girders or columns, usually more massive than those in buildings. Conformity with crack control requirements in the Standard will effectively preclude bundling of bars larger than 35 mm as tensile reinforcement. The Standard phrasing “bundled in contact, assumed to act as a unit”, is intended to preclude bundling more than two bars in the same plane. Typical bundle shapes are triangular, square, or L-shaped patterns for three or four-bar bundles. As a practical provision, bundles more than one bar deep in the plane of bending may not be hooked or bent as a unit. Where end hooks are required, it is preferable to stagger them. Bending and hooking of bundles must be established in this manner, even at supports.

C8.3.5 *Spacing of principal reinforcement in walls and slabs*

These maximum spacing limits have remained essentially unchanged for many years. The spacing of reinforcement in topping slabs shall be such that effective anchorage required for diaphragm action, in accordance with 13.3.5, is assured.

C8.3.6 *Spacing of outer bars in bridge decks or abutment walls*

Experience has shown that the spacing of reinforcement at greater than 300 mm centres in the exposed surfaces of bridge decks or abutment walls is likely to result in long-term maintenance problems, due to shrinkage effects of direct exposure to the weather and the fatigue effects of live loading. Two cases are given which would permit the spacing to be increased to a maximum of 450 mm. An example of (a) would be the soffits of cantilever slabs, while an example of (b) would be the earth face of abutment retaining walls.

C8.3.7 *Spacing between longitudinal bars in compression members*

These requirements for minimum bar spacing, like those in 8.3.1, were developed originally to provide access for concrete placing in columns. Use of the bar diameter as a factor in establishing the minimum spacing permitted is an extension of the original provision to larger bars.

C8.3.8 *Spacing between splices*

Commentary C8.3.1 and C8.3.7 are applicable here. See also 8.3.5.

C8.3.9 *Spacing between pretensioning reinforcement*

These requirements are provided to prevent weakened planes for splitting bond failure developing in the cover concrete in the anchorage zones. Provision has been made for reducing the clear distance for hollow-core flooring systems.

C8.3.10 *Bundles of ducts for post-tensioned steel*

When ducts for post-tensioning steel in a beam are arranged closely together vertically, provision must be made to prevent the reinforcement, when tensioned, from breaking through the duct. Horizontal disposition of ducts must allow proper placement of concrete. Generally a clear spacing of $1^1/_3$ times the nominal maximum size of the coarse aggregate, but equal to or greater than 25 mm, proves satisfactory. Where concentration of reinforcement or ducts tends to create a weakened plane in the concrete cover, reinforcement should be provided to control cracking.

C8.4 Bending of reinforcement

C8.4.2.1 *Minimum bend diameters for main bars*

The minimum bend diameters given in Table 8.1 are generally twice the bend test diameters specified in AS/NZS 4671.

When large steel stresses need to be developed in the bend, radial bearing and splitting stresses in the concrete may become excessive. Equation 8–1 controls the diameter of the bend when there is a combination of high tensile stress at the bend, large bar diameter and low concrete strength.

The quantities shown in Table 8.1 are based on the assumption that $s_b = d_b + 40$. The requirements for the development length may be difficult to satisfy when beam bars are anchored in exterior columns in accordance with 9.4.3.2.5. In such cases the means by which the development length of hooked bars can be reduced, as permitted in 8.4.2.1, should be investigated, or alternatively the diameter of bend must be increased.

Clause 8.4.2.1 also permits the addition of transverse bars to allow the use of Table 8.1 values for d_i in cases where Equation 8–1 in would call for larger values.

The arrangement of the transverse bars is shown in Figure C8.1, and is based on the fact that excessive bearing stresses will not extend past the first 60° of bend that is closest to the critical section.

The transverse bars should extend for a distance of at least 3 d_b beyond the centreline of the outermost bars in each layer.

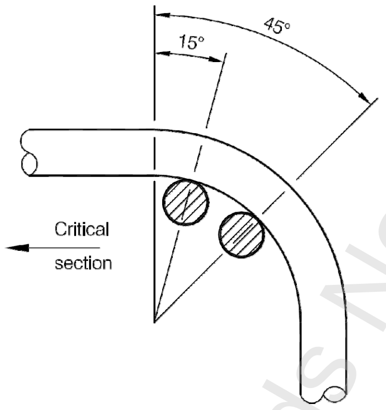


Figure C8.1 – Arrangement of additional transverse bars to reduce bearing stress

C8.4.2.2 *Minimum bend diameter in fatigue situations*

Bends in primary reinforcement should be avoided in regions of high stress range. The minimum diameter of bend of slab reinforcing bars, for example, of cranked transverse reinforcement in bridge deck slabs, is increased to the bar bend diameter equal to or greater than that specified in 2.5.2.2. High localised areas of stress concentration, which occur in tight radius bends, can cause fatigue failure to propagate from these locations. A2

C8.4.2.3 *Stirrup and tie bends*

It is not intended that a tie is to have different bend diameters should it pass round longitudinal bars of different diameters.

C8.4.3 *Bending of welded wire fabric*

Welded wire fabric of plain or deformed wire can be used for ties and stirrups. The wire at welded intersections does not have the same uniform ductility and ease of bending as in areas which were not heated. These effects of the welding temperature are usually dissipated over a length of approximately four wire diameters. Minimum bend diameters permitted are in most cases the same as those required in the bend test for wire material. The requirements of 8.4.3 for welded wire fabric are shown in Figure C8.2.

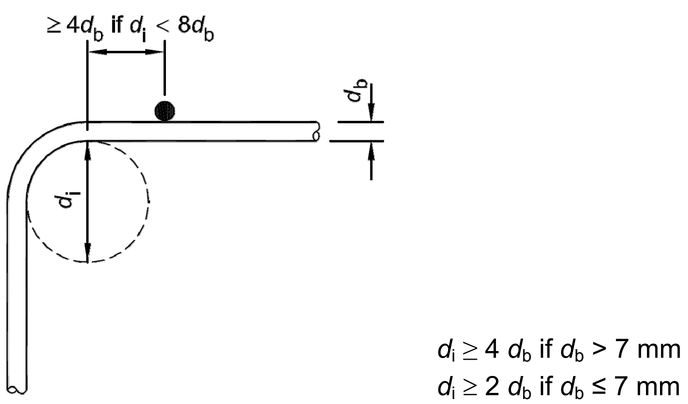


Figure C8.2 – Bends in welded wire fabric for stirrups and ties

C8.5 Welding of reinforcement

C8.5.1 *Compliance with AS/NZS 1554: Part 3*

Reinforcing steels not conforming to AS/NZS 4671 will require different welding techniques and the designer and fabricator must become familiar with these techniques before designing a weld or attempting to weld the steel.

Due to the low carbon metallurgy of reinforcing steel manufactured to AS/NZS 4671, the steel is considered readily weldable. However, AS/NZS 4671 permits a range of manufacturing processes for the production of steel reinforcement. Due care must be exercised for welding of such reinforcement because the welding process can alter the metallurgy and microstructure of the as-rolled bars. In certain situations this may result in lower yield strengths and lower ultimate tensile strengths in the heat affected zones of the weld sites. This may lead to detrimental behaviour with loss of ductility in the bar and fracture of the bar may occur. Refer to AS/NZS 1554:Part 3 and the reinforcement manufacturer's recommendations for details of appropriate welding techniques.

In line quenched and tempered bars are subject to loss of strength when welded.

Steel which has been heavily strained can become embrittled if heated, particularly into the critical range of approximately 250 °C – 450 °C. This can be avoided by welding (or any other heating procedure) at some distance from bends or, provided the steel is not quenched and tempered, by a stress relieving heat treatment of the bend zone.

C8.5.2 *In-line quenched and tempered steel bars*

Welding of in-line quenched and tempered bars can have detrimental effects on the strength and ductility of the bars and associated connection. AS 3600 requires designers to assume that the strength of such reinforcement has a design strength of 250 MPa when raised to the temperatures associated with welding, galvanising or hot bending. Such a requirement is considered inappropriate in a seismically active country where concentration of yielding at a weld position would be undesirable and could result in brittle failure.

C8.6 Development of reinforcement

C8.6.1 *Development of reinforcement – General*

The development length concept for anchorage of reinforcement was first introduced in the 1971 ACI Building Code, to replace the dual requirements for flexural bond and anchorage bond contained in earlier editions of the ACI Building Code. It is no longer necessary to consider the flexural bond concept which placed emphasis on the computation of nominal peak bond stresses. Consideration of an average bond resistance over a full development length of the reinforcement is considered more meaningful, partly because most bond tests consider average bond resistance over a length of embedment of the reinforcement, and partly because unpredictable extreme variations in local bond stresses exist near flexural cracks [8.1].

The term "development" used in this Standard implies the development of the required strength of the reinforcement at a critical section. This may be the computed tensile stress, yield strength, f_y, or breaking stress.

The development length concept is based on the attainable average bond stress over the length of embedment of the reinforcement. In application the development length concept requires the specified minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. In flexural members such peak stresses generally occur at the points specified in 8.6.12.2. This development length or anchorage is necessary on both sides of such peak stress points.

Often the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the shorter distance; for example, the negative moment reinforcement continuing through a support to the middle of the next span. However, bars often need to be extended by a greater distance to satisfy special requirements applicable to tension zones of flexural members in accordance with 8.6.12, 8.6.13 and 8.6.14.

C8.6.3 *Development length of deformed bars and deformed wire in tension*

A combination of recommendations from ACI 318 and other research [8.3] have been used to form the equations in 8.6.3.2 and 8.6.3.3.

C8.6.3.2 *Basic development length in tension*

Concrete cover, clear distance between bars in a layer and bar diameter are the principal quantities which determine the basic development length of a bar. Transverse reinforcement, crossing splitting cracks, will, to a certain extent, improve anchorage. Accordingly, empirical expressions have been derived to determine the basic development length L_{db}. Additional parameters, which may beneficially influence development, are then taken into account separately in 8.6.3.3.

The basic development length, L_{db}, in this clause is proportional to the lower characteristic yield stress of the bar. Equation 8–2 includes α_a to recognise that for top reinforcement the reduction in the quality of bond because the excess water used in the mix for workability and air entrapped during the mixing and placing operations may rise toward the top of the finished concrete before setting is complete. Entrapped below bars, this water and air leaves the bar less bonded to the concrete on the underside. For horizontal top bars in a structural member, bond resistance reflects this weakened underside restraint because the loss can be of the order of 50 % in extreme cases.

The symbol f'_c is limited to 70 MPa because data on the development and bond of bars at concrete strengths above 70 MPa is not readily available to date.

To allow designers to reduce L_{db}, if desired, a more rigorous determination of L_d may be undertaken using 8.6.3.3.

C8.6.3.3 *Refined development length in tension*

To reduce the development length specified in 8.6.3.2, three factors may be considered:

(a) Because the development length required is proportional to the tensile stress to be developed, the full development length may be reduced proportionally when the stress is lower than the yield strength. This is achieved by the modification factor A_{sr} /A_{sp}. It should be noted, however, that this reduction must not be used at or near critical sections of members subjected to earthquake forces, nor should the area of reinforcement required only to control shrinkage and temperature effects in restrained members be omitted when computing A_{sr}.

(b) Concrete splitting is a common mode of failure when the strength of bars is developed, and when the surrounding concrete cannot sustain the circumferential tensile stresses induced by the bond stresses. As either cover or clear distance between bars is increased, improved resistance to concrete splitting is achieved.

Equation 8–5 recognises that an increased cover or clear distance between bars will result in increased bond strength. Equation 8–5 is based entirely on test results. Figure C8.3 indicates typical splitting cracks along embedded bars. c_m is the smallest of c_b, c_s and c_p.

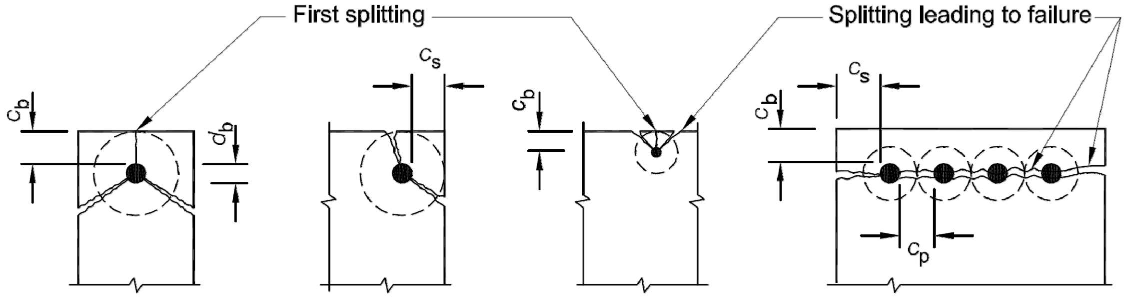


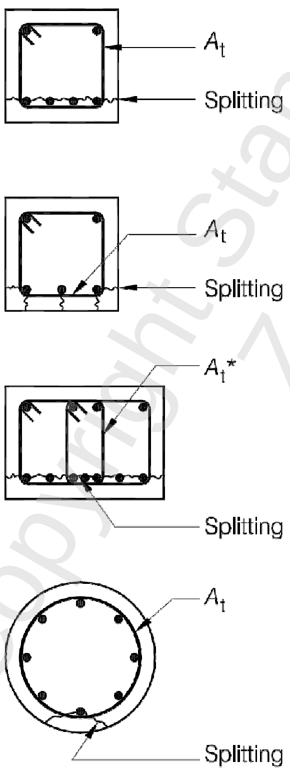
Figure C8.3 – Definition and significance of distances c_b, c_s and c_p

The beneficial effect of transverse reinforcement [8.4], which may control the opening of splitting cracks as shown in Figure C8.3, is expressed by the area of transverse reinforcement A_{tr}. The effectiveness of ties, stirrups, hoops or spirals in crossing a potential splitting crack is illustrated in Figure C8.4. For such reinforcement to be effective, at least three bars must cross a potential crack over the development length. However, transverse reinforcement used for any other purpose, such as shear resistance or to provide stability for compression bars or confinement of concrete, may be included for this purpose.

In the case of Figure C8.4 (a), A_{tr} is effective only for the outer bars. The designer would have several choices in this case. A different L_d could be calculated for the inner and outer longitudinal bars, or the effect of transverse reinforcement could be ignored, or A_{tr} could be taken into account as an average for the bars, using total area of transverse bars crossing the plane of splitting divided by the number of longitudinal bars in the layer, n. The last approach was checked [8.3] using data reported by Untrauer and Warren [8.5] and it was found to give a reasonable estimate of measured values. This approach is incorporated in the definition of A_{tr} in this section.

Where a number, n, of longitudinal bars are enclosed by a spiral, the value of A_{tr} to control splitting, as shown for a circular member in Figure C8.4 is given by:

$$A_{tr} = \frac{6A_t}{n} \leq A_t \quad \text{(Eq . C8–1)}$$



(a) For horizontal splitting through the layers of bars can put $A_{tr} = \frac{\Sigma A_t}{n}$

$$= \frac{2A_t}{4}$$

(b) For splitting through the cover concrete

$$A_{tr} = A_t$$

(c) For wide sections multiple stirrups are effective

$$A_{tr} = \frac{\Sigma A_t}{n}$$

$$= \frac{2\left(A_t + A_t^*\right)}{7}$$

(d) Spiral

$$A_{tr} = \frac{6A_t}{n} \leq A_t$$

Figure C8.4 – Basis for calculation of A_{tr}

The 300 mm minimum development length shall not be multiplied by the α_a, α_b, α_c or α_d factors.

Because the multiplier in 8.6.3.3(c), which allows for the beneficial effects of transverse reinforcement, involves additional calculations, the designer may always assume that A_{tr} is zero, so that the multiplier becomes unity.

C8.6.4 *Development length of plain bars and plain wire in tension*

As required by 5.3.1, plain bars other than those explicitly listed should only be used when special verifiable reasons exist. The development of plain bars in tension must not rely on straight development length.

C8.6.5 *Development length of deformed bars and deformed wire in compression*

These provisions are similar to those of the previous structural Concrete Standard, NZS 3101:1995.

The weakening effect of flexural tension cracks is not present for bars in compression and usually end bearing of the bars on the concrete is beneficial. Therefore, shorter development lengths have been specified for compression than for tension. The development length may be reduced by up to 25 % according to 8.6.5.3 when the compression reinforcement is enclosed within a column by spiral or rectangular ties, hoops or supplementary ties, or an individual spiral around each bar or group of bars is used. The interpretation of the effective transverse reinforcement area in the calculation of A_{tr} for 8.6.5.3 is defined in the notation of the Standard and illustrated in Figure C8.4.

C8.6.7 *Development of bundled bars*

An increased development length for individual bars is required when three or four bars are bundled together. The extra extension is needed because the grouping reduces the effective surface area over which bond stresses to the surrounding concrete can be transferred.

The designer should also note 8.3.4 relating to the cutoff points of individual bars within a bundle and 8.7.2.2 relating to splices of bundled bars.

C8.6.8 *Development of welded plain and deformed wire fabric in tension*

C8.6.8.1 *Development length of wire fabric*
The requirements of either 8.6.8.2 or 8.6.8.3 may be used to calculate the development length, L_d, required for plain or deformed wire fabric in tension.

C8.6.8.2 *Development length of welded wire fabric – cross wires considered*
Figure C8.5 shows the development requirements for wire fabric with the development primarily dependent on the location of the cross wires rather than the bond characteristics of the plain or deformed wire. An embedment of at least two cross wires 50 mm or more beyond the point of critical section is adequate to develop the yield strength of anchored wires [8.6].

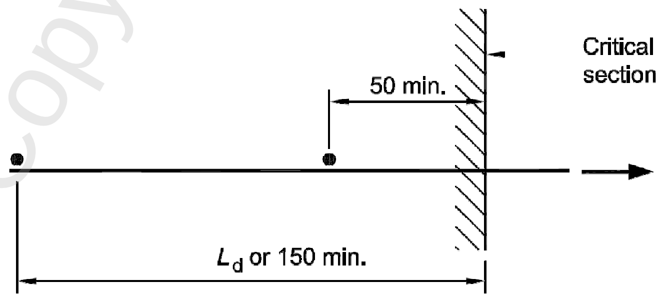


Figure C8.5 – Development of welded wire fabric

C8.6.8.3 *Development length of welded wire fabric – cross wires not considered*
If no cross-wires are present or assumed to be present the development of the plain or deformed wire will be dependent upon bond. Hence the requirements of 8.6.3 or 8.6.4 will govern, except that the minimum development length is reduced to 200 mm.

C8.6.9 *Development of prestressing strand*

The development requirements for prestressing strand are intended to provide bond integrity for the strength of the member. The provisions are based on tests performed on normal density concrete members with a minimum cover of 50 mm. These tests may not represent the behaviour of strand in low water-cement ratio, no-slump concrete. Fabrication methods should ensure consolidation of concrete around the strand with complete contact between the steel and concrete. Extra precautions should be exercised when low water-cement ratio, no-slump concrete is used. In general, this clause will control only for the design of cantilever and short-span members. The requirement of doubled development length for strand not bonded through to the end of the member is also based on test data [8.7].

The expression for development length L_d may be rewritten as:

$$L_d \geq \frac{\left[\frac{f_{se}}{3} d_b + \left(f_{ps} - f_{se}\right) d_b\right]}{7} \quad \text{(Eq . C8–2)}$$

where L_d and d_b are in mm, and f_{ps}, and f_{se} are in MPa. The first term represents the transfer length of the strand, that is, the distance over which the strand must be bonded to the concrete to develop the prestress f_{se} in the strand. The second term represents the additional length over which the strand must be bonded so that a stress f_{ps} may develop in the strand at nominal strength of the member.

The variation of strand stress along the development length of the strand is shown in Figure C8.6.

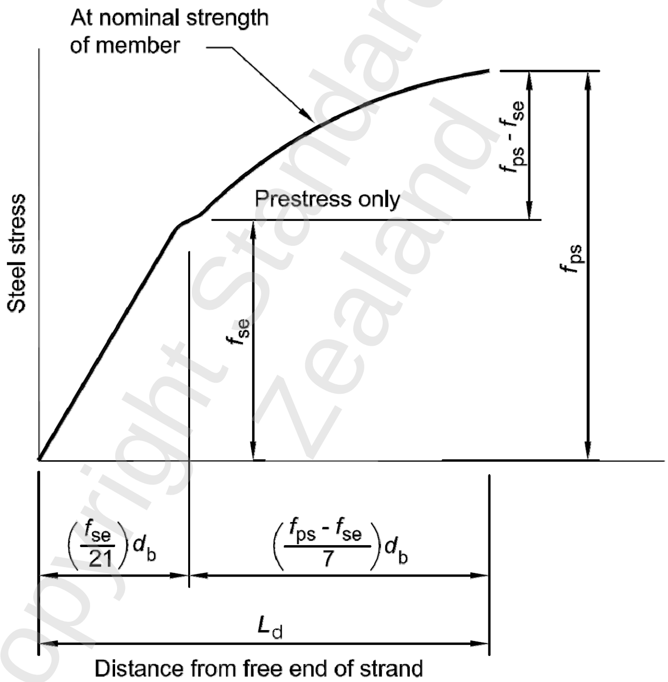


Figure C8.6 – Variation of steel stress with distance from free end of strand

The expressions for transfer length and for the additional bonded length necessary to develop an increase in stress of (f_{ps} – f_{se}) are based on tests of members prestressed with clean 8 mm, 9 mm and 12 mm diameter strands for which the maximum value of f_{ps} was 1980 MPa [8.7, 8.8, 8.9].

The transfer length of strand is a function of the perimeter configuration area and surface condition of the strand, the stress in the strand and the method used to transfer the strand force to the concrete. Strand with a slightly rusted surface can have an appreciably shorter transfer length than clean strand. Gentle release of the strand will permit a shorter transfer length than abruptly cutting the strands.

The provisions of 8.6.9 do not apply to plain wires nor to end anchored tendons. The length for smooth wire could be expected to be considerably greater due to the absence of mechanical interlock. Flexural bond failure would occur with plain wire when first slip occurred.

C8.6.9.3 *Prestressing strand transfer length* A3

The design transfer length is based on the stress in the strand after prestress loss. However, it depends on the concrete strength at transfer. The expression for transfer length is valid provided f'_c equals or exceeds 21 MPa. An increase in concrete strength reduces the transfer length as does a slower rate of transfer. It should be noted that the transfer and development lengths given in 8.6.9.1 and 8.6.9.3 allow for the potential increase in development length with time[8.15].

C8.6.10 ***Standard hooks***

Clause 8.6.10 is based on the recommendations of ACI Committee 408 [8.3, 8.10]. Refer to Figure 8.1 for a diagrammatic explanation of L_{dh}.

One or more bars anchored by standard hooks with a development length according to Equation 8–12, and in close proximity to each other, should develop the strength of the bars provided the bars are included in a viable "strut and tie" mechanism. A viable mechanism consists of equilibrating internal actions where the bond stresses along the hook and the bearing stresses in the bend of the hook are balanced by (i) compression fields in the surrounding concrete and (ii) tension fields produced by reinforcement bounding and passing through the volume of concrete in which the bars are anchored.

Note that Reference 8.11 recommends that for the strut developed inside of the bend of the hook the angle of the strut to the straight shaft of the hook (length L_b, in Figure 8.1) should not be greater than 55°. If the angle is greater than 55° then the pull-out of a cone of concrete, before the yield strength of the bar is reached, is likely. This failure mode should be avoided. A typical situation where a concrete cone pull-out can occur at the connection of a floor to a wall, is where starter bars are anchored with a standard hook close to the adjacent face of the wall.

Typical situations where the "strut and tie" mechanism exists include: beam-column joints, column and beam stubs (for anchoring bars outside the beam-column joints) and longitudinal bars terminated by standard hooks at (i) the end of cantilevers elements (slabs, beams and foundation pads) and (ii) where curtailment of the longitudinal bars occurs within elements, where the traditional shear "truss" or "strut and tie" mechanisms exist.

Where a "strut and tie" mechanism does not exist, the failure mode of the bar under tension may be the pull-out of a cone of concrete, before the yield strength of the bar is reached. It is possible to prevent the pull-out of a concrete cone and the bar embedded in it, by tying back the cone into the "strut and tie" mechanism with appropriate tension reinforcement.

Meshes or grillages of reinforcement in the plane of the concrete element, such as a wall panel, are ineffective in preventing a cone type of failure [8.12]. One method for determining adequate embedment or development lengths terminated with standard hooks (not complying with 8.6.10.3) may be found in Reference 8.12.

A study of the failures of hooked bars indicates that splitting of the cover parallel to the plane of the hook is the primary cause of failure and that the splitting originates at the inside of the hook where stress concentrations are very high. For this reason, Equation 8–12 is a function of d_b which governs the magnitude of compressive stresses on the inside of the hook. Only standard ACI hooked bars were tested and the influence of a larger radius of bend was not evaluated. The test results indicate that as the straight lead length increases, the lateral splitting force which develops in the side cover decreases; this is reflected in an improvement in hook capacity.

The recommended provisions include adjustments to reflect the resistance to splitting provided by enclosure in transverse reinforcement. If the side cover is large so that side splitting is effectively eliminated, as in mass concrete, the product of the factors α_b, α_1 and α_2 as given in 8.6.10.3 may be used. Minimum values of L_{dh} are indicated to prevent failure by direct pullout in cases where the standard hook may be located very near the critical section. No distinction is made between top bars and other bars.

In many cases where the value of L_{dh} given by Equation 8–12 is used, the value of d_i required will be greater than that given in Table 8.1 as it will be governed by Equation 8–1. In such cases, if it is desired to reduce the value of d_i to that given in Table 8.1, the value of L_{dh} will have to be increased above that given to be used by Equation 8–12 in order to give an increased value for the lead length L_b as shown in Figure 8.1 which will allow a reduced value of d_i from Equation 8–1.

C8.6.10.1 *Standard hooks – definition*

The standard hooks defined in this section are shown in Figure 8.1.

C8.6.10.3 *Development length of standard hooks in tension*

The required development length L_{dh} for hooked bars in tension in accordance with 8.6.10, may be larger than what might be available in a column when the requirements of 9.4.3.2 shown in Figure C9.18 are to be satisfied. In such situations it is better to improve the bearing conditions in the bend than to provide extra straight anchorage length beyond the 90° bend. When transverse bars, as shown in Figure C8.1, are provided, a 20 % reduction in the development length L_{dh} of Figure 8.1 may be made. When beam bars are anchored within column bars in the core of a beam-column joint, the application of the multiplier α_1 = 0.7 in 8.6.10.3(b) is appropriate.

The bars placed in the bend help reduce the local bearing stresses and reduce the tendency for splitting cracks to form in the plane of the bend. The extension of these bars by $3d_b$ beyond the plane of the bar does not imply any limit on the spacing of adjacent bent bars.

When the same bar is required to develop yield strength in compression, the bent portion of the anchorage must be disregarded in satisfying the requirements of 8.6.5.1. However, when bars are anchored in column cores, as described above, the confinement may be considered to be equivalent to that implied in 8.6.5.3. The development of bars in compression will commence closer to the inner face of exterior columns.

C8.6.11 ***Mechanical anchorage***

A3 Mechanical end anchorages and bar couplers should be made adequate for strength both for prestressing tendons and for reinforcing bars.

Mechanical anchors and couplers need to be resistant to brittle fracture under their normal service temperature conditions. Anchors and couplers manufactured by processes involving heat treatment of the steel and bars to be coupled that have their ends enlarged for threading by cold forging will have their mechanical properties including their brittle fracture resistance altered by these processes. Therefore their manufacturing processes need to be subjected to appropriate testing of the finished products' mechanical properties and quality control procedures need to be employed to provide quality assurance of mechanical anchor and coupler systems.

Mechanical anchors and couplers manufactured from Grade 500/7 spheroidal graphite iron are not to be used. This material may contain casting defects and is likely to be prone to brittle failure under impact loading at normal service temperatures[8.14].

C8.6.12 *Development of flexural reinforcement*

C8.6.12.2 *Critical sections*

Critical sections for a typical continuous beam are indicated in Figure C8.7, together with the different criteria which determine where bars may be cut off.

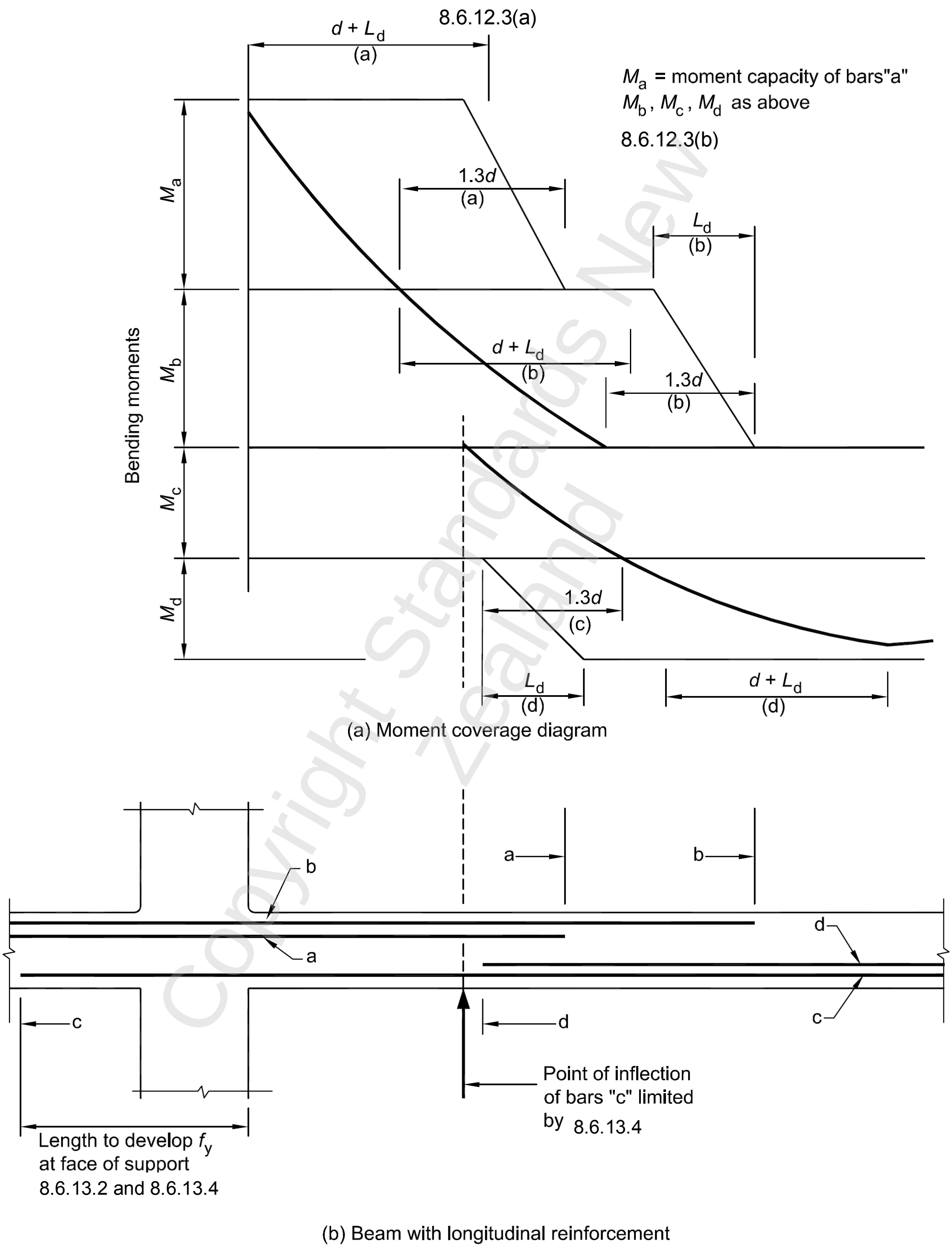


Figure C8.7 – Development of flexural reinforcement in a typical continuous beam

C8.6.12.3 *Extension of tension reinforcement*

The moment diagrams customarily used in design are approximate; some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, unaccounted lateral forces or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance *d* towards a point of zero moment. When stirrups are provided, this effect is less severe, although still present to some extent.

To provide for shifts in the moment demand, the Standard requires the extension of reinforcement by a distance 1.3*d* beyond the point at which it is theoretically no longer required to resist flexure, and by *d* beyond the length L_d. Cut-off points of bars to meet this requirement are illustrated in Figure C8.7.

When bars of different sizes are used, the extension should be in accordance with the diameter of bar being terminated. A bar bent to the opposite face of a beam and continued to the point where the bar crosses the mid-depth of the member may logically be considered effective in satisfying this clause.

The same principles apply to the curtailment of vertical reinforcement in walls, as implied by the design moment envelope for cantilever walls shown in Appendix D, Figure CD.7.

C8.6.12.4 *Termination in a tension zone*

Evidence of reduced shear strength and consequent loss of ductility when bars are cut off in a tension zone, as in Figure C8.7, has been reported by several investigators [8.13]. As a result, the Standard does not permit flexural reinforcement to be terminated in a tension zone unless special conditions are satisfied. Flexural cracks tend to open early wherever any reinforcement is terminated in a tension zone. If the steel stress in the continuing reinforcement and the shear strength are each near their limiting value, diagonal tension cracking tends to develop prematurely from these flexural cracks. Diagonal cracks are less likely to form where shear stress is low (see 8.6.12.4(a)). Diagonal cracks can be restrained by closely spaced stirrups (see 8.6.12.4(b)). Tension bars bent into the web at an angle not exceeding 45° and terminating at a distance of at least *d*/2 away from the tension face may be considered exempt from the requirements of this clause, because such bars do not terminate in a tension zone. These requirements are not intended to apply to tension splices which are covered by 8.7.2 and the related 8.6.3.

C8.6.12.5 *End anchorage in flexural members*

Members such as brackets, members of variable depth and others where steel stress f_s does not decrease linearly in proportion to a decreasing moment require special consideration for proper development of the flexural reinforcement. For the bracket shown in Figure C8.8 the stress in the reinforcement at nominal strength is almost constant at approximately f_y from the face of support to the load point. In such a case, development of the flexural reinforcement depends largely on the anchorage provided at the loaded end. A welded cross bar of equal diameter may be used as a means of providing effective end anchorage. An end hook in the vertical plane, with the minimum diameter bend, is not totally effective because an unreinforced concrete corner may exist near loads applied close to the corner.

A3

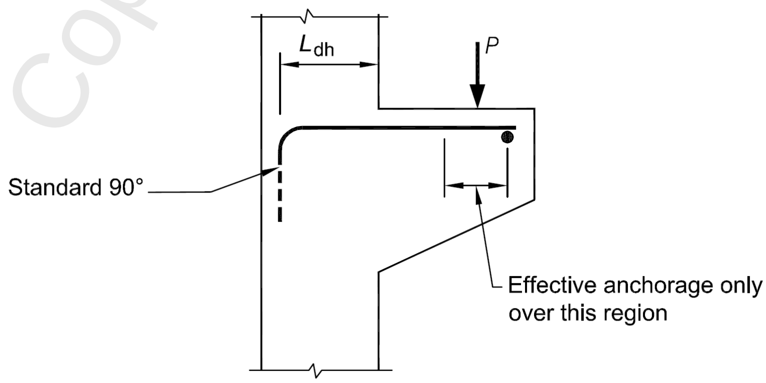


Figure C8.8 – Consideration of the critical anchorage for a special member

C8.6.12.6 *Anchorage of flexural reinforcement in external beam-column joints* A3

A major proportion of the joint zone shear force is resisted by a diagonal compression force which acts from the compression corner of the tension zone corner of the joint zone. Where beam flexural tension reinforcement is anchored by a 90° hook the diagonal compression force bears against the hook, inducing a vertical tension force in the vertical leg of the hook. To enable the joint zone to reach its full strength the tension force in the vertical leg must be transferred by bond to the adjacent vertical reinforcement in the column. To enable this force transfer to occur, reinforcement that encloses both the vertical leg of the hooked bar and the longitudinal column reinforcement is required.

C8.6.13 ***Development of positive moment reinforcement in tension***

C8.6.13.1 *Limitation in area of bars*

Certain proportions of the maximum positive moment reinforcement are required to be carried into the support to provide for some shifting of the moment due to changes in loading, settlement of supports, lateral forces and other causes.

C8.6.13.2 *Critical sections*

When a flexural member is part of a primary lateral force-resisting system, forces greater than those anticipated in design may cause reversal of moment at supports; therefore the required positive reinforcement should be well anchored into the support. This anchorage is to assure ductility of response in the event of unexpected overstress, such as from blast or earthquake. It is not sufficient to use more reinforcement at lower stresses. A3

C8.6.13.3 *Limitation in diameter of bars at simple supports*

Flexural bond considerations require the anchorage length to be checked in regions of members where the bending moment is zero, that is at simple supports and at points of contraflexure. In such regions the area of tension reinforcement provided may be small and the shear force relatively large, resulting in high flexural bond stresses. Clauses 8.6.13.3 and 8.6.13.4 ensure bond failure will not occur. In Figure C8.9, M_n is the nominal moment supplied by the reinforcement at the support. M_n/V^* can be increased by 30 % if the reaction confines the end of the reinforcement.

Figure C8.9 and Figure C8.10 illustrate the use of the provisions of 8.6.13.3 and 8.6.13.4.

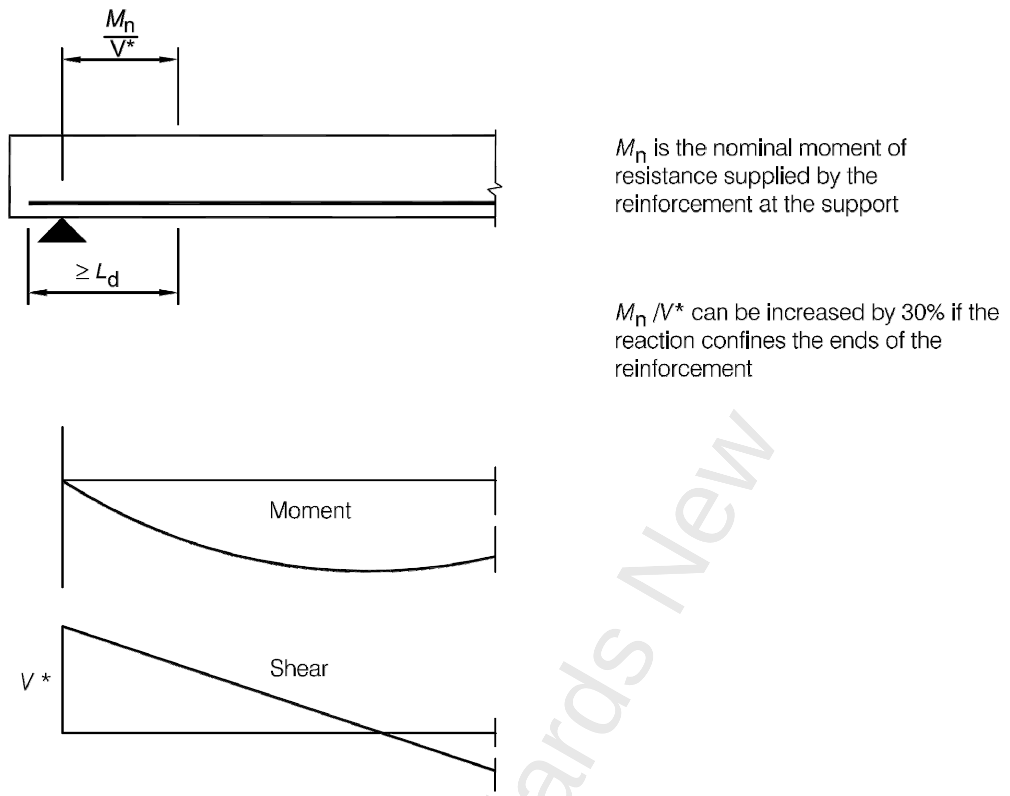


Figure C8.9 – Procedure for determining maximum size bar at simple support

Figure C8.10 illustrates the use of the provisions of 8.6.13.4.

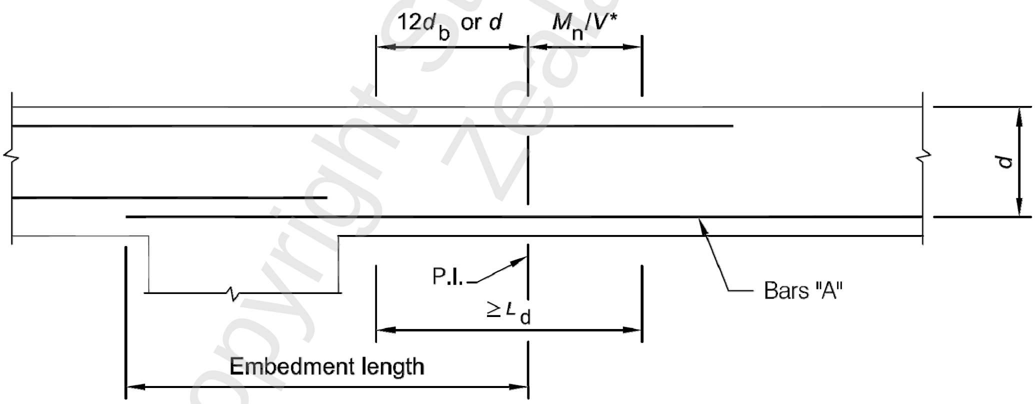


Figure C8.10 – Procedure for determining the maximum size of bars "A" at a point of inflection for positive reinforcing

In routine design it may often be found that M_n /V^* > L_d and hence no further check need then be made. When a requirement of 8.6.13.3 or 8.6.13.4 is not satisfied, the designer should either reduce the diameter of bars, whereby L_d is reduced, or increase the area of positive reinforcement at the section considered, whereby M_n is increased, or undertake both of these steps.

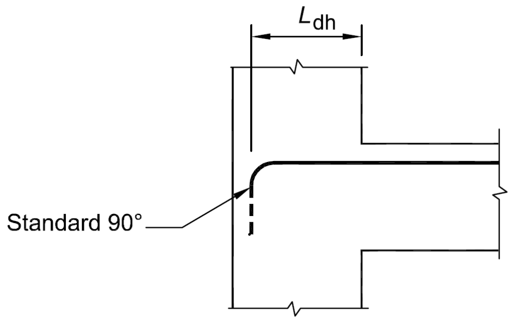


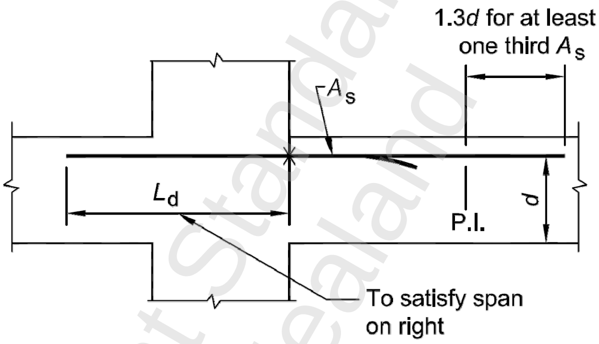
Figure C8.11 – Anchorage into exterior column

C8.6.14 *Development of negative moment reinforcement in tension*

Figure C8.11 and Figure C8.12 illustrate two methods of satisfying requirements for anchorage of tension reinforcement beyond the face of support. For anchorage of reinforcement with hooks, see commentary discussion C8.6.10.

Clause 8.6.14.3 provides for possible shifting of the moment diagram at a point of inflection, as discussed under C8.6.13.3. This requirement may exceed that of 8.6.13.3 and the more restrictive of the two provisions governs.

The principles involved in 8.6.14.4 are the same as those considered in C8.6.13.3.



NOTE – Usually anchorage in the column becomes part of the adjacent beam reinforcement.

Figure C8.12 – Anchorage into adjacent beam

C8.7 Splices in reinforcement

For ductility of a member, lap splices should be adequate to develop more than the yield strength of the reinforcement; otherwise a member may be subject to sudden splice failure when the yield strength of the reinforcement is reached. The lap splice lengths specified in the Standard satisfy this ductility requirement for members.

Splices should, if possible, be located away from points of maximum tensile stress.

The use of welded splices or mechanical connections with capacity less than the actual breaking strength is permitted if the design criteria of 8.7.5.4 are met. Therefore, lap welds of reinforcing bars, either with or without back-up material, welds to plate connections, and end-bearing splices are allowed under certain conditions.

C8.7.1 *General*

The designer's written approval should be obtained for any welding as what seems to be an unimportant weld to a site operative could affect a critical member.

C8.7.2 *Lap splices of bars and wire in tension*

C8.7.2.1 *Bar sizes of lap splices*

Research on lap splices with bars of diameter greater than 40 mm is limited. There is insufficient data to establish lap lengths for either tensile or compressive lap splices for these bars.

C8.7.2.2 *Lap splices of bundled bars*

The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars. Where the factors in this clause are applied it is not intended that the factors in 8.6.7 should also be applied.

C8.7.2.3 *Length of lap splices of deformed bars or wire*

This clause follows the recommendations of ACI Committee 408 [8.10, 8.3]. The recommendation that splice and development for deformed bars and wire are the same is also adopted by ACI 318. Statistical studies have shown that no additional factors are necessary for splices . Straight plain bars shall not be spliced except with hooks or mechanical anchorages.

In determining the required splice length, L_s, the distance c_p to be used is illustrated in Figure C8.13. Where all bars are spliced at the same location, c_p is the clear distance between bars. Where the splices are staggered and the overlap is less than L_s, c_p reflects this improvement. With staggered splices, the spacing between bars generally will not be as critical as is the cover to the centre of the bar.

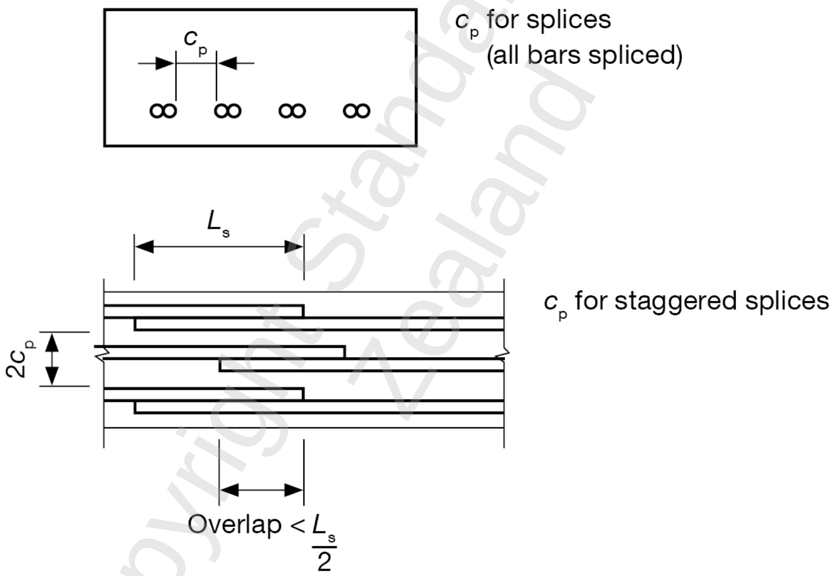


Figure C8.13 – Definition of c_p for splices

C8.7.2.4 *Length of lap splices of plain bars or wire*

This clause will apply when, under 5.3.1, there are specific or special necessities to use plain bars for other than ties, stirrups, spirals or hoops. The required standard hooks, as required by 8.6.10.1 and 8.4.2, such as shown in Figure 8.1, are not intended to pass around and engage other reinforcement. When the hooks are located near the surface of a member, the hooks should be embedded in the core concrete of the member. One application of lap splices in plain bars is detailed in 8.7.2.8.

C8.7.2.5 *Length of non-contact lap splices*

To ensure that the effective splice length, L_s, is maintained in splices with transverse spacings, s_L, of bars larger than $3d_b$, L_{ds} is introduced which assumes an approximately 33° diagonal compression field as illustrated in Figure C8.14.

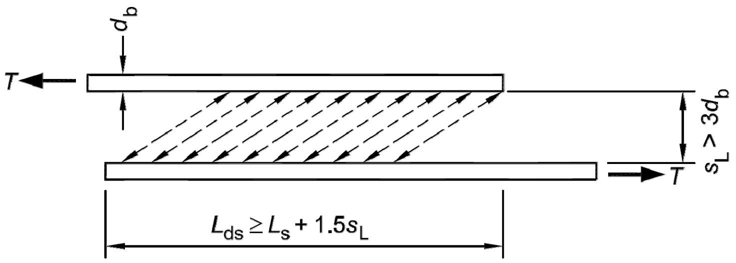


Figure C8.14 – The spacing of spliced bars

C8.7.2.8 *Lap splices of stirrups, ties and hoops*

Deformed bars and plain bars used for stirrups, ties or rectangular hoops may be spliced when necessary, provided standard hooks as required by 8.6.10.1 and 8.4.2 such as shown in Figure 8.1, are used. The hooks must be embedded in the concrete core, that is, the plane of the hook must not be in the cover concrete. Hooks anchored through and inside the core may be oriented to suit convenience in construction. Hooks required for lap splicing of transverse reinforcement need not engage a longitudinal bar for anchorage.

Bond forces generate hoop tensile stresses in the concrete surrounding the reinforcement. With closely spaced bars the hoop tensile stresses generate tensile stresses which under certain conditions may lead to spalling of the concrete. Such spalling may destroy the development of the lapped bar and the effectiveness of the stirrup or tie leg. A3

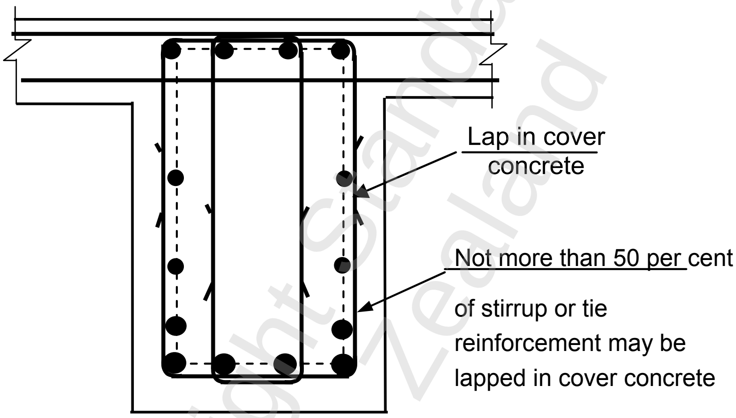


Figure C8.15 – Laps in stirrups and ties

Locating 50 % or more of the lapped stirrups in the core concrete reduces the tendency for bond stresses in the stirrup lap zone to result in spalling of the cover concrete.

C8.7.3 ***Lap splices of bars and wires in compression***

Recent bond research has been primarily related to bars in tension. Bond behaviour of compression bars is not complicated by the problem of transverse tension cracking and thus compression splices do not require provisions as strict as those specified for tension splices.

C8.7.3.2 *Lap splices in compression with stirrups and ties*

Effective tie legs included in the evaluation of A_{tr} are those which cross a potential splitting crack which develops in the plane at which two spliced bars might be in contact with each other. An example is shown in Figure C8.17. A3

C8.7.3.3 *Lap splices in compression with spiral reinforcement*

Compression lap lengths may be reduced when the lap splice is enclosed throughout its length by spirals because there is increased splitting resistance. Spirals should meet requirements of 10.3.10.7, 10.3.10.8 and 10.3.10.5. Because spirals do not cross a potential splitting crack when spliced bars in contact are aligned radially, they are less efficient in confining a splice. Therefore the area of the spiral is required to be N/6 times longer than that of a tie crossing a crack at 90° assumed in 8.7.3.2. Potential radial splitting cracks, developing when all spliced bars touch a circular spiral may eventually link up with circumferential cracks because the diameter of the strained spiral increases. The two mechanisms are illustrated in Figure C8.17.

A3

C8.7.4 *Welded splices and mechanical connections*

Designers should avoid the need to weld reinforcing steel if possible as follows:

(a) Where butt jointing is required there is a good range of coupling devices available. Lapping, particularly of smaller bars, may also be an option;

(b) Tack welding of stirrups or ties to main bars may result in a reduction in capacity of the main bar, either through metallurgical changes, or the generation of notches due to undercut if the procedures of AS/NZS 1554:Part 3 are not followed;

(c) Where welds are required to provide lightning protection, care should be taken to choose a route through non-critical members.

Welds complying with 8.7.4.1(a) can withstand the most severe strain or stress cycles. Hence they are acceptable in all locations, in particular, for splicing main longitudinal reinforcement in plastic hinge regions and in beam-column joints. Weld quality should comply with the requirements of AS/NZS 1554: Part 3, Section 9 for “Direct Butt Splices”.

The categories of splices in 8.7.4.1(b) will be adequate for large bars in main members outside plastic hinge regions and for welded splices in stirrups, ties, spirals or hoops. The limit of the breaking strength of the bar will ensure that the strength of the connection will be greater than the maximum design force in the bar. Weld quality should comply with the requirements of AS/NZS 1554:Part 3, Section 9 for “Other splices”.

C8.7.4.2 *Limitations on the classification of welded splices for grade > 450 MPa reinforcement*

The current Standard for welding of reinforcement, AS/NZS 1554.3, allows the use of welding consumables with a minimum strength of 550 MPa (E5518) or 620 MPa (E6218). It is considered unlikely that the use of the lower strength electrode will provide an appropriately high probability that the full strength of a Grade 500 bar will be achieved. Whether the full strength of the bar in the upper characteristic range for Grade 500 reinforcing can be achieved with the higher strength electrode requires verification.

Yielding of the weld is undesirable as the plastic deformation is limited to a short length. This can greatly reduce the ductility of the bar and lead to a brittle type of failure of a member. For this reason welded Grade 500 reinforcement should be approached with caution where plastic deformation may be required.

C8.7.5.2 *Performance requirements for classification as a “high-strength” mechanical connection*

A stiffness criterion is imposed on mechanical splices of C8.7.5.2(b) to ensure that large premature cracks are not produced by excessive extensions in splicing devices. Accordingly the displacement of the spliced bars relative to each other and measured in a test over the length of the connector, should not exceed twice the elongation of the same size of unspliced bar over the same measured distance when subjected to 0.7 f_y.

C8.7.5.3 *Use of welded splices and mechanical connections*

See commentary on 8.7.4.1(c). This clause describes the situation where welded splices or mechanical connections with capacity less than the actual breaking strength of the bar may be used. It provides a relaxation in the splice requirements where the splices or connections are staggered and an excess reinforcement area is available. The criterion of twice the computed tensile stress is used to cover sections containing partial tensile splices with various percentages of the total reinforcement continuous.

C8.7.6 *Splices of welded plain or deformed wire fabric*

The strength of lap splices is dependent on either the anchorage obtained from the cross-wires, as shown in Figure C8.16 and as detailed in 8.6.8.2, or the development of the individual wires as detailed in 8.6.8.3.

A3

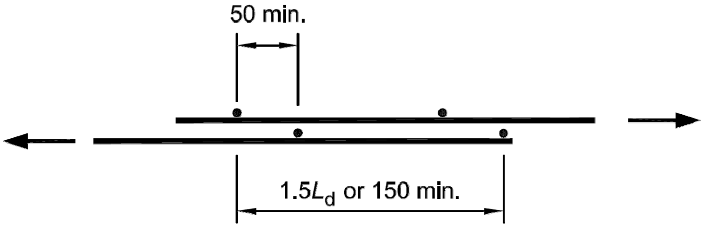


Figure C8.16 – Lap splice of welded fabric A3

C8.8 Shrinkage and temperature reinforcement

So-called shrinkage and temperature reinforcement is required at right angles to the principal reinforcement to prevent excessive cracking and to tie the structure together to assure behaviour as assumed in the design. The amount specified ($0.7/f_y$) is empirical but follows closely values which have been used satisfactorily for many years.

A3

It should be kept in mind that the reinforcement ratios given in this clause are minimum values and apply to the situation where restraint against shrinkage has been minimised. It is well known that if the slabs are fully restrained against shrinkage and temperature movement, much higher reinforcement ratios are required to avoid severe cracking. In most cases it is possible to select structural form, construction joint positions and pouring sequences to minimize restraint in suspended slabs, and this Standard has followed the practice of most leading national codes in giving reinforcement ratios appropriate to this situation.

For the condition of full restraint, first principles require that the yield strength of the reinforcement passing through any potential crack position should be greater than the ultimate tensile strength of the corresponding cross-sectional area of concrete during the period after initial setting. This would require, for example, a reinforcement percentage of the order of 0.45 % for the case of a specified 28 day concrete compressive strength f'_c of 25 MPa and characteristic yield strength of reinforcement f_y of 300 MPa.

Splices and end anchorages must be designed for the full specified yield strength.

C8.9 Additional design requirements for structures designed for earthquake effects

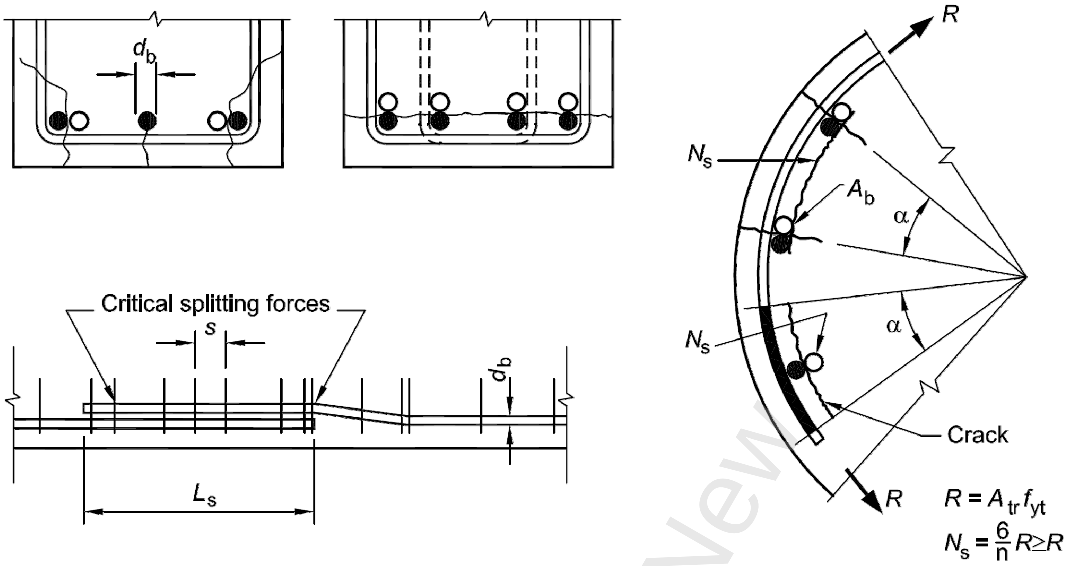
C8.9.1 *Splices in reinforcement*

C8.9.1.1 *Placement of splices*

Splices other than those described by 8.7.4.1(a) should not be used in potential plastic hinge regions or in beam-column joints where anchorage conditions may be very critical. Therefore splices should be located away from critical sections of potential plastic hinges by the distances specified. In a column, if plastic hinges form, their location will be at the top and bottom ends of storey heights, adjacent to beams or footings. When plastic hinge development is not expected because columns, designed in accordance with Appendix D, Method A, have considerable reserve flexural strength, splices may be located also in the top and bottom ends of storey heights, the preferred position usually being immediately above a floor.

C8.9.1.2 *Lap splices in regions of reversing stresses*

Transverse reinforcement provided around splices with Grade 430 bars in accordance with Equation 8–18, was found to ensure that at least 85 % of the nominal strength of a column section with all bars spliced could be sustained in at least 20 cycles of reversed loading without distress. Such splices were found to sustain even a few limited excursions beyond yield. In determining the splice length from 8.6.3, the beneficial effect of this transverse reinforcement may be utilised for A_{tr}. Transverse ties must cross potential sliding failure planes between two spliced bars as shown in Figure C8.17. A3



A3 **Figure C8.17 – Bar force transmission by shear-friction at lapped splices**

At locations along beams and the mid-height of columns, where it can be shown that reversing stresses do not exceed $0.6f_y$, in tension or compression, transverse reinforcement provided to satisfy other requirements, such as 8.7.3.2 and 8.7.3.3, may be considered to ensure satisfactory splice performance.

C8.9.1.3 *Requirements for welded splices or mechanical connections*

In members that are subjected to seismic forces, welded splices or mechanical connections are required to develop the breaking strength of the bars. This is due to the consideration of the severe consequences for the structure if failures of such classes of connections do occur. The requirement is analogous to lap splices of bars being required to develop more than the yield strength of the bars and not being reduced in length because more reinforcement is provided than that required at the lap location (8.6.3.3 and 9.4.3.2.3).

In determining the criteria for welded splices and mechanical connections, standard of workmanship, difficulty of inspection, and the final reliability of the splice in service has been taken into account. Even so, designers should be aware of the necessity for a site testing programme to ensure that these splices meet
A3 the requirements of 8.7.5.2. and 8.9.1.3.

Splices conforming to 8.7.4.1(a) and 8.7.4.1(b) may be located in the same plane or section.

REFERENCES

8.1 ACI Committee 408, “Bond Stress – The State of the Art”, ACI Journal, Proceedings Vol. 63, No. 11, Nov. 1966, pp. 1161-1188. Also “ACI Manual of Concrete Practice”, American Concrete Institute, Detroit, 1979, Part 2.

8.2 “Standard Specifications for Highway Bridges”. American Association of State Highway and Transportation Officials, Washington, D.C., 11th edition, 1974, 284 pp. Interim Specifications, 1973.

8.3 Jirsa, J.O., Lutz, L.A. and Gergely, P., ”Rationale for Suggested Development, Splice and Standard Hook Provisions for Deformed Bars in Tension”, Concrete International, Vol. I, No.7, July 1979, pp. 47-61.

8.4 Paulay, T. and Priestley, M.J.N., “Seismic Design of Reinforced Concrete and Masonry Buildings”, John Wiley, New York, 1992.

8.5 Untrauer, Raymond E. and Warren, George E., "Stress Development of Tension Steel in Beams", ACI Journal, Proceedings Vol. 74, No. 8, Aug. 1977, pp. 368 - 372.

8.6 Anderson, A.R., “Bond Properties of Welded Wire Fabric”, ACI Journal, Proceedings Vol. 50, No. 4, April 1952, pp. 681-692.

8.7 Kaar, P. and Magura, D., “Effect of Strand Blanketing on Performance of Pretensioned Girders”, Journal, Prestressed Concrete Institute, Vol. 10, No. 6, Dec. 1965, 15 pp.

8.8 Hanson, N.W. and Kaar, P.H., “Flexural Bond Tests Pretensioned Beams”, ACI Journal, Proceedings Vol. 55, No. 7, Jan. 1959, pp. 783-802. Also, Development Department Bulletin D28, Portland Cement Association, 1959, 20 pp.

8.9 Kaar, P.H., La Fraugh, R.W. and Mass, M.A., “Influence of Concrete Strength on Strand Transfer Length” Journal, Prestressed Concrete Institute, Vol. 8, No. 5, Oct. 1963, pp. 47-67. Also, Development Department Bulletin D71, Portland Cement Association, Oct. 1963, 21 pp.

8.10 ACI Committee 408, “Suggested Development, Splice and Standard Hook Provisions for Deformed Bars in Tension”, Concrete International, Vol. I, No. 7, July 1979, pp. 45-46.

8.11 Comite European du Beton, Commission “Strength of Reinforced and Prestressed Concrete Beams CEB Approach”, ACI Symposium 1976, Philadelphia, U.S.A.

8.12 Restrepo, J.I., Crisafulli, F.J. and Park, R., "Earthquake Resistance of Structures: The Design and Construction of Tilt-up Reinforced Concrete Buildings", Research Report 96-11, Department of Civil Engineering, University of Canterbury, New Zealand, 1996.

8.13 Ferguson, Philip M. and Matloob, Farid N., “Effect of Bar Cut-off on Bond and Shear Strength of Reinforced Concrete Beams”, ACI Journal, Proceedings Vol. 56, No. I, July 1959, pp. 5-24.

8.14 Kirkcaldie, D.K. and Lloyd, J.N., “Some Concrete Topics Being Addressed in New Zealand Transport Agency Bridge Manual Revisions – Shrinkage and Creep in Concrete: Mechanical Coupling of Reinforcing Steel”, NZ Concrete Industry Conference, Queenstown, October 2013. A3

8.15 Dang, C.N., Royce, W.F., Hale, W.M. and Mariti-Vargas, J.R., “Measured Transfer Length in 17.8mm Strand for Pretensioned Beams”, ACI Structural Journal, Vol. 113, No.1, January 2016.

NOTES

C9 DESIGN OF REINFORCED CONCRETE BEAMS AND ONE-WAY SLABS FOR STRENGTH, SERVICEABILITY AND DUCTILITY

C9.1 Notation

The following symbols, which appear in this section of the commentary, are additional to those used in Section 9 of the Standard:

b_c width of column, mm
b_e effective width of tensile flange of beam for strength calculations, mm
$b_{e,o}$ width of flange used for calculation of overstrength, mm
b_f width of slab on one side of web contributing to b_e or $b_{e,o}$, mm
c depth of neutral axis at flexural strength, mm
j ratio of lever arm of concrete compression force and steel tension force to beam depth
L_{AB} distance as specified in Figure C9.16
M'_{oA} positive flexural overstrength at A in Figure C9.16, N mm
M_{oA}, M_{oB} negative flexural overstrength at the column faces at A and B in Figure C9.16, N mm
M_{slab} dead and long-term live load bending moment in slab, N mm/m
T_p tension force contributing to overstrength due to precast member in slab, N
V_{GA}, V_{GB} applied total shear force at A and B in Figure C9.16 due to dead load, N
V^*_A V^*_B design shear force at A and B in Figure C9.16 to resist earthquake and gravity effect, N
V_{QuA}, V_{QuB} applied total shear force at A and B in Figure C9.16 due to live load, N
ε_c strain in extreme compression fibre of concrete at flexural strength
ϕ_u curvature at ultimate, 1/mm
ϕ_y curvature at first-yield of tension reinforcement, 1/mm

C9.3 General principles and design requirements for beams and one-way slabs

C9.3.1 General

C9.3.1.1 *Moments at supports for beams integral with supports*

Beam moments obtained at the centrelines of columns may be reduced to the moments at the face of supports for design of beam members. The assumption to neglect the width of the beam in analysis of slabs should be considered carefully with unusually wide beams. In some circumstances torsional effects could make this assumption unsound. The span lengths to be used in the design of two-way slab systems are specified in Section 12.

For short members such as half hinges, the assumption of the critical section being at the face of the member, can lead to the design moment being significantly below the correct value. In such cases the design moment should be determined from equilibrium considering the actual location of the forces in the supporting member.

C9.3.1.2 and **C9.3.1.3** *Effective width resisting compression of T-beams and L-beams* and *Effective moment of inertia in T-beams and L-beams* A3

The provisions for T- beam construction have been adapted from the provisions in previous editions of NZS 3101. Special provisions related to T-beams and other flanged members are stated in 7.6.1.8 with regard to torsion. Provisions for flanges in tension under seismic conditions are given in 9.4.1.6. These requirements have been changed to make them consistent with the requirements for the contribution of slab reinforcement to the flexural tensile strength. A3

For monolithic T- and L- beams, one-half only of the effective over-hanging parts of flanges, used for the evaluation of flexural strength in accordance with 9.3.1.2, should be included in the evaluation of the moment of inertia of the section. With this allowance flanged members with uniform depth, such as beams cast together with floor slabs, may be assumed to be prismatic. This assumption is intended to compensate for the fact that the effective widths of flanges will vary along the span and that tension may prevail in the flange area over a considerable length.

C9.3.1.4 *Contribution of slab reinforcement to design strength of T and L-beams*

The strain imposed on reinforcement in a slab, which is located near a beam, depends on the rotation sustained by the beam, the beam's depth and the distance of the reinforcement from the beam web. As the plastic rotation increases so the width over which reinforcement can contribute to the flexural strength increases. It follows that as curvatures in plastic regions increase so the width of slab over which reinforcement can contribute to strength increases. Hence different criteria are given for determining design strength, which involves limited plastic rotation, to overstrength, which involves plastic regions sustaining high rotations (see 9.4.1.6.1 and 9.4.1.6.2).

Figure C9.1 illustrates some of the criteria listed in 9.3.1.4 for defining the slab reinforcement, which may be assumed to act with the beam to increase the design flexural strength.

A3 Two criteria are given in (a) and (b) for assessing the contribution of longitudinal reinforcement in the outstanding portions of flanges to the flexural tensile strength of beams. Where relatively little reliance is placed on the tensile strength of the reinforcement in the outstanding portion of the flange or flanges the criterion given in (a) may be used. However, where the tensile strength contribution of the reinforcement in one of the outstanding portions of a flange exceeds 10 % of the total flexural tensile strength of the reinforcement it is necessary to ensure that the flange reinforcement is adequately tied into the web so that this contribution can be relied upon. In this situation the criteria given in (b) needs to be satisfied.

To satisfy the criteria in (b) sufficient transverse reinforcement is required to tie the outstanding flange into the beam web to resist the shear flow between the web and longitudinal reinforcement in the flange. To meet this condition this transverse reinforcement (normal to the axis of the beam) is required either to be anchored into the core of the web (that is, the concrete enclosed by the stirrups) or either extend between layers of the flexural tension reinforcement or be located on the web side of the flexural tension reinforcement, see Figure C9.3. Transverse reinforcement that is either not enclosed by the longitudinal beam reinforcement or anchored into the core of the web is likely to be ineffective, as a shear failure between the slab and the web, as illustrated in Figure C9.2, may develop.

The portion of the shear flow in the web that extends into the outstanding portion of the flange, q_f, is given by: $q_{\mathrm{f}} = \dfrac{V^{*} A_{\mathrm{f}\ell} f_{\mathrm{y}\ell}}{\phi d A_{\mathrm{s}} f_{\mathrm{y}}}$,

where $A_{f\ell}$ is the area of longitudinal reinforcement in the outstanding portion of the flange with a yield stress of $f_{y\ell}$. Assuming diagonal struts develop at an angle of $\tan^{-1}0.5$ to the axis of the beam in the slab the area of a bar of transverse reinforcement required is given by:

$$A_{\mathrm{ft}} = \frac{q_{\mathrm{f}} s_{\mathrm{ft}}}{2 f_{\mathrm{yt}}}$$

where A_{ft} is the area of transverse reinforcement in the flange with a yield stress of f_{yt} and a spacing of s_{ft}.

In considering the development of the longitudinal reinforcement in the flange and in the location where transverse reinforcement is placed allowance should be made for the tension lag associated with diagonal cracking in both the web and the outstanding portion of the flange. For bars in the outstanding portion of the flange a reasonable allowance for tension lag associated with diagonal cracking could be made by extending the reinforcement beyond the point where the reinforcement is no longer theoretically required by standard theory (plane sections remain plane) by a distance of d + the distance from the face of the web to the reinforcing bar in the flange.

There is a major additional advantage in tying slabs into beam webs. This arrangement gives a more robust structure. With this detail tensile membrane action can develop in the slab in the event that the support to a beam is lost due to the failure of a column, or other structural element. The tensile membrane action can transfer the load to adjacent columns or walls, thus preventing a local failure spreading over a large area.

In major T- beams, the distribution of the negative moment tension reinforcement for control of crack widths at service load should take into account two considerations:

(a) Wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web, and
(b) Close spacing near the web leaves the outer regions of the flange unprotected.

To avoid possible formation wide cracks in the flanges of T-beam construction some reinforcement should be spread over regions which may be subjected to tension due to bending of the beam.

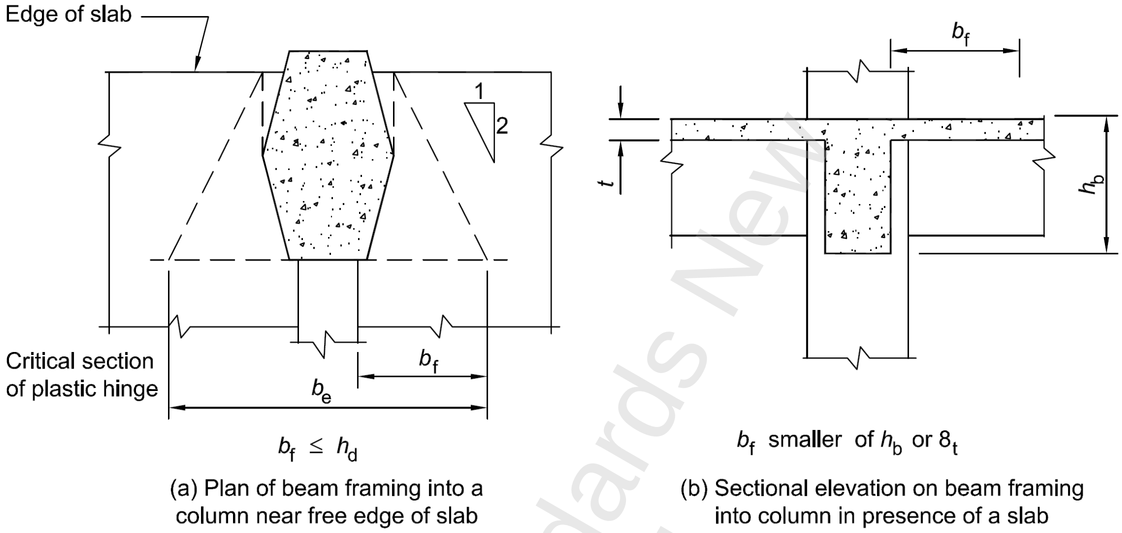


Figure C9.1 – Effective flange width of beams used for calculating nominal negative moment flexural strength concrete floor systems [9.1]

C9.3.1.5 *Floor finishes*

This Standard does not specify an additional thickness for wearing surfaces subjected to unusual conditions of wear. Whether or not the separate finish is structural, the need for added thickness for unusual wear is left to the discretion of the designer.

It is permissible to include a separate finish for strength purposes in the structural thickness if composite action is ensured in accordance with Section 18 and diaphragm action, where required, is ensured in accordance with Section 13.

All floor finishes may be considered for non-structural purposes such as cover and fireproofing. Provisions should be made, however, to ensure that the finish will not spall off, thus causing decreased cover.

C9.3.1.6 *Deep beams*

This Standard does not contain detailed requirements for the design of deep beams for flexure but states that non-linearity of strain distribution and lateral buckling must be considered.

Suggestions for the design of deep beams for flexure are given in References 9.2, 9.3, 9.4. The strut and tie approach is a particularly useful method for designing deep beams.

C9.3.5 ***Distance between lateral supports of beams***

Tests have shown that laterally unbraced reinforced concrete beams of any reasonable dimensions, even when very deep and narrow, will not fail prematurely by lateral buckling provided the beams are loaded without lateral eccentricity that could cause torsion [9.4].

Laterally unbraced beams are frequently loaded off-centre (lateral eccentricity) or with slight inclination. Stresses and deformations set up by such loading become detrimental for narrow, deep beams, more so as the unsupported length increases. Lateral supports spaced closer than 50*b* may be required by actual loading conditions.

C9.3.6 *Control of flexural cracking*

C9.3.6.1 *General*

Unsightly cracking and cracking likely to lead to corrosion of reinforcement should be avoided. Wide cracks can also reduce the shear strength of a member.

C9.3.6.2 *Beams and one-way slabs*

Flexural cracking is particularly important when reinforcement with a yield strength greater than 400 MPa is used. Extensive laboratory work [9.5, 9.6, 9.7, 9.8 and 9.9] has shown that crack width at service loads is proportional to steel stress. Significant variables affecting the detailing were found to be the thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar. Better crack control is obtained when the reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

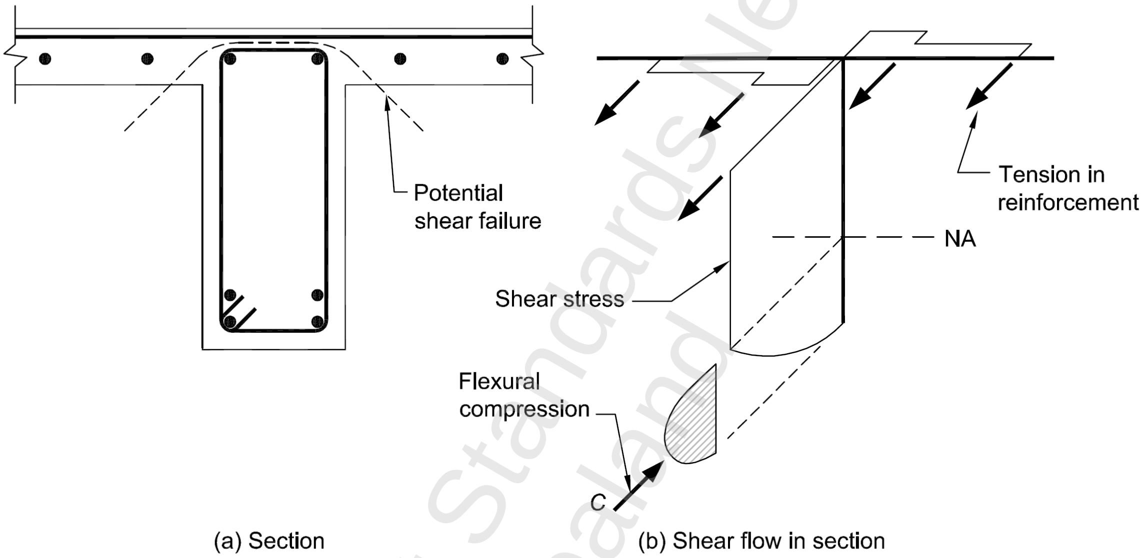


Figure C9.2 – Potential shear failure surface and shear flows

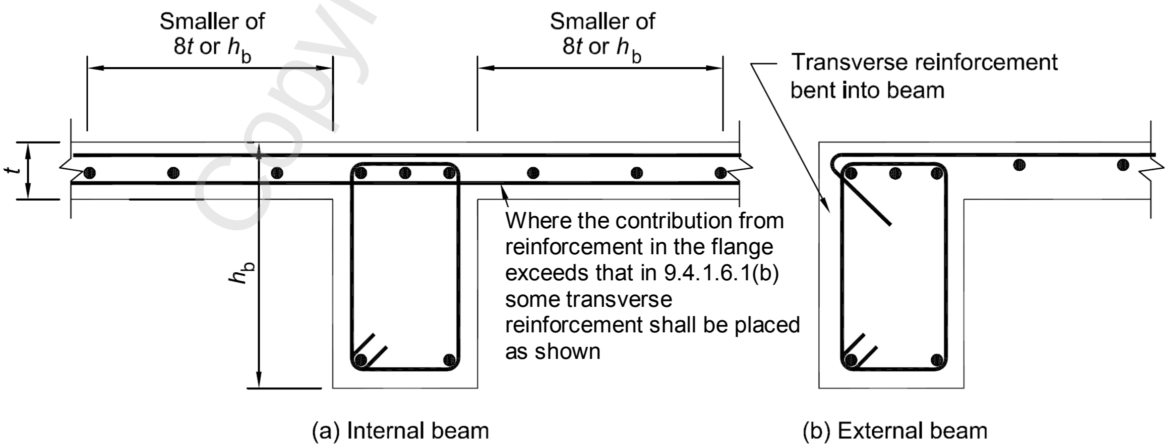


Figure C9.3 – Effective reinforcement providing slab shear connection to beam

C9.3.6.3 *Skin reinforcement*

For relatively deep flexural members some reinforcement should be placed near the vertical faces of the tension zone to control cracking of the web [9.10]. Without this reinforcement wide cracks can form its webs and these can lead to a significant reduction in the shear strength.

C9.3.7 *Control of deflections*

C9.3.7.1 *Minimum thickness*

Deflections can be controlled by either the minimum thickness requirements or by calculating deflections and ensuring that they do not exceed stipulated allowable values.

C9.3.8 Longitudinal reinforcement in beams and one-way slabs

C9.3.8.1 *Maximum longitudinal reinforcement in beams and one-way slabs*

The maximum value of the neutral axis depth, c, of beams and one-way slabs at the ultimate limit state is limited to 0.75 of the neutral axis depth at balanced strain conditions, c_b (see 7.4.2.8), in order to ensure a level of ductile behaviour. The ductility of a member is dependent on the value of ε_c /c achieved at the critical section, where ε_c is the ultimate concrete compressive strain, and hence a relatively small value for c indicates ductile behaviour. At the ultimate limit state it is important that $c < c_b$ so that the strain in the main tension reinforcement will have exceeded the yield strain of the steel when the concrete strain reaches its ultimate compressive value. This will ensure that the member will not fail until a relatively large deflection is reached and with wide cracks in the tension zone giving ample warning of impending failure (a ductile failure condition). If at the ultimate limit state $c > c_b$ the member will fail with consequent small deflection and little warning of impending failure since the tension reinforcement will not have yielded and the crack widths will be small (a brittle failure condition).

The magnitude of the neutral axis depth, c, depends on the shape of the cross section of the member, the areas and location of the reinforcement and the material strengths f'_c and f_y. The requirement of 9.3.8.1, that c should not exceed $0.75c_b$, will govern the maximum permitted amount of tension reinforcement in a member. Members with compression reinforcement can contain greater amounts of tension reinforcement since only that portion of the total tension steel balanced by compression in the concrete will need to be limited.

It is considered that the requirement $c \leq 0.75\ c_b$ will provide sufficiently ductile behaviour of members without axial compression for most designs. One condition where greater ductile behaviour may be required is in design for redistribution of moments in continuous beams, two-way slabs and frames. Moment redistribution is dependent on adequate ductility being available at plastic hinge regions and in 6.3.7.2(f) the maximum amount of tension reinforcement is controlled by relating the neutral axis depth to the amount of moment redistribution permitted.

Another reason for limiting c to less than $0.75\ c_b$ is it ensures that small variations in the actual concrete strength have little influence on the flexural strength.

C9.3.8.2 *Minimum longitudinal reinforcement in beams and one-way slabs*

C9.3.8.2.1 *and* **C9.3.8.2.2** *Minimum reinforcement in beams* A2

The provisions for a minimum amount of tension reinforcement applies to beams, which for architectural or other reasons, are much larger in cross section than required by strength considerations. With a very small amount of tension reinforcement, the computed moment strength as a reinforced concrete section becomes less than that of the corresponding unreinforced concrete section computed from its modulus of rupture. Failure in such a case can be sudden.

To prevent such a failure, a minimum of tension reinforcement is required and 9.3.8.2.1 takes into account the possible use of high strength concrete and the different requirement for a T-beam with the flange in tension. This requirement is to ensure that the flexural strength of the section (after cracking) is at least equal to the moment when cracking first occurs computed using the modulus of rupture of the concrete. The requirement applies to both the positive and negative moment regions of a beam. Table C9.1 contains values for $p_{min} = A_s/b_wd$ given by 9.3.8.2.1 for rectangular beams for the usual range of concrete and steel strengths.

Table C9.1 – Values of p_{min} given by 9.3.8.2.1 for rectangular beams

f'_c (MPa)	f_y = 300 MPa	f_y = 500 MPa
25	0.0047	0.0028
30	0.0047	0.0028
40	0.0053	0.0032
50	0.0059	0.0035

A2 **C9.3.8.2.3** *Reduced minimum reinforcement*

The minimum reinforcement required by 9.3.8.2.1 and 9.3.8.2.2 must be provided except where both positive and negative reinforcement are one-third greater than required for flexural strength by analysis. This exception provides sufficient additional reinforcement in large members where an area given by
A2 9.3.8.2.1 and 9.3.8.2.2 would be excessive. This reduction is not permitted for beams where their flexural strength contributes to the lateral strength of the structure. The reinforcement in this situation is required to ensure that beams have adequate ductility to sustain actions not normally considered in analysis, such as vertical seismic actions.

C9.3.8.2.4 *Minimum reinforcement in slabs and footings*

The minimum reinforcement required for slabs is somewhat less than that required for beams, since an overload would be distributed laterally and a sudden failure would be less likely. The structural reinforcement should, however, be at least equal to the shrinkage and temperature reinforcement, as required by 8.8.1.

Soil supported slabs, such as slabs on grade, are not considered to be structural slabs in the context of this clause, unless they transmit vertical loads from other parts of the structure to the soil. Reinforcement, if any, in soil-supported slabs should be proportioned with due consideration of all design forces. Raft foundations and other slabs which help support the structure vertically should meet the requirements of this clause.

C9.3.8.4 *Maximum diameter of longitudinal beam bar in internal beam-column joint zones*

(a) Where the critical load combination for flexure in a beam at the face of a column includes earthquake actions the limiting bar size is limited by Equation 9–2 to prevent premature slipping of the bar. This equation is a modified form of that in 9.4.3.5.2, but the coefficient has been modified to recognise the change in strength reduction factor, structural performance factor, overstrength factors and the reduced number of peak stress cycles that occur in a nominally ductile structure compared with a ductile structure.

(b) Where earthquake load combinations are not critical the bar diameter may be increased in some cases above that given in Equation 9–2. Equation 9–3 is based on the average bond stress being limited to a maximum value of 1.5 $\alpha_f \sqrt{f'_c}$.

A2 **C9.3.8.5** *Anchorage of beam bars*

The hook on a beam bar anchored in a beam-column joint must be given the maximum possible development length. In all cases the development length measured from the face of the column where the bar enters the joint zone, must be greater than the development length defined in 8.6.10.

C9.3.9 ***Transverse reinforcement in beams and one-way slabs***

C9.3.9.1 *General*

Transverse reinforcement is required in beams to prevent inelastic buckling of compressed longitudinal bars in beams and one-way slabs and to resist shear and torsion.

C9.3.9.2 *Diameter and yield strength of transverse reinforcement*

Limiting the design yield strength of shear and torsion reinforcement to 500 MPa provides a control on diagonal crack width.

C9.3.9.3 *Design for shear*

C9.3.9.3.1 *Design shear force adjacent to supports*

The closest inclined crack to the support of the beam in Figure C9.4 will extend upwards from the face of the support reaching the compression zone about *d* from the face of the support. If loads are applied to the top of this beam, the stirrups across this crack are stressed by loads acting on the lower freebody in Figure C9.4. The loads applied to the beam between the face of the column and the point *d* away from the face are transferred directly to the support by compression in the web above the crack. Accordingly, the Standard permits design for a maximum shear force V^* at a distance *d* from the support for non-prestressed members. Two things are emphasised: first, stirrups are required across the potential crack designed for the shear at *d* from the support, and second, a tension force exists in the longitudinal reinforcement at the face of the support.

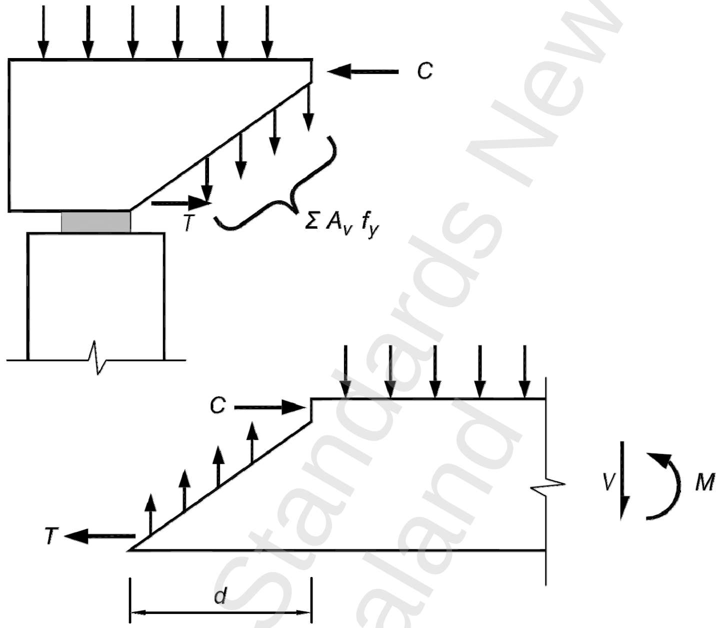


Figure C9.4 – Free body diagrams of each end of a beam

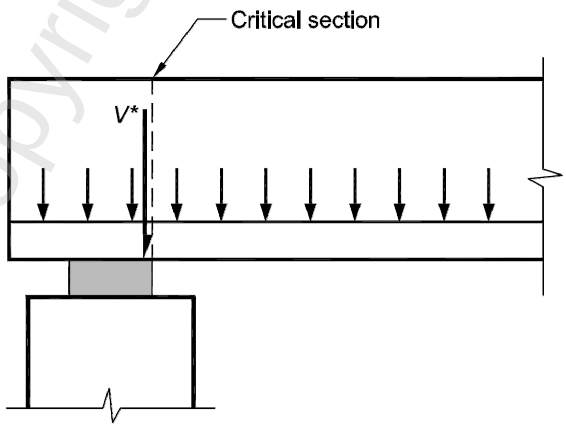


Figure C9.5 – Location of critical section for shear in a member loaded near bottom

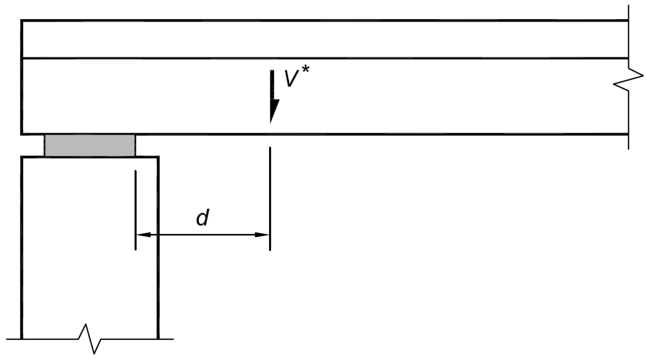


Figure C9.6 – Typical support conditions for locating factored shear force V^* (a)

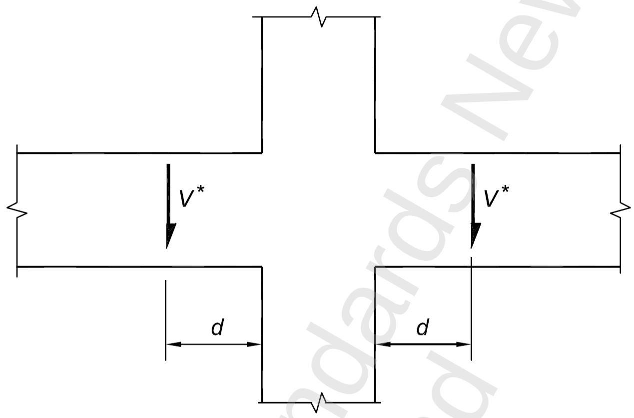


Figure C9.7 – Typical support conditions for locating factored shear force V^* (b)

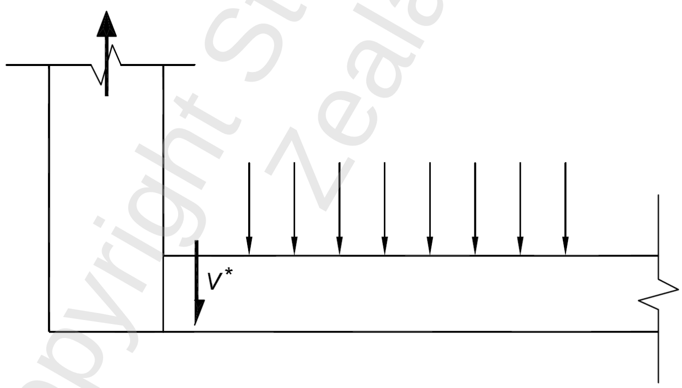


Figure C9.8 – Typical support conditions for locating factored shear force V^* (c)

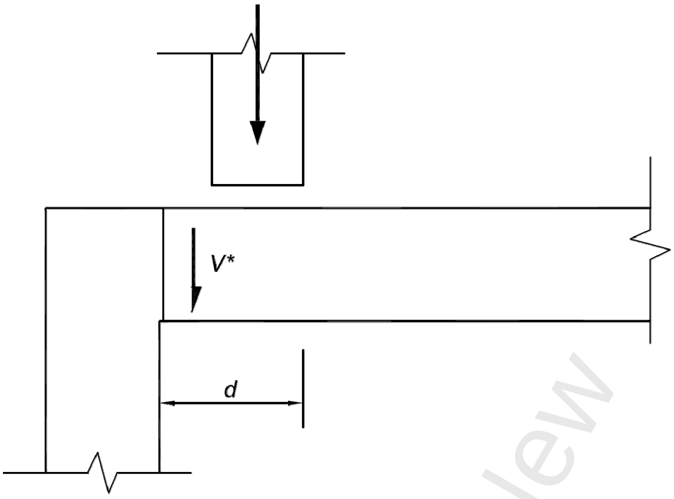


Figure C9.9 – Typical support conditions for locating factored shear force V^* (d)

In Figure C9.5, loads are shown acting near the bottom of a beam. In this case, the critical section is taken at the face of the support. Loads acting near the support should be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section should include all loads applied below the potential inclined crack.

Typical support conditions where the shear force at a distance *d* from the support may be used include:

(a) Members supported by bearing at the bottom of the member, such as shown in Figure C9.6; and
(b) Members framing monolithically into another member as illustrated in Figure C9.7.

Support conditions where this provision should not be applied include:

(a) Members framing into a supporting member in tension, such as shown in Figure C9.8. For this case, the critical section for shear should be taken at the face of the support. Shear within the connection should also be investigated and special corner reinforcement should be provided;
(b) Members for which loads are not applied at or near the top of the member. This is the condition referred to in Figure C9.5. For such cases the critical section is taken at the face of the support. Loads acting near the support should be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section should include all loads applied below the potential inclined crack;
(c) Members loaded such that the shear at sections between the support and a distance *d* from the support differs radically from the shear at distance *d*. This commonly occurs in brackets and in beams where a concentrated load is located close to the support, as shown in Figure C9.9 or in footings supported on piles. In this case the shear at the face of the support should be used.

C9.3.9.3.2 *Design of shear reinforcement*

The design shear force is assumed to be resisted by the contribution of the concrete, V_c, and by the shear reinforcement, V_s.

C9.3.9.3.3 *Maximum nominal permissible shear strength and effective shear area*

To guard against diagonal compression failure of the web mainly due to truss action, the total nominal shear strength, V_n is limited.

C9.3.9.3.4 *Nominal shear strength provided by concrete for normal density concrete, V_c*

Tests on reinforced concrete beams have shown that the shear stress sustained at failure decreases as the size of the beam increases and as the size of the aggregate particles decrease [9.10]. The effect is more marked in beams without shear reinforcement than in those with shear reinforcement. This decrease in shear stress sustained at failure (or diagonal tension failure) has been known for a considerable period of time [9.11]. This effect is allowed for in a number of codes of practice [9.12, 9.13 9.14], while being ignored in others such as former editions of NZS 3101 and ACI 318. Neglecting the decrease in shear strength with size can lead to members being designed with a factor of safety well below unity. Tests have shown that in some cases[9.10] the measured shear strength was less than half the value predicted by NZS 3101:1995.

The shear stress that can be sustained in the flexural tension zone of a beam depends on the shear transfer across cracks. This action is known as aggregate interlock action or interface shear transfer. As crack widths increase shear transfer decreases consequently the shear stress that can be sustained at failure decreases. It is found that as beam depths increase the crack width in the mid-depth region of the flexural tension zone increases and this leads to the observed decrease in shear stress at failure [9.10]. The use of either longitudinal reinforcement or stirrups in the web of the beam helps control crack widths, and hence this reinforcement reduces the loss in v_c, as the depth of the beam is increased.

The value of concrete strength, which may be used to calculate V_c is limited to 50 MPa. This limit is imposed as it has been found that with high strength concrete the larger aggregates can split in tension, which reduces shear transfer by aggregate interlock action and hence reduces the shear strength.

In beams that contain either shear reinforcement in excess of the nominal value, as indicated in 9.3.9.4.15, or longitudinal reinforcement spread through the flexural tension zone as indicated in 9.3.9.3.4(d) the loss in shear resistance with increasing size is small. This occurs as the shear and/or longitudinal reinforcement reduces the crack widths in mid-region of the flexural tension zone, thus maintaining a higher level of shear resistance by aggregate interlock action. As most beams contain either nominal shear reinforcement (9.3.9.4.12) or nominal longitudinal reinforcement (2.4.4.5) the decrease in shear stress sustained by the concrete with increasing depth is small. However, the depth factor, k_d, has a major influence on thick slabs such as are found in footing, as these elements are exempt from the requirement for nominal shear reinforcement when the design shear force, V^* lies between $0.5\phi V_c$ and ϕV_c, see 9.3.9.4.13.

C9.3.9.3.5 *Nominal shear strength provided by the concrete for lightweight concrete*

Two alternative procedures are provided to modify the provisions for shear and torsion when lightweight aggregate concrete is used. The lightweight concrete modification applies only to the terms containing $\sqrt{f_c'}$ in the equations for shear and torsion:

(a) The first alternative bases the modification on laboratory tests to determine the relationship between splitting tensile strength f_{ct} and the compressive strength f_c' for the lightweight concrete being used. For normal density concrete, the splitting tensile strength f_{ct} is approximately equal to $\sqrt{f_c'}/1.8$ [9.15, 9.16].

(b) The second alternative bases the modification on the assumption that the tensile strength of lightweight concrete is a fixed fraction of the tensile strength of normal weight concrete[9.17]. The multipliers are based on data from tests [9.18] on many types of structural lightweight aggregate concrete.

C9.3.9.3.6 *Nominal shear strength provided by shear reinforcement*

Considerable research has indicated that if certain assumptions are made regarding the inclination of diagonal compression forces in the web of beams, shear reinforcement is only required to resist the shear which exceeds that causing diagonal cracking (Reference 9.19).

C9.3.9.4 *Design of shear reinforcement in beams*

Equations 9–7, 9–8 and 9–9 are for the contribution of shear reinforcement to shear strength are based on the assumption that the diagonal compression forces in the web will develop at an angle of $\tan^{-1}j$ to the flexural tension force. The value of j is equal to the ratio of the internal lever-arm to the effective depth. The value of V_c given in 9.3.9.3.4 is associated and consistent with this assumption.

The alternative is to use the strut and tie method of designing shear reinforcement. This approach allows the angle of the diagonal compression forces in the web to range between $\tan^{-1}(2.0)$ to $\tan^{-1}(0.5)$, but in this case the contribution of the shear resisted by the concrete must be disregarded ($V_c = 0$). In using the strut and tie method caution should be exercised. The use of diagonal compression forces approaching the minimum permitted inclination can, in some cases, lead to wide diagonal cracks developing in the serviceability limit state.

Equations 9–7, 9–8 and 9–9 are presented in terms of shear strength V_s attributed to the shear reinforcement. Research [9.20, 9.21] has shown that shear behaviour of wide beams with substantial flexural reinforcement is improved if the transverse spacing of stirrup legs across the section is reduced.

Equations 9–7 and 9–8 are consistent with the assumption that the diagonal compression forces in the web are inclined at an angle of $tan^{-1} j$ to the longitudinal axis of the member.

C9.3.9.4.8 Angle of shear reinforcement not parallel to applied shear

In a circular member adjacent stirrups, hoops or spirals, will cross a diagonal tension crack at different angles. Only the component of the force that this reinforcement can resist in the direction of the applied shear force contributes to the shear strength. To allow for this effect the shear resistance provided by hoops or spirals in circular members is given by: A2

$$V_s = \frac{\pi}{2} A_h f_y \frac{d''}{s} \qquad \text{(Eq. C9–1)}$$

Where A_h is the cross section of the bar in the hoop (one leg) or spiral and s is the spacing of this reinforcement along the axis of the member and d'' is the centre-to-centre dimension of the outside stirrup, hoop or spiral. A2

C9.3.9.4.9 *Stirrups required where beam frames monolithically into side of girder* A3

Tests have shown that where a beam is supported by a girder of about the same depth failure at the joint tends to develop on a surface similar to the one shown in Figure C9.10A. To prevent this type of failure, this clause requires hanger stirrups capable of transferring the full reaction across the failure surface. These stirrups should be located in the supporting member (girder) within a distance equal to the width of the beam plus $d/2$ on each side of it. In addition, it is good practice for the longitudinal reinforcement in the supported member to pass over that in the supporting member.

The requirement for hanger stirrups is waived if the shear stresses at the end of the supported member are low as may be the case for one-way joists. Major inclined cracking in the beam will not occur in that situation, and the shear stresses will be introduced to the girder over the full depth of the beam. Similarly, if the girder is significantly deeper than the beam or if the girder is supported at the joint, the inclined thrust which develops in the beam can be resisted without tearing away the bottom of the girder.

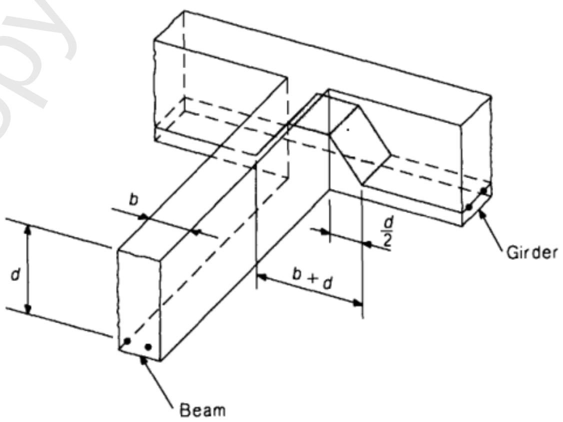


Figure C9.10A – Cracking at region where a beam is supported on a girder

C9.3.9.4.10 *Location and anchorage of shear reinforcement*

It is essential that shear (and torsion) reinforcement be adequately anchored at both ends to be fully effective on either side of any potential inclined crack. This generally requires a hook or bend at the end of the reinforcement.

C9.3.9.4.12 *Spacing limits for shear reinforcement*

It is very important that the spacing of shear reinforcement is close enough to ensure that it all crosses potential diagonal tension cracks.

C9.3.9.4.13 *Minimum area of shear reinforcement*

A minimum area of shear reinforcement is required in most beams where the design shear force, V^* A2 exceeds half the design shear strength provided by the concrete, ϕV_c. Exceptions to this requirement are allowed for beams, slabs, and rib construction and floor slabs constructed with pretensioned floor units, where these have a total thickness and web spacing width equal to or less than the appropriate limits stated in the 9.3.9.4.13.

A3 A negative moment with or without axial tension near support locations in composite floors, which are made up of precast pretensioned units with *in situ* concrete topping, can reduce the shear strength compared to the corresponding value for positive flexure. Generally, the requirement for a minimum A2 amount of shear reinforcement to be contained in pretensioned units used in floors when the design shear action, V^*, exceeds half the design shear strength is waived, provided the overall depth of construction is less than 400 mm and the clear gap between webs is less than 750 mm. However, shear reinforcement is still required when V^* is equal to or greater than the design shear strength provided by the concrete.

A3 It should be noted that where continuity reinforcement connects the topping and the precast unit to the supporting structure shrinkage of the concrete, or relative rotation between the composite unit and support, can induce tension together with negative moments and shear into the composite member near the supports. This action can result in a major decrease in shear strength compared with that obtained in tests where the composite member is supported on simple supports where no negative moments can develop. Further information may be obtained from Reference 9.43.

C9.3.9.6.1 *Extent of transverse reinforcement*

Compression reinforcement in beams or girders must be enclosed to prevent buckling. It is considered good practice to enclose all longitudinal bars where practicable.

C9.3.10 *Special provisions for deep beams*

C9.3.10.2 *Design methods*

The behaviour of a deep beam is discussed in References 9.22, 9.23 and 9.24. For a deep beam supporting gravity loads, this section applies if the loads are applied on the top of the beam and the beam is supported on its bottom face. If the loads are applied through the sides or bottom of such a member, the design for shear should be the same as for ordinary beams.

The longitudinal reinforcement in deep beams should be extended to the supports and adequately anchored by embedment, hooks, or welding to special devices. Bent-up bars are not recommended.

Deep beams can be designed using strut-and-tie models, regardless of how they are loaded and supported. Clause 9.3.1.6 allows the use of non-linear stress fields when proportioning deep beams. Such analyses should consider the effects of cracking on the stress distribution.

C9.3.10.3 and C9.3.10.4 *Vertical and horizontal shear reinforcement*

Tests[9.22, 9.23, 9.24] have shown that vertical shear reinforcement is more effective than horizontal shear reinforcement. The maximum spacing of bars has been reduced from 450 mm to 300 mm because this steel is provided to restrain the width of the cracks.

C9.3.11 *Openings in the web*

C9.3.11.1 *General*

These recommendations have been largely restricted to the restatement of general principles in “good engineering practice”. There is relatively little in the literature [9.25] that is relevant to seismic conditions, for which, in principle, more stringent rules should apply.

C9.3.11.2 *Location and size of openings*

Openings must be located in such a way that no potential failure planes, passing through several openings, can develop. In considering this the possible reversal of the shear forces, associated with the development of the flexural overstrength of the members, should be taken into account.

Small openings with areas not exceeding those specified are considered not to interfere with the development of the strength of the member. However, such openings must not encroach into the flexural compression zone of the member. Therefore the edge of a small opening should be no closer than 0.33d to the compression face of the member, as required by 9.3.11.4. Where two or more small openings are placed transversely in the web the distance between the outermost edges of the small openings should be considered as being equivalent to the height of one large opening and the member should be designed accordingly.

C9.3.11.3 *Larger openings*

Parts of the web adjacent to an opening, larger than that permitted by 9.3.11.2, should be subjected to rational analysis to ensure that failure of the member at the opening cannot occur under the most adverse load conditions. This will require the design of orthogonal or diagonal reinforcement around such openings.

C9.3.11.4 *Location and size of large openings*

More severe restrictions apply where the largest dimension of an opening in the web exceeds 0.25d. Openings of this size are not permitted in areas of the member where the nominal shear stress exceeds $0.4\sqrt{f_c'}$ or in a region closer than 1.5h to the critical section of a plastic hinge. The dimension of the opening at right angles to the axis of the member must not exceed 0.4d. The horizontal clear distance between adjacent large openings in a beam should not be less than twice the length of the opening or the depth of the member.

C9.3.11.5 *Reinforcement in chords adjacent to openings*

Rational analysis should be used to assign appropriate fractions of the total shear force to each of the chords above and below the opening through the web of a beam. In this the effects of axial forces and consequent cracking on the stiffness of each chord should also be considered[9.25]. Alternatively it may be assumed that the stiffness of the tension chord is negligible and therefore the entire shear resistance may be assigned to the compression chord. This approximation is implied in the example shown in Figure C9.10. The amounts, locations and anchorages of the longitudinal reinforcement which may be required in addition to the primary flexural reinforcement of the beam, should be determined from first principles so as to resist 1.5 times the moment induced in the chords only by the shear force across the opening. Similarly, shear reinforcement in the chords adjacent to the opening must resist 150 % of the design shear force. This is to ensure that no failure should occur as a result of the local weakening of the member due to the opening. Effective diagonal reinforcement above or below the opening, resisting 1.5 times the shear and moment, may also be used.

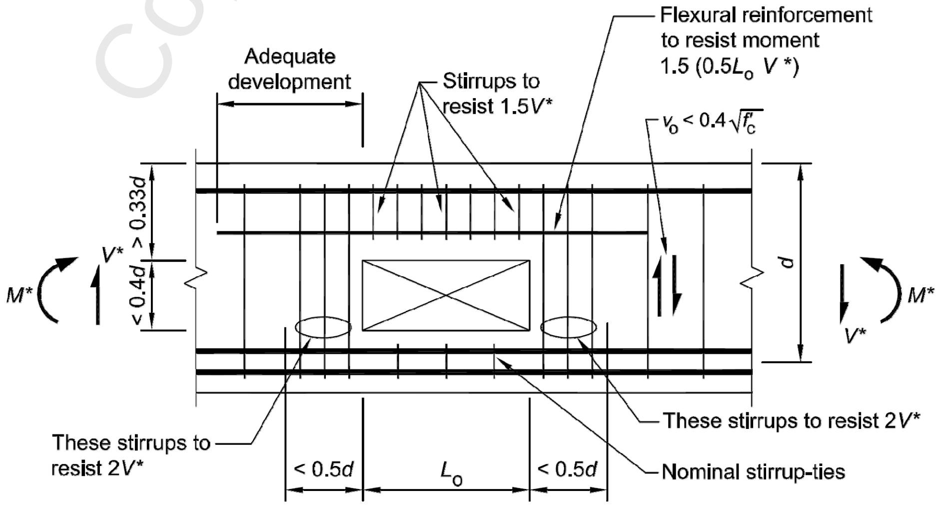


Figure C9.10 – Detail of requirements at a large opening in the web of a beam

C9.3.11.6 *Reinforcement in webs adjacent to openings*

At either side of an opening where the moments and shear forces are introduced to the full section of a beam, horizontal splitting or diagonal tension cracks at the corners of an opening are to be expected. To control these cracks transverse reinforcement, resisting at least twice the design shear force, must be provided on both sides of the opening. Such stirrups can be distributed over a length not exceeding 0.5d at either side immediately adjacent to the opening.

Typical details of reinforcement around a large opening in the web of a beam subjected to predominant positive moment, complying with these requirements, are shown in Figure C9.10(a).

C9.4 Additional design requirements for structures designed for earthquake effects

C9.4.1 *Dimension of beams*

C9.4.1.2 *Beams with rectangular cross sections*

The criteria for the relationship between clear span, depth and breadth of rectangular flexural members are based on dimensional limitations of the British Code of Practice CP 110 (1972) [9.26]. It was recognised, however, that stiffness degradation occurs in a flexural member during reversed cyclic loading in the yield range. Hence only one-half of the maximum slenderness ratios in CP 110 have been allowed. It has also been assumed that a continuous beam subjected to end moments due to lateral forces is equivalent to a cantilever with a length equal to two-thirds of that of the continuous beam and having an effective length factor of 0.75. The correspondingly adjusted CP 110 recommendations result in Equations 9–11 and 9–12.

C9.4.1.3 *Cantilevered beams*

For cantilevers a similar procedure was used. In this case the true length of the member with an effective length factor of 0.85 was considered, and the free end was not considered to be restrained against lateral movement. For bridge piers the criteria stated in Equations 9–13 and 9–14 will not be appropriate if diaphragm action of the superstructure can be relied upon. However, if these equations are not used for bridge piers special studies should be conducted to establish that lateral buckling will not be a problem.

C9.4.1.4 *T- and L- beams*

The contribution of flanges, built integrally with a web, to the stability of T- and L-beams has been recognised by allowing the maximum values of the length to breadth ratio, L_n/b_w, for rectangular flexural members to be increased by 50 %. Note that the restrictions of Equations 9–12 and 9–14 remain the same as for rectangular beams.

The breadth to depth ratios and depth to length ratios are shown in Figure C9.11 as functions of the length to breadth ratio. These rules allow a more uniform design approach to beam, column and wall sections.

C9.4.1.5 *Width of compression face of members*

The minimum width of the compression face of flexural members is specified as 200 mm.

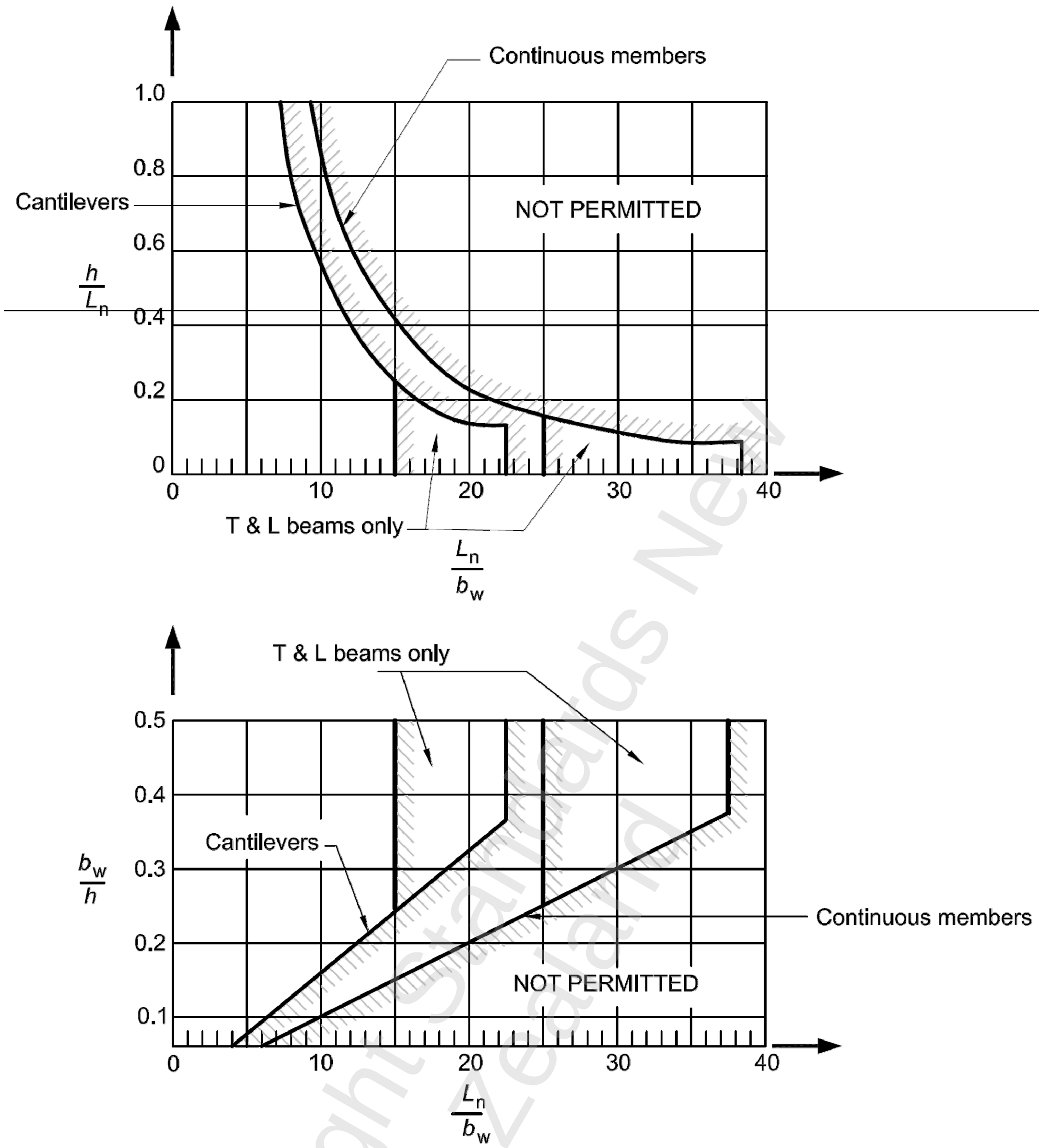


Figure C9.11 – Dimensional limitations for members

C9.4.1.6.1 Contribution of slab reinforcement to design strength of beams

For reinforcement in an outstanding flange of a ductile plastic region containing an appreciable proportion of reinforcement to act reliably under repeated inelastic cyclic loading conditions it must be effectively tied into the beam to enable the shear force to be sustained between the flange and beam web. A3

The width of slab that is mobilised to act with the beam increases with the curvature. The requirements in this clause are intended to give the design strength that can be sustained at relatively small section ductility levels.

The way in which reinforcement in a reinforced concrete slab can act to increase the strength of a beam in a moment resisting frame is outlined in references 9.27 and 9.28.

C9.4.1.6.2 Contribution of slab reinforcement to overstrength of plastic region in a beam

A3 A major gap exists in our knowledge of the performance of T and L-beams reinforced with post-tensioned cables in the beams or in the slabs. The prestress introduced into the slab is likely to increase the contribution that the slab makes to the flexural strength of the beam-slab members beyond that implied in (a) to (g) in 9.4.1.6.2. This is a particularly critical issue in determining the overstrength of potential plastic hinge zones. For this reason where this form of construction is used a special study is required to determine overstrength actions if capacity design requires plastic hinges to form in the beams.

The remainder of this clause deals with the case where the slab is reinforced with non-prestressed reinforcement, or alternatively the slab consists of *in situ* concrete with non-prestressed reinforcement supported by pretensioned units.

This is an area of active research and the full mechanisms of the interaction of beams and floor slabs is not well understood. It is hoped that future research will increase our understanding. Consequently the requirements in this clause are tentative.

Tests have shown that the interaction of floor slabs, particularly when they contain prestressed units, can greatly increase the bending moment resisted by the beam. [9.1, 9.28, 9.29, 9.30 and 9.31]. The extent of this strength increase is considerably greater than has been implied in earlier editions of NZS 3101. It is particularly important that this aspect is considered in capacity design so that non-ductile failure mechanisms can be avoided.

To capture the maximum likely overstrength moment under high section ductility levels additional reinforcement in the flanges, compared to that assumed to contribute to the design strength, needs to be included in calculations. For this reason, different criteria are given for determining the effective flange widths for strength than for overstrength.

Figure C9.12 illustrates some assumptions made in assessing the overhanging flange width made in overstrength calculations at plastic regions corresponding to 9.4.1.6.2(a), (b) (c) and (e).

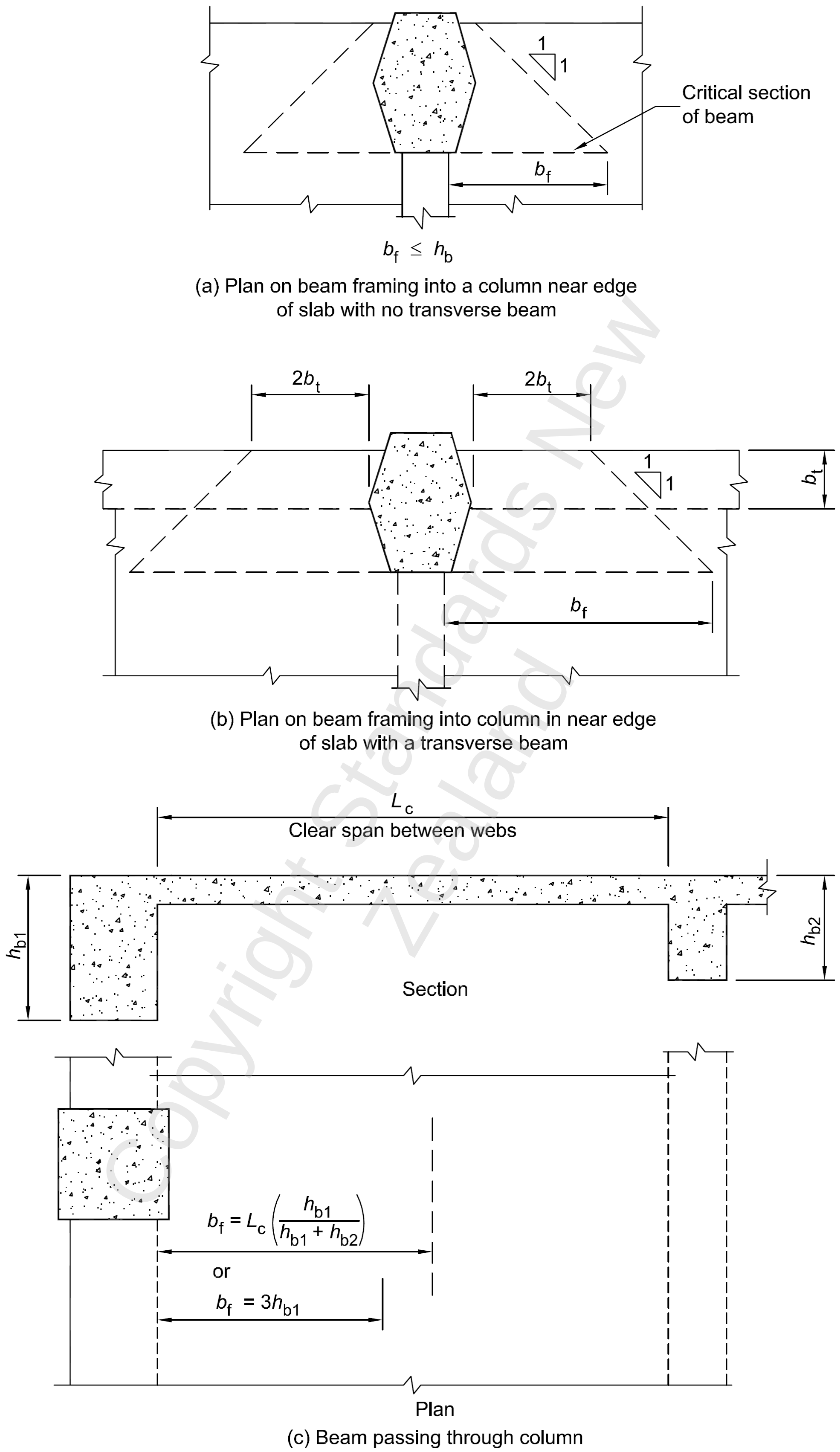


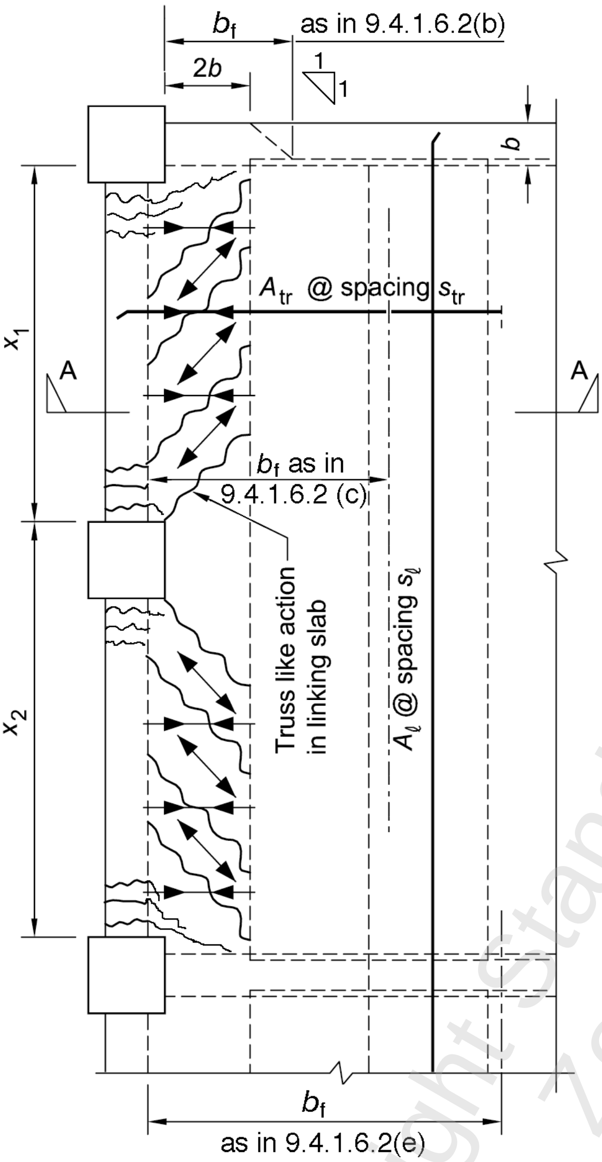
Figure C9.12 – Flange widths for calculating overstrength moments

Parts (b) and (e) of 9.4.1.6.2 deal with the situation where an outstanding flange to a beam meets a transverse beam. Elongation in plastic a plastic region in the beam generally creates wide cracks in the A2 concrete at the interface of the overhanging flange and transverse beam. Consequently, reinforcement crossing this interface may be stressed close to its ultimate stress. Hence the overstrength force in the reinforcement at such sections is taken as 1.1 $\phi_{o,fy}$ f_y. The situation described by 9.4.1.6.2 (e) is complex, and a rational method of assessing the likely overstrength at such locations has not been developed[9.32].

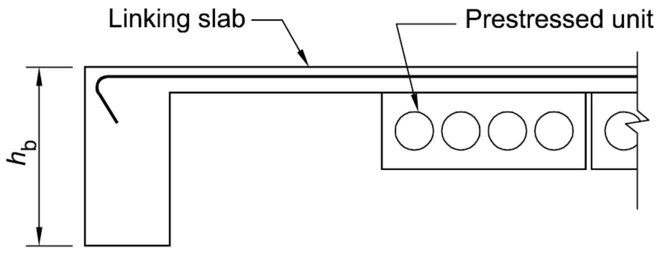
Part (d) of 9.4.1.6.2 considers the case where the effective overhanging flange contains prestressed units, which span past the plastic region or regions in the beam.

The overall mechanism of interaction of an outstanding flange containing prestressed units, where these units span past a column, is illustrated in Figure C9.13. Part (a) of this figure shows how the slab restrains elongation of the beam due to the formation of plastic hinge zones adjacent to the central column. The linking slab acts to restrain the elongation by a truss like manner. Diagonal compression forces develop between diagonal cracks with the tension force normal to the beam being resisted by reinforcement. The area of each bar resisting the transverse component of the diagonal compression forces is shown as A_{tr} on the figure. The total area of transverse reinforcement, A_t, in Equation 9–17, is equal to the sum of all the A_{tr} areas within the distance *x*. Likewise the reinforcement area, A_ℓ, in the same equation is equal to the total area of longitudinal reinforcement in the effective overhanging flange width. Tests have indicated that the cracks develop at close to 30° to the axis of the beam. The transfer of shear by this truss like action is illustrated in part (c) of the figure. The horizontal shear transfer across the linking slab applies a tension force to the prestressed units at mid-height of the topping concrete. This places the prestressed unit and associated topping concrete member in negative flexure, causing it to hog up. The differential vertical movement results in the linking slab being subjected to flexure and consequently transferring vertical shear between the beam and prestressed unit. The beam is pulled up and the prestressed unit is pulled down by this shear as illustrated in Figure C9.13 (d).

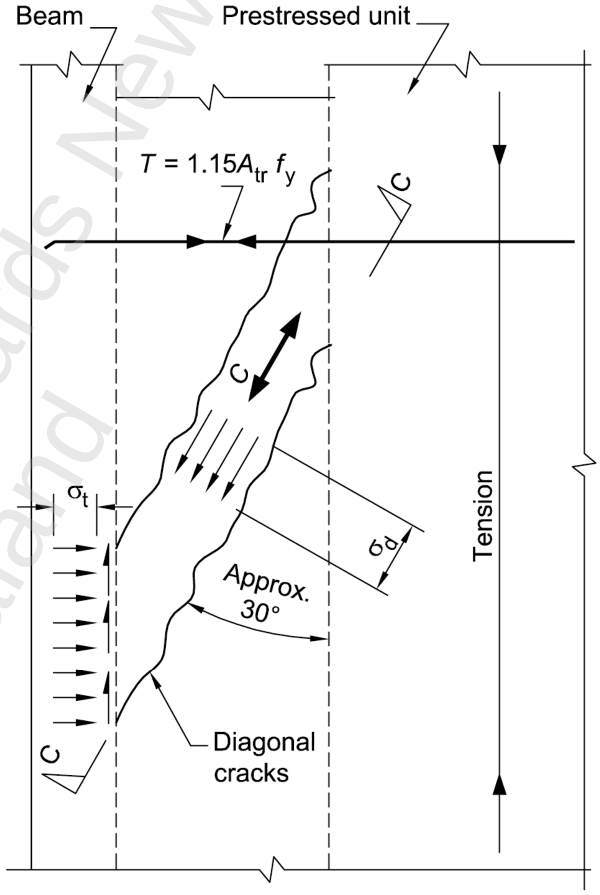
A2



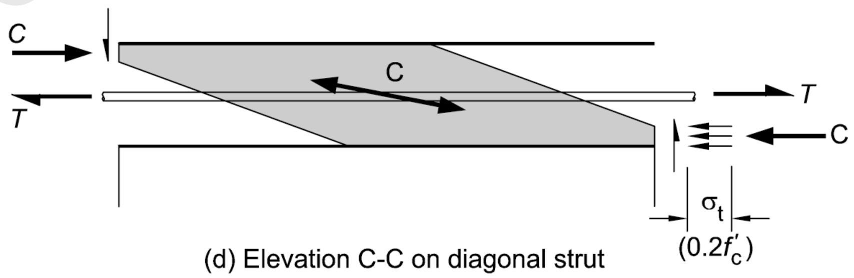
(a) Plan on floor



(b) Section A-A



(c) Strut and tie action in linking slab



(d) Elevation C-C on diagonal strut

Figure C9.13 – Transfer of horizontal and vertical shear forces across linking slab

The tensile capacity of composite overhanging flange can be assessed in two components as outlined in (a) and (b) below.

(a) The tensile capacity of longitudinal reinforcement in the topping concrete is based on an assumed upper stress limit of $1.1f_y$. This value is intended to represent an average stress level in reinforcement in the *in situ* topping concrete away from the ends of the prestressed units. Strain levels are not sufficient to cause appreciable strain hardening. The 1.1 allows for the fact that f_y is a lower characteristic strength and hence the stress level needs to be increased to correspond to an average value.

(b) The tensile resistance provided by the precast units is calculated for a tension force, T_p, acting at the mid-height of the *in situ* topping concrete, as this is the level where the actions are transferred to the beam. The prestressed unit on its own has virtually no capacity to resist any tension force acting in *in situ* concrete. In precast prestressed units, such as hollow-core, stem or tee beams, which are designed basically to provide resistance to gravity loads, the pretensioned reinforcement is located close to the bottom surface of the unit. Hence there is virtually no room for the compression force to drop below the prestressed reinforcement and there is virtually no negative moment capacity, which results in the member not being able to sustain a tension force in the *in situ* concrete. In the unloaded unit the flexural compression force is coincident with the prestressing force (internal level-arm is zero as the bending moment is zero) as illustrated in Figure C9.14(a). However, when a bending moment acts on the composite precast *in situ* member the centroid of the compression force rises above the prestressing steel, as illustrated in Figure C9.14(b) so that the product of compression force and lever-arm is equal to the bending moment. Part (c) of this figure illustrates what happens when a tension force, T_p, acts at the mid-height of the *in situ* concrete. It follows that T_p can be found from equilibrium requirements assuming that the limiting position of the compression force centroid is coincident with the prestressing force. On this basis the value of T_p is given by equation 9–15 where the bending moment is divided by the distance between the tension force, T_p, and the centroid of the prestressing force.

The bending moment, M_f, resisted by the outstanding flange, is found assuming the composite *in situ* concrete and precast units act as an equivalent beam, which has a span equal to the length of the precast units. The loads acting on this equivalent beam consist of;

(i) The dead load of the outstanding flange and the long-term live load acting on it;

(ii) The vertical component of the shear force that can be transmitted between the web of the beam and the column face to the first precast unit by the slab linking these elements;

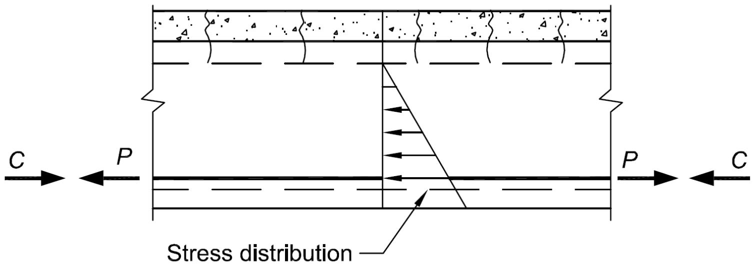
(iii) The end moments, which are applied to the precast units at their support points.

The vertical component of the shear force in the slab, which links the beam web and column face to the first precast unit, arises due to the differential vertical displacement between the beam web and the prestressed units and its value is found from the flexural strength of the linking slab.

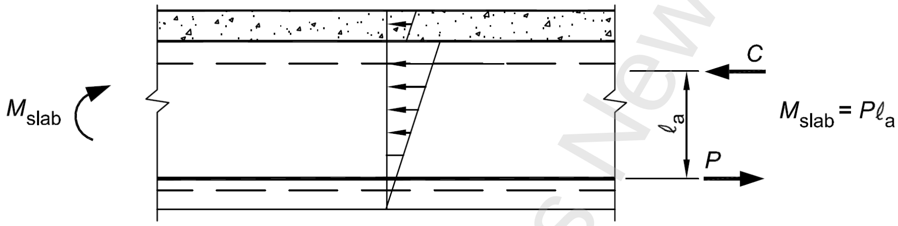
To determine the flexural strengths of the linking slab the interaction of the truss like action transferring horizontal shear and the flexural actions due to differential vertical movement needs to be considered. These two actions are illustrated in Figure C9.13 (c) and (d). The compressive strength is reduced by the diagonal cracks and hence the compressive strength available for resisting the diagonal compression forces has been assumed to be reduced from $0.85f'_c$ to $0.6f'_c$. With this limit the compression stress in the concrete normal to the web of the beam corresponds to 0.15 times the concrete strength. However, as f'_c corresponds to a lower characteristic strength a value of 0.2 f'_c is used to bring it up to the likely strength of concrete. In the linking slab between the column face and the first precast unit there is no horizontal diagonal transfer and consequently in this location a stress in the concrete of 0.8 f'_c is appropriate. Some reduction from 0.85 f'_c is made as cracks can be expected in this zone. With these concrete stress limits, as illustrated in Figure C9.13 (d), the flexural strengths can be found from conventional flexural theory. In the calculations the yield stress of the reinforcement is taken as 1.1 f_y, with the 1.1 factor increasing the stress to the likely average yield stress value.

Where there is reinforcement located in both the topping concrete and in the bottom of the precast units at their support points, positive moments can be applied to the precast units. This situation arises with hollow-core units where some of the cells are reinforced at their supports (see Figure C18.4 and Figure C18.5). This reinforcement allows a positive moment to be applied to the outstanding flange, which is equal to the critical force carried by the top or bottom reinforcement times the lever-arm to the other layer of reinforcement. As wide cracks are expected in the support locations of the precast units the critical reinforcement stress is taken as $1.1\phi_{o,\,fy}\,f_y$ A2

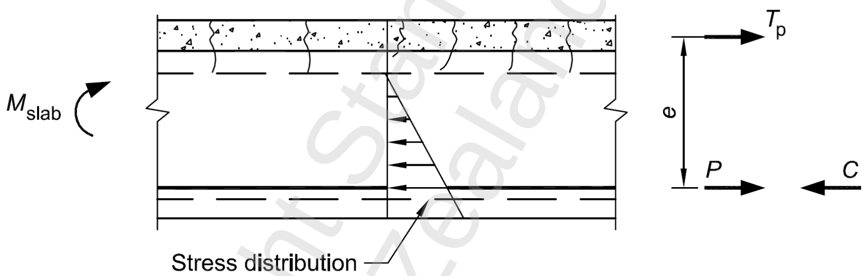
The capacity of an outstanding flange to contribute the flexural overstrength is limited either by its tension capacity, as found following 9.4.1.6 (a), (b), (c), (d), (e) or (f), or by the horizontal shear strength of the linking slab. Clause 9.4.1.6(g) establishes this shear strength limit. The first term in Equation 9–17 gives the tension force that is transmitted between the beam supporting the precast units and the overhanging flange and the second term gives the horizontal force that can be transmitted by the horizontal shear strength of the linking slab. The first term on the right hand side of the equation gives the tension force transferred to the slab from the supporting beam. All the reinforcement connecting the slab to the supporting beam that is located within the overhanging flange (area A_t equal to the sum of bar areas A_{tr}) contributes to this force. The width of the overhanging (b_f) depends upon whether it is an external or internal member, as illustrated in Figure C9.13(a). The shear strength given by the second term is based on a strut angle of 30° and a reinforcement strength, which is a few percent above the design yield strength, to allow for the average strength being higher than the design value and for limited strain hardening. A2



(a) Actions in floor slab when no dead or live load acts on the floor



(b) Actions in floor slab when dead and live loads act on the floor



(c) Actions on floor slab when dead and live lmoment acts on the floor together with tension restraint force per beam

Figure C9.14 – Calculation of tension force from pretensioned units which contribute to flexural overstength of beam

C9.4.1.6.3 Diameter and extent of slab bars

Principal beam bars may be situated in layers adjacent to columns, that is, in the areas shown in Figure C9.1. The diameter of these bars is limited to one fifth of the slab thickness because it would be difficult to prevent the inelastic buckling of larger size bars. Moreover, it is more difficult to ensure the force transfer from larger bars in the slab to the column core under predominantly earthquake actions. In any case, sufficient transverse reinforcement should be present in such slabs to ensure effective transfer of bond forces to the column core.

C9.4.1.7 *Narrow beams and wide columns*

The effective width to be considered for wide columns and the treatment of eccentric beam-column connections are discussed in C15.4.6 and C15.4.7. Frame details in which the axes of the beams and columns do not coincide should be avoided.

C9.4.1.8 *Wide beams at columns*

Figure C9.15 illustrates this requirement which is intended to ensure that the beam is not greatly wider than the column in order to ensure that the longitudinal beam steel needed for seismic forces is kept reasonably close to the column core.

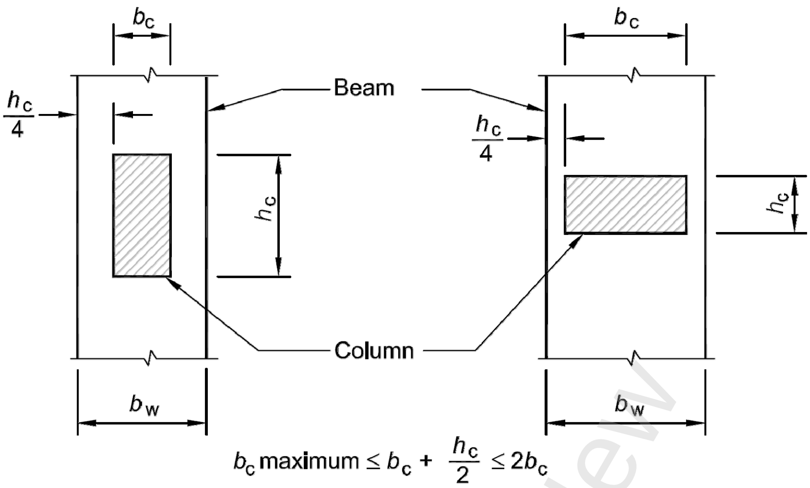


Figure C9.15 – Maximum width of beams

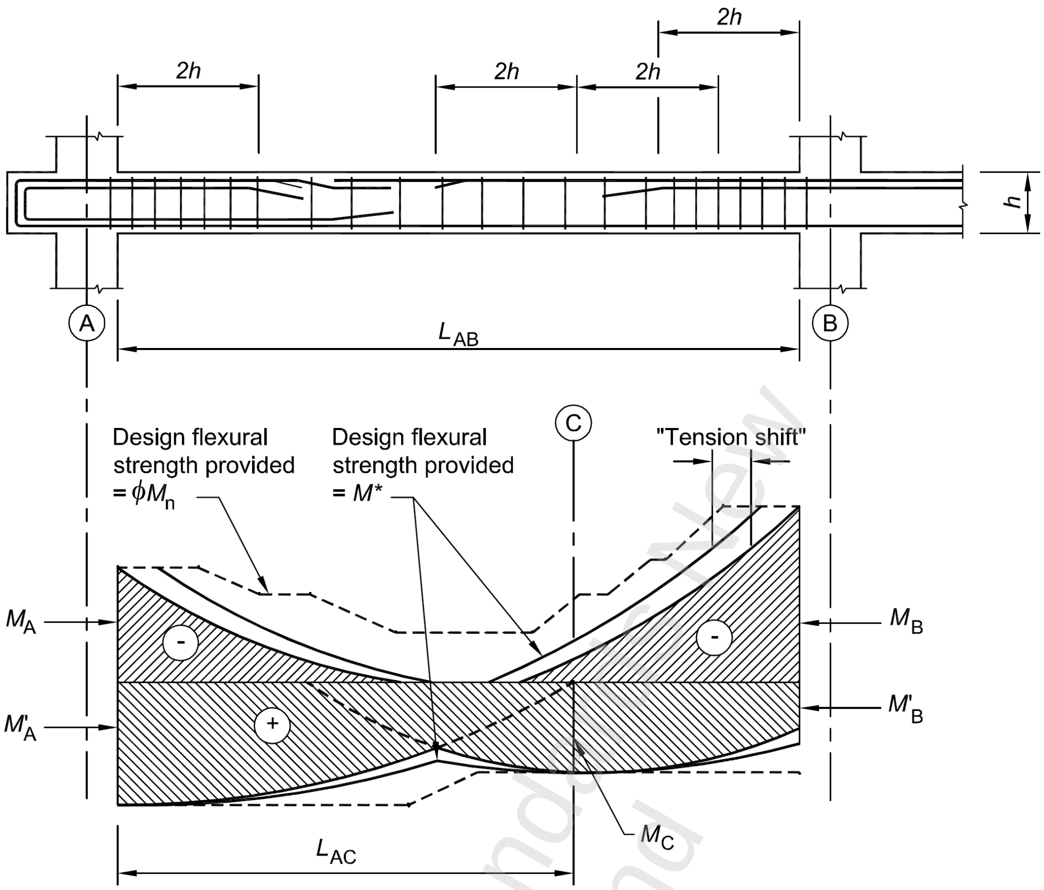
C9.4.2 *Ductile detailing lengths*

As indicated in C2.6.1.3 yielding of reinforcement, due to the formation of a plastic hinge, extends for an appreciable distance along a beam. Over this length, which is referred to as the potential yielding region, special detailing is required to prevent bars from buckling, confine the concrete and eliminate possible anchorage failure of bars. The three regions where plastic hinging could occur in beams are discussed below:

(a) Regions adjacent to supporting columns, where both the top and the bottom reinforcement can be subjected to yielding in tension and compression due to reversed flexure (see Figure C9.16).

(b) When a potential plastic hinge is deliberately relocated from a column face it should be designed so that its critical section is at least a distance equal to the member depth *h* or 500 mm away from the column face. This section will occur where the flexural reinforcement is abruptly terminated by bending it into the beam, or where a significant part of the flexural reinforcement is bent diagonally across the web, or where the narrow end of a haunch occurs. It is considered that under reversed loading yielding can encroach into the zone between the critical section and the column face. Therefore special transverse reinforcement must be placed at least 0.5*h* or 250 mm before that section and extended over a distance of 2*h* to a point 1.5*h* past the critical section into the span. Two examples are given in Figure C9.17. The detailing of such regions requires particular attention [9.4, 9.32].

(c) A plastic region may form in the positive moment region within the span of a beam where a negative moment plastic hinge cannot develop (see Figure C9.16 at section C). In this region the danger of buckling of the top compression bars is far less, since those bars will not have yielded in tension in a previous load cycle. Moreover such a plastic hinge is likely to be well spread and under yield conditions it will carry very low shear forces. Because of the variability of gravity loads during a major earthquake the position of the critical section of such plastic hinges may not be able to be determined with precision.



NOTE – Tension shift accounts for the actual tension force in the flexural reinforcement, at a given section, being greater than that required to resist the bending moment at that section; being a function of the slope of the inclined crack or diagonal compression (typically assumed to be 45° in beams), which is part of the truss mechanism that resists shear [9.4].

Figure C9.16 – Localities of plastic hinges where stirrup-ties are required

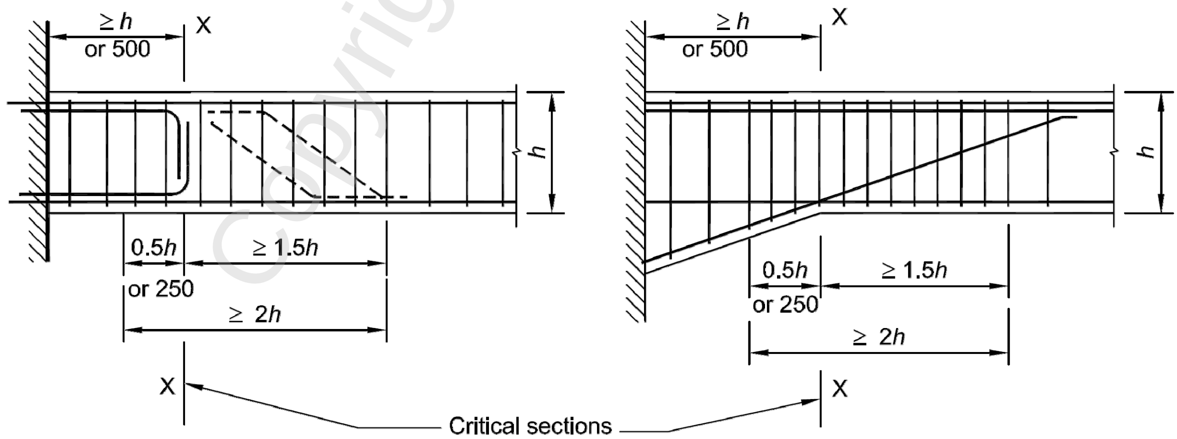


Figure C9.17 – Plastic hinges located away from column faces

C9.4.3 *Longitudinal reinforcement in beams of ductile structures*

C9.4.3.1 *Development of beam reinforcement*

The bending moment envelope to be used is that corresponding to the formation of two plastic hinges in each span under the combined effects of seismic forces and gravity load. The moments at the plastic hinges are to be based on the flexural overstrengths of the sections as detailed. To ensure that the curtailed reinforcement is adequate for the moment demand between plastic hinges, the envelope should also take into account the possibility of overstrength being developed at one plastic hinge of the beam while only the nominal moment is developed at the other plastic hinge.

In some circumstances, when flexural overstrength is developed at the critical section of a plastic hinge region, some sections outside the plastic hinge region may develop greater than nominal flexural strength. The reinforcement should not be increased beyond the hinge to meet such a condition. However, reinforcement provided at the critical sections of the plastic hinges should not be terminated unless the continuing bars provide nominal flexural strength at least as great as the moment demand resulting when flexural overstrength is attained at either or both of the critical sections in the plastic hinge regions.

C9.4.3.2 *Anchorage of beam bars in columns or beam stubs*

Because of yield penetration from the face of a column toward its core, the length available for the development of the strength of beam bars is gradually reduced during cyclic reversals of earthquake actions. To ensure that the beam capacity is maintained after several excursions of the structure into the inelastic range, half the column depth or $8d_b$, whichever is less, is required to be disregarded for the purpose of anchorage. This ineffective development length to be assumed is illustrated in Figure C9.18.

When bars are anchored short of the external column bars the area of the bars on the inside column face should be increased by 10 % above that required from strength calculations at the column face. Stopping the beam bars short of the column bars reduces the strength of the column for flexure associated with tension on the inside face of the column.

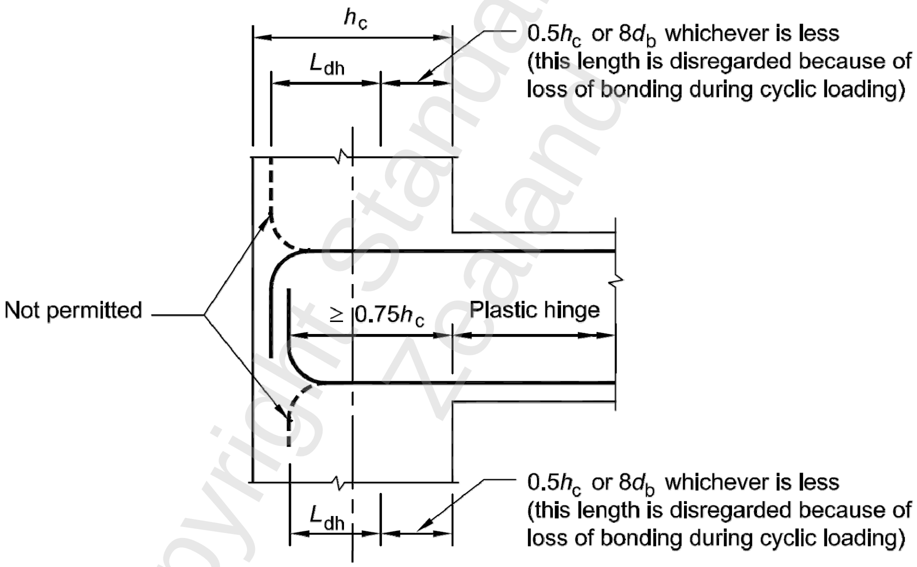


Figure C9.18 – Anchorage of beam bars when the critical section of the plastic hinge forms at the column face

When the flexural reinforcement is curtailed in such a way that the critical section of a potential plastic hinge is at a distance from the column face of at least the beam depth or 500 mm, whichever is less, progressive yield penetration into the column is not expected. Only in this case may the development length for the beam bar be assumed to commence at the column face where the beam bar enters. This case is shown in Figure C9.19.

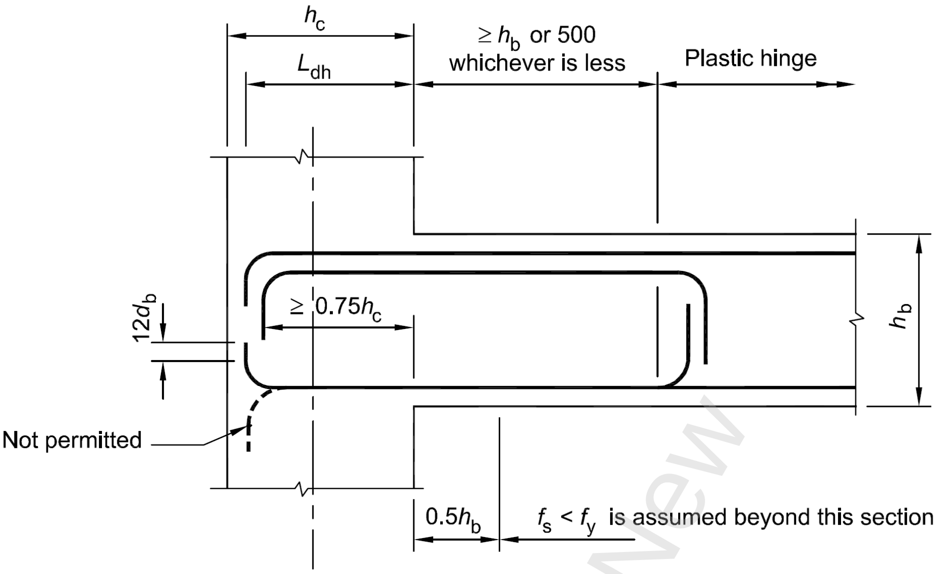
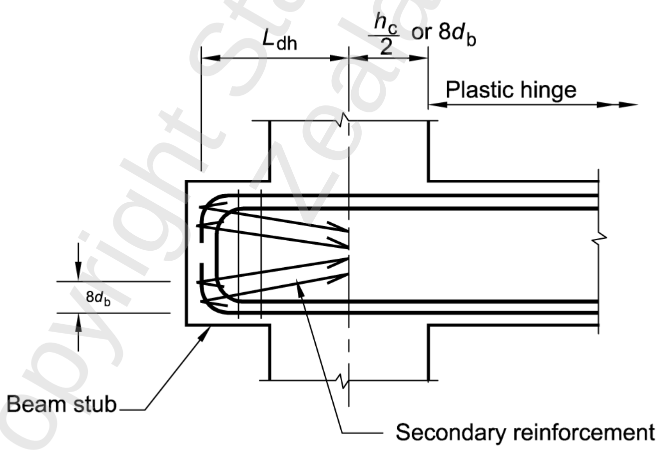


Figure C9.19 – Anchorage of beam bars when the critical section of the plastic hinge is at a distance from the column face of at least the beam depth or 500 mm, whichever is less

C9.4.3.2.2 *Reinforcement of beam stubs*

The sloping bars or secondary reinforcement in Figure C9.20 indicate one-way by which anchorage of the beam bars can be boosted when in compression. Mechanical anchorage devices, such as plates welded to the end of the beam bars, while performing well when the bar to be anchored is in tension, should be tied back into the column core where the development length is inadequate to develop the strength of the bars in compression without the anchorage device.

A2



NOTE –
Secondary reinforcement placed within a vertical distance of 8 d_b measured from the longitudinal axis of the bar.

Figure C9.20 – Anchorage of beam bars in a beam stub

C9.4.3.2.3 *Development length*

The provisions of 8.6.3.3(a) which reduce the anchorage length to less than that required for a bar at yield stress are unsafe for laps in regions adjacent to plastic hinge regions and for the lap zones at the ends of columns that are protected by capacity design. In such columns, the magnitude of the stresses in the longitudinal bars may approach yield.

C9.4.3.2.4 Anchorage of diagonal bars in coupling beams

Where diagonal or horizontal bars in a coupling beam are anchored in adjacent structural walls, the development length must be increased. This is in consideration of the likely adverse effect of reversed cyclic loading on the anchorage of a group of bars and the fact that the concrete in the wall may be subjected to tension transverse to the anchored bars[9.33].

C9.4.3.2.5 *Bars to terminate with a hook or anchorage device*

These requirements at exterior columns are illustrated in examples of Figure C9.18 and Figure C9.19.

When bars are anchored in or near a column core, the bearing stress developed in the bend is required to be directed towards the core to ensure sufficient force transfer within the joint. Therefore, the bending of bars away from the core, as illustrated by dashed lines in Figure C9.18 and Figure C9.19 is not permitted.

When the moment demands, particularly those involving bottom reinforcement, are different at opposite faces of an interior column, some of the beam bars may be terminated at the interior column. This will enable the unnecessary boosting of flexural capacity to be avoided. Anchorage within the joint core of interior columns is permitted, provided that a standard hook located adjacent to the opposite face of the column is employed, see Figure C9.21.

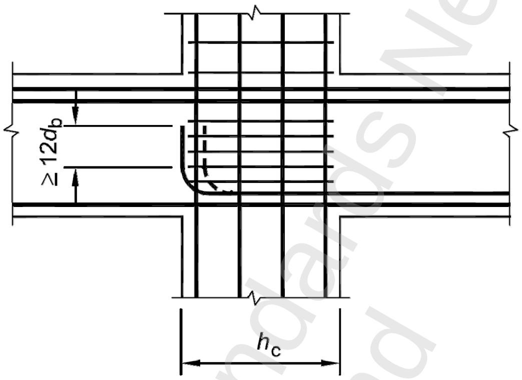


Figure C9.21 – Termination of beam bars at an interior joint

C9.4.3.3 *Maximum longitudinal reinforcement in beams containing ductile plastic regions*

The ductility of a plastic hinge region in a beam is dependent on the value of the curvature ductility factor, ϕ_u/ϕ_y , which can be achieved at the critical section, where $\phi_u = \varepsilon_c /c$ is the ultimate curvature at the section, ε_c is the ultimate concrete compressive strain, c is the neutral axis depth at the ultimate limit state, and ϕ_y is the curvature at first yield at the section [9.4]. Moment-curvature analysis has shown that, with the other variables held constant, the available curvature ductility factor ϕ_u/ϕ_y is increased if the tension steel ratio p is decreased, the compression steel ratio p' is increased, the steel yield strength f_y is decreased, and the concrete compressive strength f'_c is increased.

Equation 9–18 will ensure that when the extreme fibre concrete compression strain is 0.004, an adequate curvature ductility factor of at least seven can be attained when either Grade 300 or 500 steel is used in a rectangular cross section with a compression reinforcement area equal to one-half of the tension reinforcement area [9.34]. An extreme fibre concrete compression strain of 0.004 is a lower bound for the commencement of crushing of the cover concrete when normal strength concrete is used. The confinement of the concrete core of the beam provided by stirrup-ties in the potential plastic hinge regions will ensure that the concrete core can sustain much higher strains than 0.004 and hence permit much higher ϕ_u/ϕ_y values (at least 20) to be reached accompanied by spalling of the cover concrete.

Values of p_{max} given by Equation 9–18 are shown in Table C9.2. Equation 9–18 indicates that p_{max} increases with concrete strength. However, the analysis on which the equation was based was only conducted for concrete strengths up to about 35 MPa [9.34]. It is specified that p_{max} is not to exceed 0.025 since that steel ratio is regarded as a practical maximum. It is also to be noted that to control the quantity of shear reinforcement required in beam-column joints it is expedient to limit the tension steel ratio in beams to much less than 0.025.

C9.4.3.4 *Minimum longitudinal reinforcement in beams containing ductile plastic regions*

(a) The area of compression reinforcement should be at least equal to one-half of the area of tension reinforcement, in order to ensure adequate ductility at potential plastic hinge regions, and to ensure that a minimum of tension reinforcement is present for moment reversal. With less compression reinforcement the tension reinforcement ratio would have to be reduced considerably, in order to ensure that reasonable curvature ductility is available. When the area of the longitudinal compression reinforcement A'_s is greater than one-half of the area of the longitudinal tension reinforcement A_s, the p_{max} given by Equation 9–18 could be increased. For example, if A'_s = 0.75 A_s, p_{max} given by Equation 9–18 could be increased by at least 30 % [9.34]. The requirement that A'_s shall at least equal 0.5A_s need not be complied with for T- or L- beams subjected to positive bending moment, with the compression flange being part of a cast-in-place floor slab, due to the large width of compressed area of concrete.

Table C9.2 – Values of p_{max} given by Equation 9–18

f'_c (MPa)	f_y = 300 MPa	f_y = 500 MPa
25	0.0194	0.0117
30	0.0222	0.0133
40	0.0278 (1)	0.0169
50	0.0333 (1)	0.0200

NOTE —
(1) p_{max} is not permitted to exceed 0.025.

(b) It is required that the tension reinforcement ratio should at least equal $\sqrt{f'_c}/(4f_y)$ over the full length of the beam to avoid a sudden failure at first cracking (see C9.3.8.2.1).

(c) It is required that the top reinforcement along the beam should not be reduced to less than one-quarter of the top reinforcement at either end. Also at least two reasonable size longitudinal bars should exist in both the top and the bottom of the beam throughout its length. This is to ensure continuity of reinforcement and some positive and negative moment capacity throughout the beam, in order to allow for unexpected deformations and moment distributions from severe earthquake actions.

C9.4.3.5 *Maximum diameter of longitudinal beam bars passing through interior joints of ductile structures*

At interior beam-column joints, such as shown in Figure C9.21, extremely high bond stresses can develop when a frame sustains large inelastic deformations due to seismic motions. Beam bars may be forced to yield in tension at one column face and be subject to a high compressive stress at the opposite column face. Also, yield penetration along a beam bar from either face of an interior column may considerably reduce the effective anchorage length of the bar.

Thus the limit for the ratio of bar diameter d_b to the column depth (h_c in Figure C9.21), is intended to ensure that a beam bar will not slip prematurely through the joint core during cyclic reversed inelastic displacements [9.35, 9.36]. However, when potential plastic hinges are designed so that yielding in the beam bars cannot develop nearer than half a beam depth to the column face, as shown in Figure C9.19, better bond conditions exist and consequently larger diameter beam bars may be used [9.32, 9.37, 9.38 9.39]. For paired or bundled bars, the diameter should be taken as the diameter of a single bar of equivalent area.

Tests have shown that with increased yield stress levels in reinforcement there is a decrease in the bond performance of beam bars passing through beam-column joint zones when they are subjected to cyclic conditions involving yielding. The degradation arises due to cyclic yielding of the beam reinforcement in the joint zones. The higher strains associated with high grade reinforcement result in a more rapid degradation in bond and consequently the criteria developed for Grades 300 and 430 reinforcements need to be modified for use with Grade 500 reinforcement. Analysis of test results on internal beam-column joints, published in the literature, show that the current criteria for Grade 300 reinforcement works adequately for Grade 500 reinforcement provided the inter-storey drifts are limited to 1.8 % calculated in accordance with NZS 1170.5.

Failure in bond of beam bars passing though an internal beam-column joint generally results in a very significant loss of stiffness and it can be associated with a loss in strength [9.40].

In low-rise structures in which column sidesway mechanisms are permitted, shallow columns are common. Since the beam reinforcement may be controlled by gravity loading considerations, a large excess of strength under seismic forces may exist, and beam bar stresses at the moment capacity of the columns may be of one sign (for example tensile) through the full width of the joint.

The limitations set in 9.4.3.5.2 are derived for the condition of beams hinging, at flexural overstrength, at both faces of the column, producing bar stresses ranging from tensile yield at one face of the column to compressive yield at the other. Where such conditions do not exist, such as where the bar force remains tensile through the joint, lower bond stresses will result, and consequently increased bar sizes are permitted. In addition, any loss of anchorage caused by deteriorating bond conditions within the joint may, under these conditions, be accommodated in the opposite beam without detriment to the structural performance. The relaxation permitted will alleviate the congestion caused by the need for abnormally small bar diameters otherwise required by the shallow columns.

When the criteria in 9.4.3.5.2 are difficult to satisfy, the somewhat more elaborate procedure[9.32], which considers the beneficial effects of additional parameters, may be applied. This may enable larger diameter beam bars to be used.

When the flexural compression force acting in a plastic region subjected to cyclic loading conditions is significantly greater than the force that can be resisted by the reinforcement in the compression zone, extensive yielding in both tension and compression occurs in this reinforcement. This action accelerates the break down in bond in beam-column joint zones. Allowance for this effect is made in subclause 9.4.3.5.3(d) of the clause. This situation may arise where either: A2

(a) The area of reinforcement on one side of a beam is appreciably greater than the area on the other side; or
(b) Where a beam flange contributes a significant portion of the flexural tension force to the beam plastic region under overstrength conditions.

In the second situation, a significant portion of the flexural tension force may be provided by prestressed units and reinforcement located in the effective flange width (see 9.4.1.6.2).

C9.4.3.6 *Splices in longitudinal reinforcement of beams of ductile structures*
Lap splices in beams must be located away from regions of high shear stress and away from potential plastic hinge regions where stress reversals could occur.

C9.4.4 *Transverse reinforcement in beams of ductile structures*

C9.4.4.1 *Design for shear in beams of ductile structures*

C9.4.4.1.1 *Design shear strength*
Typically two plastic hinges may form in a beam, such as at A and B in the span shown in Figure C9.16. With the corresponding flexural overstrengths, in accordance with the definitions, denoted as M_{oA} and M_{oB}, the design shear force at B will be:

$$V^*_{OB} = \frac{M'_{oA} + M_{oB}}{L_{AB}} + V_{GB} + V_{QuB} \quad \text{(Eq. C9–2)}$$

Similarly the critical shear for the same beam at A will be:

$$V^*_{OA} = \frac{M'_{oA} + M_{oB}}{L_{AB}} + V_{GA} + V_{QuA} \quad \text{(Eq. C9–3)}$$

Where V_{GB} and V_{GA} are the shear forces at B and A in Figure C9.16 due to dead load, and V_{QuB} and V_{QuA} are the shear forces at B and A due to live load.

The value of M'_{oB} must be evaluated from the flexural overstrength in the vicinity of C. It will be noted that the shear at C for this load combination is zero. The intent is to prevent a shear failure under maximum possible lateral forces. Accordingly the nominal shear strength must be equal to or larger than the shear obtained above. In accordance with 2.3.2.2, the strength reduction factor is not used, that is, $\phi = 1.0$, when, as is the case above, earthquake-induced shear forces are derived from a capacity design procedure.

Attention should be given to spandrel beams supporting a roof when earthquake induced shear forces opposite to those due to gravity loads are large. In such cases the diagonal failure plane may extend into the beam-column joint area where adequate vertical shear reinforcement must be provided. The situation is similar to that shown in Figure C9.8 with the figure turned through 180°. A2

C9.4.4.1.3 *Nominal shear strength provided by concrete in plastic hinge regions of beams*

It is assumed that the shear resistance provided by the concrete is negligible in plastic hinge regions of beams of ductile structures, since reversal of actions in yielding regions cause significant degradation of the shear resisted by the concrete mechanisms. Hence, in potential plastic hinge regions transverse reinforcement is required for the full shear demand ($V_c = 0$). In the essentially elastic regions between the potential plastic hinges shear reinforcement may be designed as in 9.3.9.3.2. Stirrup-ties should be designed using Equation 9–7.

A2 **C9.4.4.1.4** to **C9.4.4.1.6** *Shear in ductile plastic regions*

Reversing plastic hinge regions may fail in shear by two different modes; namely sliding shear and conventional shear. Conventional shear is associated with diagonal tension cracking at an angle that is typically assumed to be at about 45° to the axis of the beam. These two shear failure mechanisms are illustrated in Figure C9.22(a).

The provisions of 9.4.4.1.4 and 9.4.4.1.5 present design criteria to control sliding shear and conventional shear failure. Clause 9.4.4.1.6 specifies the minimum shear reinforcement required over the full span of beams containing ductile plastic hinges (reversing and unidirectional), and 9.4.4.1.7 sets out the requirements for diagonally reinforced coupling beams.

The provisions of 9.4.4.1.4 have been made to safeguard against sliding shear failure in reversing plastic hinges. When the top and bottom reinforcement progressively yield under cyclic inelastic actions, elongation occurs and wide full depth cracks can develop through the web. These cracks significantly reduce both the diagonal compression strength of the concrete in the web and the interface shear transfer across the cracks. As a result, sliding shear displacements can develop across a section, which is normal to the axis of the beam; see Figure C9.22(a). As stirrups do not cross the failure section they cannot contribute significantly to sliding shear strength. However, diagonal reinforcement in the web can provide restraint to sliding by acting in either tension or compression[9.41].

Clause 9.4.4.1.4(a) specifies the maximum shear force which may be sustained in reversing plastic hinges unless they are designed as diagonally reinforced coupling beams satisfying the requirements of 9.4.4.1.7. Subclauses (b) and (c) of clause 9.4.4.1.4 define how much diagonal reinforcement is required to control sliding shear, and (d), (e) and (f) give the requirements that the reinforcement must satisfy to enable the diagonal reinforcement to act effectively in the plastic region.

Figure C9.22(b) illustrates how diagonal reinforcement is fitted into a reversing plastic region.

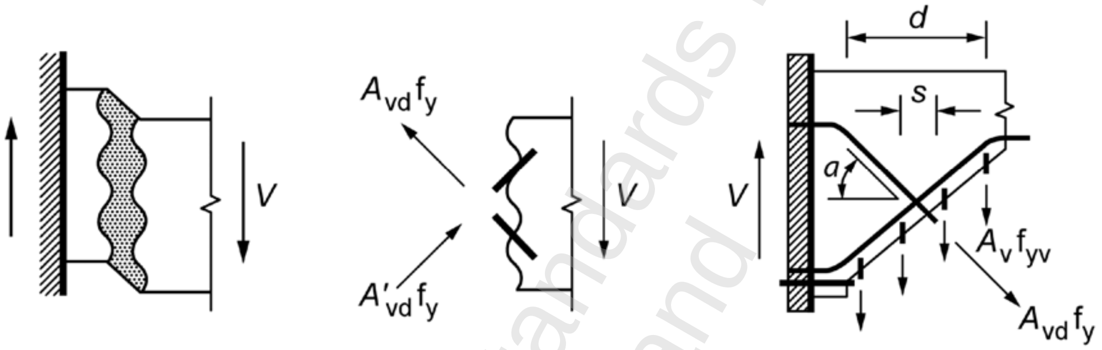
The primary purpose of diagonal web reinforcement in this case is to effectively cross every potential wide full depth crack after the flexural reinforcement in both faces of a member has yielded. The traditional truss mechanism does not prevent sliding shear failure. A rational analysis is required to show that the vertical component of the diagonal reinforcement across each section of a potential reversing plastic hinge within a distance of, d, away from the critical section of the plastic region, is equal to or larger than the shear force, V_{di}, required at that section. Examples of diagonal reinforcement are shown in Figure C9.22(b). When inclined bars are required to resist shear in both directions in a beam subjected to large earthquake shear forces and relatively small gravity shears, the vertical component of both inclined tension and compression bars may be assumed to contribute to sliding shear resistance [9.32, 9.42]. Where diagonal bars act in compression to provide resistance to sliding shear, they must be anchored to prevent them breaking out of the top and bottom surfaces of the beam. Additional stirrups may be required to resist the transverse component of the force in these bars, as specified in (f), and shown as a dashed lines in Figure C9.22(b). A2

In the evaluation of the flexural overstrength of a plastic hinge, the contribution of any diagonal reinforcement to this strength should be considered.

Conventional shear failure occurs as a result of separation along a diagonal crack, which is generally considered to develop at an angle of close to 45°, see the right hand side of Figure C9.22(a)(ii). Clause 9.4.4.1.5 considers this form of failure, and notes that diagonal reinforcement that may have been provided to resist sliding shear in compression will not intersect the diagonal failure plane, and consequently it cannot contribute to the (conventional) shear strength.

C9.4.4.1.7 *Diagonally reinforced coupling beams*

See C11.4.9.4. A3

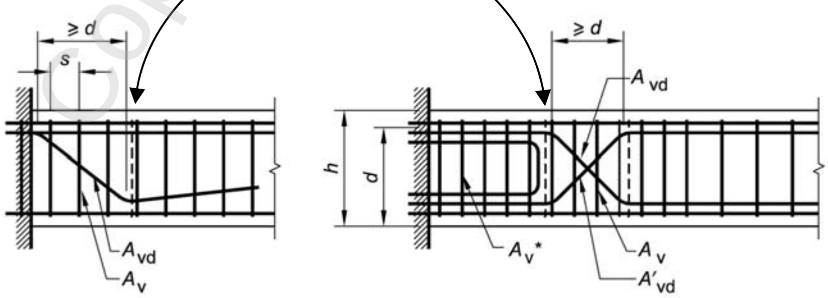


$$V_s = A_v f_{yv} \frac{d}{s} + A_{vd} f_y \sin\alpha$$

(i) Sliding shear and action of diagonal reinforcement

(ii) Conventional shear

(a) Sliding shear and conventional shear in ductile plastic regions



$$V_{di} = A_{vd} f_y \sin\alpha \qquad V_{di} = (A_{vd} + A'_{vd})\, f_y \sin\alpha$$

(b) Diagonal web reinforcement to control sliding shear

Figure C9.22 – Shear failure in reversing plastic hinges

A2

A3

C9.4.5 *Buckling restraint of longitudinal bars in potential ductile and limited ductile plastic regions*

To ensure that compression bars in beams cannot buckle when subjected to yield stress, they must be restrained by a 90° bend of a stirrup-tie as shown in Figure C9.23(a). It is seen that bars numbered 1 and 2 are well restrained. Bar 3 need not be tied because the centre-to-centre distance between adjacent tied bars is less than 200 mm. This, however, will affect the size of the ties holding bars numbered 2 as stipulated in ΣA_b in Equation 9–28.

It is considered that the capacity of a tie in tension should not be less than one-sixteenth of the force at yield in the bar or group of bars it is to restrain at $6d_b$ centres. For example the area of the tie restraining the corner bars shown in Figure C9.23(a) should be $A_{te} = A_1/16$ assuming the yield strength for all bars is the same. However, the area of the inner ties must be $A^*_{te} = (A_2 + 0.5A_3)/16$ because they must also give some support to the centrally positioned bar marked 3. In computing the value of ΣA_b the tributary area of the unrestrained bars should be based on their position relative to the two adjacent ties.

Figure C9.23(b) shows a beam with eight bottom bars of the same size, A_b. Assuming again that $f_y = f_{yt}$, the area of the identical ties will be $A_{te} = 2A_b/16$ because the second layer of bars is centred at less than 75 mm from the horizontal inside legs of the stirrup-ties. The inner vertical ties for the bars shown in Figure C9.23(c), however, need only support one longitudinal bar because the second layer is centred more than 75 mm from the inside of stirrup-ties.

The outer bars situated in the second or third layers in a beam may buckle outward if they are situated too far from a horizontal leg of a stirrup-tie. This situation is illustrated in Figure C9.23(c), which shows a single horizontal tie in the third layer, because these outer bars are further than 100 mm from the bottom horizontal leg of the stirrup-ties. The inner four bars need not be considered for restraint because they are situated further than 75 mm from any tie leg. The outer bars in the second layers shown in Figure C9.23(b) and (c) are considered satisfactorily restrained against horizontal buckling as long as they are situated no further than 100 mm from the horizontal leg at the bottom of the stirrup-ties.

The limitations on maximum spacing are to ensure that longitudinal bars are restrained adequately against buckling and that the concrete has reasonable confinement. The limitations are more severe if longitudinal bar yielding can occur in both tension and compression, for the reasons explained previously.

In potential plastic hinge regions in the ends of beams considerable stirrup reinforcement may be required to resist shear. All full-depth vertical legs of stirrup-ties required according to should also be considered to contribute to shear resistance.

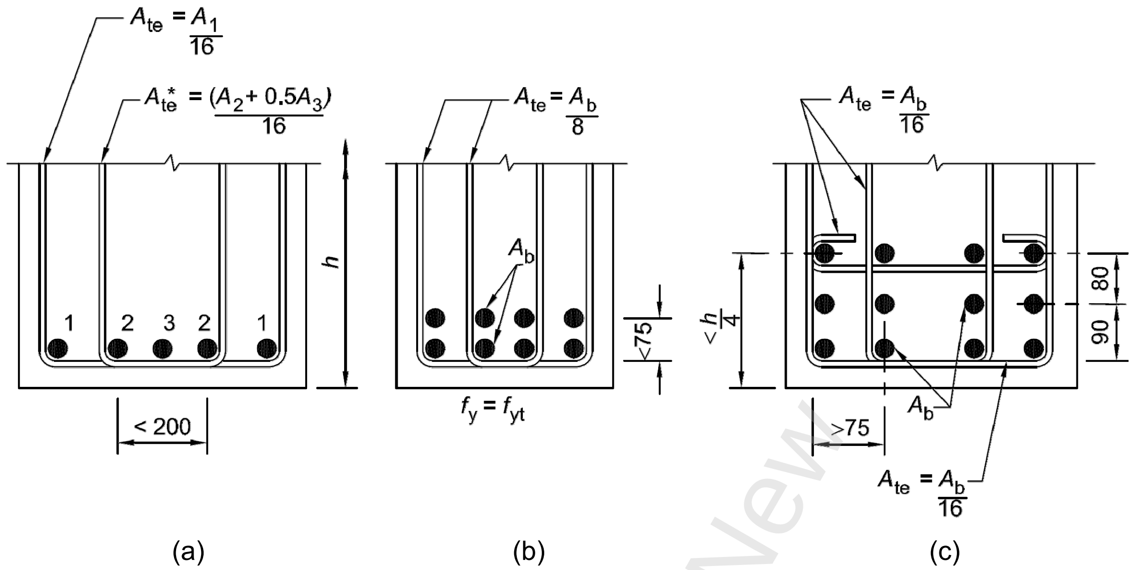


Figure C9.23 – The arrangement and size of stirrup-ties spaced at 6d_b between centres in potential plastic hinge regions

REFERENCES

9.1 Cheung, P.C., Paulay, T. and Park, R., "Mechanisms of Slab Contributions in Beam-column Subassemblages", Design of Beam-Column Joints for Seismic Resistance, Special Publication SP-123, ACI, Detroit, 1991, pp. 259-289.

9.2 Chow, L., Conway H. and Winter, G., "Stresses in Deep Beams," Transactions, ASCE, Vol. 118, 1953, pp. 686-708.

9.3 "Design of Deep Girders", ISO79D, Portland Cement Association, Skokie, IL, 1946, p. 10.

9.4 Park, R. and Paulay, T., "Reinforced Concrete Structures", Wiley-Inter-Science, New York, 1975, 769 pp.

9.5 Gergely, P. and Lutz, L.A., "Maximum Crack Width in Reinforced Concrete Flexural Members, Causes Mechanism, and Control of Cracking in Concrete", SP-20, American Concrete Institute, Farmington Hills, MI, 1968, pp. 87-117.

9.6 Kaar, P.H., "High Strength Bars as Concrete Reinforcement, Part 8: Similitude in Flexural Cracking of T-Beam Flanges", Journal, PCA Research and Development Laboratories, Vol. 8, No. 2, May 1966, pp. 2–12.

9.7 Base, G.D., Reed, J.B., Beeby, A.W. and Taylor, H.P.J., "An Investigation of the Crack Control Characteristics of Various Types of Bar in Reinforced Concrete Beams", Research Report No. 18, Cement and Concrete Association, London, Dec. 1966, 44 pp.

9.8 Beeby, A.W., "The Prediction of Crack Widths in Hardened Concrete", The Structural Engineer, Vol. 57A, No. 1, Jan. 1979, pp. 9–17.

9.9 Frosch, R.J., "Another Look at Cracking and Crack Control in Reinforced Concrete", ACI Structural Journal, Vol. 96, No. 3, May–June 1999, pp. 437-442.

9.10 Collins, M.P. and Kuchma, D., "How Safe are our Large Lightly Reinforced Concrete Beams, Slabs and Footings?", ACI Structural Journal, Vol. 96, No. 4, July-Aug. 1999, pp. 482-490.

9.11 Kani, G.N.J., "How Safe are our Large Reinforced Concrete Beams?", ACI Journal Proceedings, Vol. 64, No. 3, Mar. 1967, pp. 128-141.

9.12 British Standards, BS 8110- 1985, "Structural Use of Concrete", British Standards Institution, London.

9.13 AS 3600, 1994, "Concrete Structures", Standards Australia, Sydney.

9.14 Commission of European Communities, "Eurocode 2: Design of Concrete Structures", ENV 1-5:1996.

9.15 Johnson, M.K. and Ramirez, J.A., "Minimum Amount of Shear Reinforcement in High Strength Concrete Members", ACI Structural Journal, Vol. 86, No. 4, July–Aug. 1989, pp. 376–382.

9.16 Ozcebe, G., Ersoy, U. and Tankut, T., "Evaluation of Minimum Shear Reinforcement for Higher Strength Concrete", ACI Structural Journal, Vol. 96, No. 3, May–June 1999, pp. 361–368.

9.17 Ivey, D.L. and Buth, E., "Shear Capacity of Lightweight Concrete Beams", ACI Journal, Proceedings Vol. 64, No. 10, Oct. 1967, pp. 634-643.

9.18 Hanson, J.A., "Tensile Strength and Diagonal Tension Resistance of Structural Lightweight Concrete", ACI Journal, Proceedings Vol. 58, No. 1, July 1961, pp. 1-40.

9.19 ACI Committee 426, "The Shear Strength of Reinforced Concrete Members", Proceeding of American Society of Civil Engineers, Vol. 99, ST6, June 1973, pp 1091-1187, and Vol. 100 ST8, Aug. 1994, pp. 1543-1591.

9.20 Anderson, N.S. and Ramirez, J.A., "Detailing of Stirrup Reinforcement", ACI Structural Journal, Vol. 86, No. 5, September-October 1989, pp. 507–515.

9.21 Leonhardt, F. and Walther, R., "The Stuttgart Shear Tests", C & CA Translation No. 111, Cement and Concrete Association, London, 1964, 134 pp.

9.22 Rogowsky, D.M. and MacGregor, J.G., "Design of Reinforced Concrete Deep Beams," Concrete International: Design and Construction, Vol. 8, No. 8, Aug. 1986, pp. 46–58.

9.23 Marti, P., "Basic Tools of Reinforced Concrete Beam Design," ACI Journal, Proceedings, Vol. 82, No. 1, Jan.–Feb. 1985, pp. 46–56.

9.24 Crist, R.A., "Shear Behaviour of Deep Reinforced Concrete Beams," Proceedings, Symposium on the Effects of Repeated Loading of Materials and Structural Elements (Mexico City, 1966), Vol. 4, RILEM, Paris, 31 pp.

9.25 Beattie, G.J., "Reinforced Concrete Beams with Large Penetrations under Cyclic Loading", Proceedings New Zealand Concrete Society 1991 Annual Technical Conference, Wairakei, pp. 90–100.

9.26 "Code of Practice for the Structural Use of Concrete. Part 1, Design Materials and Workmanship", (CP 110: Part 1, Nov. 1972) British Standards Institution, London, 1972, 154 pp.

9.27 Cheung, P.C., Paulay, T. and Park, R., "Seismic Design of Beam-Column Joints with Floor Slab", Research Report 91-4, Department of Civil Engineering, University of Canterbury, October 1991, 328 pp.

9.28 Qi, Xr, and Pantasopoulou, S.J., "Response of RC frames under Lateral Loads", Journal of Structural Engineering, ASCE, 1991, Vol. 117, No. 4, pp. 1167-1188.

9.29 MacPherson, C., "Seismic performance and forensic analysis of a precast concrete hollow-core floor super-assembledge", ME thesis, University of Canterbury, 2005.

9.30 Fenwick, R C., Davidson, B J. and Lau, D B N., "Interaction Between Ductile RC Perimeter Frames and Floor Slabs Containing Prestressed Units", NZSEE Technical Conference, March 2005, paper No. 23, 13 pp.

9.31 Fenwick, R.C. Davidson, B.J. and McBride, A., "The influence of slabs on elongation in ductile seismic resistant frames", NZ National Society for Earthquake Engineering Technical Conference Proceedings, Rotorua, March 1995, pp. 36-43.

9.32 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced Concrete and Masonry Buildings", Wiley Interscience, New York, 1992, 774 pp.

9.33 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley and Sons, 1992, p. 767.

9.34 Park, R. and Dai Ruitong., "Ductility of Doubly Reinforced Concrete Beam Sections", Structural Journal, ACI, Vol. 85, No. 2, March-April 1988, pp. 217-225.

9.35 Blakeley, R.W.G., Megget, L.M. and Priestley, M.J.N., "Seismic Performance of Two Full-Size Reinforced Concrete Beam-Column Joint Units", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 8, No. I, March 1975, pp. 38-69.

9.36 Blakeley, R.W.G., Edmonds, F.D., Megget, L.M. and Wood, J.H., "Cyclic Load Testing of Two Refined Reinforced Concrete Joints", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 3, September 1979, pp. 238-255.

9.37 Park, R. and Dai Ruitong, "A Comparison of the Behaviour of Reinforced Concrete Beam-column Joints Designed for Ductility and Limited Ductility", Bulletin of the New Zealand National Society for Earthquake Engineering. Vol. 21, No. 4, December 1988, pp. 255-278.

9.38 Cheung, P.C., Park, R. and Paulay, T., "Some Possible Revisions to the Seismic Provisions of the New Zealand Concrete Design Code of Moment Resisting Frames", Pacific Conference on Earthquake Engineering, Proceedings, Vol. 2, Auckland, November 1991, pp. 79-90.

9.39 Xin, X.Z., Park, R. and Tanaka, H., "Behaviour of Reinforced Concrete Interior Beam-column Joints Designed Using High Strength Concrete and Steel," Research Report 92-3, Department of Civil Engineering, University of Canterbury, New Zealand, 1992.

9.40 Hakuto, S., Park, R. and Tanaka, H., "Effect of Deterioration of Bond of Beams Bars Passing through Interior Beam-Column on Flexural Strength and Ductility", Structural Journal of ACI Vol. 96, No. 5, September–October, 1999, pp. 858–864.

9.41 Paulay, T. and Bull, I.N., "Shear Effects on Plastic Hinges of Earthquake Resisting Reinforced Concrete Frames", Comité Euro-International du Béton, Bulletin D'Information No. 132, April 1979, pp. 165-172.

9.42 Restrepo, J.I., Park, R. and Buchanan, A.H., "Design of Connections of Earthquake Resisting Precast Reinforced Concrete Perimeter Frames of Buildings, Journal of PCI, Vol. 40, No. 5, September–October, 1995, pp. 68–77.

9.43. Fenwick, R., Bull D. and Gardiner, D., "Assessment of Hollow-Core Floors for Seismic Performance", Research Report 2101-2, Civil and Natural Resources Engineering, University of Canterbury, 2010, p. 151. A3

Table C9.3 – Design of reinforced concrete beams (excluding deep beams)

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy	
Material limitation applicable to the detailing described in this table	Range limitation on concrete compressive strength, f'_c	25 to 100 MPa (5.2.1)	25 to 70 MPa for ductile elements (5.2.1)	
	Limitation on longitudinal reinforcement yield strength, f_y	Not greater than to 500 MPa (5.3.3)	Same as for nominally ductile	
	Limitation on transverse reinforcement yield strength, f_y	Not greater than 500 MPa for shear and 800 MPa for confinement (5.3.3)	Same as for nominally ductile	
	Reinforcement class as per AS/NZS 4671	Class E, unless conditions for Class N are satisfied (5.3.2.3)	Same as for nominally ductile	
Ductility	Curvature ductility achievable through tabled detailing	Depends whether flexure or shear controls strength - see 2.6.1.3	$K_d \dfrac{2f_y}{E_s h}$ with $f_y \le 425\text{MPa}$; For K_d see Table 2.4(a) and (b)	
Dimensional limitations	For stability	Spacing of lateral supports shall not exceed 50 times the least lateral dimension. Effects of lateral eccentricity of loads need to be taken into account (9.3.5)	Rectangular beams	$b_w \ge L_n/25$ or $b_w \ge \sqrt{(L_n h/100)}$ (9.4.1.2)
			Cantilevered rectangular beams	$b_w \ge L_n/15$ or $b_w \ge \sqrt{(L_n h/60)}$ (9.4.1.3)
			T - , L - beams	$b_w \ge L_n/37.5$ (9.4.1.4)
			Cantilevered T - , L - beams	$b_w \ge L_n/22.5$ (9.4.1.4)
	Minimum depth	Refer Table 2.1 and Table 2.2 unless calculations of deflections indicate a lesser thickness may be used without adverse effects (2.4.3)	Same as nominally ductile	
Plastic hinge region	Extent of plastic hinge for detailing purposes, ductile detailing length	Not applicable	Location A and B (See Notes (1) and (2)) – $2h$ (9.4.2) Location C (see Note (3)) – $4h$ at end of table	
Strength reduction factors	Strength reduction factors	Refer 2.3.2.2	ϕ = 1.0 when actions derived from overstrengths (2.3.2.2)	

Table C9.3 – Design of reinforced concrete beams (excluding deep beams) (Continued)

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy
Material limitation applicable to the detailing described in this table	Range limitation on concrete compressive strength, *f´c*	25 to 100 MPa (5.2.1)	25 to 70 MPa for ductile elements (5.2.1)
	Limitation on longitudinal reinforcement yield strength, *fy*	Not greater than to 500 MPa (5.3.3)	Same as for nominally ductile
	Limitation on transverse reinforcement yield strength, *fy*	Not greater than 500 MPa for shear and 800 MPa for confinement (5.3.3)	Same as for nominally ductile
	Reinforcement class as per AS/NZS 4671	Class E, unless conditions for Class N are satisfied (5.3.2.3)	Same as for nominally ductile
Ductility	Curvature ductility achievable through tabled detailing	Depends whether flexure or shear controls strength - see 2.6.1.3	$K_d \frac{2 f_y}{E_s h}$ with $f_y \leq 425\text{MPa}$; For *Kd* see Table 2.4(a) and (b)
Dimensional limitations	For stability	Spacing of lateral supports shall not exceed 50 times the least lateral dimension. Effects of lateral eccentricity of loads need to be taken into account (9.3.5)	Rectangular beams: $bw \geq Ln/25$ or $bw \geq \sqrt{(L_n h/100)}$ (9.4.1.2) Cantilevered rectangular beams: $bw \geq Ln/15$ or $bw \geq \sqrt{(L_n h/60)}$ (9.4.1.3) T - , L - beams: $bw \geq Ln/37.5$ (9.4.1.4) Cantilevered T - , L - beams: $bw \geq Ln/22.5$ (9.4.1.4)
	Minimum depth	Refer Table 2.1 and Table 2.2 unless calculations of deflections indicate a lesser thickness may be used without adverse effects (2.4.3)	Same as nominally ductile
Plastic hinge region	Extent of plastic hinge for detailing purposes, ductile detailing length	Not applicable	Location A and B (See Notes (1) and (2)) – 2h (9.4.2) Location C (see Note (3)) – 4h at end of table
Strength reduction factors	Strength reduction factors	Refer 2.3.2.2	ϕ = 1.0 when actions derived from overstrengths (2.3.2.2)

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy
Overstrength factors for reinforcement and concrete		Not applicable	Clause 2.6.5.5 Reinforcement Grade 300E 1.25 f_y Grade 500E 1.35 f_y Concrete (f'_c+15)
T-beams-(slab and web built integrally or otherwise effectively bonded)	Width of slab assumed to be effective as a T-beam compression flange resisting compressive stress due to flexure	Clause 9.3.1.2(a), lesser of – $^1/_4$ the span length of the beam; plus width of web, or The effective overhanging slab width on each side of the web shall not exceed: (i) Eight times the slab thickness, nor (ii) Total depth of beam (iii) The clear distance to the next web multiplied by $\left(\frac{h_{b1}}{h_{b1}+h_{b2}}\right)$	Same as for nominally ductile
	Width of slab used to calculate effective moment of inertia or cracked section	One-half of that above (9.3.1.3)	Same as for nominally ductile
L-beams – (slab and web built integrally or otherwise effectively bonded)	Width of slab assumed to be effective as a L-beam compression flange resisting compressive stress due to flexure	Clause 9.3.1.2(b), the width of overhanging slab shall not exceed the lesser of: (i) One-eighth the span length of the beam, nor (ii) Eight times the slab thickness, nor (iii) The depth of the beam (iv) The clear distance to the next web multiplied by $\left(\frac{h_{b1}}{h_{b1}+h_{b2}}\right)$	Same as for nominally ductile
	Width of slab used to calculate effective moment of inertia of cracked sections	One-half of that above (9.3.1.3)	Same as for nominally ductile

<table>
<tr>
<td>T - and L - beams (slab and web built integrally or otherwise effectively bonded)</td>
<td>Width of slab within which effectively anchored longitudinal slab reinforcement shall be considered to contribute to negative moment flexural strength of the beam in addition to those longitudinal bars placed within the web width of the beam</td>
<td>See clause 9.3.1.4
Lesser of:
(a) by rational analysis with slab reinforcement tied into beam web, or
(b)(i) contribution of reinforcement in overhang to flexural strength shall be less than 15 % per overhanging flange.
(ii) lesser of
(A) beam depth
(B) 8 times slab thickness
(C) clear distance between beams
multiplied by $\left(\frac{h_{b1}}{h_{b1}+h_{b2}}\right)$
(c) Where beam frames into free edge refer 9.3.1.4.</td>
<td colspan="2">For strength
Same as nominally ductile, but contribution of reinforcement in each overhanging flange reduced to 10 %, 9.4.1.6.1.

For overstrength
Refer 9.4.1.6.2</td>
</tr>
<tr>
<td rowspan="3">Longitudinal reinforcement detailing</td>
<td rowspan="2">Minimum longitudinal reinforcement</td>
<td rowspan="2">$A_s = \frac{\sqrt{f_c'}}{4f_y} b_w d$ but greater than $1.4\, b_w d / f_y$
(9.3.8.2.1)
Unless A_s is ⅓ greater than required by analysis (9.3.8.2.3)
For T - beams with flange in tension replace b_w with smaller of $2b_w$ or flange width. (9.3.8.2.2)
Where beams resist seismic actions minimum of 2 bars or ¼ of maximum negative moment reinforcement. (9.3.8.2.1)</td>
<td colspan="2">Greater of:
(i) $p_{min} = \frac{\sqrt{f_c'}}{4f_y}$ (9.4.3.4(b))
(ii) At least $^1/_4$ of the larger of the top reinforcement at each end shall continue or two 16 mm diameter bars provided in both top and bottom throughout the length of the beam (9.4.3.4(c))</td>
</tr>
<tr>
<td>Compression reinforcement in plastic hinge region</td>
<td>$A_s' \geq 0.5\, A_s$ for DPR,
$A_s' \geq 0.38\, A_s$ for LDPR
Refer to 9.4.3.4(a) for exceptions</td>
</tr>
<tr>
<td>Maximum longitudinal reinforcement</td>
<td>Must limit neutral axis depth to $0.75c_b$ (9.3.8.1)</td>
<td>Tension reinforcement in ductile detailing length</td>
<td>The smallest of:
$p_{max} = \frac{f_c' + 10}{6f_y}$
or $p_{max} = 0.025$ where p shall be computed using the width of the web (9.4.3.3)</td>
</tr>
</table>

			Outside plastic hinge region	Same as for nominally ductile
	Spacing between longitudinal reinforcement	Shall be well distributed in tension zone (9.3.6.2) and shall satisfy 2.4.4.	Same as for nominally ductile	
	Minimum spacing between individual, bundles, or the bundled bars in a contact splice	Clear distance to exceed greater of 25 mm d_b, or 1.33 times the nominal maximum aggregate size (8.3.1 and 8.3.2)	Same as for nominally ductile	
	Spacing requirements for layers	Place directly above bars in lower layer with clear distance between layers of greater of 25 mm or d_b (8.3.3)	Same as for nominally ductile	
	Maximum numbers of bars in a bundle	4 (8.3.4)	Same as for nominally ductile	
	Maximum diameter of bars that can be bundled	32 mm (8.3.4)	Same as for nominally ductile	
	Maximum bar diameters at simple supports	The positive tension reinforcement at simple supports shall be limited in diameter to enable the bars extending to the free end of the member to be fully developed from a point M_n/V^* from the centre of the support. The value of M_n/V^* shall be calculated at the centre of the support and may be increased by 30 % when the ends of reinforcement at the support are confined by a compressive reaction. (8.6.13.3)	Same as for nominally ductile	
	Bar diameters at points of inflection	Bar diameters at points of inflection shall be limited to ensure that: $L_d \leq \frac{M_n}{V^*} + 12d_b$ and $L_d \leq \frac{M_n}{V^*} + d$ (8.6.13.4)	Same as for nominally ductile	

	Bar diameters through interior beam-column joints	$\frac{d_b}{h_c} = 4\alpha_t \frac{\sqrt{f_c'}}{f_y}$ (9.3.8.4) or as given by Equation (9–3) where earthquake load cases do not govern.	Where hinges form in the beams at the column face and $\delta_c \leq 1.8$ % or f_y = 300 MPa $\frac{d_b}{h_c} \leq 3.3\alpha_f \alpha_d \frac{\sqrt{f_c'}}{1.25 f_y}$ (9.4.3.5.2) Multiply by $\gamma = (1.53 - 0.29\delta_c)$ where δ_c > 1.8 % and f_y = 500 (See 9.4.3.5.1 and 9.4.3.5.2 for details)
	Distribution of bars in beams greater than 1 m deep	Refer (9.3.6.3)	Same as for nominally ductile
	Distribution of tension reinforcement in T- beams	Where the flanges of T - beams are in tension, part of the flexural reinforcement may be placed in the smaller of the effective width defined in this table, or one tenth the span (9.3.1.4)	Refer 9.4.1.6
Splicing of reinforcement	Splices within plastic hinge regions	Not applicable	Splicing of reinforcement in plastic hinge regions or beam-column joints shall not be permitted with the exception that reinforcement may be spliced in these areas by full strength butt welds (8.9.1.1 and 8.7.4)
	Splices in areas of reversing stress	Not applicable	Reinforcement shall not be spliced by lapping in a region where reversing stresses at the ultimate limit state may exceed 0.6 f_y in tension or compression unless each spliced bar is confined by stirrup-ties so that: $\frac{A_{tr}}{s} \geq \frac{d_b f_y}{48 f_{yt}}$ (8.9.1.2)
Curtailment and anchorage of longitudinal reinforcement	Curtailment of longitudinal reinforcement within span	Reinforcement shall extend the greater of the following distances: (a) $L_d + d$ beyond where full strength of the reinforcement is required for: (b) 1.3d past the point where the reinforcement in question is no longer required to resist flexure (8.6.12.3)	The distribution and curtailment of the longitudinal flexural reinforcement shall be such that the flexural overstrength of a section can be attained at critical sections in potential plastic hinge regions (9.4.3.1)

	Curtailment in a tension zone, with the exception of tension splices, only allowed if one of the following is satisfied:	(a) Shear at the cut-off point is less than $^2/_3$ of the shear strength provided by the concrete; or (b) The shear strength provided by the web reinforcement , V_s, measured for a distance of 1.3d along the terminating bar from the cutoff point is equal to or greater than $V_s = 1.2\frac{\sqrt{f_c'}}{16} b_w d$ and the spacing, s, of stirrups or ties is equal to or less than the smaller of d/2 or $\frac{d}{8\beta_b}$. (8.6.12.4)		Same as for nominally ductile
	Curtailment of bundled bars	Individual bars in a bundle cut off within the span of flexural members shall terminate at different points with at least 40 bar diameter stagger (8.3.4)		Same as for nominally ductile
	Amount of positive reinforcement that shall extend along the same face of the member into the support	Simply supported members	At least ⅓ of the maximum positive moment reinforcement (8.6.13.1)	Simply supported, As for nominally ductile.
		Continuous members	At least $^1/_4$ of the maximum positive moment reinforcement (8.6.13.1)	Continuous members Same as for nominally ductile, but not less than two 16 mm bars in the top and bottom of beam and requirements in (9.4.3.4)
	Anchorage of positive moment reinforcement	Positive moment reinforcement shall extend at least 150 mm into the support, or if part of the primary horizontal force-resisting system, shall be anchored to develop f_y in tension at the support. (8.6.13.1 and 8.6.13.2)		Reinforcement shall be anchored to ensure that plastic deformation is confined to potential plastic regions flexural overstrengths can be attained in plastic hinge regions (9.4.3.1)

	Development of positive moment reinforcement at simple supports	The positive tension reinforcement at simple supports shall be limited in diameter to enable the bars extending to the free end of the member to be fully developed from a point M_n/V^* from the centre of the support. The value of M_n/V^* shall be calculated at the centre of the support and may be increased by 30 % when the ends of reinforcement at the support are confined by a compressive reaction (8.6.13.3)	Same as nominally ductile	
	Development of positive and negative reinforcement at points of inflection	Bar diameters at points of inflection shall be limited to ensure that: $L_d \le \frac{M_n}{V^*} + 12d_b$ and $L_d \le \frac{M_n}{V^*} + d$ (8.6.13.4)	Same as nominally ductile	
	Curtailment of negative moment reinforcement	At least ⅓ the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection, for a distance equal to or greater than 1.3d (8.6.14.3). See 9.3.8.2.1 for minimum longitudinal reinforcement throughout the length.	The greater of $^1/_4$ of the maximum negative moment reinforcement in the beam, 2x 16 mm bars or the value given by Equation 9–1, shall be continued throughout its length (9.4.3.4(c) and 9.3.8.2.1).	
	Anchorage of beam bar in columns considered to commence at:	The point of maximum stress, normally the face of the column (8.6.12.2)	Where beam plastic hinge forms at column face	Anchorage is deemed to commence lesser of half column depth or 8d_b (9.4.3.2.1)
			Where beam plastic hinge is located greater than 500 mm or beam depth from column face	At column face (9.4.3.2.1)
Transverse reinforcement, <u>outside</u>	Minimum diameter for transverse reinforcement	Shall be at least 5 mm (9.3.9.2)	Same as for nominally ductile	

plastic hinge regions	Compression longitudinal bars requiring restraint	Support – (i) Each corner bar (ii) Bars greater than 150 + d_b from a restrained bar. (iii) At least every alternative bar for spacing less than above (9.3.9.6.3)	
	Maximum spacing of stirrups for anti-buckling	Smaller of 16d_b or least lateral dimension (9.3.9.6.2)	
	Maximum spacing for shear reinforcement	0.5d, or 600 mm, or where b_w exceeds 0.5d, larger of 0.5d or 250 mm (9.3.9.4.12 (a) and (b)). Half this spacing when V_s exceeds $0.33\sqrt{f'_c}\, b_w d$ (9.3.9.4.12(d))	
	Maximum spacing of torsional reinforcement	Lesser of p_o/8 or 300 mm (7.6.3.2)	
	Area of transverse reinforcement	Dictated by above clauses and considerations for shear, torsion, and anti-buckling (9.3.9.1)	
	Shear reinforcement	$A_v = \dfrac{V_s s}{f_{yt} d}$ (9.3.9.4.2)	
	Minimum shear reinforcement	When required by 9.3.9.4.13 $A_v = \dfrac{1}{16}\sqrt{f'_c}\,\dfrac{b_w s}{f_{yt}}$	For beams $A_v = \dfrac{1}{12}\sqrt{f'_c}\,\dfrac{b_w s}{f_{yt}}$
	Shear carried by reinforcement	$V_s = V_n - V_c$, where $V_n \leq 0.2 f'_c\, b_w d$ or 8 $b_w d$ (7.5.2)	

	Shear strength provided by concrete	$v_c = k_d k_a v_b$ where $v_b = (0.07+10p_w)\sqrt{f_c'}$, but is not less than $0.08\sqrt{f_c'}$, or greater than $0.2\sqrt{f_c'}$, except where d is less than 400 mm when v_c can be increased (see 9.3.9.3.4 for details). k_a = 1.0 for 20 mm aggregate and 0.85 for 10 mm aggregate. k_d = 1.0 where effective depth < 400 mm or minimum shear reinforcement provided, otherwise refer to (9.3.9.3.4)	With minimum spacing of 12 d_b or d/2, where d_b, is diameter of longitudinal reinforcement. (9.4.4.1.6). As for nominally ductile, but v_c = 0 in ductile plastic regions, and in limited ductile plastic regions, half the corresponding value for nominally ductile plastic regions (9.4.4.1.3).
	Shear reinforcement details	Refer (7.5.6)	
	Torsion reinforcement required for equilibrium or compatibility See 7.6.1.2 for axial load or prestress	Torsional reinforcement required if $T^* \geq 0.1\phi A_{co} t_c \sqrt{f_c'}$, provided stiffness reduction neglected for compatibility torsion.	
	Equilibrium torsion, reinforcement required (7.6.4.2). See 7.6.1.2 for axial load or prestress	$A_t = \dfrac{T_n s}{2 A_o f_{yt}}$ & $A_\ell = \dfrac{T_n p_o}{2 A_o f_y}$ T_n greater of T^*/ϕ or $0.44 A_{co} t_c \sqrt{f_c'}$ Where A_ℓ and A_t are added to reinforcement required for flexure and shear	
	Compatibility torsional reinforcement See 7.6.1.2 or axial load or prestress	T_n smaller of T^*/ϕ or $0.44 A_{co} t_c \sqrt{f_c'}$ Where T^* is calculated neglecting decrease in stiffness due to cracking, and $\left(\dfrac{A_t f_{yt}}{s} \dfrac{A_\ell f_y}{p_o}\right)^{0.5} \geq \dfrac{T_n}{2A_o}$ with limits on ratio of A_ℓ to A_t (see 7.6.4.2). Areas of A_ℓ and A_t may include reinforcement for flexure and shear.	
	Spacing of longitudinal reinforcement	Longitudinal reinforcement shall be distributed around the perimeter and closed stirrups with a maximum spacing of 300 mm (7.6.3.3)	

	Minimum diameter of longitudinal bars or strands in corners of stirrups	Greater than 10 mm or *s*/16 times stirrup spacing (7.6.3.4)	
Transverse reinforcement, <u>inside</u> plastic hinge regions-	Minimum diameter for transverse reinforcement	Shall be at least 5 mm (9.3.9.2)	Shall be at least 5 mm (9.4.5 (b))
	Anti-buckling reinforcement (restraint of compression reinforcement)	Same as outside plastic hinge region	$A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ (9.4.5 (b))
	Longitudinal bars requiring restraint	Same as outside plastic hinge region	All bars in top and bottom face, with exception of those between restrained bars at less than 200 mm centres (9.4.5(a))
	Maximum longitudinal spacing of stirrups	Anti-buckling – Same as outside plastic hinge region	Locations A and B plastic hinges (see Notes (1) and (2) Lesser of: (i) $d/4$; (ii) 6d_b (longitudinal bar) in DPR and 10 d_b for LDPR (9.4.5(d)) Location C plastic hinges. (See Note (3)) Lesser of: (i) $d/3$; (ii) 10d_b (longitudinal bar) (9.4.5(e))
		Shear – Same as outside plastic hinge region	Same as for nominally ductile
		Torsion – Same as for outside plastic hinge region	Same as for nominally ductile
	Area of transverse reinforcement	Dictated by considerations of shear, torsion and anti-buckling	Same as for nominally ductile
	Minimum shear reinforcement	Same as outside plastic hinge region	$A_v = \frac{\sqrt{f'_c}}{12 f_y} b_w s$ (9.4.4.1.6)

	Shear carried by reinforcement	Same as outside plastic hinge region	$V_s = \frac{V^*}{\phi} - V_c$, and V^* shall not exceed 0.85 $\sqrt{f'_c}\ b_w d$ unless entire force resisted by diagonal reinforcement (9.4.4.1.4(a)) and ϕ = 1.0 and capacity design used to generate design shears (2.3.2.2(a)). Where entire force resisted by diagonal reinforcement maximum same as for nominally ductile, unless requirements of 9.4.4.1.7 are satisfied, see 9.4.4.1.4 for details.
	Shear strength provided by concrete	Same as outside plastic hinge region	V'_c = 0 for DPR V'_c = 0.5 V_c for LDPR where V_c same as nominal ductile (9.4.4.1.3)
	Diagonal shear reinforcement required when	Not applicable	$V^*_o > 0.25\ (2+r)\ \sqrt{f'_c}\ b_w d$ (9.4.4.1.4 (b))
	Design for torsion	Same as outside plastic hinge region	Same as outside plastic hinge region
Serviceability consideration	Control of cracking	Control by limiting longitudinal tensile stress, distributing reinforcement, or assessing crack widths (2.4.4.1)	Same as for nominally ductile
	Deflection of beams by calculation-short-term	Refer 6.8	Same as for nominally ductile
	Additional long-term deflection	K_{cp} = 2/(1 + 50 p'/p) (6.8.3(b))	Same as for nominally ductile
	Deflection deemed to comply span to depth ratios	Refer 2.4.3	Same as for nominally ductile

NOTE –
Definition of plastic hinge locations (9.4.2)

(1) Location A Where the critical section is located at the face of a supporting column, wall or beam: over a length equal to twice the beam depth measured from the critical section toward mid-span, at each end of the beam where a plastic hinge may develop.

(2) Location B Where the critical section is located at a distance equal to or greater than either the beam depth h or 500 mm away from a column or wall face: over a length that commences between the column or wall face and the critical section, at least either 0.5h or 250 mm from the critical section, and extends at least 1.5h past the critical section toward mid-span.

(3) Location C Where, within the span, yielding of longitudinal reinforcement may occur only in one face of the beam as a result of inelastic displacements of the frame: over the lengths equal to twice the beam depth on both sides of the critical section.

NOTES

C10 DESIGN OF REINFORCED CONCRETE COLUMNS AND PIERS FOR STRENGTH AND DUCTILITY

C10.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 10 of the Standard:

k_c a factor applied to the ratio of ultimate curvature to curvature at first yield
N_c buckling load, N
ϕ_k stiffness reduction factor
ϕ_u ultimate curvature
ϕ_y curvature at first yield of tension reinforcement
μ structure ductility factor
ψ ratio of $\Sigma\, EI/L_u$ of columns to $\Sigma\, EI/L_n$ of beams in a plane at one end of columns

C10.3 General principles and design requirements for columns A3

C10.3.1 *Strength calculations at the ultimate limit state*

Corner and other columns subjected to known moments about each axis simultaneously should be designed for bi-axial bending and axial load.

Columns must be designed for the loadings which produce the most adverse combinations of axial loads and moments. For gravity loads, the combination of ultimate limit state loads on all floors above, which produces maximum axial force, and factored live load on a single adjacent span of the floor under consideration which produces the maximum bending moment is often a critical load combination. In addition, it is required to consider the case which produces the maximum ratio of moment to axial compression load. This is generally the chequerboard loading pattern in multi-storey structures which results in maximum column moments but at a somewhat lower than maximum axial force. Because of the non-linear nature of the column interaction relationship, both cases need to be examined to find which governs the design of the column.

In structures where the loading patterns and type of structural system result in bi-axial bending in compression members, the effect of moments about each of the principal axes must be considered and many programs have the facility to design sections with bi-axial actions. Where P-delta actions may be significant amplification of the bending moments may be required.

C10.3.2 *Slenderness effects in columns* A3

C10.3.2.1 *Design considerations for columns*

In a first order analysis the influence of the deflection of members on the bending moments that are sustained are not considered. However, this P-delta effect, which causes non-linear behaviour, is included in a second order analysis.

In a second order analysis for P-delta effects in a member, any reduction in stiffness due to creep, flexural cracking, non-linear behaviour of a section due to tension yielding of reinforcement or compression strains in the concrete appreciably exceeding the linear range, should be included. In general, the analysis should be based on a moment curvature relationship for each section. Such analyses may be carried out using appropriate software or by hand using successive approximations until convergence is achieved. When carrying out an analysis of slenderness effects (P-delta actions) tension stiffening of concrete should be ignored and the design loading should be based on the ultimate loads divided by the strength reduction factor. The structure is stable if the design actions are less than the nominal strengths.

A conservative alternative approach to using a full moment curvature relationship is to assume elastic behaviour with the elastic section properties modified to allow for the effects of creep and flexural cracking and ignoring tension stiffening. In this case the member may be assumed to be stable if the bending moment and axial load at all sections are less than those which cause either tension yielding or reinforcement or imply that compression stresses are sustained in the concrete stresses in excess of the compressive strength of the concrete.

A3 **C10.3.2.2** *Evaluation of slenderness effects in columns braced against sidesway*

As an alternative to the refined second-order analysis of 10.3.2.1, design may be based on elastic analyses and the moment magnifier approach [10.1].

C10.3.2.3 *Approximate evaluation of slenderness effects*

A3 This clause describes an approximate slenderness-effect design procedure for columns braced against sidesway and based on the moment magnifier concept. The moments computed by an ordinary first-order frame analysis are multiplied by a "moment magnifier" which is a function of the design axial load N^* and the critical buckling load N_c for the column. The design procedure embodies some of the similar design provisions for steel beam-columns in NZS 3404:1997, Structural Steel Standard. A first-order frame analysis is an elastic analysis that does not include the internal force effects resulting from deflections.

A3 Only columns braced against sidesway are considered, since columns and piers in unbraced frames will be subject to seismic forces and must not be designed using 10.3.2.3. The method given here allows only for elastic deformations with an allowance for concrete creep and does not account for plastic hinging which is specifically considered in earthquake frame design. Hence the method should not be used for members in which plastic hinging is expected to occur. Examples of compression members braced against sidesway are those located in a storey in which bracing elements such as structural walls carry almost all of the lateral seismic forces. With these stiff elements the lateral deflection of the storey, even when the stiff elements yield and deflect into the inelastic range, should not be great enough to cause the braced compression members to yield. The provisions of this clause are only applicable if the members braced against sidesway are not expected to yield under seismic forces.

Reference 10.2 gives some design aids for slender columns.

C10.3.2.3.2 *Effective length factor*

The moment magnifier equations were derived for pin ended columns and should be modified to account for the effect of end restraints. This is done by using an effective length kL_u in the computation of N_c.

The primary design aid to estimate the effective length factor k is the Jackson and Moreland Alignment Charts (Figure C10.1), which allow a graphical determination of k for a column of constant cross section in a multibay frame [10.3, 10.4].

The effective length is a function of the relative stiffness at each end of the compression member. Studies have indicated that the effects of varying beam and column reinforcement percentages and beam cracking should be considered in determining the relative end stiffnesses. In determining ψ for use in evaluating the effective length factor k, the flexural rigidity of the flexural members may be calculated on the basis of $0.35I_g$ for flexural members to account for the effect of cracking and reinforcement on relative stiffness, and $0.70I_g$ for compression members.

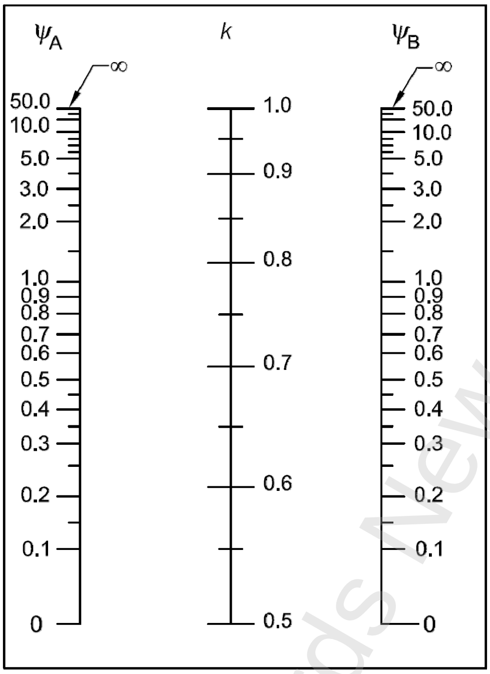
Simplified equations for computing the effective length factors for braced compression members have since been recommended by the 1972 British Standard Code of Practice [10.5]. An upper bound to the effective length for braced compression members may be taken as the smaller of the following two expressions according to that British Code:

$$k = 0.7 + 0.05\,(\Psi_A + \Psi_B) \le 1.0 \qquad \text{(Eq. C10–1)}$$

$$k = 0.85 + 0.05\,\Psi_{min} \le 1.0 \qquad \text{(Eq. C10–2)}$$

where Ψ_A and Ψ_B are the values of Ψ at the two ends of the column and Ψ_{min} is the smaller of the two values.

The use of the chart in Figure C10.1 or Equations C10–1 or C10–2 may be considered as satisfying the requirements of this Standard to justify k less than 1.0.



Where ψ = ratio of Σ (EI/L_u) of columns to Σ (EI/L_n) of beams in a plane at one end of a column

k = effective length factor

Figure C10.1– Effective length factors for braced frames

C10.3.2.3.4 *Consideration of slenderness*

Equation 10–2 is derived from Equation 10–4 assuming that a 5 % increase in moments due to slenderness is acceptable.[10.1]. The derivation did not include ϕ in the calculation of the moment magnifier. As a first approximation, k may be taken equal to 1.0 in Equation 10–2.

C10.3.2.3.5 *Design actions including slenderness effects*

The slenderness ratio limits, below which slenderness effects need not be considered in design, indicate that many stocky and sufficiently restrained compression members can essentially develop the full cross-sectional strength. The lower limit was determined from a study of a wide range of columns and corresponds to lengths for which a slender member strength of at least 95 % of the cross-sectional strength can be developed.

(a) The ϕ factors used in the design of slender columns represent two different sources of variability. First, the stiffness reduction ϕ factors in the magnifier equations in the 1989 and earlier ACI building codes were intended to account for the variability in the stiffness EI and the moment magnification analysis. Second, the variability of the strength of the cross section is accounted for by strength reduction ϕ factors for columns. Studies reported in Reference 10.6 indicate that the stiffness reduction factor ϕ_K, and the cross-sectional strength reduction ϕ- factors do not have the same values, contrary to the assumption in the 1989 and earlier ACI Building Codes. These studies suggest the stiffness reduction factor ϕ_K for an isolated column should be 0.75 for both tied and spiral columns. The 0.75 factor in Equation 10–4 is a stiffness reduction factor ϕ_K and replaces the ϕ factor in this equation in earlier codes. This has been done to avoid confusion between a stiffness reduction factor ϕ_K in Equation 10–4, and the cross-sectional strength reduction ϕ factors;

(b) In defining the critical load, the main problem is the choice of a stiffness EI that reasonably approximates the variations in stiffness due to cracking, creep, and the non-linearity of the concrete stress-strain curve. Equation 10–6 was derived for small eccentricity ratios and high levels of axial load where the slenderness effects are most pronounced.

Creep due to sustained load will increase the lateral deflections of a column and hence the moment magnification. This is approximated for design by reducing the stiffness EI used to compute N_c and hence δ by dividing EI by $(1 + \beta_d)$. Both the concrete and steel terms in Equation 10–6 are divided by $(1 + \beta_d)$. This reflects the premature yielding of steel in columns subjected to sustained load.

Either Equation 10–6 or 10–7 may be used to compute EI. Equation 10–7 is a simplified approximation to Equation 10–6. It is less accurate than Equation 10–6.[10.7]. Equation 10–7 may be simplified further by assuming $\beta_d = 0.6$. When this is done Equation 10–7 becomes $EI = 0.25 E_c I_g$.

For non-sway frames β_d is the ratio of the design axial sustained load to the total design axial load.

(c) The factor C_m is an equivalent moment correction factor. The derivation of the moment magnifier assumes that the maximum moment is at or near mid-height of the column. If the maximum moment occurs at one end of the column, design should be based on an equivalent uniform moment $C_m M_2$ that would lead to the same maximum moment when magnified [10.1].

In the case of compression members that are subjected to transverse loading between supports, it is possible that the maximum moment will occur at a section away from the end of the member. If this occurs, the value of the largest calculated moment occurring anywhere along the member should be used for the value of M_2 in Equation 10–3. In accordance with the last sentence of 10.3.2.3.5(c) C_m is to be taken as 1.0 for this case.

(d) In the Standard, slenderness is accounted for by magnifying the column end moments. If the design column moments are very small or zero, the design of slender columns should be based on the minimum eccentricity given in this section. It is not intended that the minimum eccentricity be applied about both axes simultaneously.

The column end moments from the structural analysis are used in Equation 10–8 in determining the ratio M_1/M_2 for the column when the design should be based on minimum eccentricity. This eliminates what would otherwise be a discontinuity between columns with computed eccentricities less than the minimum eccentricity and columns with computed eccentricities equal to or greater than the minimum eccentricity.

C10.3.2.3.6 *Bending about both principal axes*

When bi-axial bending occurs in a compression member, the computed moments about each of the principal axes must be magnified. The magnification factors δ are computed considering the critical buckling load N_c about each axis separately, based on the appropriate effective lengths (kL_u) and the related stiffness (EI). The clear column height may differ in each direction, and the stiffness ratios $\Sigma(EI/L_u)$ of columns to $\Sigma(EI/L_n)$ of flexural members may also differ, where L_u and L_n are the span lengths for columns and beams, respectively. Thus, the different buckling capacities about the two axes are reflected in different magnification factors. The moments about each of the two axes are magnified separately, and the cross section is then proportioned for axial load and bi-axial bending. Note that the moment, $M_c = \delta M_2$, refers to the "larger end moment" with respect to bending about one axis. It will usually be necessary, therefore, to magnify the moments at both ends of a column subjected to bi-axial bending, and to investigate both conditions at both ends.

A3 **C10.3.3** ***Design cross-sectional dimensions for columns***

The minimum sizes for compression members are not specified and thus reinforced concrete compression members of small cross section may be used in lightly loaded structures, such as low rise residential and light office buildings. The engineer should recognise the need for careful workmanship, as well as the increased significance of shrinkage stresses with small cross sections.

C10.3.4.1 *General assumptions for flexural and axial force design*

Normally the flexural strength of the member will be based on the cracked cross section, including the concrete cover in compression outside the transverse reinforcement. The assumptions of 7.4 apply. References 10.8, 10.2, 10.9 and others give theory and design aids. The use of an extreme fibre concrete compressive strain of 0.003, as specified in 7.4.2.3, will generally result in a satisfactory prediction for the flexural strength of a beam but may lead to a significant underestimate of the flexural strength of a column, particularly if high strength steel reinforcement is used. Design equations are not given here as they have become part of standard theory.

C10.3.4.2 *Limit for design axial force N^*, on columns* A3

The design axial load strength in compression with or without eccentricity is limited to 85 % of the design axial load strength without eccentricity in order to account for accidental eccentricities not considered in the analysis that may occur in a compression member and to recognise that at sustained high loads the concrete strength may be less than f_c'. The 85 % value approximates the axial load strength at an e/h ratio of 0.05, where e is the eccentricity of load parallel to the axis of the column measured from the centroid of the section and h is the section depth. The same axial load limitation applies to both cast-in-place and precast concrete compression members.

C10.3.5 *Transmission of axial force through floor systems*

C10.3.5.1 *Transmission of load through floor system*

The requirements of this section are based on a paper on the effect of floor concrete strength on column strength [10.10]. The provisions mean that when the column concrete strength does not exceed the floor concrete strength by more than 40 %, no special precautions need be taken. For higher column concrete strengths, methods in 10.3.5.2 or 10.3.5.3 should be used for corner or edge columns. Methods in 10.3.5.2, 10.3.5.3 or 10.3.5.4 should be used for interior columns with adequate restraint on all four sides.

C10.3.5.2 *Placement of concrete in floor*

Application of the concrete placement procedure described in 10.3.5.2 requires the placing of two different concrete mixtures in the floor system. The lower strength mixture should be placed while the higher strength concrete is still plastic and should be adequately vibrated to ensure the concretes are well integrated. This requires careful co-ordination of the concrete deliveries and the possible use of retarders. In some cases, additional inspection services will be required when this procedure is used. It is important that the higher strength concrete in the floor in the region of the column be placed before the lower strength concrete in the remainder of the floor to prevent accidental placing of the low strength concrete in the column area. It is the designer's responsibility to indicate on the drawings where the high and low strength concretes are to be placed.

C10.3.5.4 *Strength of columns laterally supported on four sides*

Research [10.11] has shown that heavily loaded slabs do not provide as much confinement as lightly loaded slabs when ratios of column concrete strength to slab concrete strength exceed about 2.5. Consequently, a limit is placed on the concrete strength ratio assumed in detail.

C10.3.6 *Perimeter columns to be tied into floors*

Research [10.12] has demonstrated the need to tie columns back into the floor to prevent outward movement of the column. In most instances, beams framing into the column will provide sufficient restraint. However, in situations such as illustrated in Figure C10.2, where beams are not present in a direction perpendicular to the floor edge, extra tie reinforcement is required in the topping as illustrated.

Effective tension reinforcement connecting columns of one-way frames to precast floor systems which they support must be provided at each level. To prevent separation of columns from the diaphragm when the lateral design forces are applied, such ties must be effectively anchored in the beam-column floor joint region to both the column and the floor. Anchorages in the floor should be of sufficient length to allow effective dissipation of the design tension force within the diaphragm. The required area of the reinforcement should be based on the maximum column forces derived for the storey below the level considered. An example of such ties is given in Figure C10.2.

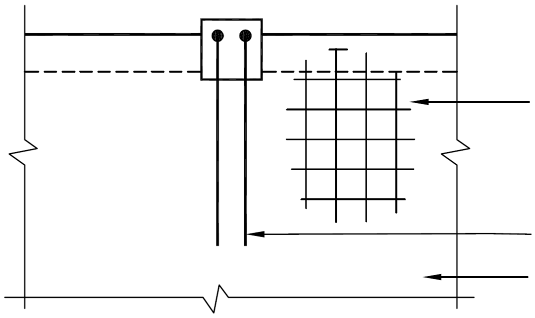


Figure C10.2 – Reinforcement to tie exterior columns to floors

A3 **C10.3.8 *Longitudinal reinforcement in columns***

C10.3.8.1 *Limits for area of longitudinal reinforcement*

A3 This clause prescribes the limits on the amount of longitudinal reinforcement for columns. If the use of high reinforcement ratios would involve practical difficulties in the placing of concrete, a lower percentage and hence a larger column, or higher strength concrete or reinforcement, should be considered. The percentage of reinforcement in columns should usually not exceed 4 % if the column bars are required to be lap spliced.

Minimum reinforcement

Since the design methods for columns incorporate separate terms for the load carried by concrete and by reinforcement, it is necessary to specify some minimum amount of reinforcement to ensure that only reinforced concrete columns are designed by these procedures. Reinforcement is necessary to provide resistance to bending, which may be introduced whether or not computations show that bending exists, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Tests have shown that creep and shrinkage tend to transfer load from the concrete to the reinforcement, with a consequent increase in stress in the reinforcement, and that this increase is greater as the ratio of reinforcement decreases. Unless a lower limit is placed on this ratio, the stress in the reinforcement may increase to the yield level under sustained service loads. This phenomenon was emphasised in the report of ACI Committee 105 [10.13] and minimum reinforcement ratios of 0.01 and 0.005 were recommended for spiral and tied columns, respectively. A minimum reinforcement ratio of 0.008 for both types of laterally reinforced columns is recommended in this Standard.

Maximum reinforcement

Extensive tests of the ACI column investigation [10.13] included reinforcement ratios no greater than 0.06. Although other tests with as much as 17 % reinforcement in the form of bars produced results similar to those obtained previously, it is necessary to note that the loads in these tests were applied through bearing plates on the ends of the columns and the problem of transferring a proportional amount of the load to the bars was thus minimised or avoided. It is considered that 0.08 is a practical maximum for reinforcement ratio in terms of economy and requirements for placing.

C10.3.8.3 *Spacing of longitudinal bars*

The spacing limits are set to ensure that the column has some ductility. While this clause gives the spacing limits bars contained in a spiral or hoop, or for cross linked bars in a rectangular section, it should be noted that 2.4.4.5 requires the spacing of longitudinal reinforcement (not necessarily cross linked) to be equal to or less than 300 mm in the region of any member subjected to tension.

A2

C10.3.9.2 *Offset column faces*

This requirement for lap spliced dowels with column faces offset 75 mm or more, together with 8.7.2.1, precludes offsetting 75 mm or more in columns reinforced with bars larger than 40 mm since lap splices are prohibited for such bars.

C10.3.10 *Transverse reinforcement in columns* A3

C10.3.10.1 *General*

Transverse reinforcement is required to resist shear, confine the concrete and prevent premature buckling of compressed longitudinal bars. Each set of criteria should be checked and reinforcement provided for the critical set.

The nominal shear strength is taken as the sum of the shear resistance provided by the concrete and the shear resistance provided by shear reinforcement.

C10.3.10.2.1 Maximum permissible nominal shear force and effective shear area

The maximum permissible shear stress is limited to prevent diagonal compression failure.

C10.3.10.2.2 Method of design for shear

Either the strut and tie method may be used, see 7.5 and Appendix A, or the shear resisted by the concrete and web reinforcement may be found from 10.3.10.3 and 10.3.10.4.

C10.3.10.3.1 Nominal shear strength provided by the concrete for normal density concrete

Equation 10–11 is similar to that used for beams. However, for columns longitudinal reinforcement is required to be spaced at relatively close centres around the perimeter of the column (10.3.8). This reinforcement controls crack widths and consequently the depth factor (k_d in 9.3.9.3.4) for beams can be taken as 1.0 for columns.

Equations 10–14 and 10–15 for k_n make a conservative allowance for the influence of axial load on shear resistance provided by the concrete. In designing for shear it is important to note the detrimental influence that axial tension may have on the shear resistance provided by the concrete. Designers should be aware that axial tension may be induced due to shrinkage of concrete or temperature change where members are restrained against longitudinal movement.

With increasing axial load on a column, up to a level of about $0.3A_g f'_c$, the shear resisted by the concrete increases. However, above this axial load level the increase is small in relatively slender columns, and above a level of $0.5A_g f'_c$ the shear strength resisted by concrete starts to decrease. For this reason the value of the limit of 0.3 is used with Equation 10–14. A3

The increase in shear resistance with axial load is associated with the inclination of the compression force relative to the axis of the member. At the point of contra-flexure the centroid of the compression force is on the axis of the column, while at the critical section its location can be found by standard flexural theory. These two locations determine the average inclination of the compression force associated with the axial load. The lateral component of this force contributes to the shear resistance of the column.

Low levels of axial tension often occur due to volume changes, but are not important in structures with adequate expansion joints and minimum reinforcement. It may be desirable to design shear reinforcement to carry total shear if there is uncertainty about the magnitude of axial tension.

A2

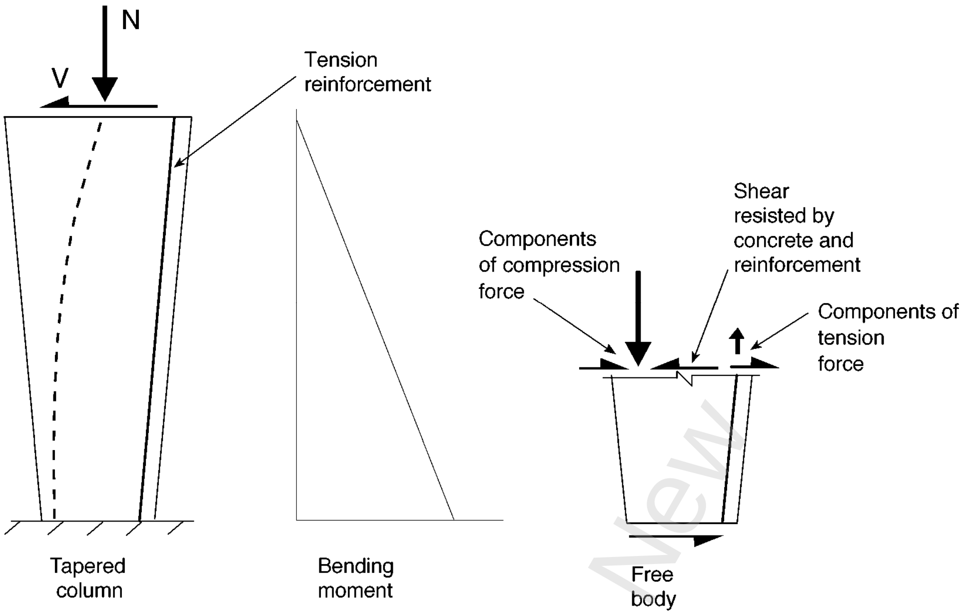


Figure C10.3 – Effect of column taper on shear strength

C10.3.10.3.2 Change in shear strength in members where sides are not parallel to the longitudinal axis

Figure C10.3 illustrates the case where the depth of a column increases in the direction of decreasing moment. In this situation, the lateral transverse components of the compression and tension forces in the section act against the shear force to be resisted by the concrete and reinforcement. The reduction in nominal shear strength due to the lateral component of these forces may be calculated from the inclinations of the compression and tension forces, relative to the axis of the member. Conservatively, the inclination of the compression may be assumed to be parallel to the side subjected to the highest compression strain. The inclination of the tension force is defined by the inclination of the reinforcement. Where a designer wishes to allow for the beneficial effect of the inclination of the flexural forces, where the section depth decreases with decreasing moment, the strut and tie method of design should be used (see 10.3.10.2.2 and Appendix A).

C10.3.10.3.3 Nominal shear strength provided by the concrete for lightweight concrete

See C9.3.9.3.5.

C10.3.10.4 Shear reinforcement

C10.3.10.4.2 Nominal shear strength provided by shear reinforcement

The equations 10–17 and 10–18 are consistent with an angle of $\tan^{-1} j$ for the diagonal compression forces in the web (see C9.3.9.4.) The values of V_c given by 10.3.10.3 have been determined from test results and the assumption that the contribution of shear resistance provided by the concrete is given by equations 10–17 and 10–18. If other inclinations of diagonal compression force are assumed there is uncertainty as to the correct value of V_c which should be used.

Shear reinforcement may be designed as permitted in 7.5.9 and 10.3.10.2.2 using the “strut and tie” method.

C10.3.10.4.3 Maximum spacing of shear reinforcement

The recommended spacing ensures that the diagonal cracks are crossed by shear reinforcement.

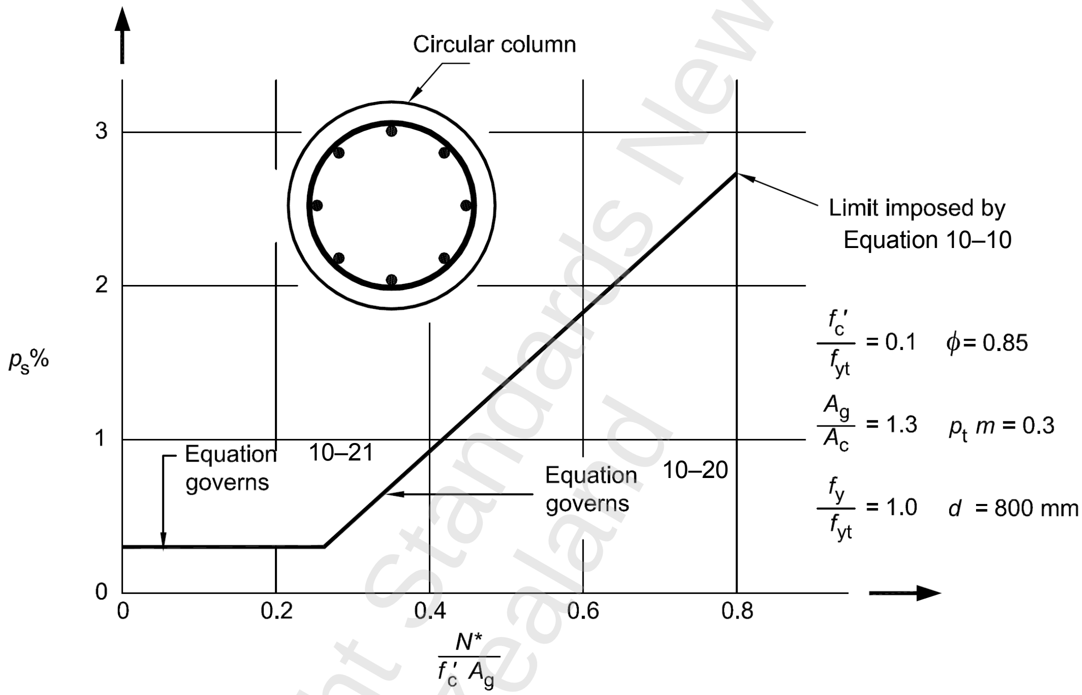
C10.3.10.4.4 Minimum shear strength provided by shear reinforcement

The minimum area of shear reinforcement as for beams is recommended (see 9.3.9.4.13 to 9.3.9.4.15).

C10.3.10.5 and C10.3.10.6 *Design of spiral or circular hoop, or rectangular hoop and tie transverse reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

Spiral or circular hoop, or rectangular hoop and tie reinforcement, is needed to ensure ductile behaviour of the column in the event of overload or unexpected displacements. Equations 10–20 and 10–22 are to ensure that the concrete is confined. Equations 10–21 and 10–23 are to ensure that premature buckling of longitudinal bars does not occur.

Figure C10.4 illustrates examples of the application of Equations 10–20 to 10–23 to circular and rectangular columns.



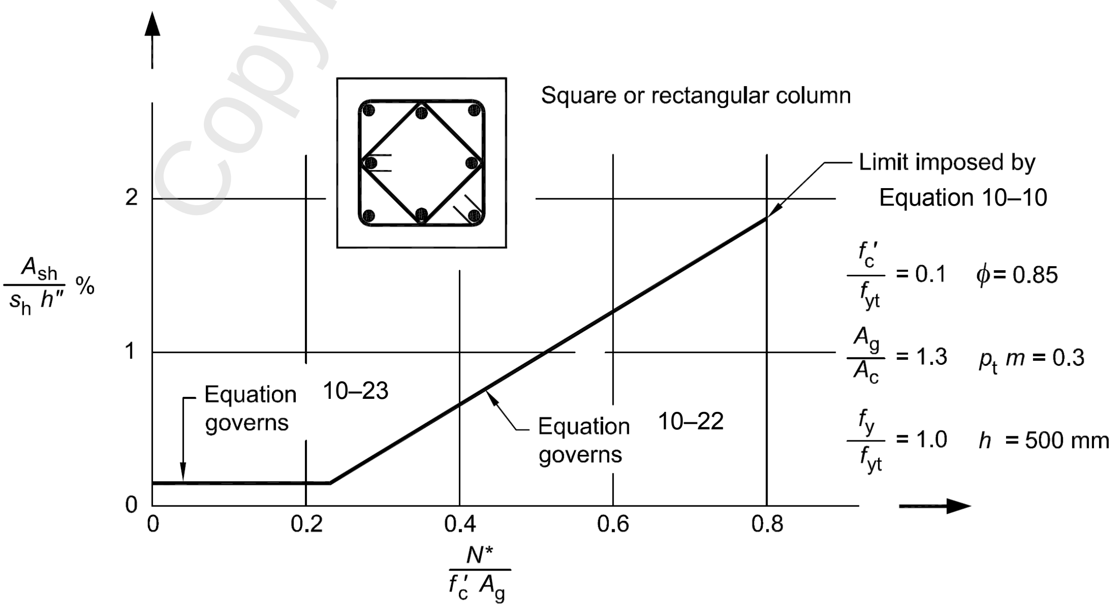


Figure C10.4 – Example of application of Equations 10–20 to 10–23

The arrangement of transverse reinforcement should ensure that the ratio A_g / A_c does not exceed 1.5 unless it can be shown that the design strength of the core of the column, including the beneficial effect of the enhancement in the concrete compressive strength due to confinement if necessary, can resist the design actions given by the design loading combinations including earthquake effect. In that case $A_g = A_c$ and the value of $A_g/A_c = 1.0$ should be substituted in Equations 10–20 and 10–22. If the gross area of the section A_g is used to resist the design actions, the limitation of $A_g / A_c \leq 1.5$ means that there is a practical minimum size of core concrete. This limitation on reduction of core area, as compared to the gross area of the section, may become critical for members with relatively small cross-sectional areas in conjunction with relatively large covers to the transverse reinforcement.

The limitation on $p_t m$ means that the maximum value of $p_t m$ that can be used in Equations 10–20 and 10–22 is 0.4. This is not a physical limitation on $p_t m$. The selection of non-prestressed longitudinal reinforcement content p_t, f_y and f'_c may result in the actual $p_t m$ ratio exceeding 0.4.

The quantities of transverse reinforcement specified by Equations 10–20 to 10–23 are aimed at ensuring ductile behaviour and continued load carrying ability of the column at large inelastic deformations when the shell (cover) concrete has spalled off. The derivation of Equations 10–21 and 10–23 is given elsewhere[10.14]. The equations were determined to ensure that while carrying constant axial load the moment of resistance of the column does not diminish by more than 20 % during cycles of flexure to curvature ductility factors ϕ_u / ϕ_y of at least ten, where ϕ_u is the ultimate curvature and ϕ_y is the curvature at first yield of the column. Note that the required quantity of transverse reinforcement increases with the axial load level. The maximum centre-to-centre spacing of the transverse reinforcement permitted in 10.3.10.5.2 and 10.3.10.6.2 is that considered necessary to restrain buckling of longitudinal steel and for adequate confinement of the concrete. Too great a spacing would not provide adequate lateral restraint or confinement; too small a spacing would not allow aggregate particles to pass between the transverse bars when concrete is being placed. Note that this transverse reinforcement is required without reduction up the full height of the column, since failure could occur away from the ends of the column, as the bending moment may be more critical elsewhere when gravity load effects dominate.

Equations 10–20 and 10–22 result in quantities of transverse reinforcement that are approximately 70 % of that required by Equations 10–38 and 10–40. For columns, where the provisions of 10.3.10 govern, it is expected that the degree of restraint of longitudinal bars in compression (once spalling of cover concrete has occurred) need not be as large as that required for columns, governed by the requirements of 10.4.

The requirement for the minimum transverse reinforcement in columns stems from assuring a reasonable capability for inelastic deformation. The predominant situation seen as producing these deformations is seismic loading. Therefore the combination of factored loads for the ultimate limit state for determining the quantity of transverse reinforcement for Equation 10–20 and Equation 10–22 N* should be taken as the maximum compression axial load in and load combinations involving either seismic actions or wind forces, or any other load combination in which appreciable lateral force is applied to the structure.

C10.3.10.9 *Set out of transverse reinforcement at column ends*

A provision has been included in this Standard requiring ties above the termination of the spirals in a column if enclosure by beams or brackets is not available on all sides of the column. These ties are chosen to enclose the longitudinal column reinforcement and the portion of bars from beams bent into the column for anchorage. The Standard allows spirals to be terminated at the level of lowest horizontal reinforcement framing into the column. However, if one or more sides of the column are not enclosed by beams or brackets, ties are required from the termination of the spiral to the bottom of the slab or drop panel. If beams or brackets enclose all sides of the column but are of different depths, the ties should extend from the spiral to the level of the horizontal reinforcement of the shallowest beam or bracket framing into the column.

C10.3.11 *Composite compression members*

C10.3.11.1 *General*

Composite columns are defined without reference to classifications of combination, composite, or concrete-filled pipe column. Reference to other metals used for reinforcement has been omitted because they are seldom used in concrete construction.

C10.3.11.2 *Strength*

The same rules used for computing the axial load – moment interaction strength for reinforced concrete sections can be applied to composite sections. Interaction charts for concrete-filled steel tubes would have a form identical to those for reinforced concrete sections.

C10.3.11.3 *Axial load strength assigned to concrete*

Direct bearing or direct connection for transfer of forces between steel and concrete can be developed through lugs, plates, or reinforcing bars welded to the structural shape or tubing before the concrete is cast. Flexural compressive stress need not be considered a part of direct compression load to be developed by bearing. A concrete encasement around a structural steel shape may stiffen the shape, but it would not necessarily increase its strength.

C10.3.11.5 *Slenderness effects*

Equation 10–24 is given because the rules of 10.3.2.3.3 for estimating the radius of gyration are overly conservative for concrete filled tubing and are not applicable for members with enclosed structural shapes.

In reinforced concrete columns subject to sustained loads, creep transfers some of the load from the concrete to the steel, increasing the steel stresses. In the case of lightly reinforced columns, this load transfer may cause the compression steel to yield prematurely, resulting in a loss in the effective EI. Accordingly, both the concrete and steel terms in Equation 10–25 are reduced to account for creep. For heavily reinforced columns or for composite columns in which the pipe or structural shape makes up a large percentage of the cross section, the load transfer due to creep is not significant. Accordingly in Equation 10-9 only EI of the concrete is reduced for sustained load effects.

C10.3.11.6 *Structural steel encased concrete core*

Steel encased concrete sections should have a metal wall thickness large enough to attain longitudinal yield stress before buckling outward.

C10.4 Additional design requirements for structures designed for earthquake effects

C10.4.2 *Protection of columns at the ultimate limit state*

In accordance with the requirements of NZS 1170.5, with few exceptions in low-rise buildings, multi-storey frames subjected to earthquake forces must exhibit a strong column-weak beam plastic mechanism to ensure that so-called "soft storeys" do not develop.

This Standard requires capacity design procedures to be used to achieve this aim as far as possible. These procedures take into account possible beam overstrength, concurrent seismic forces and magnification of column moments due to dynamic effects (see 2.6.5).

C10.4.3 *Dimensions of columns*

The derivation of Equations 10–26 to 10–29 relating permissible depth, width and clear height of columns is similar to that for beams discussed in C9.4.1.2. It was recognised that stiffness degradation occurs during cyclic loading. A3

For bridge piers the criteria stated in Equations 10–28 and 10–29 will not be appropriate if diaphragm action of the superstructure can be relied upon. However, if these equations are not used for bridge piers special studies should be conducted to establish that lateral buckling will not be a problem.

C10.4.3.6 *Narrow beams and wide columns*

The effective width to be considered for wide columns and the treatment of eccentric beam-column connections are discussed in Commentary Clauses C15.4.6 and C15.4.7. Frame details in which the axes of the beams and columns do not coincide should be avoided.

A3 **C10.4.4** ***Limit for design axial force on columns***

An upper limit of $0.7N_{n,max}$ is placed on the axial compressive load in columns because for heavily loaded sections a large amount of confining reinforcement is required to make the section adequately ductile. The upper limit applies both to columns or piers which are protected against plastic hinging and to columns which are designed for deliberate plastic hinging (for example, in the case of one or two storey buildings where Section 2 allows column sidesway mechanisms, or in bridge piers) since it is considered that columns detailed according to this Standard will have adequate ductility to enable the structure to deform to the required displacement ductility factor. That is, because the amount of transverse reinforcement increases with the axial compressive load level, there is no need to place a more severe limit on axial load level for these cases than the limit given in 10.4.4.

When loads on columns and frames are derived from capacity design principles the value of ϕ in the expression for $0.7\ \phi N_{n,max}$ should be taken as unity.

C10.4.5 ***Ductile detailing length***

The length over which yielding of the reinforcement may occur, referred to as the ductile detailing length, ℓ_y, which is to be confined in the end region of a column or pier is listed in Table C10.1. It is taken as the greater of a multiple of the longer cross section dimension or diameter or where the moment exceeds a percentage of the maximum moment. The bending moment diagram for a column or pier is known quite accurately in statically determinate cases and in the case of low frames where higher mode effects are not significant. In tall frames where higher mode effects are significant, the moment diagram will be different from that given by equivalent static seismic forces. In lieu of more accurate analysis to determine the length of the potential plastic hinge region in a column or pier, the following assumptions should be made with regard to the column bending moment diagram, obtained from an equivalent static analysis of lateral forces or a first mode analysis in a response spectrum analysis, to which the appropriate percentages of maximum moment are to be applied:

(a) When the bending moment diagram of the column or pier contains a point of contraflexure, the column bending moment diagram to be used to determine the length of the potential plastic hinge region can be considered to extend from the maximum moment at the end under consideration to zero moment at the centre of the beam, at the other end of the column in that storey.

(b) When the column or pier is dominated by cantilever action and a point of contraflexure does not occur in the storey being considered, then the column bending moment diagram to be used may be assumed to commence with the maximum moment at the end of the column being considered and shall have 80 % of the gradient of the column bending moment diagram resulting from the determination of maximum moments.

For (a) and (b) above, the maximum moment in the columns shall be determined with regard to dynamic magnification and overstrength actions. Appendix D has appropriate methods for obtaining the maximum moments (M_{col}) and column bending moment diagrams. In (a) and (b), the maximum moments are those at the top or the soffit of the beams in the storey under consideration (equally applicable to the base of the column).

The recommendations of 10.4.5 were derived as the result of analysis and tests [10.15].

Note that for heavily loaded columns in frames the whole height of the column may be in the top and bottom potential plastic hinge regions. For example, this will be so if $N_o^* \geq 0.5\phi f'_c A_g$ when the ratio of the clear height of column to larger lateral dimension is 6.0 or less.

A3

Table C10.1 – Length of potential plastic hinge region at end of columns

Axial load N^*_o (N)	The larger of: Multiple of longer cross section dimension	The larger of: Where moment exceeds percentage of maximum moment accounting for type of bending moment diagram; see C10.4.5(a) and (b) (%)
$0 - 0.25\phi f'_c A_g$	1.0	80
$0.25\phi f'_c A_g - 0.5\phi f'_c A_g$	2.0	70
$0.5\phi f'_c A_g - 0.7\phi N_o$	3.0	60

A3

C10.4.6 *Longitudinal reinforcement in columns*

C10.4.6.1 *Longitudinal reinforcement*

The minimum area of longitudinal reinforcement is the same as that specified for members not designed for seismic forces given by 10.3.8.1.

C10.4.6.2 *Maximum area of longitudinal reinforcement*

The maximum areas are more restrictive than for members not designed for seismic loading, and are less for higher grade steel in view of the higher yield strength of that steel. Limits are also placed on the maximum reinforcement area at lap splices.

C10.4.6.3 *Spacing of longitudinal bars in plastic hinge region*

The requirement concerning the spacing of longitudinal bars in potential plastic hinge regions is to ensure that bars are distributed reasonably uniformly around the perimeter of the section in order to assist the confinement of concrete. The bars between the corner bars in rectangular columns or between bars at the sides in circular columns can also act as vertical shear reinforcement in beam-column joints if required (see 15.4.5.2). In wide columns with narrow beams some concentration of the effective flexural reinforcement may be required in accordance with 15.4.6.

C10.4.6.6 *Maximum longitudinal column bar diameter in beam-column joint zones*

Generally columns are given protection against the simultaneous formation of plastic regions on each side of a joint zone. Consequently the bond conditions for longitudinal bars are considerably better than for the corresponding condition for beam bars, where simultaneous yield of bars in compression and tension may be expected on each side of the joint zone. Hence less restrictive bar diameter to beam depth ratios may be used for column bars. Where a high level of protection against plastic hinge formation occurs in columns (as in method A in Appendix D) the maximum permitted bar diameter is further increased.

Elongation of beams can force plastic hinges to occur in columns immediately above or below the joint zone at the first elevated level in moment resisting frames. For this reason equation 10–32 should not be used for bar diameters in the columns adjacent to the first level.

C10.4.6.8 Splices of longitudinal reinforcement

The centre of lap splices in longitudinal column bars must be within the middle quarter of the storey height unless it is shown that plastic hinges cannot develop at the column ends. This condition may be assumed where columns are designed by method A in Appendix D in the region above mid-depth of the second storey. Stirrup-ties must confine the lap splice if reversing stresses in the longitudinal bars in the lap exceed 0.6 f_y.

A3

C10.4.7 *Transverse reinforcement in columns*

C10.4.7.2.1 Design shear force

Design shear forces in members containing potential plastic regions are determined by capacity design principles. The objective is to ensure a ductile failure mode develops in preference to non-ductile failure modes, such as shear failure or buckling of longitudinal reinforcement. Appendix D contains two methods that may be used to determine capacity design actions in multi-storey, ductile, moment resisting frames. However, the shear force determined by either of these actions is not allowed to be less than the appropriate values listed in this clause.

It should be noted that elongation of beams, which is associated with the formation of plastic hinge zones, can induce plastic hinges at the base of columns as well as either just above or just below the beams at the first level above the base. This is the reason for (c) in clause 10.4.7.2.1.

A2

C10.4.7.2.2 *Types of potential plastic hinges in columns*

This clause identifies where potential ductile plastic regions should be used and where potential limited ductile plastic regions may be used in columns in multi-storey buildings adjacent to beams.

Method B in Appendix D provides a high level of protection against the formation of a column sway mechanism, but it accepts that some plastic deformation may occur in the columns. Consequently, where this design method is used, ductile detailing should be provided in all the potential plastic regions.

Method A provides a high level of protection against plastic deformation in the columns except in the first one and a half levels above the primary plastic regions at the base of each column. In this region, elongation of beams, which is generally ignored in analysis, can induce plastic hinges in the columns immediately below or above the beams in the first elevated level. For this reason, potential plastic regions below the mid-height of the first elevated storey should be detailed as for ductile plastic regions.

C10.4.7.2.3 *Design of shear reinforcement*

Special provisions are given in 10.4.7.2.4 and 10.4.7.2.5 for shear design in ductile and limited ductile potential plastic regions. Outside these zones the provisions in 10.3.10.2, 10.3.10.3 and 10.3.10.4 may be followed, except the minimum shear reinforcement is increased by 33 % as specified in 10.4.7.2.7.

C10.4.7.2.4 Strut and tie method for shear design

When axial compression force acts on a column sustaining a bending moment which varies over its length, the resultant compression force is inclined to the axis of the column. This is illustrated in Figure C10.5. The resultant compression force may be envisaged as being made up from two components, one due to flexure, which varies with distance, and one due to axial load, which is constant with distance. The approximation is illustrated in the figure. From this it can be seen that the average shear resisted by the inclined axial force, N^*_o, is given by $N^*_o \tan \alpha$, where α is the angle sustained between the centroidal axis of the member and the line formed by joining the centroid of the section at the point of inflection to the centroid of the compression force in the concrete at the point of maximum moment. The remainder of the shear is resisted by shear reinforcement and diagonal compression forces in the web of the member.

Within the ductile detailing length the inclination of the diagonal compression forces in the flexural tension zone to should be equal to or more than 45° to the flexural tension reinforcement. This recommendation is made as smaller angles of diagonal compression force can lead to premature yielding of ties, which results in the longitudinal reinforcement being kinked, hence reducing the resistance to the bucking of these bars. Smaller angles of inclination may be used in the column between ductile detailing regions.

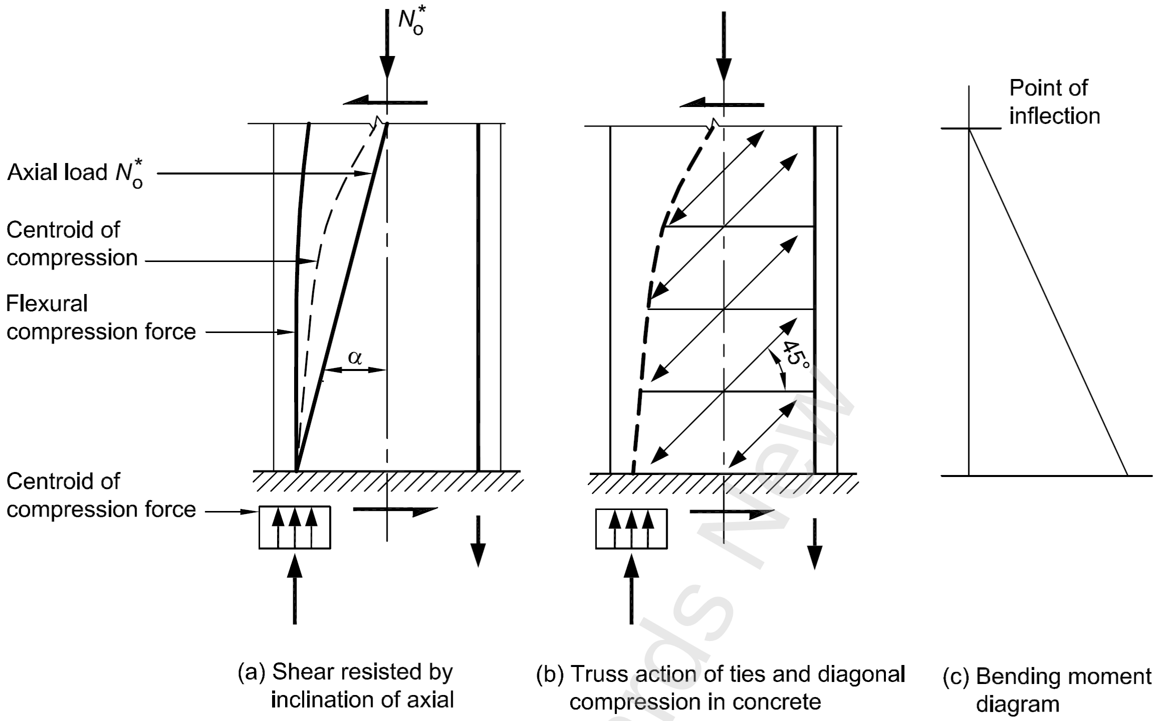


Figure C10.5 – Strut and tie design for shear

C10.4.7.3 *Alternative design methods for concrete confinement and lateral restraint of longitudinal bars*

Where columns contain a significant amount of transverse confining reinforcement they exhibit considerable ductility at high strains at the ultimate limit state when the concrete shell outside the core concrete spalls off. This ductility is due to the increased strength and ductility of the concrete core, and to the restraint against buckling of the longitudinal reinforcement, provided by the transverse confining reinforcement [10.8].

Clause 10.4.7.3 permits alternative methods to be used to determine the transverse reinforcement required for confinement and lateral restraint of bars.

A great deal of experimental testing and analysis of reinforced concrete columns subjected to simulated seismic forces has been conducted in New Zealand (for example, see References 10.14, 10.15, 10.16, 10.17, 10.18, 10.19, 10.20 and 10.21). This research has provided improved information on the cyclic stress-strain characteristics of concrete confined by various amounts and arrangements of confining reinforcement. As a result, design charts have been derived by Zahn et al[10.20] to relate the available curvature ductility factor ϕ_u/ϕ_y of reinforced concrete column and pier-cross sections to the magnitude of the confining stress applied by transverse spiral or hoop steel, and to determine the flexural strength of those confined sections. The design charts were derived from theoretical studies of the cyclic moment-curvature behaviour of reinforced concrete column sections, using analyses that included the cyclic stress-strain relationships for confined and unconfined concrete and the longitudinal reinforcing steel and transverse confining steel. The cyclic stress-strain relationships used for confined concrete, due to Mander et al[10.19, 10.22], include the effects of various quantities and arrangements of the transverse confining reinforcement. In the analysis the ultimate curvature ϕ_u is obtained by imposing four identical cycles of bending moment to peak curvatures of equal magnitude in each direction.

The available ultimate curvature is considered to have been reached when one of the following limit conditions is reached:

(a) The peak moment resisted in the last cycle has reduced to 80 % of the maximum theoretical flexural strength;

(b) The strain energy accumulated in the confining reinforcement at the end of four cycles has become equal to its strain energy capacity and the transverse steel fractures;

(c) The tensile strain in the longitudinal reinforcing steel has reached that at the ultimate tensile strength; or

(d) The compression strain in the longitudinal reinforcing steel has reached such that significant inelastic buckling occurs.

The first of these four limits conditions to be reached and defines the available ultimate curvature, ϕ_u. Generally either limit condition (a) or (b) was found to govern[10.20]. A range of design charts for the available curvature ductility factor ϕ_u/ϕ_y of circular and rectangular reinforced concrete columns were derived[10.20], where ϕ_y is the curvature at first yield. The design charts plot the axial load ratio N_o^*/A_g against the curvature ductility factor ϕ_u/ϕ_y for various ratios of effective lateral confining stress, f_ℓ, to concrete compressive strength, f_ℓ/f_c' and for various $p_t m$ values, where p_t is the longitudinal steel ratio and $m = f_y/0.85$. The effective lateral confining stress is dependent on the spacing, area and yield strength of the transverse bars. Design charts were also derived to determine the flexural strength of column sections including the influence of the increase in the concrete compressive strength and ductility capacity due to confinement.

Watson et al [10.14, 10.15] have used the design charts for ductility derived by Zahn et al [10.20] to obtain refined design equations for the quantities of transverse confining reinforcement required in the potential plastic hinge regions of reinforced concrete columns. Typical ranges of the axial load ratio N_o^*/A_g, the concrete compressive strength, the mechanical reinforcing ratio $p_t m$, and the cover ratios c/h for square and rectangular columns or c/D for circular columns, were considered, where c = concrete cover thickness and h and D = overall depth and diameter of rectangular or square and circular cross sections, respectively. The 95 % upper-tail values of the area of transverse reinforcement obtained from the design charts and a regression analysis were used to obtain the best-fit equations by the least squares method. The Equations C10–3 and C10–4 are based on corresponding equations developed by Watson et al. However, an additional factor, k_c, which is applied to the curvature ratio, ϕ_u/ϕ_y, has been introduced to make this ratio consistent with the method of calculation and the curvature limits given in Section 2.6. Where the curvatures are calculated by the method set out in 2.6 the appropriate value of k_c is equal to $270/f_y$ but with an upper limit of 0.75 ($270/f_y < 0.75$). The modified equation for rectangular column cross sections is as follows:

$$\frac{A_{sh}}{s_h h''} = \left\{ \frac{A_g}{A_c} \frac{\{(k_c \phi_u/\phi_y) - 33 p_t m + 22\}}{111} \frac{f_c'}{f_{yt}} \frac{N_o^*}{\phi f_c' A_g} \right\} - 0.006 \quad \text{(Eq. C10–3)}$$

For circular column cross sections the modified equation becomes:

$$p_s = 1.4 \left\{ \frac{A_g}{A_c} \frac{\{(k_c \phi_u/\phi_y) - 33 p_t m + 22\}}{111} \frac{f_c'}{f_{yt}} \frac{N_o^*}{\phi f_c' A_g} \right\} - 0.0084 \quad \text{(Eq. C10–4)}$$

In Equation C10–3

A_{sh} is the total effective area of transverse bars in direction under consideration within centre-to-centre spacing of hoop sets

s_h, h'' is the dimension of core of rectangular or square column at right angles to direction of transverse bars under consideration measured to the centreline of the perimeter hoop

A_g is the gross area of column, A_c = core area of column

ϕ_u/ϕ_y is the curvature ductility factor

p_t is the A_{st}/A_g, A_{st} = total area of longitudinal column reinforcement

m is the $f_y/0.85$, f_y = lower characteristic strength yield strength of longitudinal steel

f_{yt} is the lower characteristic strength yield strength of transverse steel,

f_c' is the concrete compressive cylinder strength

N_o^* is the axial compressive load on column derived from capacity considerations

ϕ is the strength reduction factor

p_s is the ratio of volume of transverse circular hoop or spiral steel to volume of concrete core of column.

The refined equations have had experimental verification[10.14, 10.15].

When applying the refined equations it should be ensured that for the arrangement of transverse reinforcement that the ratio A_g/A_c does not exceed 1.5 unless it can be shown that the design strength of the core of the column, including the beneficial effect of the enhancement in the concrete compressive strength due to confinement if necessary, can resist the design axial load given by the design loading combinations including earthquake effect. In that case the actual value of A_g/A_c should be substituted in Equations 10–38 and 10–40. This means that there is a practical minimum size of core concrete. This limitation on reduction of core area, as compared to the gross area of the section, may become critical for members with relatively small cross-sectional areas in conjunction with relatively large covers to the transverse reinforcement.

Also $p_t m$ should not be taken larger than 0.4. The limitation on $p_t m$ means a maximum value of 0.4 could be used in Equations 10–38 and 10–40. This is not a physical limitation on $p_t m$. The selection of non-prestressed longitudinal reinforcement p_t, f_y and f'_c may result in the actual $p_t m$ ratio exceeding 0.4.

C10.4.7.4.1 In ductile potential plastic hinge regions

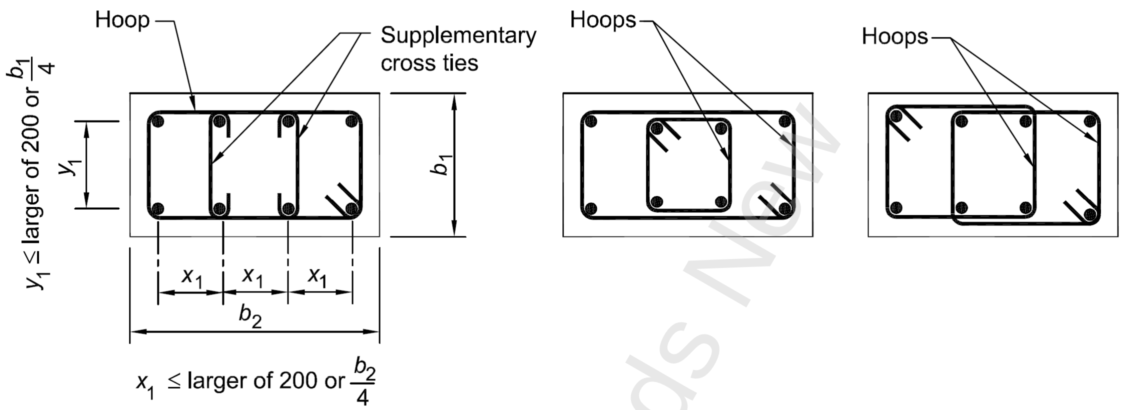
A value of curvature ductility factor $\phi_u/\phi_y = 20/k_c$ could be used in the above equations when plastic hinging of ductile columns is expected in a severe earthquake. For example, at the bottom storey of ductile building frames, or in the columns of one or two storey ductile frames where strong beam-weak column design is permitted, or in ductile bridge piers where plastic hinging is expected in a severe earthquake. Equations 10–38 and 10–40 were obtained from the above equations by substituting $\phi_u/\phi_y = 20$. Unless special studies are undertaken the maximum curvature ductility factor ϕ_u/ϕ_y should not exceed or assume to be greater than $20/k_c$. A3

At low axial load levels the need of transverse reinforcement for concrete confinement becomes less and the provision of sufficient transverse reinforcement to prevent buckling of the longitudinal reinforcement becomes more critical. The quantity of transverse reinforcement required to prevent buckling of longitudinal reinforcement is given by Equation 10–39 for spiral or circular hoop reinforcement and by Equation 10–41 for rectangular hoops or supplementary cross-ties. The transverse reinforcement should not be less than the greater of that required for concrete confinement and restraint against bar buckling.

The permitted centre-to-centre vertical spacing of transverse steel of not greater than one-quarter of either the least lateral dimension or the diameter of the column or pier is also to ensure adequate confinement of concrete. This maximum spacing is kept reasonably small. This is because the concrete is confined mainly by arching between the spiral or hoops and hence if the vertical spacing is too large a significant depth of unconfined concrete will penetrate into the concrete core between the spirals or hoops and thus reduce the effective confined concrete section. This maximum spacing is a function of the column dimension, and hence the spacing is greater for larger sections than for smaller sections, since a greater penetration of unconfined concrete between the transverse steel has a less significant effect on strength for larger sections. The requirements that the vertical spacing should not exceed six longitudinal bar diameters is to prevent buckling of longitudinal steel when undergoing yield reversals in tension and compression consistent with the attainment of a curvature ductility factor of at least $20/k_c$. It is well known that such stress reversals in the yield range cause a reduction in the tangent modulus of the steel at relatively low stresses, due to the Bauschinger effect, and therefore closely spaced transverse reinforcement providing lateral support is required to prevent buckling of the longitudinal reinforcement.

In most rectangular sections a single rectangular peripheral hoop will be insufficient to properly confine the concrete and to laterally restrain the longitudinal bars against buckling. Therefore an arrangement of overlapping rectangular hoops or supplementary cross-ties or both, will be necessary. It would appear to be better to use a number of overlapping rectangular hoops rather than a single peripheral hoop and supplementary cross-ties. An example of alternative details and the preferred arrangement is shown in

Figure C10.6. Note from Figure C10.6(a) that a supplementary cross tie will normally engage a longitudinal bar. That is, the concrete is confined by arching between hoops, supplementary cross-ties and longitudinal bars. In a set of overlapping hoops it is preferable to have one peripheral hoop enclosing all the longitudinal bars together with one or more hoops covering smaller areas of the section. This is because such a detail is easier to construct, since the longitudinal bars are held more firmly in place if they are all enclosed by one hoop. Thus the detail in Figure C10.6(b), which has a hoop enclosing all bars and a smaller hoop enclosing the middle four bars, is to be preferred to the detail in Figure C10.6(c), which has two hoops each enclosing six bars.



(a) Single hoop plus two supplementary cross-ties bent around longitudinal bars

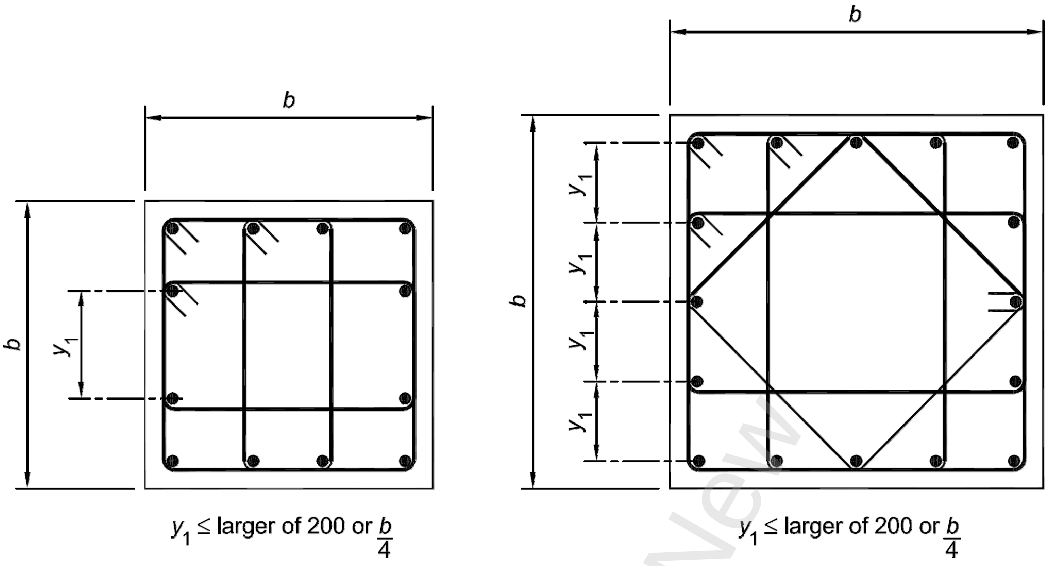
(b) Two overlapping hoops – preferred detail

(c) Two overlapping hoops – not preferred to (b)

Figure C10.6 – Alternative details using hoops and supplementary cross ties

Figure C10.7 illustrates examples of the use of overlapping hoops for column sections with a greater number of longitudinal bars. It is to be noted that the inclined hoop surrounding the four bars at the centre of each face in Figure C10.7(b) can be counted on making a contribution to A_{sh} in Equation 10–40 by determining the equivalent bar area of the component of forces in the required direction. For example, two such hoop legs inclined at 45° to the section sides could be counted as making a contribution of $\sqrt{2}$ times the area of one perpendicular bar in assessing A_{sh}. That is, in Figure C10.7(b), A_{sh} may be taken as 5.41 A_{te}, where A_{te} is the area of each hoop bar.

The legs of rectangular hoops and supplementary cross-ties should not be too widely spaced across the section if concrete confinement and restraint against buckling of longitudinal bars is to be adequate. However, not all longitudinal bars need to be laterally supported by a bend in a transverse hoop or cross-tie.

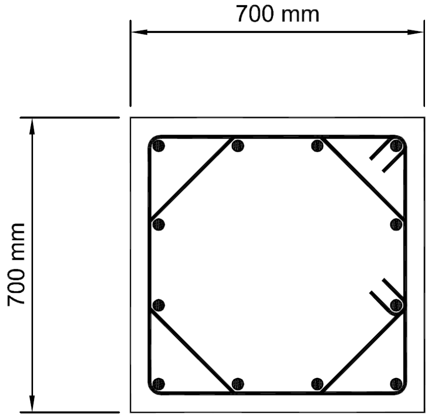


(a) Three overlapping hoops (b) Four overlapping hoops

Figure C10.7 – Typical details using overlapping hoops

If bars or groups of bars which are laterally supported by bends in the same transverse hoop or cross-tie are not further apart than the larger of 200 mm and one-quarter of the adjacent lateral dimension of the cross section any bar to bundle of bars between them need not have effective lateral support from a bent transverse bar, as is demonstrated in Figure C10.7(a). Also, bars which lie within the core of the section centred more than 75 mm from the inside face of the peripheral hoop need no special lateral support.

Figure C10.8 illustrates the difference between the provisions for the transverse reinforcement required for confinement of concrete and lateral support of longitudinal bars in the potential plastic hinge regions of a 700 mm square column according to this Standard NZS 3101 when ϕ = 1 and the Building Code of the American Concrete Institute (ACI 318). It is evident that the quantities of transverse reinforcement required for concrete confinement by Equation 10–40 reduces significantly with the decrease in axial compression load until the transverse reinforcement required by Equation 10–41 to restrain lateral buckling of longitudinal bars becomes critical.



NOTE – $A_{sh} = 2\,(1 + \frac{1}{\sqrt{2}})A_b = 3.414\,A_b$

where

A_b = area of transverse bar
f_{yt} = 300 MPa f_y = 500 MPa
f'_c = 30 MPa ρ_t = 0.02
ϕ = 1.0 s_h = 100 mm
Cover to hoops = 40 mm

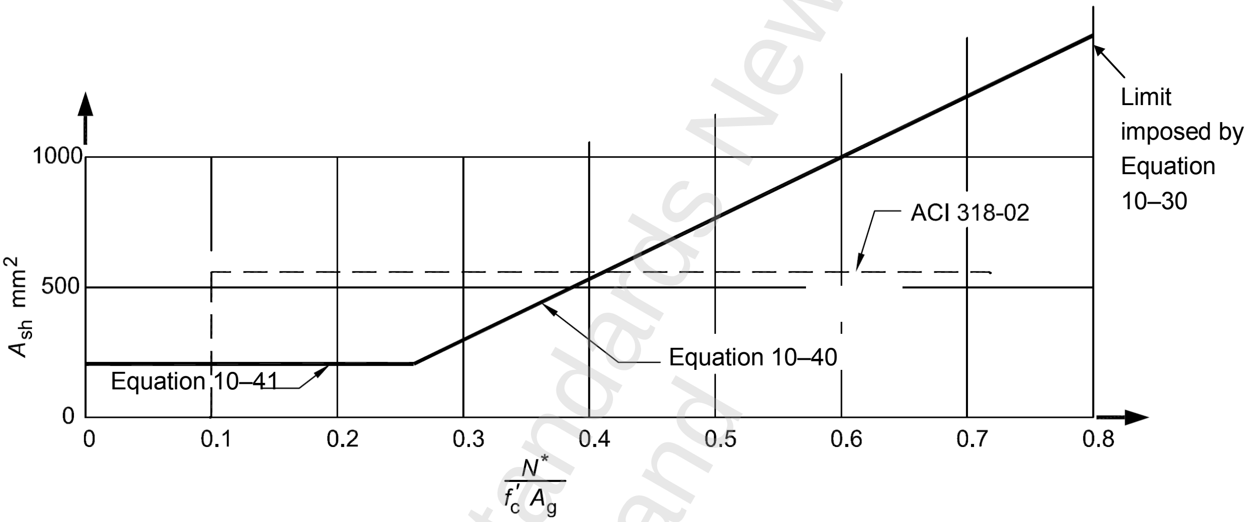


Figure C10.8 – Example of quantities of transverse reinforcement required in the potential plastic hinge region of a reinforced concrete column

Equations 10–38 and 10–40 were derived for concrete with compressive strength up to 40 MPa. However they have been shown to apply approximately to concrete with compressive strength up to 100 MPa[10.21]. Note that the tests on columns with concrete compressive strength of 100 MPa have shown that very high strength concrete is extremely brittle when not confined adequately [10.21] and that the required confinement will be considerably greater than for normal strength concrete columns. More recent tests and analytical study [10.23] have shown that the equations also apply approximately to columns produced from lightweight aggregate concrete.

A3 **C10.4.7.4.2, C10.4.7.5.1, C10.4.7.5.2, C10.4.7.4.4 and C10.4.7.5.4** *Regions protected against plastic hinging and outside potential plastic hinge regions*

In frames where the capacity design procedure, method A in Appendix D, is used, there is a high level of protection against the formation of plastic regions in the columns above the mid-height of the second storey. In recognition of the reduced inelastic demand on the plastic regions in these columns the quantity of confining reinforcement can be reduced to 70 % of that required by Equations 10–38 and 10–40. This should enable the column to achieve a curvature ductility factor of at least ten under repeated cyclic loading of 15 under earthquake attack. However, as protection against bar buckling is still required, and some confinement of the concrete is necessary, there is no corresponding reduction in the requirements given by Equations 10–39 and 10–41.

This reduction in transverse reinforcement for confinement does not apply below the mid-height of the second storey. Beam elongation associated with the formation of plastic hinges in the beams can force plastic hinges to form in the columns in the region below the mid-height of the second storey. Nor does this reduction apply to columns where plastic hinging is expected to occur, such as in one or two storey frames, or the top storey of multi-storey frames, or in bridge piers, which are deliberately designed for plastic hinging. This reduction in the quantity of transverse reinforcement does not apply if plastic hinge regions are in close proximity. For example, for a column in the lowest storey with $N^* > 0.5\phi A_g$ in a frame where the ratio of the clear height of column to the larger lateral cross section dimension is six or less, the whole height of column is in the two potential plastic hinge regions and will need full confinement.

Subclause (b) in 10.4.7.5.1 is intended to cover the case which frequently arises in bridge piers. In Section 10 bridge piers are treated as columns. In these structural elements the predominant seismic forces often act in the plane of the pier, inducing high compression stresses in the end zones of the pier section. Consequently in the ductile detailing length these zones need to be confined to ensure that ductile behaviour can be sustained. However, the mid-regions of the pier are only required to sustain relatively low compression stresses and consequently the level of confinement to the concrete in this area may be reduced. It should be noted that all longitudinal bars in the pier should be restrained against buckling as required in subclause (c). In addition, the minimum requirements of 10.3.5.2 apply over the whole section and the full length of the column. A3

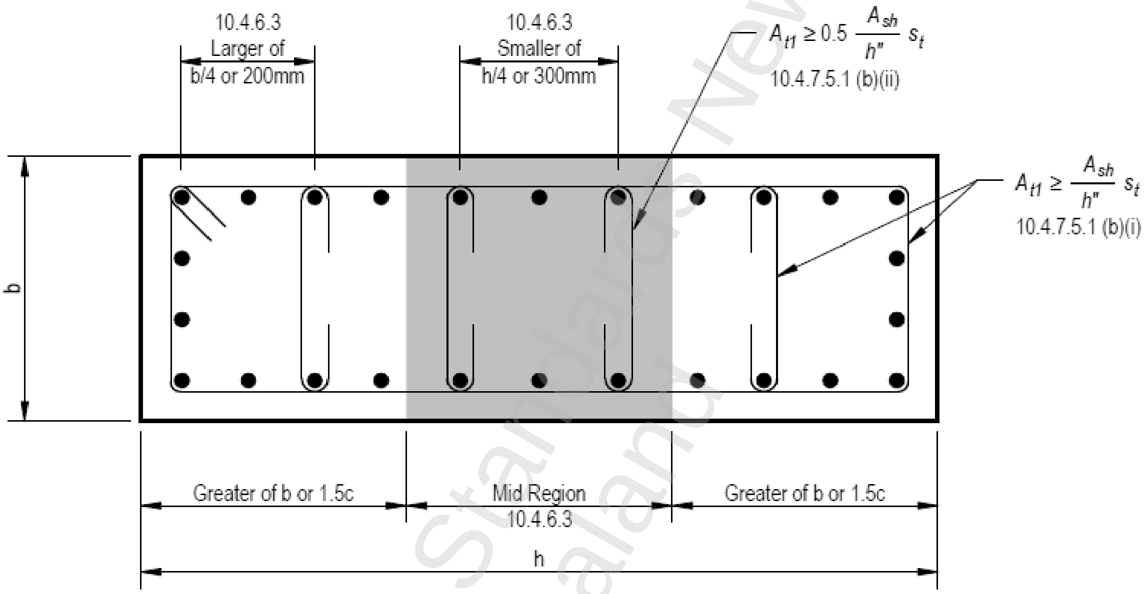


Figure C10.9 – Reduction in cross-tying of column bars in ductile potential plastic hinge regions

The reduction in cross-tying allowed in the cross-sectional mid-region of a column of high length to width ratio, for ductile design, is in line with the increased spacing of cross-tied bars allowed in mid-regions in 10.3.8.3 (see Figure C10.9).

REFERENCES

10.1 MacGregor, J.G., Breen, J.E. and Pfrang, E.O., "Design of Slender Concrete Columns", ACI Journal, Proceedings Vol. 67, No. 1, Jan. 1970, pp.6-28.

10.2 "New Zealand Reinforced Concrete Design Handbook", Cement and Concrete Association of New Zealand, Wellington 1992. (Updated May 2000)

10.3 "Guide to Design Criteria for Metal Compression Members," 2nd Edition, Column Research Council, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Penn., 1966.

10.4 ACI Committee 340, Design Handbook in Accordance with the Strength Design Method of ACI 318-77, V.2 – Columns, SP-17A(78), American Concrete Institute, Farmington Hills, MI, 1978, 228 pp.

10.5 "Code of Practice for the Structural Use of Concrete. Part 1, Design Materials and Workmanship", (CP 110: Part 1, Nov. 1972) British Standards Institution, London, 1972, 154 pp.

10.6 Mirza, S.A., Lee, P.M. and Morgan, D.L., "ACI Stability Resistance Factors for RC Columns, ASCE Structural Engineering, Vol. 113, No. 9, September 1987, pp. 1963–1976.

10.7 Mirza, S.A., "Flexural Stiffness of Rectangular Reinforced Concrete Columns", ACI Structural Journal, Vol. 87, No. 4, July–August 1990, pp. 425–435.

10.8 Park, R. and Paulay, T., “Reinforced Concrete Structures”, John Wiley & Sons, 1975, 769 pp.

10.9 Paulay, T. and Priestley, M.J.N., “Seismic Design of Reinforced Concrete and Masonry Buildings”, Wiley Interscience, New York, 1992, 774 pp.

10.10 Bianchini, A.C., Woods, Robert E. and Kesler, C.E., “Effect of Floor Concrete Strength on Column Strength,” ACI Journal, Proceedings, Vol. 56, No. 11, May 1960, pp. 1149–1169.

10.11 Ospina, C.E. and Alexander, S.D.B., “Transmission of Interior Concrete Column Loads through Floors,” ASCE Journal of Structural Engineering, Vol. 124, No. 6, 1998.

10.12 Matthews J., Bull D. and Mander J., "The Performance of Hollow-core Floor Slabs and the Effects They Have on Their Supporting Beams following a Severe Earthquake", Proceedings fib Symposium on Concrete Structures in Seismic Regions, Athens, 2003

10.13 “Reinforced Concrete Column Investigation Tentative Final Report of Committee 105”, ACI Journal, Proceedings Vol. 29, No. 5, Feb. 1933, pp. 275-282.

10.14 Watson, S., Zahn, F.A. and Park, R., “Confining Reinforcement for Concrete Columns”, Journal of Structural Engineering, ASCE, Vol. 120 , No.6 , June 1994.

10.15 Watson, S. and Park, R., “Simulated Seismic Load Tests on Reinforced Concrete Columns”, Journal of Structural Engineering, ASCE, Vol. 120, No. 6, June 1994.

10.16 Potangaroa, R.T., Priestley, M.J.N. and Park, R., “Ductility of Spirally Reinforced Concrete Columns Under Seismic Loading”, Research Report 79-8, Department of Civil Engineering, University of Canterbury February 1979.

10.17 Gill, W.D., Park, R. and Priestley, M.J.N., “Ductility of Rectangular Reinforced Concrete Columns with Axial Load”, Research Report 79-1, Department of Civil Enginemen, University of Canterbury, February 1979.

10.18 Ang, B.G., Priestley, M.J.N. and Park, R., “Ductility of Reinforced Concrete Bridge Piers Under Seismic Loading”, Research Report 81-3, Department of Civil Engineering, University of Canterbury, February 1981, 109 pp.

10.19 Mander, J.B., Priestley, M.J.N. and Park, R., “Seismic Design of Bridge Piers”, Research Report 84-2, Department of Civil Engineering, University of Canterbury, February 1984, 483 pp.

10.20 Zahn, F.A., Park, R. and Priestley, M.J.N., “Design of Reinforced Concrete Bridge Columns for Strength and Ductility”, Research Report 86-7, Department of Civil Engineering, University of Canterbury, March 1986, 330 pp.

10.21 Li Bing., Park, R. and Tanaka. H., “Strength and Ductility of Reinforced Concrete Members and Frames Constructed Using High Strength Concrete”, Research Report 94-5, Department of Civil Engineering, University of Canterbury, May 1993, 373 pp.

10.22 Mander, J.B., Priestley, M.J.N. and Park, R., “Theoretical Stress-strain Model for Confined Concrete”, Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1804-1826.

10.23 Allington, C.J., “Seismic Performance of Moment Resisting Frame Members Produced from Lightweight Aggregate Concrete”, PhD Thesis, Department of Civil Engineering, University of Canterbury 2003, p. 459.

Table C10.2 – Design of reinforced columns

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy
Material limitation applicable to the detailing described in this table	Range limitation on concrete compressive strength, f'_c	20 to 100 MPa (5.2.1)	25 to 70 MPa for ductile elements (5.2.1)
	Limitation on longitudinal reinforcement yield strength, f_y	Not greater than 500 MPa (5.3.3)	Same as for nominally ductile
	Limitation on transverse reinforcement yield strength, f_{yt}	Not greater than 500 MPa for shear and 800 MPa for confinement (5.3.3)	Same as for nominally ductile
	Reinforcement class as per AS/NZS 4671	Class E	Same as nominally ductile
Ductility	Curvature ductility achievable through tabled detailing	Clause 2.6.1.3.4 Table 2.4(a) and (b)	Clause 2.6.1.3.4 Table 2.4 (a) and (b)
Slender columns		(10.3.2)	Refer dimensional limitations below
Compressive load limitations	Maximum axial compressive load	$0.85\phi N_{n,max}$ where, $\phi = 0.85$ (10.3.4.2)	$0.7\phi N_{n,max}$, use $\phi = 1.0$ when column actions determined by capacity design (10.4.4)
Dimensional limitations	Dimension of column	Refer slender column requirements	$b_w \geq L_n/25$ (10.4.3) $b_w \geq \sqrt{(L_n h/100)}$ (10.4.3) refer 10.4.3.3 for cantilevered columns
Column plastic hinge detailing	Extent of ductile detailing length, ℓ_y, for detailing purposes	Not applicable	$N^*_o \leq 0.25\ \phi f'_c A_g$ (10.4.5): Greater of h, diameter or where moment exceed $0.8\ M_{max}$ $0.25\phi f'_c A_g < N^*_o \leq 0.5\ \phi f'_c A_g$ (10.4.5): Greater of 2 times (h, diameter) or where moment exceed $0.7\ M_{max}$ $N^*_o > 0.5\ \phi f'_c A_g$ (10.4.5): Greater of 3 times (h, b_w diameter) or where moment exceed $0.6\ M_{max}$
	Provide required special detailing in potential plastic hinge regions when	Not applicable	Column hinging is expected. If column hinging is prevented use reduced detailing requirements.
Strength reduction factors	Strength reduction factors	(2.3.2.2)	$\phi = 1.0$ when actions are derived from overstrength (2.3.2.2)
Overstrength factors		Not applicable	Where column hinging is expected use 2.6.5.5 and 2.6.5.6

Table C10.3 – Design of reinforced columns (Continued)

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy
Longitudinal reinforcement detailing	Minimum longitudinal reinforcement ratio	0.008 A_g (10.3.8.1)	Same as for nominally ductile
	Maximum longitudinal reinforcement ratio	0.08 A_g (10.3.8.1)	18 A_g/f_y (10.4.6.2)
	Maximum longitudinal reinforcement ratio at splices	0.08 A_g (10.3.8.1)	24 A_g/f_y (10.4.6.2)
	Limitations on the position of lap splices in columns	No limitations	Central quarter unless can show high degree of protection against plastic hinges forming in columns (10.4.6.8.2) refer also Method A of Appendix D
	Minimum number of longitudinal bars	8 bars, but may be reduced 6 or 4 if clear spacing is less than 150 mm and $N^* \leq 0.1\phi f'_c A_g$ (10.3.8.2)	Same as for nominally ductile (10.3.8.2)
	Maximum spacing between longitudinal bars requiring restraint	Circular columns, larger of one quarter of a diameter or 200 mm Rectangular, larger of one third of column dimension in direction of spacing or 200 mm, spacing can be increased in centre of column when $h/b > 20$ (10.3.8.3)	Larger of one-quarter of the column dimension (or diameter) in direction of spacing or 200 mm (10.4.6.3) In protected plastic hinge regions and outside plastic hinge regions use same as nominally ductile (10.4.6.4)
	Maximum longitudinal column bar diameter	No limitations	$\frac{d_b}{h_b} \leq 3.2\frac{\sqrt{f'_c}}{f_y}$ (10.4.6.6) Bar diameter can be increased by 25 % when plastic hinges are not expected to develop in column end zones and need not be met when bars remain in tension or compression over the length of the joint.
Transverse reinforcement <u>outside</u> of the potential plastic hinge region	Minimum diameter for transverse reinforcement	Rectangular hoops and ties (10.3.10.7.1) 5 mm for $d_b < 20$ 10 mm for $20 \leq d_b \leq 32$ 12 mm for $d_b > 32$ or bundled bars Spirals or hoops of circular shape	Same as for nominally ductile

Table C10.3 – Design of reinforced columns (Continued)

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy
		(10.3.10.7.2) 5 mm	
	Maximum vertical spacing of ties	Smaller of $h/3$, $b_w/3$, diameter/3, or $10d_b$ (10.3.10.5.2 and 10.3.10.6.2)	Same as for nominally ductile (10.4.7.4.4 or 10.4.7.5.4)
	Anti-buckling reinforcement	Rectangular hoops and ties $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}} \frac{s}{d_b}$ (10.3.10.6.1) Spirals or hoops of circular shape $p_s = \frac{A_{st}}{155\,d''} \frac{f_y}{f_{yt}} \frac{1}{d_b}$ (10.3.10.5.1)	Same as elastic (10.4.7.4.4 or 10.4.7.5.4)
	Confinement reinforcement	Rectangular hoops and ties $A_{sh} = \frac{(1-p_t m)s_h h''}{3.3} \frac{A_g}{A_c} \frac{f'_c}{f_{yt}} \frac{N^*}{\phi f'_c A_g} - 0.0065 s_h h''$ (10.3.10.6.1) Spirals or hoops of circular shape $p_s = \frac{(1-p_t m)}{2.4} \frac{A_g}{A_c} \frac{f'_c}{f_{yt}} \frac{N^*}{\phi f'_c A_g} - 0.0084$ (10.3.10.5.1)	Same as nominally ductile (10.4.7.4.4 or 10.4.7.5.4)
	Minimum shear reinforcement	$A_v = \frac{1}{16}\sqrt{f'_c}\frac{b_w s}{f_{yt}}$ (10.3.10.4.4)	$A_v = \frac{1}{12}\sqrt{f'_c}\frac{b_w s}{f_{yt}}$ (10.4.7.2.7)
	Maximum shear force	$V_n \le 0.2 f'_c b_w d$, or $8\, b_w d$ (10.3.10.2.1)	Same as nominally ductile
	Shear strength provided by concrete	Refer to 10.3.10.3	Refer to 10.3.10.3

Table C10.3 – Design of reinforced columns (Continued)

	Design issue	**Nominally ductile seismic design philosophy**	**Ductile seismic design philosophy**
Transverse reinforcement <u>within</u> potential plastic hinge region	Minimum diameter for transverse reinforcement	Same as outside plastic hinge region	Same as for nominally ductile
	Maximum vertical spacing of ties		In DPRs smallest of $h/4$, $b/4$, diameter/4 or 6 d_b (10.4.7.4.5(a), 10.4.7.5.5 (a)) In LDPRs smallest of $h/4$, $b/4$, diameter/4 or 10 d_b (10.4.7.4.5(b), 10.4.7.5.5 (b))
	Anti-buckling reinforcement		For rectangular hoops and ties In DPRs & LDPRs $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s_h}{d_b}$ (10.4.7.5.1) Regions protected from hinging $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}} \frac{s_h}{d_b}$ (10.4.7.5.3) Spirals or circular hoops In DPRs & LDPRs $p_s = \frac{A_{st}}{110 d''} \frac{f_y}{f_{yt}} \frac{1}{d_b}$ or (10.4.7.4.1) In regions protected from hinging $A_{te} = \frac{A_{st} f_y}{155 d'' f_y d_b}$ (10.4.7.4.4)
	Confinement reinforcement		For rectangular hoops and ties In DPRs $A_{sh} = \frac{(1.3 - p_t m) s_h h''}{3.3} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N_o^*}{\phi f_c' A_g} - 0.006\, s_h h''$ (10.4.7.5.1) In LDPRs and regions protected from hinging use 70 % of this area (10.4.7.5.2 and 10.4.7.5.3) Spirals or circular hoops In DPR $p_s = \frac{(1.3 - p_t m)}{2.4} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N_o^*}{\phi f_c' A_g} - 0.0084$ (10.4.7.4.1) In LDPRs and regions protected from hinging use 70 % of this area

Table C10.3 – Design of reinforced columns (Continued)

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy
			(10.4.7.4.2 and 10.4.7.4.3)
	Shear resisted by concrete	V_c given by 10.3.10.3.1	DPR $v_c = 3v_b\left[\frac{N_o^*}{A_g f'_c} - 0.1\right]$ LDPR $v_c = v_b\left[0.5 + \left(\frac{N_o^*}{A_g f'_c} - 0.1\right)\right] \geq 0.0$ Outside ductile detailing lengths as for 10.3.10.3.1
	Minimum shear reinforcement	Same as outside plastic hinge region	Same as for nominally ductile design
	Maximum shear force	Same as outside plastic hinge region	Same as outside plastic hinge region
	Shear reinforcement	Same as outside plastic hinge region	Refer 10.4.7.2

NOTES

C11 DESIGN OF STRUCTURAL WALLS FOR STRENGTH, SERVICEABILITY AND DUCTILITY

C11.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 11 of Part 1. A3

M_E moment at the base of a wall resulting from lateral earthquake forces specified by AS/NZS 1170 and NZS 1170.5, N mm

M_o overstrength moment of resistance of section at the base of a cantilever wall, N mm

t_1 thickness of boundary element in the direction of wall length, mm A3

V_E shear at the base of a wall resulting from lateral earthquake forces specified by AS/NZS 1170 and NZS 1170.5, N

C11.2 Scope

Section 11 requires that walls be designed to resist loads to which they are subjected, including eccentric gravity loads and lateral forces due to wind or earthquake, which may result in actions in the plane of, or transverse to the wall[11.1]. In general this section applies to walls spanning vertically between horizontal supports.

Walls need to be designed for combined flexure, axial load and shear according to 11.3.11, considering the wall to be a member subject to axial force and flexure, while also satisfying the requirements of 11.3.12 with respect to vertical and horizontal reinforcement. A3

C11.3 General principles and design requirements for structural walls

This clause lists well established general requirements generally adopted from those in ACI 318. Walls, being relatively stiff elements, will in general be subjected to earthquake forces in New Zealand. Accordingly, most of the general requirements here will need to be supplemented or modified in accordance with 11.4. A3

C11.3.1.1 *General*

In the majority of buildings out-of-plane moments in walls, or in parts of walls such as flanges, are induced by compatibility requirements. The magnitude of lateral forces resisted by these actions is generally small. Consequently, redistribution of the lateral shears can be used to transfer these forces to in-plane actions in other walls or to other stiffer lateral force-resisting structural components.

C11.3.1.5 *Singly reinforced walls*

Although research and some experiences in the Canterbury Earthquake sequence have demonstrated ductility of singly reinforced walls, these walls generally lack the robustness and ductility to sustain significant damage while retaining lateral stability, particularly when considering multi-directional actions. To allow for the uncertain performance of singly reinforced walls in earthquakes, designing these walls for nominal ductility (μ = 1.25) combined with a strength reduction factor of 0.7 (in 2.3.2.2) essentially results in this type of wall behaving elastically in the ultimate limit state, and such walls should sustain only limited inelastic deformation in the maximum considered earthquake.

For singly reinforced walls, the reinforcement content is required to satisfy 11.3.12.3 to ensure adequate ductility for out-of-plane actions.

A3 **C11.3.1.6** *Maximum axial actions*

The axial loading on walls in buildings is difficult to determine with any level of accuracy. To ensure adequate performance of nominally ductile walls the axial load level has been limited as indicated in the clause.

C11.3.5.1.2 *Design moment and P-delta effects – simplified method*

Section 11.3.5.1.2 is based on the corresponding requirements in the ACI 318-02 and experimental research.

Panels that have windows or other large openings are not considered to have constant cross section over the height of the panel. Such walls are to be designed taking into account the effects of openings.

Equation 11–3 is derived by adopting the following assumption:

(a) The panel is simply supported at the top and bottom
(b) The deflected shape approximates that of a parabola
(c) The wall is cracked sufficiently that the effective moment of inertia approaches that of I_{cr}.

In many instances the wall may have some fixity at the base which may mean that the assumption of simple fixity introduces a degree of conservation. However, foundation flexibility or yielding of the reinforcement of single reinforced walls under seismic action may mean that the base restraint rapidly approaches that of a pin.

In low seismicity regions, where the design force load moment is low compared to the moment to cause cracking, the use of a rational analysis to calculate the design moment including incorporation of the P-delta effect may be more appropriate.

A3 **C11.3.5.2.1** *Limitation on use of method*

The axial load ratio of 0.015 ($N^{*}/ f'_c A_g$) is the extent of axial load ratio in the experimental data set used as the basis for this clause.

C11.3.5.2.2 *Prevention of flexural torsional buckling of walls loaded in-plane with low axial loads*

The effective length of a wall for flexural torsional buckling is a function of the wall length and the degree of restraint from rotation and lateral movements at the support points. When considering the in-plane effective length it is also important to consider the out-of-plane design philosophy. If walls are designed to be ductile for out-of-plane loads, hinges are likely to form at the supports. The development of these hinges will reduce the rotational restraint capability of these support points under in-plane loads. Table 11.1 recognises the reduction of rotational restraint available to the wall by specifying that k_{ft} = 1.0 when plastic hinges form at the base of a wall designed to contain nominally ductile plastic hinge regions for in-plane loads.

If a plastic hinge was to form in the mid-height of a wall subjected to seismic face loads, the flexural torsional performance of the wall under subsequent in-plane loads could be compromised by initial eccentricities being present at the mid-height of the wall, or due to the loss of buckling resistance due to the formation of hinges at the top, bottom, and mid-height of the wall. Hence 11.3.5.2.1(c) limits the design method to walls in which plastic hinges do not form at the wall mid-height under face loads.

The design equations provided in 11.3.5.2.2 where developed from tests conducted on walls subjected to in-plane loads with low axial loads. The axial load limitations applicable to this method in 11.3.5.2.1(a), ensures that the design equations are not used beyond the available test data base.

The equations provided in 11.3.5.1.2 relate to the prevention of flexural torsional buckling of walls with low axial loads when subjected to in-plane lateral loads. Such walls are common in warehouse type structures.

Equation 11–8 represents a modification of the Vaslov flexural torsional buckling equation. The upper limit on the maximum height to thickness ratio ensures that the equation is used within the bounds of the available test data.

The denominator of Equation 11–8 reflects the size of the concrete compression zone due to in-plane loads at the ultimate limit state. As the size of the compression block increases, so does the possibility of flexural torsional buckling. The size of the compression block is a function of the wall self weight, applied axial load, and amount of reinforcement in the wall. Clause 11.3.5.2.2 (a) will govern when the yielding of the reinforcement is expected under in-plane loads at the ultimate limit state. However, in structures with a considerable length of wall the minimum reinforcement requirement may dictate the volume of reinforcement and yielding of the reinforcement at the ultimate limit state may not occur in these instances, then Clause 11.3.5.2.2 (b) may govern. 11.3.5.2.2 (b) has been derived by considering the area of reinforcement required in an elastically responding wall with evenly distributed reinforcement.

C11.3.6.2 *Design for actions causing bending about the minor axis* A3

The limits provided for the height to thickness ratio below which moment magnification is not needed to be considered have been determined by rearranging the formula provided in 10.3.2.3.5. Where the calculated height to thickness ratio exceeds that stipulated in Equation 11–11 the thickness of the wall can be increased to ensure compliance with the limit, or the walls evaluated using the moment magnifier method outlined in 10.3.2.

The equations provided in this section refer to the consideration of Euler buckling due to an imposed axial load. The α_m factor considers the degree of restraint from rotational movement at each end, and whether the walls are prevented from sidesway.

The subscript "e" to the effective length factor k_e, is provided as the factor relates to Euler buckling.

C11.3.7 *Walls with high axial loads* A3

Walls may be subjected to both in-plane and out-of-plane actions during a seismic event. It is therefore desirable to provide a degree of robustness when axial loads are high. The minimum thickness requirement of 11.3.8 relates to the entire length of the compression zone. Walls designed for curvature ductilities consistent with limited or fully ductile structures may require larger thicknesses in the boundary zone.

C11.3.8 *Minimum thickness for compression flanges of walls*

When resistance of the design actions requires the development of compressive forces in the flange remote from the web, limitations are placed on the effective height to thickness ratio to prevent premature buckling and deterioration of the flanges, as flanges remote from the restraint of the web are prone to buckling. Flanges near the web are prevented from premature buckling by the stabilising influence of the web.

C11.3.11.3 *Design for shear in the plane of a wall*

Shear in the plane of a wall, as a design consideration, is primarily of importance for walls with a small height to length ratio. The design of higher walls, particularly walls with uniformly distributed reinforcement, will probably be controlled by flexural considerations. It is, therefore, essential that the flexural strength of walls be computed, along with their shear strength.

C11.3.11.3.2 *Maximum shear stress* A3

Although the wall length to wall thickness ratio is less than that for ordinary beams, tests [11.2] on walls with a thickness equal to L_w /25 imply that in the absence of earthquake-induced forces, the limitations of 11.3.11.3.2 for maximum shear stress are relevant. In conformity with the requirement of 11.3.11.3.2 the maximum shear stress is limited in terms of the compression strength, f'_c. The thinnest section of the wall, which may occur where horizontal recesses reduce the thickness, must be considered in computing the shear stress. A3

The total nominal shear stress, v_n, at any section, including the base of the wall, is limited in accordance with 11.3.11.3.2. A3

A3 **C11.3.11.3.4 and C11.3.11.3.5** *Concrete shear strength – simplified and detailed*

Equation 11–15 predicts the inclined cracking strength at any section through a wall, and corresponds approximately to the occurrence of a flexural tensile stress of $0.05\sqrt{f_c'}$ at a section $L_w/2$ above the section being investigated. As the term $\left(\frac{M^*}{V^*} - \frac{L_w}{2}\right)$ decreases, Equation 11–15 will control before this term becomes very small or negative.

A3 **C11.3.11.3.6** *Shear design of sections near base of walls*

The values of V_c computed from Equation 11–15 at the section located a distance $L_w/2$ or $h_w/2$ above the base apply to that and all sections between this section and the base.

Unlike what may be permitted in slabs, it is envisaged that, irrespective of the concrete contribution to shear resistance, a minimum amount of shear reinforcement will be provided in all wall elements.

A3 **C11.3.11.3.8** *Design of horizontal shear reinforcement*

In the design for shear strength of walls, sufficient horizontal shear reinforcement is required to carry the shear exceeding V_c. The minimum horizontal and vertical reinforcement ratio should not be less than $0.7/f_y$. This is 0.23 % when steel with f_y = 300 MPa is used and it is more than that required for shear in beams in accordance with Equation 9–10.

A3 The maximum spacing of reinforcement in reinforced concrete walls is 300 mm to be consistent with the requirements of 2.4.5, where 300 mm is the maximum allowable spacing for crack control requirements.

C11.3.11.3.9 *Detailing of vertical shear reinforcement*

In almost all walls, the vertical reinforcement required for shear will be less than that required for flexure. The longitudinal flexural reinforcement will provide the required shear reinforcement as well, and additional shear reinforcement will not be required, except in some squat walls (see 11.3.12.8). That is, the requirements for vertical shear and flexural reinforcement are not additive.

C11.3.12.3 *Minimum and maximum area of reinforcement*

If the quantity of tension reinforcement is too small, the moment strength of a reinforced concrete section can be less than that of the corresponding cracking moment, resulting in a possible sudden non-ductile failure. To prevent such a failure, a minimum amount of tension reinforcement is required to ensure that the flexural strength of the section (after cracking) is at least equal to the moment when cracking first occurs computed using the modulus of rupture of the concrete.

It should be noted that f_c' is the specified 28-day concrete strength, and that the actual concrete strength may be significantly higher. Particular attention should be given to situations where a higher than specified concrete strength may be used, such as in precast construction and when using self-compacting concrete.

The minimum vertical reinforcement requirements in 11.3.12.3 are only suitable for walls designed for nominal ductility and are insufficient to ensure well-distributed secondary cracks in plastic hinge regions[11.8,11.9]. Additional vertical reinforcement is required to ensure walls have sufficient ductility when designed for limited ductile or ductile response, as outlined in 11.4.4.2.

Table C11.1 – Minimum values of p_ℓ given by 11.3.12.3 for walls

f_c' (MPa)	f_y = 300 MPa	f_y = 500 MPa
30	0.0046	0.0027
40	0.0053	0.0032
50	0.0059	0.0035

C11.3.12.5 *Anchorage of shear reinforcement at wall ends* A3

Anchoring of horizontal reinforcement requires vertical bars at each corner.

The ends of wall segments should be locally confined to provide development of horizontal reinforcement, and transfer of loads through a diagonal compression strut into the end vertical (longitudinal) bars. A continuous U-shaped bar which encloses the two outermost corner bars, which is effectively lapped with the horizontal shear reinforcement in the wall, is considered to provide this anchorage effectively. See Figure C11.1A below.

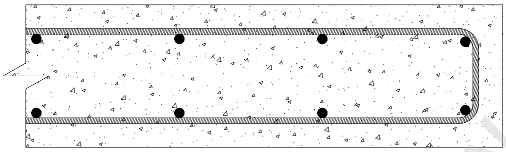


Figure C11.1A – U-shaped bar for end anchorage

When U-shaped bars are used for anchorage, horizontal bars should be lapped with overlapping hooks, anchored around vertical reinforcement, as shown in Figure C11.1B.

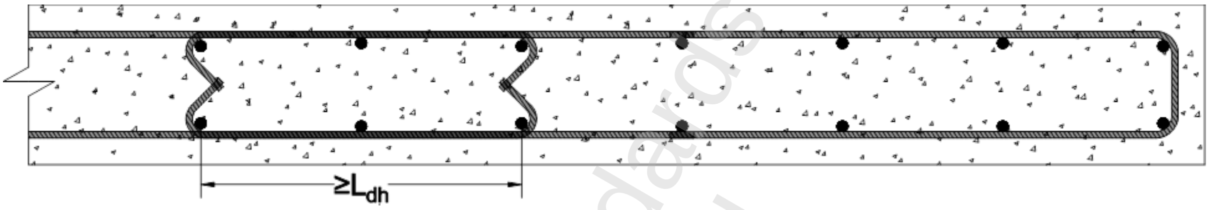


Figure C11.1B – Horizontal bar laps

Instead of using continuous U-shaped bars, effective anchorage may be achieved by enclosing the two outermost corner bars with 135° hooks, and by locally enclosing those two corner bars with a tie, as shown in Figure C11.1C.

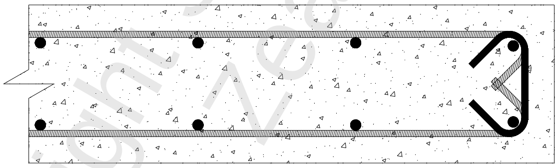


Figure C11.1C – Corner bars anchored with hooks and local ties

Effective anchorage may also be provided by hooking the horizontal shear reinforcement with 90° bends around the two outermost corner bars, and additionally enclosing at least the four outermost bars with a closed stirrup cage. The cage is required to have the equivalent area and vertical spacing as the horizontal bars which are being anchored. See Figure C11.1D. The distance between the innermost face of the enclosed cage and the hook end should be at least $12d_b$.

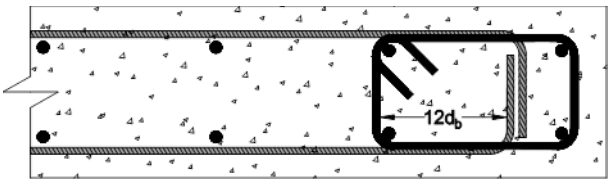


Figure C11.1D – Horizontal reinforcement anchored at wall end with 90° hooks and cage enclosing four outermost bars

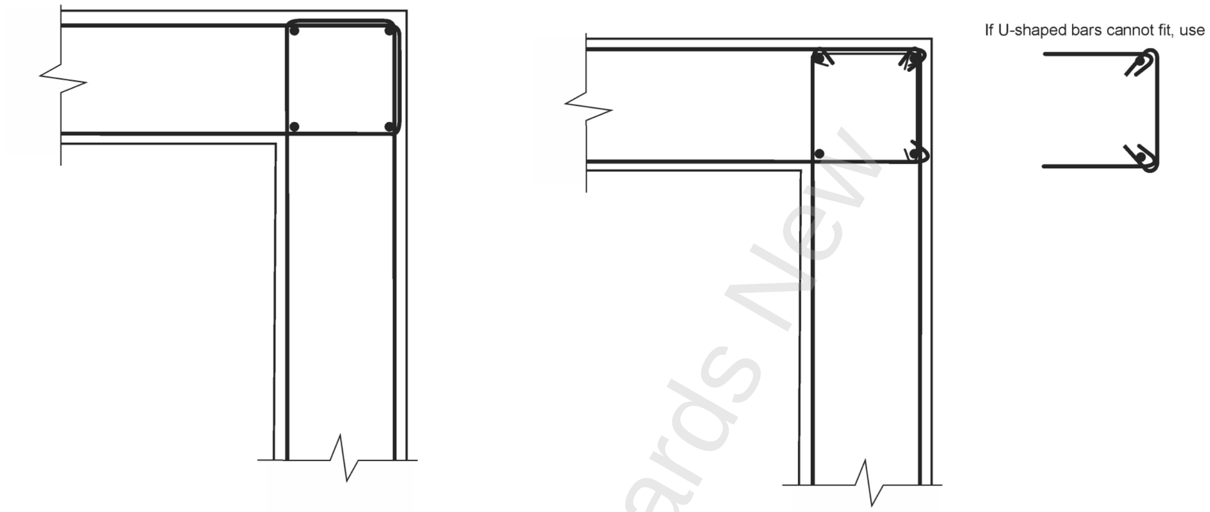
C11.3.12.6 *Anchorage of shear reinforcement at intersections of walls*

At web/flange intersections of a wall, the horizontal reinforcement will be anchored with 90° hooks inside the core concrete at the outermost face of the intersecting wall. At the junction of a T-shaped wall, only the

A3 innermost bar is effective in resisting shear. Bar hooks located in the cover concrete are not effectively anchored.

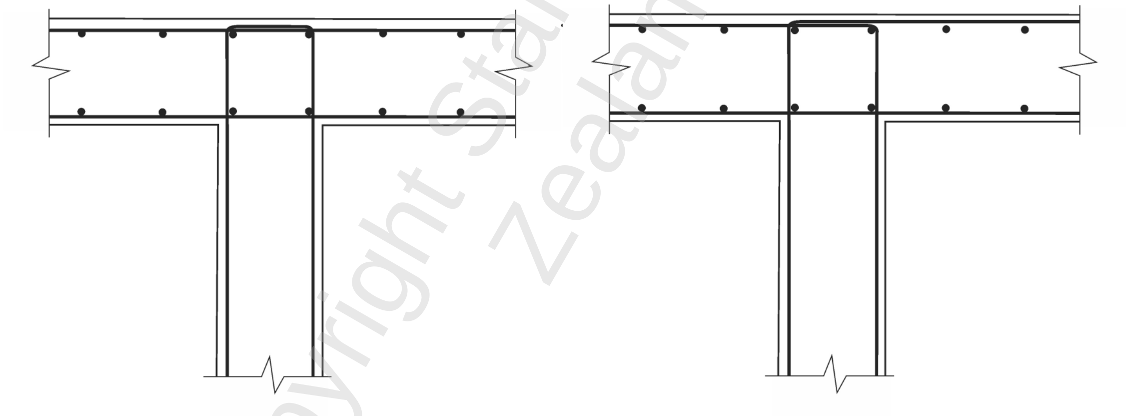
The use of U-shaped bars is effective also, as shown in Figure C11.1E for L-shaped walls, and in Figure C11.1F for T-shaped walls.

The 90° hook may extend into the vertical plane where the wall width does not permit the hook to be horizontal.



(a) (b)

Figure C11.1E – Intersection of bars at L intersection



(a) (b)

Figure C11.1F – Intersection of bars at T-shaped wall

C11.3.12.7 *Curtailment of flexural reinforcement*

The design flexural demand envelope for a given wall will be influenced by the interaction with other parts of the structure, walls and frames. This interaction occurs, typically, via floor diaphragms.

Allowance should be made for higher mode effects or dynamic magnification influencing the flexural demand up the wall.

Additional actions introduced into the wall through connections such as ground anchors, transfer beams, ramps, stairs and linkages from other structures will modify the bending moment profiles for walls. Such effects should be accounted for rationally.

C11.3.12.8 *Vertical reinforcement in squat walls* A3

In structures where the aspect ratio is low, such as can occur in podium buildings, the vertical flexural reinforcement may be insufficient to maintain shear resistance. A strut and tie analysis may be carried out in this situation. However, a simpler solution is to ensure the vertical proportion of reinforcement at any location along the wall is equal to or greater than given by Equation 11–19. It should be noted that A_{wv} corresponds to a minimum area of reinforcement for shear. If the reinforcement provided for flexure exceeds this value then no additional vertical reinforcement is required for shear.

In structural walls where the aspect ratio (h_w/L_w) is small the flexural design criteria of plane sections remaining plane is no longer valid. For this reason the strut and tie method should be used. With this approach the shear stress resisted by concrete, v_c, is taken as zero. The strut angles of the compression forces to the horizontal plane, which are used to resist shear and flexure, are required to equal or exceed 30°. The limit of 25° given in A4.5 is increased to 30° to allow for the inelastic cyclic loading that occurs in earthquakes. The equations in 11.2.12.8 are based on a strut angle of 45°.

C11.4 Additional design requirements for members designed for ductility in earthquakes

C11.4.1 *General seismic design requirements*

C11.4.1.1 *Maximum axial actions for ductile walls* A3

The peak axial load that is sustained in a wall during an earthquake is difficult to determine due to the opening up of flexural cracks causing the wall to elongate. This action becomes particularly significant when a plastic hinge forms. The increase in height of the wall is generally partially restrained by beams and slabs and other vertical elements in the structure. This restraint induces compression in the wall and tension in surrounding vertical gravity load resisting elements, namely the columns and other walls. A major difficulty exists in the calculation of the induced axial force as the level of restraint depends on:

(a) The flexural and torsional stiffness values of the beams and slabs, and of the tensile and compressive membrane actions in these elements;

(b) The tensile and axial compressive stiffness of the vertical load resisting elements.

The major difficulty arises in establishing the appropriate values for the flexural and torsional stiffness of slabs, beams and columns, as these values vary with the level of flexural cracking during an earthquake. This makes it difficult to carry out an accurate analysis. A further complication arises as the majority of practical methods of analysis do not model elongation in structural elements.

C11.4.1.2 *Design of ductile walls*

Clause 11.4 makes provisions for the design of ductile structural walls in buildings at the ultimate limit state. Therefore the general requirements with respect to analysis, design forces, ductilities and capacity design procedures, in accordance with 2.6.8 must also be considered.

C11.4.1.3 *Effective flange projections for walls with returns*

In all walls, nominally ductile, limited ductile and fully ductile, the nominal moment capacity of a wall, M_n, should be based on the one horizontal to two vertical spread of flange action for including tension reinforcement, according to 11.3.1.3[11.3]. A3

In walls where the overstrength flexural action of the walls needs to be considered, it is expected that more of the flange will be engaged in tension. This means that the overstrength moment for a flanged wall is determined by the reinforcement in tension over a flange width based on a 45° spread from the top of the wall. The effective width of each flange is therefore taken as 1 times the height of the wall above the critical section[11.3]. A3

C11.4.2 *Ductile detailing lengths* A3

For the potential plastic hinge zone, where the contribution of the concrete towards shear resistance is to be evaluated from 11.3.11.3.2, modifications for walls relative to columns are introduced to take into account the relative dimensions of cantilever or coupled walls which may be different from those of columns. The potential plastic hinge zone normally extends from the level of the wall at which the critical base moment can develop.

A3 For reasons outlined for columns in NZS 3101:1995, diagonal shear reinforcement in potential plastic hinge regions of walls will very seldom be required. Squat walls may be an exception, as discussed in C11.4.6.4. The maximum shear stress in walls is usually limited by Equation 11–28 rather than by 9.4.4.1.4(a).

Outside the potential plastic hinge region the full value of V_c, as given in 11.3.11.3.4 or 11.3.11.3.5, may be used in determining the horizontal shear reinforcement, provided that no yielding of the flexural reinforcement is likely to occur. This is discussed at the end of C11.4.5.5.

The ductile detailing length for coupled walls should extend over a height of two storeys. The usual criteria given in 11.4.2 do not apply as the restraint from the foundation beam can prevent elongation occurring at the first level coupling beam. This can result in a greatly increased shear resistance in this coupling beam resulting in plastic deformation occurring above this coupling beam.

C11.4.3 *Dimensional limitations*

Theoretical and experimental research[11.3] indicates that the potential for out-of-plane buckling in the plastic hinge region of ductile walls arises after the critical boundary region has been subjected to large inelastic tensile strains. Upon reversal of the earthquake forces, previously formed wide cracks must close before the flexural rigidity of the section, necessary for stability, can be restored. As a result of uneven closure of cracks at this stage, out-of-plane buckling has been observed.

The major parameters that affect wall instability under such circumstances are:
(a) Maximum steel tensile strains gauged by the curvature ductility demand;
(b) The thickness of the wall in the critical boundary region;
(c) Arrangement of the wall reinforcement, i.e. one or two layers of bars;
(d) The quantity of vertical reinforcement present in the boundary region. As the reinforcement content, p_ℓ, increases, the closure of previously formed cracks is delayed;
(e) The probable buckling length.

Although the relationship between these parameters is relatively simple, expressions derived from first principles do not lend themselves readily for routine design without incorporation into an appropriate computer program. Therefore to facilitate easy use, a number of simplifications, mainly involving approximations with linear relationships where non-linear relationships[11.3] exist, were introduced without significant loss of accuracy.

A3 **C11.4.3.1** *Prevention of buckling of thin walls loaded in-plane*

To safeguard against premature buckling, the thickness in the boundary region of the wall section, where
A3 under reversing moments large inelastic strains may be generated, should not be less than t_m given by Equation 11–20. Curvature ductility demands, and hence maximum tensile strains, are estimated with the structural ductility factor μ used in establishing the magnitude of the lateral design forces for the ultimate limit state, and the aspect ratio $A_r = h_w/L_w$ of the wall. The parameter ξ given by Equation 11–22 gauges the effect of the quantity of vertical tension reinforcement in the boundary region of the wall section in
A3 restraining the closure of cracks upon moment reversals. When the potential plastic hinge region of the section is heavily reinforced so that $p_\ell \geq 0.04$, instability becomes insensitive to the reinforcement content and hence for these cases $\xi = 0.1$ may be used. In the majority of cases for nominally rectangular ductile walls, shear requirements will govern the web thickness of the wall, and boundary elements will not be needed.

Where the buckling length, assumed to be equal to the theoretical length of the plastic hinge, approaches or exceeds the unsupported height of the wall in the first storey, the limitation of Equation 11–20 becomes overly severe. In such cases, which are encountered when the length of a wall relative to the height of the first storey becomes large, it is assumed that the buckling length is equal to 80 % of the clear unsupported height of the wall. This is accounted for by Equation 11–21.

In Equation 11–22, p_ℓ is to be calculated for the vertical reinforcement in the boundary region only.

The term α_r is added to Equation 11–20 as it was found[11.3] that when the criteria of 11.4.3.2 are applied to walls with a single central layer of vertical reinforcement, at least a 25 % increase of the wall thickness is required to prevent instability due to out-of-plane buckling. When large ductilities are to be developed these requirements will generally necessitate wall thicknesses in excess of 200 mm, for which 11.3.12.2 requires the placement of two layers of reinforcement. A3 A3

Where a wall interacts with frames in ductile dual structures, considered in 6.9.1.4, the maximum value of the ratio $\frac{M_e}{V_e L_w}$ may be substituted in Equations 11–20 and 11–21 for the wall aspect ratio, A_r. A3

C11.4.3.3 *Dimensions of enlarged boundary element*

When stability criteria govern the geometry of the wall section, it will be necessary to thicken the wall in boundary regions. This is readily achieved by providing flange elements with sufficient dimensions so as to provide adequate flexural rigidity at the end of the wall section. Equation 11–23 specifies the minimum dimensions for such elements and Figure C11.1 summarises possible applications.

A3

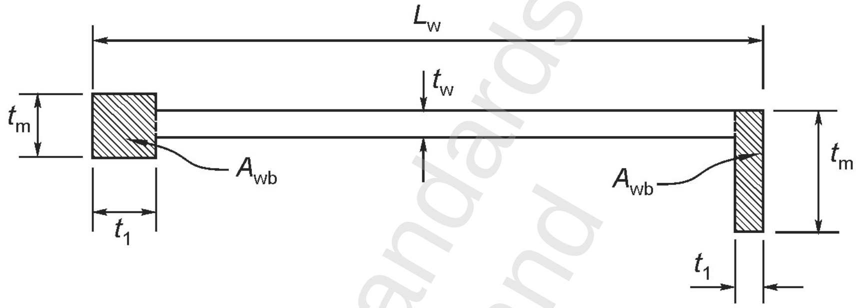


Figure C11.1 – Minimum dimensions of boundary elements of wall sections in plastic hinge regions

C11.4.3.4 *Flange thickness for prevention of instability within plastic hinge region* A3

This clause relates to stability requirements and is separate from the effective flange width for determining the flexural capacity. The area of a flange intended to stabilise the stem of a wall should be determined from Equation 11–23. The limitation of 11.3.7 safeguards against out-of-plane buckling of thin and wide flanges. If the reinforcement ratio in the flange is large and the flange width is greater than three times its thickness, Equation 11–20 may control flange thickness.

C11.4.4.1 *Reinforcement diameters*

Large diameter bars relative to member thickness can cause splitting cracks in the concrete. To guard against this bar sizes are restricted. The maximum diameter of bars is restricted to avoid the use of large bars in thin walls. The maximum diameter of the bar is to be based upon the thickness of the wall at the location of that bar.

C11.4.4.2 *Minimum area of reinforcement*

In addition to ensuring that the flexural strength of the section is at least equal to the moment when cracking first occurs, walls designed for a limited ductile or ductile response require additional vertical reinforcement to ensure that the design ductility can be achieved. If too little vertical reinforcement is used in walls, there is insufficient tension generated to replace the tensile resistance provided by the surrounding concrete after a crack forms, resulting in a reduced number of cracks in the critical moment region, large crack widths, and possible premature fracture of the reinforcing steel. Well-distributed flexural cracks in the plastic hinge region can be achieved by placing additional reinforcement at the end zone of the wall to initiate secondary cracks[11.10].

The requirement for minimum vertical reinforcement in the end zone was determined by equating the tensile force provided by the vertical reinforcement in the end zone with the expected average long-term

A3 tensile strength of the concrete in the end zone. The tensile force in the vertical reinforcement was calculated based on the average yield strength of G300 and G500 reinforcement with allowance for strength enhancement due to dynamic loading rates expected during earthquakes. The average long-term concrete tensile strength was based on the direct tensile strength given in C5.2.4 with allowance for the average supplied concrete compressive strength, long-term aging, shrinkage, and strength enhancement due to dynamic loading rates expected during earthquakes. Use of the average instead of the upper characteristic tensile strength was considered appropriate due to a number of factors that reduce the tensile force required to cause secondary cracking, including stirrups in the end zone, which act as stress raisers.

Development of minimum reinforcement equation

The end zone, requiring additional vertical reinforcement, extends $0.15L_w$ along the wall length at each end of rectangular walls, or from the ends of flanges and webs in non-rectangular walls. The end zone definitions are shown in Figure C11.2A for different wall geometry, and Figure C11.2B shows the minimum reinforcement requirements with a wall elevation.

In addition to concentrated reinforcement in the end zones, it is important to ensure that there is sufficient distributed reinforcement through the web region of the wall to allow the secondary cracks to propagate along the wall length[11.11]. If the quantity of vertical reinforcement in the end zone is significantly greater than the quantity of distributed vertical reinforcement in the web region, the wall can be susceptible to widely spaced cracks in the web which can result in poor shear resistance and higher shear deformations. To prevent this behaviour, the amount of distributed reinforcement in the web region should be at least 30% of the area of vertical reinforcement in the end zone. Table C11.2 gives values of reinforcement ratios determined for 11.4.4.2.

Table C11.2 – Minimum values of p_ℓ and $p_{\ell e}$ given by 11.4.4.2 for walls

f'_c (MPa)	f_y = 300 MPa		f_y = 500 MPa	
	p_ℓ	$p_{\ell e}$	p_ℓ	$p_{\ell e}$
30	0.0046	0.0092	0.0027	0.0054
40	0.0053	0.0106	0.0032	0.0064
50	0.0059	0.0118	0.0035	0.0070

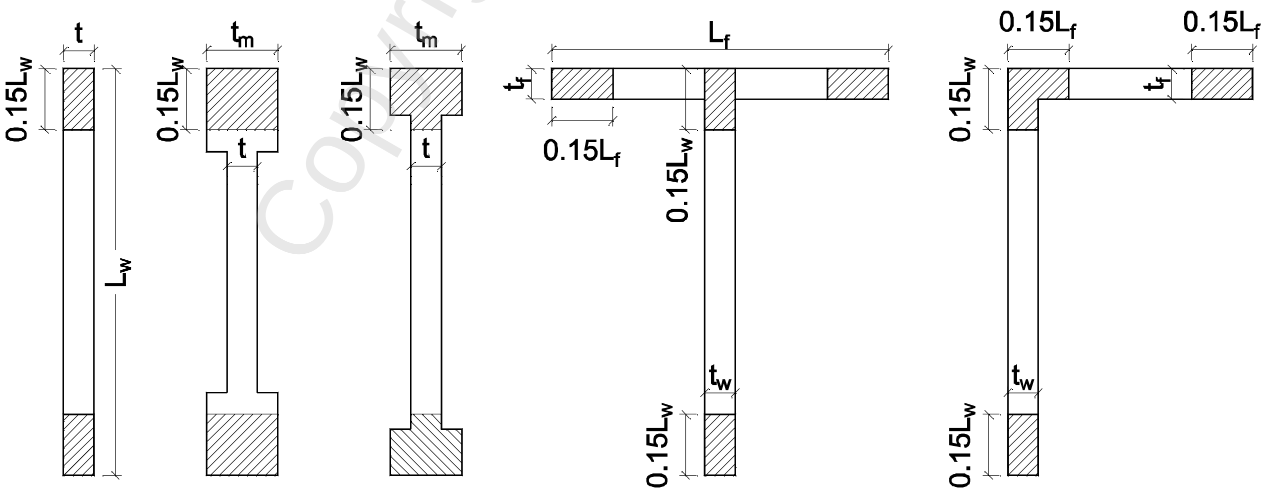


Figure C11.2A – End zone definition for different wall geometry

A3

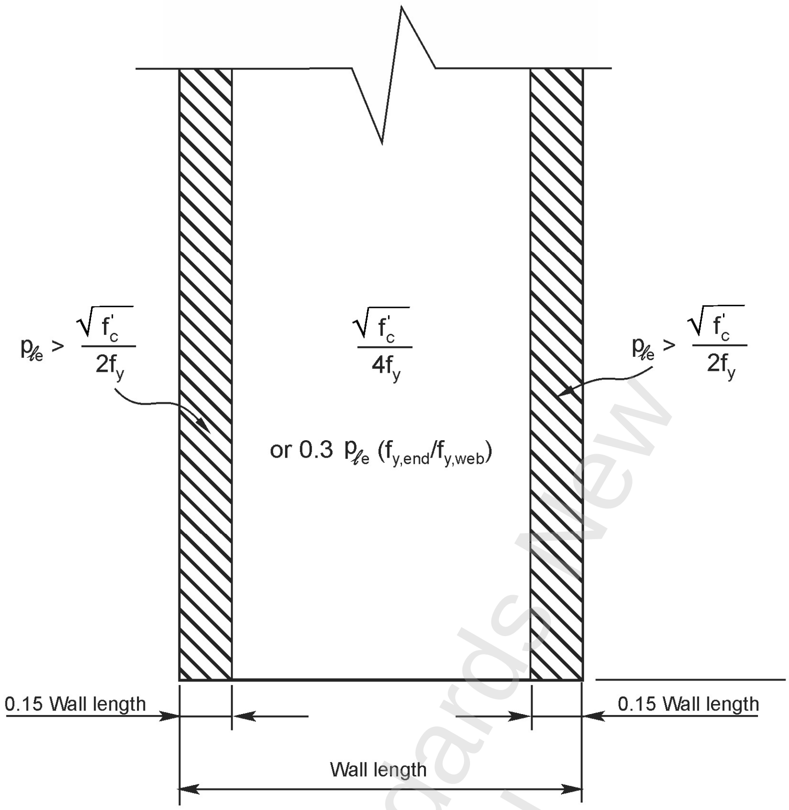


Figure C11.2B – Wall elevation – minimum reinforcement requirements

C11.4.5 ***Transverse reinforcement***

C11.4.5.1 *Transverse reinforcement requirements*

The requirements for spacing, quantity and bar sizes in wall webs are similar for both the vertical and horizontal reinforcement.

The requirements for tie reinforcement at any location in a wall section are not additive, such that only the most stringent requirements for anti-buckling, confinement or shear reinforcement apply. In no case should there be any less transverse reinforcement than required for anti-buckling or confinement.

C11.4.5.2 *Transverse tie reinforcement for lateral restraint in plastic hinge regions – compression region of wall*

This clause is intended to ensure that the principal longitudinal reinforcement receives adequate lateral support, taking the Bauschinger effect into account, to enable it to be strained beyond compression yield. The requirements extend to areas, both horizontally and vertically, where yielding of the longitudinal reinforcement could occur. Every vertical bar should be assumed to be subjected to alternating yielding in tension and compression. The vertical extent of potential yielding is defined in 11.4.5.5(f).

The detailed requirements for tie shapes, tie leg area and spacing, as set out in 11.4.5.2(a), (b) and (c), are similar to those for potential plastic hinge regions of flexural members as given in 9.4.1.6. The interpretation of these requirements is illustrated in Figure C11.2, which shows a small flange and a typical boundary element, containing the bulk of the longitudinal flexural reinforcement for a shear wall.

C11.4.5.3 *Transverse tie reinforcement for lateral restraint in plastic hinge regions – central region of wall*

The longitudinal reinforcement in the mid-region of the wall, between the regions where the lateral ties comply with 11.4.5.2, which corresponds to the region outside the neutral axis depth, is to be restrained against buckling unless the concrete surrounding the reinforcement remains intact so that it can provide the necessary restraint. The concrete may not be assumed to be intact and provide the necessary restraint if any one of the limits set out in (a) to (d) of 11.4.5.3 is exceeded. The logic behind each of the

A3 four conditions is briefly outlined below:

(a) The maximum first mode shear force sustained when overstrength actions act, which is the ultimate limit state design shear force stress, multiplied by the overstrength factor, ($\phi_{ow}V^*$) is greater than $0.075 f_c' \times 0.8L_w t$. This shear stress level has been set to correspond to a diagonal compression stress associated with shear of about $0.3 f_c'$. This shear stress limit has been set on the basis that flexural cracking due to out-of-plane bending moments reduces the width of the wall web that can resist in-plane shear stresses. In addition, in setting this limit an allowance was made for the non-uniform distribution of shear stress along the wall. This clause relates to a plastic hinge that is yielding. While yielding may possibly occur when higher mode actions arise the deformation associated with these higher mode actions is small and hence not significant for the issue being considered here. Therefore, it is the first mode type actions that need to be considered. On this basis the shear force can be taken as the shear associated with the development of the ductile overstrength collapse mechanism when the plastic hinge sustains its flexural overstrength, which is essentially the first translational mode shear, or equivalent static shear scaled by M_o/M_e where M_o is the overstrength moment capacity and M_e is the equivalent static shear or the first mode shear;

(b) The average lateral spacing of vertical reinforcement in the mid-region of the wall is smaller than $5d_b$. This limit is used as a close spacing of bars and can result in cracks forming in the plane of the bars, which increases the potential for spalling;

(c) The clear cover of the longitudinal reinforcement is less than $1.5d_b$. Small cover distances can result in bond cracks forming along the bars, which reduces the protection that concrete provides against spalling;

(d) Provided spalling or wide cracks do not form, the concrete will provide restraint against buckling of the vertical reinforcement. This condition is met if the curvature demand in the plastic region is equal to or less than the maximum permitted material strain for limited ductile plastic regions, see 2.6.1.3.4. This corresponds to a tensile strain due to inelastic deformation equal to or less than 0.007. If the tension strain in the reinforcement is less than this value, then when the bar is subjected to a reversing compression strain there would be negligible tendency for the buckling to cause spalling of the concrete.

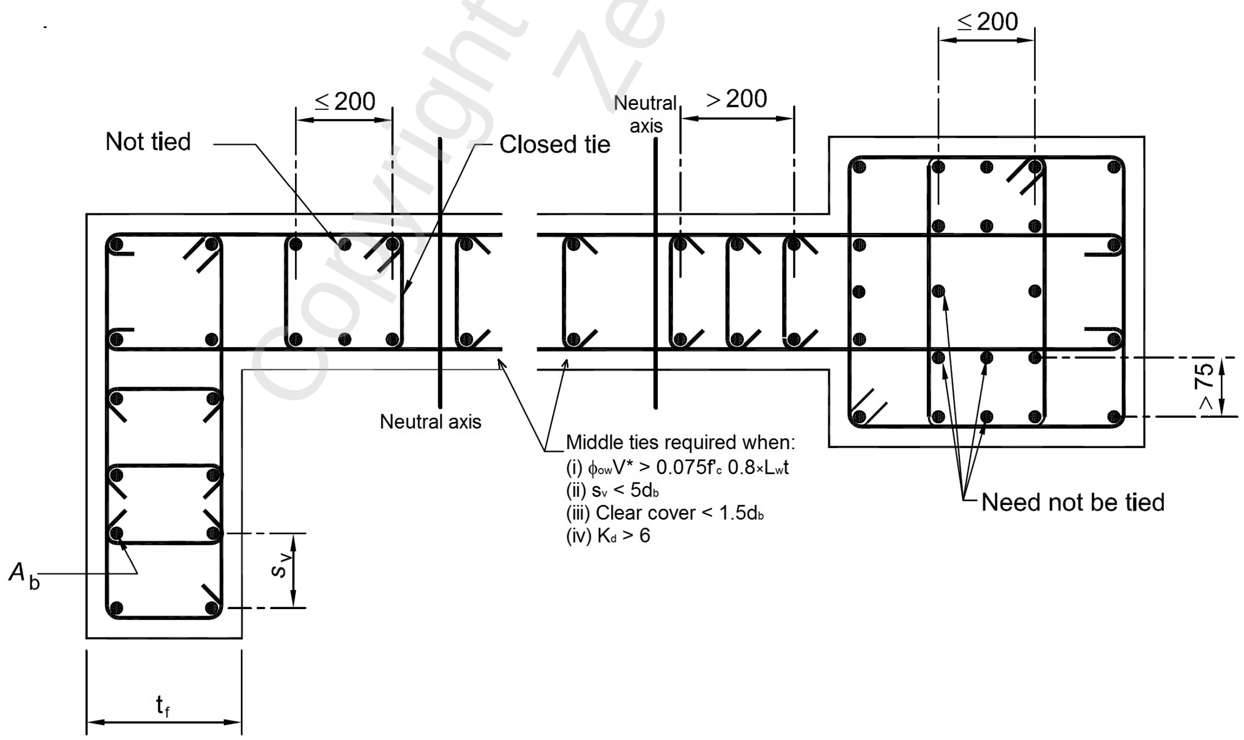


Figure C11.2 – Examples of transverse reinforcement in plastic hinge regions of walls in accordance with 11.4.5

C11.4.5.4 *Transverse reinforcement for lateral restraint of longitudinal bars outside plastic hinge regions* A3

Although bars in areas of a wall section, well above the potential height of the plastic hinge and generally in the boundary region are expected to respond elastically, some transverse reinforcement to stabilise these bars, when in compression, should be provided. This requirement is similar to that applicable to the elastic regions of columns at the ultimate limit state, as required in 10.3.10.6.

C11.4.5.5 *Confinement requirements in plastic hinge region*

Confinement of the compressed concrete, usually at the base of a wall, is required only if the compression strain at the ultimate limit state can be expected to be excessive [11.3]. Therefore these requirements are made dependent on the critical neutral axis depth, c_c, of the wall section, defined in Equation 11–25, where the wall overstrength factor is defined as:

$$\phi_o = \frac{\text{Moment of resistance at overstrength}}{\text{Moment resulting from specified earthquake forces}} = \frac{M_o^*}{M_E}$$ where both moments refer to the base of the wall. This factor recognises that when excess flexural strength has been provided, i.e. when $\phi_o > 1.25/0.85 \approx 1.5$, curvature ductility demand is likely to be reduced and hence a larger neutral axis depth can be accepted. Similar increases are justified when reduced displacement ductility capacity is relied on at the ultimate limit state. A typical critical value is $c_c = (0.3 \times 1.5/5)\ L_w = 0.09\ L_w$. In most walls the properly computed neutral axis depth c will be less and hence no confinement will be required.

(a) Equation 11–27 takes into account the modifications for the confinement of columns such as introduced in 10.3.10.6.1. The transverse confining reinforcement so computed must be distributed over a length of the compressed part of the wall section defined by Equation 11–27; A3

(b) The requirement is that this clause is similar in purpose to those for confinement of columns. As the maximum strains and degree of degradation of the compression zone of the walls are likely to be less than that of columns, the maximum permissible spacing between longitudinal bars is less restrictive; that is, the centre-to-centre spacing may be larger than that in columns;

(c) This transverse reinforcement must be placed over a height above the base equal to or greater than the length of the wall L_w or one-sixth of the total wall height, whichever is larger;

(d) Walls with a single layer of reinforcement should not be used when the limit of Equation 11–25 is exceeded. A3

The potential yield region of a wall behaving as a cantilever is assumed to be at its base. Outside this region the special requirements for hoops need not be satisfied provided that the designer ensures that yielding of the flexural wall reinforcement will not occur outside the potential yield region. This may be achieved if the flexural reinforcement is curtailed in accordance with a linear bending moment envelope, rather than the bending moment diagram derived for the lateral static forces.

A3

C11.4.6 *Shear strength*

C11.4.6.1 *General*

Generally the same principles apply as for columns. Seldom can it be readily identified whether a horizontal force is introduced primarily to the flexural compression or to the flexural tension region of a structural wall. Introduction of shear forces to walls depends greatly on the geometry and on the reaction necessary to equilibrate inertia forces in diaphragms. Diaphragm reactions are more commonly introduced in the flexural tension regions of cantilever walls, and hence diagonal struts, justifying the application of A3 9.3.9.3, cannot be readily developed unless extra reinforcement (drag bars) is provided to transfer inertia and deformation capability forces (including transfer forces) to the compression zone of the wall.

C11.4.6.3 *Shear strength provided by the concrete* A3

The provisions for shear strength of wall are similar to those of 11.3.12.3.5. Additional restrictions are required, however, in potential plastic hinge zones, where shear strength is affected by yielding of the vertical wall reinforcement during reversed cyclic rotation in a major earthquake. The restriction on the concrete shear strength, V_c, is similar to that given for columns except that the contributions of the concrete, V_c, to shear strength may be assumed even for very small axial compression loads. This has been established in tests [11.5]. Because of the distribution of the vertical reinforcement throughout the depth of a wall section, better control of diagonal crack width is expected than in beams.

Tests [11.5, 11.6] have shown that web crushing in the plastic hinge zone, at the base of cantilever walls, may occur after a only few cycles of reversed loading involving displacement ductilities of 4 or more. When the imposed displacement ductilities, μ, in these tests were only 3 or less, the shear stress levels specified by 11.3.12.3.2 could be repeatedly attained. Premature web crushing can be expected where, due to large curvature ductility demands in the plastic hinge region, the concrete carrying diagonal compression stresses is also subjected to large transverse tensile strains. To prevent web crushing due to excessive shear load, Equation 11–28 makes the maximum total shear dependent on the curvature ductility and the flexural strength that may have been provided in excess of that required by NZS 1170.5 measured by the flexural overstrength factor for walls, ϕ_{ow}. A3

C11.4.6.4 *Sliding shear of squat walls* A3

Because of the limited capacity of the foundations in resisting over turning moments, often it may be difficult to develop the full flexural strength at the base of squat walls typically with a height to length ratio of less than two. However, when adequate foundations allow a plastic hinge to develop at the base of squat walls, a sliding shear failure may occur only after a few inelastic displacement reversals. In exceptional cases when such walls are subjected to large ductility demands, diagonal reinforcement, similar to that shown in Figure C9.22 and crossing the horizontal base section, may be required [11.3].

C11.4.8 ***Special splice and anchorage requirements*** A3

C11.4.8.1 *Splicing of flexural tension reinforcement*

Because a large quantity of flexural tension wall reinforcement may have to be extended up several storeys, some splicing in potential plastic hinge regions may be unavoidable, see Figure C11.4. Such splices must be staggered so that not more than every third bar is spliced at the same level in a potential yield region, defined in 11.4.5.5(e).

C11.4.8.2 *Staggering and confining of lapped splices* A3

Because a large quantity of flexural tension wall reinforcement may have to be extended up several storeys, some splicing in potential plastic hinge regions may be unavoidable. Such splices are staggered so that not more than one-third of bars is spliced at the same level in a ductile potential yield region, and not more than one-half of bars are spliced at the same level in a limited ductile detailing length, defined in 11.4.2. See Figure C11.5.

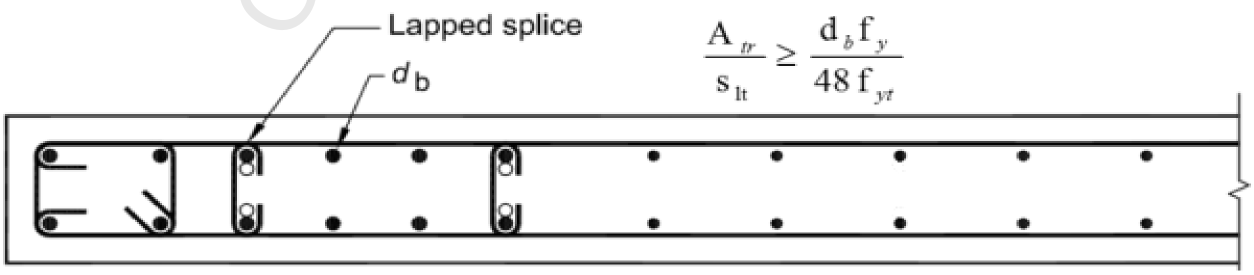


Figure C11.4 – Ties required at lapped bar splices

A3

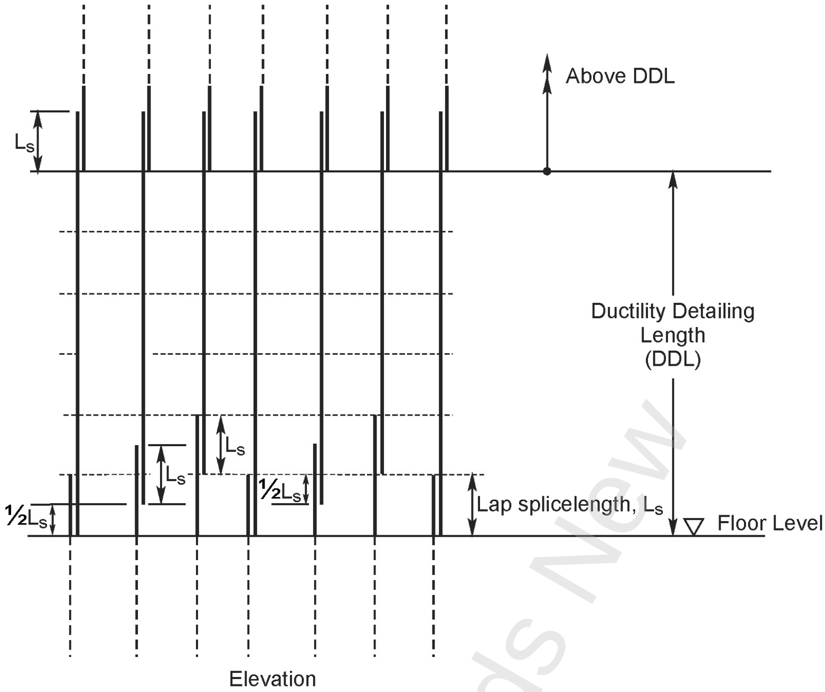


Figure C11.5 – Lap splice configuration in and above ductile detailing length

C11.4.8.3 *Welded and mechanical splices*

As a general rule splices of any kind should be staggered in potential plastic hinge regions of walls. However, welded splices and mechanical connectors with proven strengths and stiffness need not be staggered. Therefore in precast panel construction, where the staggering of splices becomes impractical, only full strength welded splices or connectors meeting the requirements of 8.9.1.3 must be used.

A3

C11.4.9 ***Coupled shear walls***

C11.4.9.1 *General*

Coupled walls are designed to resist the overstrength actions from the coupling beams. This gives the bending moments, shear forces and axial loads acting on the walls. The nominal strength of the wall should be capable of resisting the overstrength actions from all the coupling beams and in addition for each coupling beam individually applying an additional bending moment of 0.2 times the beam over-strength moment. This step is taken due to the difficulty of accurately determining axial load in the coupling beam and hence its peak strength. It is not intended that the 1.2 be applied to the simultaneous overstrength actions of all the beams

C11.4.9.2 *Overstrength of coupling beams*

The design actions in coupling beams are illustrated in Figure C11.6.

As the diagonal compression force in a coupling beam is resisted by concrete acting with the diagonal reinforcement, it is stiffer than the corresponding diagonal tension tie. Consequently the axial deformation of the diagonal compression strut will be small compared to the deformation of the diagonal tension tie. Assuming that the diagonal compression strut is stiff enables the level of deformation in the coupling beams to be determined and the shear deformation to be found from the inter-storey drift, as illustrated in Figure C11.6(a).

The elongation of the coupling beam pushes the coupled walls apart. The resultant displacement induces tension in floor slabs that are connected to the walls or to beams that are built into the walls. The resultant tension force in the floor slabs induces axial compression in the coupling beams, which increases the flexural and shear capacity of these members. This can have a very significant influence on the magnitude of the seismic actions induced in the structural walls, and it is essential that this action is considered in assessing the overstrength capacity of the coupling beams.

Detailed research has still to be completed in establishing what area of floor contributes to the peak axial compression force that acts on the coupling beams. The widths of floor given in 11.4.9.2 are believed to be appropriate. A3

The shear capacity of a diagonally reinforced coupling beam subjected to an axial force of $N_{o,c}$ can be found as detailed below:

(a) The stress in the diagonal tension tie is equal to its overstrength value of $\phi_{o,fy}\ f_y$, so that the diagonal tension force T_o is $\phi_{o,fy}\ f_y\ A_{s,d}$, where $A_{s,d}$ is the area of diagonal reinforcement;

(b) The magnitude of diagonal compression force equals T_o plus $N_{o,c}\ /Cos\alpha$, where α is the angle between the diagonal reinforcement and the axis of the coupling beam;

(c) The resultant overstrength shear force, $V_{o,c}$, resisted by the coupling beams is given by $V_{o,c} = 2T_o \sin\alpha + N_{o,c} \tan\alpha$.

Where orthogonal reinforcement is used the shear force can be found from the overstrength flexural capacity of the beam.

Particular care is required to ensure the connection zone between the coupling beam and a structural wall has sufficient strength to prevent premature failure. This will generally require a strut and tie analysis to be undertaken to ensure that the actions from the coupling beams are transferred into the internal mechanisms of the adjacent walls.

C11.4.9.3 *Diagonally reinforced coupling beams*

The limit on clear span to beam depth ratio for diagonally reinforced beams of 4 is due to a lack of tests and analytical work for aspect ratios exceeding 4.0.

Diagonally reinforced coupling beams can be used to couple walls, or in frame structures to provide a shear yielding connection in the mid-span region of a beam. In the first case L_n is equal to the clear span of the beam, and in the second case L_n is equal to the length of the diagonal reinforcement projected to the axis of the beam. The maximum length of L_n is equal to or less than four times the beam depth (h_b). The advantages of diagonally reinforced coupling beams are that they can be designed to resist high shear stress levels and they can sustain greater inelastic deformation than conventional beams. The disadvantage lies in the complexity of the reinforcement detailing required to obtain satisfactory performance.

No upper limit is placed on the shear force that may be resisted by diagonally reinforced coupling beams, but congestion of the reinforcement will limit the shear force capacity. With diagonally reinforced coupling beams, two sets of diagonal reinforcement are required which are inclined so that they intersect each other at the mid-section of the clear span. The reinforcement is designed to enable all the seismic-induced shear and moment to be resisted by the diagonal reinforcement without any contribution of concrete in the clear span of the beam. No limits are placed on the angle the diagonal reinforcement makes to the axis of the beam other than the limit of the span, L_n, being equal to or smaller than four times the overall depth.

For the diagonal reinforcement in coupling beams to work effectively, each set of diagonal bars needs to be restrained against buckling by ties at close centres and the reinforcement must be fully developed outside the span, L_n, of the coupling beam. In addition to the diagonal reinforcement, nominal longitudinal reinforcement is required at the top and bottom surfaces of the beam, together with nominal stirrups enclosing the longitudinal reinforcement.

Where a diagonally reinforced coupling beam is used in a frame, particular care is required in detailing reinforcement to ensure compression in the diagonal bars does not result in the bars breaking out of the concrete, and to ensure that yielding is confined to the diagonal bars, and that the bend in the bars is protected against yielding[11.7].

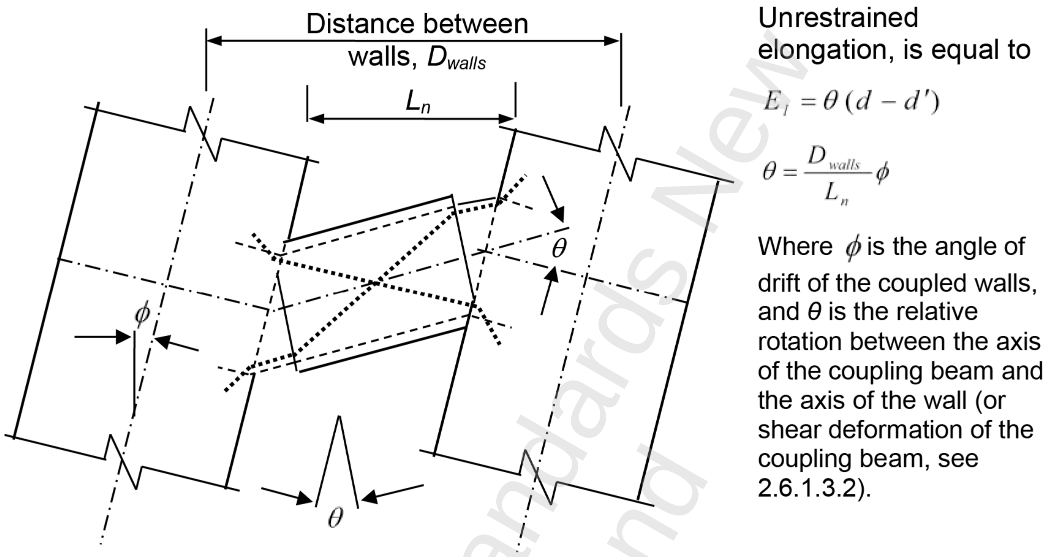
C11.4.9.4 *Reinforcement details for diagonally reinforced coupling beams*

Diagonally oriented reinforcement is generally effective if bars are placed with a large inclination.

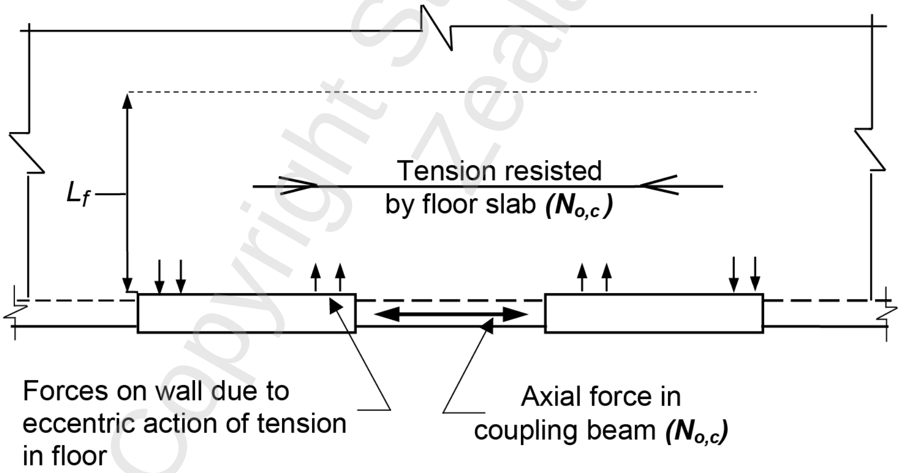
Diagonal bars should be placed approximately symmetrically in the beam's cross section, in two or more layers. The diagonally placed bars are intended to provide the entire shear and corresponding moment strength of the beam; designs deriving their moment strength from combinations of diagonal and longitudinal bars are not covered by these provisions.

A3 In the confinement option shown in 11.4.9.4(a) each diagonal element consists of a cage of longitudinal and transverse reinforcement as shown in Figure C11.7 and Figure C11.8. Each cage consists of at least four diagonal bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate toughness and stability to the cross section when the bars are loaded beyond yielding.

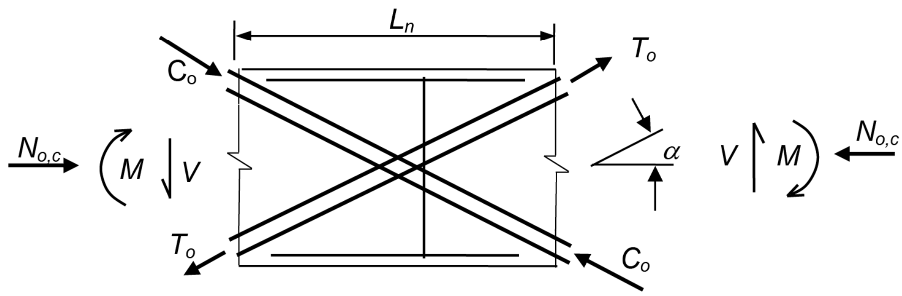
In the confinement option shown in 11.4.9.4(b), the entire beam cross section is confined, as shown in Figure C11.10, instead of confining the individual diagonals. This option can considerably simplify placement of the hoops, which can otherwise be difficult where diagonal bars intersect each other or enter the wall boundary.



(a) Elevation on coupled walls and elongation of coupling beam



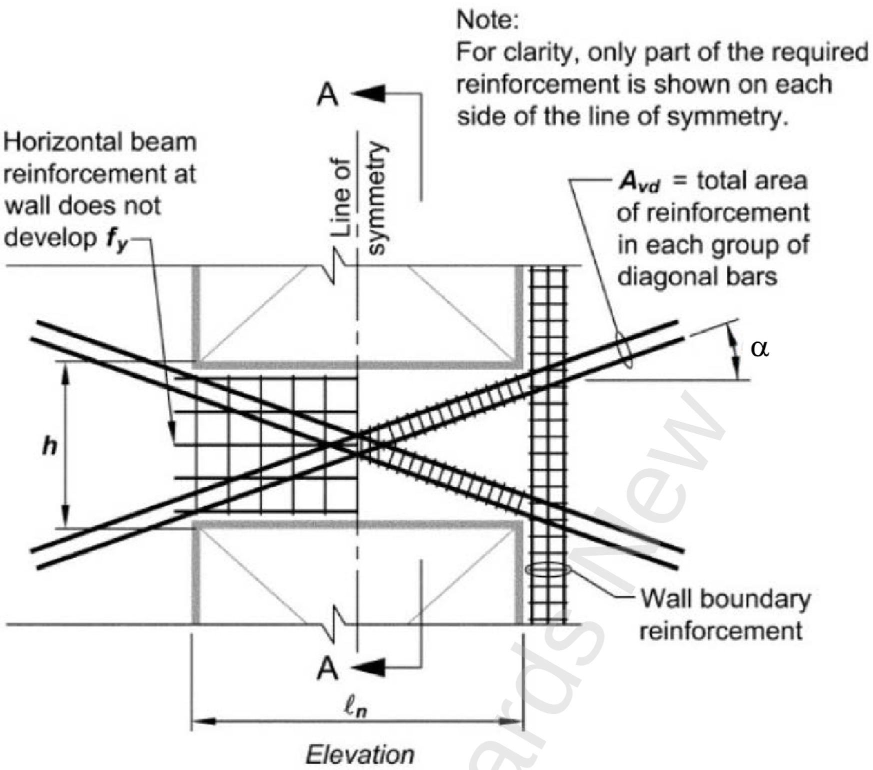
(b) Plan on floor and coupled walls



(c) Actions in coupling beam

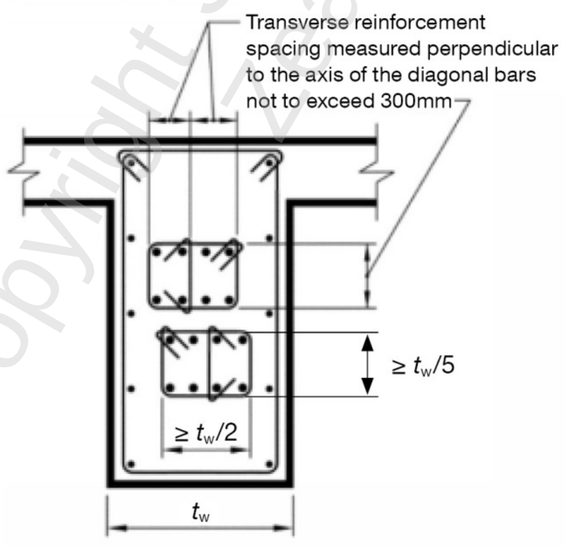
Figure C11.6 – Coupled structural walls

A3



Figures C11.7, C11.8 and C11.9 are authorised adaptations of Figure R21.9.7(a)(b) 'Coupling Beams with Diagonally Oriented Reinforcement' from the American Concrete Institute's ACI 318-08 *Building Code Requirements for Structural Concrete and Commentary*. Reproduced with permission from the American Concrete Institute.

Figure C11.7 – Confinement of individual diagonals

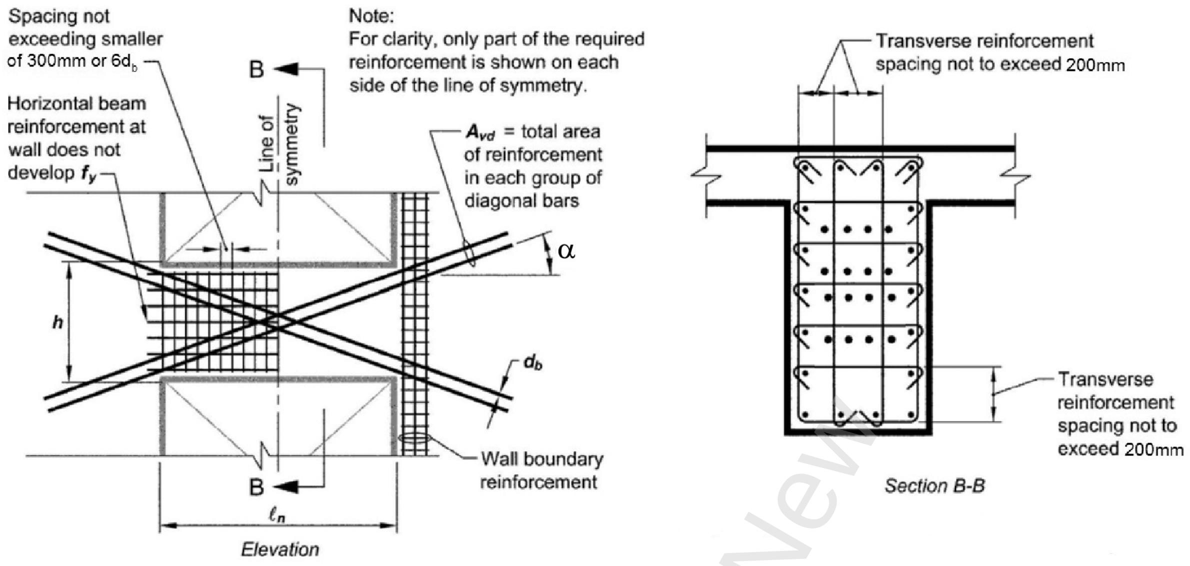


Section A-A

Figures C11.7, C11.8 and C11.9 are authorised adaptations of Figure R21.9.7(a)(b) 'Coupling Beams with Diagonally Oriented Reinforcement' from the American Concrete Institute's ACI 318-08 *Building Code Requirements for Structural Concrete and Commentary*. Reproduced with permission from the American Concrete Institute.

Figure C11.8 – Confinement of individual diagonals – section A-A

A3



Figures C11.7, C11.8 and C11.9 are authorised adaptations of Figure R21.9.7(a)(b) 'Coupling Beams with Diagonally Oriented Reinforcement' from the American Concrete Institute's ACI 318-08 *Building Code Requirements for Structural Concrete and Commentary*. Reproduced with permission from the American Concrete Institute.

Figure C11.9 – Full confinement of diagonally reinforced concrete beam section

C11.4.9.5 *Conventionally reinforced coupling beams*

In previous versions of this Standard if a reversing shear stress in a beam exceeded $0.25\sqrt{f'_c}$ diagonal reinforcement was required to prevent a sliding shear failure. This type of failure occurs due to elongation of the reinforcement leaving wide open cracks in the beam. The limit has been increased to $0.3\sqrt{f'_c}$ for two reasons. First, some axial compression is induced due to restraint to elongation, and second the beam is detailed to minimise elongation, which reduces the crack widths and increases the resistance to sliding shear.

Previously the shear stress limits were based on overstrength actions. However, for beams the strength is nearly proportional to $\phi_{o,fy} f_y$ where $\phi_{o,fy}$ is 1.35, while the ratio of nominal strength to design strength is 1.33 (inverse of strength reduction factor, ϕ of 0.75). For practical purposes this balances the 1.35, which simplifies the calculations by allowing the values to be based on ultimate limit state calculations.

In assessing the shear stress limits the axial force due to elongation is not included. This axial force is not constant and the peak value is only induced at peak lateral displacements. At other stages it may be negligible. As axial force increases shear strength also increases. Consequently it is not necessary to include it in determining the maximum critical shear strength.

Placing one sixth of the longitudinal reinforcement in the web between the top and bottom layers ensures that the flexural tension force is always larger than the flexural compression force resisted by compression zone reinforcement. This action reduces elongation and the crack widths that lead to sliding shear. This is one of the reasons why the previous limit on cyclic shear stress of $0.25\sqrt{f'_c}$ before diagonal reinforcement was required has been increased to $0.3\sqrt{f'_c}$.

REFERENCES

11.1 Oberlander, G.D. and Everard, N.J., "Investigation of Reinforced Concrete Walls", ACI Journal, Proceedings Vol. 74, No. 6, June 1977, pp. 256-263.

11.2 Cardenas, Alex E. and Magura, Donald D., "Strength of High-rise Shear Walls – Rectangular Cross-section", Response of Multi-storey Concrete Structures to Lateral Forces. SP-36, American Concrete Institute, Detroit, 1973, pp. 119-150. Also Research and Development Bulletin RDO29D, Portland Cement Association, Skokie, Ill.

11.3 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley and Sons, 1992, p. 767.

11.4 Bertero, V.V., Popov, E.P., Wang, T.Y. and Vallenas, J., "Seismic Design Implications of Hysteretic Behaviour of Reinforced Concrete Structural Walls", 6th World Conference on Earthquake Engineering, New Delhi, 1977, Preprints 5, pp. 159-165.

11.5 Oesterie, R.G., Fiorato, A.E., Johal, L.S., Carpenter, J.E., Russell, H.G. and Gorley, W.G., "Earthquake Resistant Structural Walls – Tests of Isolated Walls", Report to National Science Foundation, Portland Cement Association, Skokie, Nov. 1976, 44 pp. (Appendix A, 38 pp., Appendix B, 233pp).

11.6 Bertero, V.V., Popov, E.P., Wang, T.Y. and Vallenas, J., "Seismic Design Implications of Hysteretic Behaviour of Reinforced Concrete Structural Walls", 6th World Conference on Earthquake Engineering, New Delhi, 1977, Preprints 5, pp. 159-165.

11.7 New Zealand Concrete Society and New Zealand Society for Earthquake Engineering, "Guidelines for the Use of Structural Precast Concrete in Buildings", Centre for Advanced Engineering, University of Canterbury, 1999, p.143. A3

11.8 Lu, Y., Henry, R.S. "Numerical Modelling of Reinforced Concrete Walls with Minimum Vertical Reinforcement", Engineering Structures, 2017, p.143, pp.330-345.

11.9 Lu, Y., Henry, R.S., Gultom, R., Ma, Q.T. "Cyclic Testing of Reinforced Concrete Walls with Distributed Minimum Vertical Reinforcement", Journal of Structural Engineering, 2017, Vol.143 (5).

11.10 Lu, Y., Gultom, R.S., Ma, Q.T. "Testing of RC Walls to Investigate Proposed Minimum Vertical Reinforcement Limits in NZS 3101:2006 (A3)", Proceedings of the 2016 NZSEE Annual Conference, Christchurch, April 1 – 3, 2017.

11.11 Sritharan, S., Beyer, K., Henry, R.S., Chai, Y.H., Kowalsky, M., Bull, D. "Understanding Poor Seismic Performance of Concrete Walls and Design Implications", Earthquake Spectra, 2017, Vol. 30(1): 307-334.

Table C11.3 – Design of reinforced concrete walls

	Design issue	Nominally ductile design philosophy	Ductile seismic design philosophy
Material limitation applicable to the detailing described in this table	Range limitation on concrete compressive strength, f'_c	20 to 100 MPa (5.2.1)	Same as for nominally ductile
	Range limitation on longitudinal reinforcement yield strength, f_y	300 to 500 MPa (5.3.3)	Same as for nominally ductile
	Range limitation on transverse reinforcement yield strength, f_{yt}	300 to 500 MPa for confinement (5.3.3) 300 to 500 MPa for shear (7.5.8)	Same as for nominally ductile
	Reinforcement class as per AS/NZS 4671	Class E	Same as for nominally ductile
Ductility	Curvature ductility achieved through tabled detailing	Clause 2.6.1.3.4 Table 2.4(a) and (b)	Clause 2.6.1.3.4 Table 2.4(a) and (b)
Dimensional limitations	Minimum structural wall thickness-general	100 mm (11.3.2)	Same as for nominally ductile
	Limitations on the height to thickness ratio to prevent buckling of walls with high axial loads	$(k_e h_n/t) \le 30$ where $N^* > 0.2 f'_c A_g$ (11.3.7)	Same as for nominally ductile
	Limitations on effective height to thickness ratio to prevent flexural torsional buckling of in-plane loaded walls	$\frac{k_{ft} h_n}{t} \le 12\sqrt{\frac{h_n / L_w}{\lambda}}$ where $N^* \le 0.015 f'_c A_g$ and $\frac{h_n}{t} \le 75$ and $\frac{k_{ft} h_n}{t} \le 65$ (11.3.5.2.2)	Same as for nominally ductile, but k_{ft} factors different, refer Table 11.1
	Moment magnification required for slenderness when:	$\frac{k_e h_n}{t} \ge \frac{\alpha_m}{\sqrt{\frac{N^*}{f'_c A_g}}}$ (11.3.6.2)	Same as for nominally ductile, but k_e factors different
	Limitations on the dimensions of the compression zone within plastic hinge region	Not applicable	$t_m = \frac{\alpha_r k_m \beta (A_r + 2) L_w}{1700\sqrt{\xi}}$ (11.4.3.2)
Overstrength factors		N/A	Refer 2.6.5.5

Table C11.3 – Design of reinforced concrete walls (Continued)

	Design issue	Nominally ductile design philosophy	Ductile seismic design philosophy
Wall plastic hinge detailing	Extent of potential plastic hinge region, ℓ_y, for detailing purposes	N/A	The greater of $1.5L_w$ or $0.25\dfrac{M_e}{V_e}$ but need not be taken greater than $2L_w$ (11.4.2)
Effective flange widths for walls with returns		The vertical reinforcement placed within the flange width equal to one-half of the wall height above the design section, but not greater than $4t_f$, shall be considered effective (11.3.1.3)	Same as for nominally ductile, however for overstrength calculations, the flange width equals 1.0 times the height above the design section (11.4.1.3)
Longitudinal reinforcement detailing	Axial load limits	$N^* < 0.3\phi f_c' A_g$ (11.3.1.6)	$N_o^* < 0.3 f_c' A_g$ (11.4.1.1)
	Limitation on the use of singly reinforced walls	$p_\ell \le 0.01$ (11.3.12.5)) $t \le 200$ mm (11.3.12.2) ϕ as per 2.3.2.2	Not applicable
	Minimum longitudinal reinforcement ratio	$p_\ell > \dfrac{\sqrt{f_c'}}{4f_y}$ (11.3.12.3)	$p_\ell > \dfrac{\sqrt{f_c'}}{4f_y}$ & $0.3\, p_{\ell e}\, (f_{y,end}/f_{y,web})$ $p_{\ell e} > \dfrac{\sqrt{f_c'}}{2f_y}$ (11.4.4.2)
	Maximum longitudinal reinforcement ratio	$P_\ell < 16/f_y$ and ensure neutral axis depth is less than $0.75\, c_b$ (11.3.12.2)	Same as nominally ductile
	Maximum longitudinal reinforcement at splices	$P_\ell < 21/f_y$ (11.3.12.2)	Same as nominally ductile
	Limitations on the position of lap splices in walls	No limitations	In plastic hinge regions, of fully ductile walls, not more than $^1/_3$ of reinforcement shall be spliced at the same location where yielding occurs. For limited ductile walls not more than $^1/_2$ shall be spliced at any one location (11.4.8.1)
	Maximum spacing between longitudinal reinforcement	Lesser of $L_w/3$, $3h$, or 300 mm (11.3.11.3.8)	Same as nominally ductile
	Maximum longitudinal bar diameters	No limitations	$d_b \le \dfrac{t}{10}$ for fully ductile $\dfrac{t}{8}$ for limited ductile (11.4.4)

Table C11.3 – Design of reinforced concrete walls (Continued)

	Design issue	Nominally ductile design philosophy	Ductile seismic design philosophy
	Curtailment of reinforcement	Comply with 8.6.12 (11.3.12.7)	Refer 9.4.3.1
Transverse reinforcement Outside of the potential plastic hinge region	Minimum diameter for transverse reinforcement	No requirements	No requirements
	Maximum vertical spacing of ties	Lesser of $L_w/5$, $3t$, or 300 mm (11.3.11.3.8(c))	Same as nominally ductile
	Anti-buckling reinforcement	No requirement	Same as nominally ductile
	Confinement reinforcement	No requirement	No requirement
	Minimum shear reinforcement	$A_v = 0.7\ t_w s_2/f_{yt}$ (11.3.11.3.8(b))	Same as nominally ductile
	Shear reinforcement	Refer to 11.3.11	Refer to 11.4.6
Transverse reinforcement Within potential plastic hinge region	Minimum diameter for transverse reinforcement	Same as outside plastic hinge region	Same as for nominally ductile
	Maximum vertical spacing of ties	Same as outside plastic hinge region	Where required for anti buckling, spacing shall be less than $6d_b$ for DPR and 10 d_b for LDPR (11.4.5.2) Where required for confinement, as above but also shall be less than $t/2$ for DPR and t for LDPR (11.4.5.5(d))
	Anti-buckling reinforcement	Same as outside plastic hinge region	$A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ (11.4.5.2)
	Confinement reinforcement	Same as outside plastic hinge region	Where neutral axis depth c_c exceeds $0.1\phi_{ow}L_w$ for LDPR and $0.05\phi_{ow}L_w$ for DPR $A_{sh} = \alpha_k s_h h'' \frac{A_g^*}{A_c^*} \frac{f'_c}{f_{yh}} \left(\frac{c}{L_w} - 0.07 \right)$ (11.4.5.5)
	Cross ties in the central portion of wall	No requirements	Refer to 11.4.5.3

C12 DESIGN OF REINFORCED CONCRETE TWO-WAY SLABS FOR STRENGTH AND SERVICEABILITY

C12.1 Notation

The following symbols, which appear in this section of the Commentary are additional to those used in Section 12 of Part 1.

A_c	area of concrete section resisting shear transfer, mm^2
c_{AB}, c_{CD}	refer to Figure C12.8
d_b	nominal diameter of longitudinal reinforcing bar, mm
J_c	property of assumed critical section analogous to polar moment of inertia (see Equation C12–11) mm^4
M^*_x, M^*_y	ultimate resisting moments per unit width in the x and y directions, N mm
M_x, M_y	bending moments per unit width, N mm
M_{xy}	elastic theory solution torsional moment per unit width, N mm
v_{AB}, v_{CD}	maximum design shear stress on sections AB and CD respectively, MPa

C12.2 Scope

The fundamental design principles contained in Section 12 are applicable to all planar structural systems subjected to transverse loads. Types of slab systems which may be designed according to Section 12 include “flat slabs”, “flat plates”, “two-way slabs”, and “waffle slabs”.

True “one-way slabs”, that is slabs reinforced to resist flexural stresses in one direction only, are covered in Section 9 and excluded from Section 12. Also excluded are soil supported slabs, such as “slab on grade” which do not transmit vertical loads from other parts of the structure to the soil.

Much of Section 12 is concerned with the selection and distribution of flexural and shear reinforcement.

C12.3 General

Rectangular or spare panels supported by walls or relatively stiff beams on two opposite sides may be designed as one-way slabs, that is, as beam strips spanning in the direction perpendicular to the supports. When slabs are designed as one-way slabs, the designer must realise that true one-way action will exist only if the loads are uniformly distributed in the direction parallel to the supports and if the edges of the panel perpendicular to those supported on walls or stiff beams are themselves completely unsupported. If either of these conditions is not satisfied, transverse moments will exist and should be provided for in order to prevent the formation of large cracks and to provide adequate transverse distribution of non-uniform loads.

The provisions of this section apply only to reinforced concrete floor systems.

The provisions of this section do not apply to multi-storey flat plate or flat slab buildings which are used as seismic resisting structures, unless frames involving beams and columns or walls, or a combination of these components, are present to provide most of the strength and stiffness required to resist the horizontal seismic forces. Without such additional strengthening and stiffening elements it is doubtful whether the structure would have sufficient ductility at the critical slab-column connections to withstand a major earthquake, and also considerable inter-storey deflections may occur due to the flexibility of the structure.

Tests have shown that column slab connections may fail in a brittle mode when appreciable inter-storey drift occurs. The drift at which fracture occurs decreases as the axial load transferred to the column increases. For this reason column slab systems should not be used in buildings which may sustain appreciable sway under either wind or seismic forces.[12.1, 12.2] It is recommended that if the peak inter- A3

A3 storey drift in the ultimate limit state or a maximum shear stress on any part of the critical shear stress perimeter exceeds 0.78 MPa this type of connection should not be used.

C12.4 Design procedures

C12.4.1 *General*

This clause permits a designer to base a design directly on fundamental principles of structural mechanics, provided it can be demonstrated explicitly that all criteria at the serviceability and ultimate limit states are satisfied.

C12.4.2 *Design methods*

This clause lists the acceptable methods for the determination of the design moments and shears.

Methods that take into account the effects of membrane action in slabs may be used[12.3, 12.4, 12.5] provided that they satisfy 12.4.1. There are various factors that need to be taken into account when using approaches that take into account compression effects in membranes, and caution should be exercised. For design moments determined from the idealised frame method and from the simplified method, refer to
A3 C6.7.3 and C6.7.4. For the empirical design method for bridges refer to C12.8.2.

C12.5 Design for flexure

Loads should be as given by AS/NZS 1170 and NZS 1170.5 or other referenced loading standard.

C12.5.2 *Effective area of concentrated loads*

Tests on slabs at Auckland University[12.6, 12.7] have shown that the flexural tension force distributes out appreciably more than is provided for in earlier editions of NZS 3101. From the Auckland tests it would appear that the loading could be safely spread over a width of the loaded width plus three slab thicknesses. The area defined by Equations 12–1 and 12–2, should be assumed to be uniformly loaded. Some comments on the elastic theory analysis of slabs with concentrated loads are made in C12.5.3.

A3 The reduction in peak bending moment in the immediate vicinity of a load arises from elongation of the reinforcement under the load point. This elongation is partially restrained by the surrounding slab, and this results in a local arch type action that spreads the flexural tension force to adjacent areas in the slab. The load-spreading action does not reduce the total bending moment resisted by the slab.

C12.5.3 *Design moments from elastic thin plate theory*

General comments. The distribution of moments and shears in slab systems may be calculated on the assumption that the slabs act as thin elastic plates. Such solutions are particularly useful for slabs of unusual shape or boundary conditions where standard solutions such as given by Timoshenko and Woinowsky-Krieger [12.8], Bares [12.9], Hahn [12.10], and others are available. Also the availability of computer programmes based on the finite element method makes the elastic theory solution of complex floor systems possible. A Poisson's ratio of 0.2 is appropriate for prestressed slabs, and it may be used for reinforced concrete slabs. However, as the Poisson strain does not develop in a tension zone containing flexural cracks, a value of 0.1 is appropriate, but generally sufficient accuracy is achieved if Poisson's ratio is assumed to be zero.[12.1]

A convenient method for determining the moments induced in slabs from the action of concentrated loads is by the use of influence surfaces, such as those developed by Pucher [12.11]. The work done by Pucher has been extended substantially by work by Homberg [12.12, 12.13] who derived influence surfaces for continuous or haunched slabs or both. For vehicular loading only the wheel load at the point being considered needs to be represented by a contact area; the more remote wheels may be treated as point loads. For slab and beam bridge construction full edge fixity of a panel should not be assumed when calculating the mid-span moments due to concentrated loads unless the slab supports are sufficiently rigid to prevent rotation. In particular for slabs on longitudinal beams the transverse positive moment and the longitudinal moments used for design should be the average of those for the fully fixed and simply supported conditions at the beams. To allow for some support rotation and for the localised nature of the peak negative moment, the design transverse negative moment for interior spans may be taken as 0.8 of that for the condition of full fixity at the beams. In addition the moments induced by relative deflections and rotations of the beams should be investigated as these moments can significantly alter the local values calculated from elastic plate theory.

Reinforcement for a general moment field. In the general case of a slab with given loading and boundary conditions the elastic theory solution will provide the bending moments per unit width M_x and M_y in the x and y directions, respectively, and the torsional moment per unit width M_{xy}. Generally, reinforcing bars are provided at right angles in the x and y directions for these moments because it is impracticable for the bars to follow the directions of the principal moments over the slab. Designers have tended to ignore the torsional moment M_{xy} because of a lack of a method to account for it, but clearly this is unsafe, particularly where twists are high, such as in the corner regions of slabs. The ultimate resisting moments per unit width required for reinforcing bars placed in the x and y directions for a general design moment field, M_x, M_y and M_{xy} (all per unit width) have been derived by Hillerborg [12.14] and Wood [12.15]. The design rules for placing reinforcement based on this work can be stated as follows. At a point in a moment field where the moments per unit width are M_x, M_y and M_{xy} (the algebraic values of moments should be used), the reinforcement should be provided in the slab in the x and y directions so that the ultimate resisting moments per unit width in the x and y directions, M^*_x and M^*_y, are as follows:

(i) Bottom reinforcement:

Generally $M^*_x = M_x + |M_{xy}|$ ....(Eq. C12–1)

and $M^*_y = M_y + |M_{xy}|$ ....(Eq. C12–2)

If either M^*_x or M^*_y is found to be negative, then the negative value of moment is changed to zero and the other moment is given as follows:

Either $M^*_x = M_x + \left|\dfrac{M^2_{xy}}{M_y}\right|$ with $M^*_y = 0$ ....(Eq. C12–3)

or $M^*_y = M_y + \left|\dfrac{M^2_{xy}}{M_x}\right|$ with $M^*_x = 0$ ....(Eq. C12–4)

If negative M^*_x or M^*_y still occurs, no bottom reinforcement is required. If both M^*_x or M^*_y are negative, no bottom reinforcement is required.

(ii) Top reinforcement:

Generally $M^*_x = M_x - |M_{xy}|$ ....(Eq. C12–5)

and $M^*_y = M_y - |M_{xy}|$ ....(Eq. C12–6)

If either M^*_x or M^*_y is found to be positive, then the positive value of moment is changed to zero and the other moment is given as follows:

Either $M^*_x = M_x - \left|\dfrac{M^2_{xy}}{M_y}\right|$ with $M^*_y = 0$ ....(Eq. C12–7) or $$M_y^* = M_y - \left|\frac{M_{xy}^2}{M_x}\right| \text{ with } M_x^* = 0 \qquad \text{(Eq. C12–8)}$$

If positive M_x^* *or* M_y^* still occurs, no top reinforcement is required.

Sometimes it is difficult to place the reinforcement at right angles, which invalidates the Wood [12.13] equations, which are reproduced in the preceding paragraphs. Armer[12.16] expanded the Wood equations to cover the case where reinforcement is not placed at right angles.

C12.5.4 *Design moments from non-linear analysis*

At times a non-linear analysis taking into account inelastic effects may be appropriate. Finite element analysis can be used. One such approach has been developed by Vecchio[12.17].

C12.5.5 *Design moments from plastic theory*

General comments. A plastic theory can be used in which regard is given to the redistribution of bending moments which can occur before failure of the slab system. The design of slabs using plastic theory has been allowed by the British code of practice for reinforced concrete since 1957. The commonly used plastic theory methods are the yield line theory and the strip method. It should be noted that both of these methods give the flexural strength of the slab and the designer must also check the possibility of shear failure in the case of concentrated loads or reactions. In order to ensure that the sections of the slab are sufficiently ductile to develop the limit bending moment pattern, the tension steel reinforcement ratio p used should not exceed 0.4 p_b, where p_b *is* the ratio producing balanced conditions as defined by 7.4.2.8.

Yield line theory. The most widely used plastic theory method for slabs has been the yield line theory due to Johansen. In this method the ultimate load of the slab is determined when the ultimate moment has developed along a system of yield lines (lines of intense cracking across which the reinforcement has yielded) which convert the slab into a collapse mechanism. However, yield line theory is an upper bound limit design approach and therefore the designer should be careful to examine all possible yield line patterns to ensure that the one giving the lowest ultimate load is used, otherwise the strength of the slab may be overestimated. A comparison of test results from a wide range of slabs with predictions by yield line theory demonstrates that yield line theory gives a safe estimate of the ultimate load capacity of slabs provided that the critical yield line pattern is used. In many cases there is a substantial reserve of strength not predicted by the theory which gives added safety. The critical yield line patterns and ultimate load formulae for slabs with various shapes, boundary conditions and loading are available in the literature. The English translation of one of Johansen's publications [12.18] covers a wide variety of slabs. Other references in English by Park and Gamble, Wood, and Jones [12.3, 12.19, 12.20, 12.21] give a useful range of design information.

Cut-off points of negative moment reinforcement may be calculated by examining the alternative yield line patterns which could form as a result of the curtailment.

Yield line theory shows a strength reduction due to the formation of fans of yield lines in slab corners which can be significant if top reinforcement is absent. Both top and bottom reinforcement should be present in the corner regions of all slabs. The reinforcement present in the top and bottom should be provided for a distance of 0.2 times the longer span in each direction from the corner and should provide an ultimate positive and negative resisting moment per unit width equal to the maximum positive ultimate moment per unit width in the slab.

The supporting beams of slabs designed by yield line theory may be designed on the basis of the loads transferred to the beams from the adjacent segments of the yield line pattern, except for slabs with one or more unsupported free edges [12.3, 12.21].

Strip method . An alternative plastic theory method is the strip method due to Hillerborg. This method follows from the lower bound principles for plates which may be stated as follows: "If a distribution of moments can be found which satisfies the slab equilibrium equation and boundary conditions for a given external load, and if the slab is at every point able to resist these moments, then the given external load will represent a lower limit of the carrying capacity of the slab". In the strip method the load carried by torsion in the slab is put equal to zero and the slab is considered as if composed of systems of strips, generally in two directions at right angles, which enables the design bending moments to be calculated by simple statics involving the equilibrium of the strips. Publications in English by Park and Gamble, Hillerborg, Wood and Armer [12.3, 12.22, 12.23, 12.24] give treatments of the strip method.

Hillerborg [12.22] also introduces the "advanced strip method" which uses triangular and rectangular elements rather than strips to determine the design moments. Park and Gamble, Wood and Armer [12.3, 12.23, 12.24] also describe the concept of "strong bands" which enables beamless slabs supported on columns, and slabs with re-entrant corners and openings, to be treated more easily than using the corner supported rectangular elements introduced by Hillerborg.

The supporting beams of slabs designed by the strip method may be designed on the basis of the loads transferred to the beams by the adjacent strips or segments.

The strip method is an attractive approach to slab design since it involves simple concepts that can be relatively easily applied in the general case to obtain moment diagrams which can be used to determine lengths and quantities of reinforcement.

Arrangement of flexural reinforcement. Both upper and lower bound methods allow the designer freedom to choose arrangements of reinforcement which lead to simple detailing. However, it cannot be over-emphasised that the arrangements of reinforcement chosen should be such that the resulting distribution of ultimate moments of resistance at the various sections throughout the slab does not differ widely from the distribution of moments given by the elastic theory of thin slabs. If large differences between the distribution of ultimate resisting moments and the elastic moments do exist it may mean that the cracking of concrete at the service load is excessive because low reinforcement ratios at highly stressed sections may lead to high steel stresses and hence to large crack widths. Such regions of high steel stress at service load may also result in large deflections. Hence it is important that the designer should keep a feel for the elastic theory distribution of bending moments and use it to help decide the ratios of negative to positive ultimate resisting moments and ratios of the ultimate resisting moments to be used in the two directions. Just how far the reinforcement arrangement can differ from the bending moments given by elastic theory and still result in a serviceable slab has not been conclusively determined but the tests which have been carried out do indicate that sensible arrangements of steel result in serviceable slabs. It is recommended that ratios of negative to positive ultimate moments of resistance per unit width between one and two should be used with some account being taken of the degree of restraint at the edges. For example, if full restraint against rotation is anticipated, a value in the range of 1.5 to 2.0 could be used, but if some rotation is expected a value in the range of 1.0 to 1.5 would be more appropriate. Ratios of the ultimate moments of resistance per unit width in the two directions should take account of the direction of maximum bending moment given by elastic theory. For example, in two-way slabs the greatest ultimate resisting moment per unit width should act in the direction of the short span. At edges which have been considered as simply supported, care should be taken to provide top reinforcement to control cracking due to fortuitous restraining moments. Such reinforcement should be for approximately 0.33 to 0.5 of the maximum positive ultimate moment of resistance per unit width.

Serviceability checks. Checks of deflections are discussed in detail in Section 2. Excessive cracking should not occur providing a reasonable arrangement of reinforcement is used as discussed previously. In cases of concern, maximum crack widths can be estimated by the equation given in 2.4.4.6 and checked against allowable values. The steel stress at the serviceability limit state is required for such a check. This stress can be estimated or, in cases where the distribution or ultimate resisting moments show significant deviations from the bending moments given by elastic theory, elastic theory can be used to obtain a more accurate estimate of the steel stress at the serviceability limit state. Potential cracking due to self strain effects (shrinkage, differential temperature, temperature change) should be considered (see 2.4.4.8.)

Data on the punching shear strength of thick slabs is very limited (thicker than 600 mm). The influence on increased thickness on punching shear stress is expected to reduce the shear stress levels sustained at failure. Consequently caution should be used in the design of such elements, particularly where the members have low reinforcement contents, and the use of some shear reinforcement is advised. A3

C12.5.6 *Slab reinforcement*

C12.5.6.3 *Spacing of principal reinforcement*

The requirement that the centre-to-centre spacing of the principal reinforcement be not more than two times the slab thickness applies only to the reinforcement in solid slabs, and not to that in waffle slabs. This limitation is intended to ensure slab action, to reduce cracking and to provide for the possibility of loads concentrated on small areas of the slab.

C12.5.6.4, C12.5.6.5, C12.5.6.6 *Reinforcement at edges*

Bending moments in slabs at spandrel beams can be subject to great variation. If a slab is integrated with walls, the slab approaches complete fixity there. Without an integral wall, the slab could be largely simply supported dependent on the torsional stiffness of the spandrel beam or slab edge. These requirements provide for unknown conditions that might normally occur in a structure.

C12.5.6.7 *Reinforcement for torsional moments*

Torsional moments are particularly high in slab corners and top and bottom reinforcement should be present there to control cracking of concrete. An analysis based on first principles may be used to determine the reinforcement required to resist the combined flexural and torsional actions, as outlined in C12.5.3. Alternatively the reinforcement in corners required by yield line theory, to prevent fan mechanisms developing, can be regarded as providing the necessary resistance to torsional moments. In lieu or either of these analyses the top and bottom reinforcement described in 12.5.6.7(a), (b) and (c) is recommended. Reinforcement, which is already in the slab corners for other purposes may be considered to be part of the reinforcement for torsional resistance.

C12.5.6.9 *Integrity reinforcement for slabs supported on columns*

Failures due to punching shear are very brittle in nature. When diagonal punching shear cracks form the load carrying capacity is completely lost. There are cases described in the literature where a punching failure has led to major progressive collapses as the load carried by one column is transferred to other columns, which in turn fail due to the additional load that is thrown on them. A simple method of guarding against this form of failure has been proposed. Tests have shown that when punching shear failure occurs the slab drops and reinforcement in the bottom of the slab, which passes through the column, kinks typically to an angle of 30°. The vertical component of the tension force in the kinked reinforcement prevents collapse if its magnitude is equal to or more than the shear transferred between the slab and column [12.25]. Top reinforcement is ineffective as it is pulled out of the concrete. The kinking mechanism for bottom bars forms the basis of the requirement in 12.5.6.9 (a).

In lift slab construction reinforcement cannot be passed through or anchored in the column. However, bottom reinforcement, which passes over supporting elements that are tied into the column, it acts to resist collapse due to punching shear due to the bearing on the bottom surface of the slab from the supporting elements that are tied into the column. The location of this reinforcement is shown in Figure C12.1.

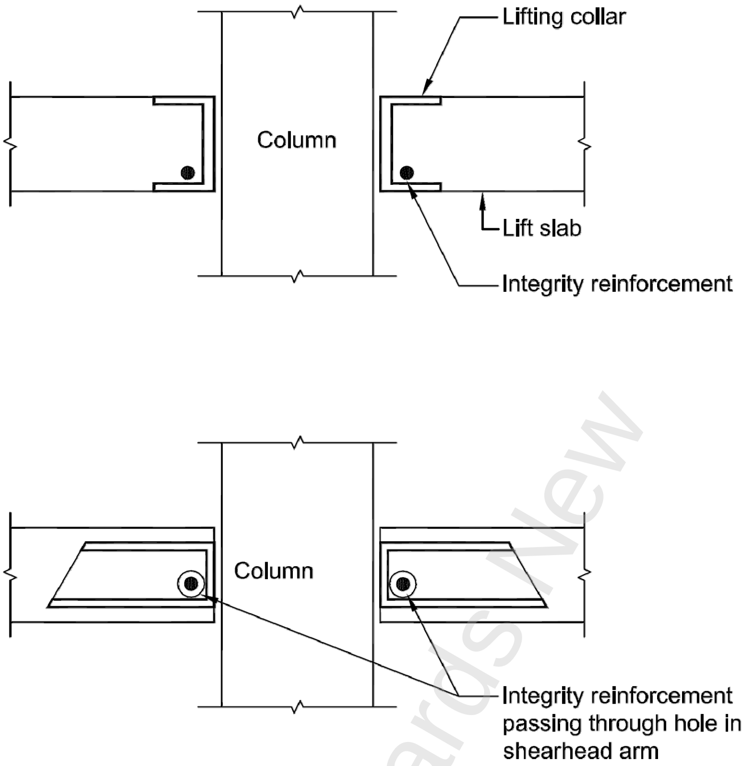


Figure C12.1 – Location of integrity reinforcement

C12.6 Serviceability of slabs

Reinforcement well distributed to limit cracking and adequate stiffness to limit deflections are important considerations. The cracking and deflection of slabs are not generally controlled by Standards such as AS/NZS 1170.0 or NZS 1170.5, although general recommendations are given. These serviceability criteria need to be evaluated and set on a case by case basis.

C12.7 Design for shear

C12.7.1 *Critical sections for shear*

Large scale test for punching shear are difficult to carry out. Consequently there are a few tests in the literature to indicate what scale affects might exist. Consequently, it is recommended that designers take a conservative approach to design for punching shear in thick members.

Differentiation must be made between a long and a narrow slab or footing acting as a beam, and a slab or footing subject to two-way action where failure may occur by 'punching' along a truncated cone or pyramid around a concentrated load or reaction area.

The critical section for shear in slabs subjected to bending in two directions follows the perimeter at the edge of the loaded area[12.26]. The shear stress acting on this section at factored loads is a function of $\sqrt{f'_c}$ and the ratio of the side dimension of the column to the effective slab depth. A much simpler design equation results by assuming a pseudo-critical section located at a distance $d/2$ from the periphery of the concentrated load. When this is done, the shear strength is almost independent of the ratio of the column size to slab depth. For rectangular columns, this critical section was defined by straight lines drawn parallel to, and at a distance $d/2$ from the edges of the loaded area. Clause 12.7.1(b) allows the use of a rectangular critical section.

For slabs of uniform thickness, it is sufficient to check shear on one section. For slabs with changes in thickness, such as the edge of drop panels, it is necessary to check shear at several sections.

For edge columns at points where the slab cantilevers beyond the column, the critical perimeter will either be three-sided or four-sided.

C12.7.3.2 *Nominal shear strength provided by the concrete,* Vc

For square columns, the shear force V_c due to ultimate loads in slabs subjected to bending in two directions is limited to $(^1/_3)\sqrt{f_c'}\,b_o d$. However, tests[12.27] have indicated that the value of $(^1/_3)\sqrt{f_c'}\,b_o d$ is unconservative when the ratio β_c of the lengths of the long and short sides of a rectangular column or loaded area is larger than 2.0. In such cases, the actual shear force on the critical section at punching shear failure varies from a maximum of about $(^1/_3)\sqrt{f_c'}\,b_o d$ around the corners of the column or loaded area, down to $(^1/_6)\sqrt{f_c'}\,b_o d$ or less along the long sides between the two end sections. Other tests [12.28] indicate that V_c decreases as the ratio b_o/d increases. Equations 12–6 and 12–7 were developed to account for these two effects. The words "interior", "edge", and "corner columns" in 12.7.3.2(b) to critical sections with four, three, and two sides, respectively.

For shapes other than rectangular, β_c is taken to be the ratio of the longest overall dimension of the effective, loaded area to the largest overall perpendicular dimension of the effective loaded area, as illustrated for an L-shaped reaction area Figure C12.2. The effective loaded area is that area totally enclosing the actual loaded area, for which the perimeter is a minimum to find the critical value.

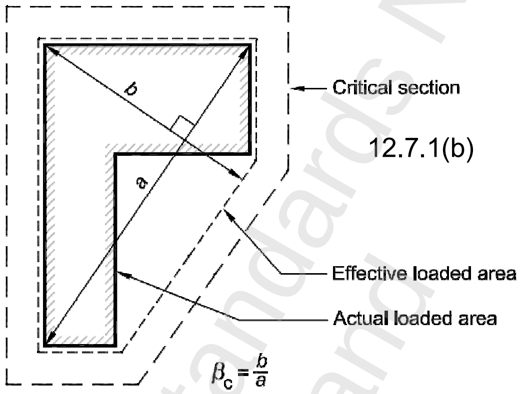


Figure C12.2 – Value of β_c for a non-rectangular loaded area

A2 Recent research (Birkle, G., and Diliger, W. H., "Influence of slab thickness on punching shear strength", ACI Structural Journal, Vol. 105, No. 2, Mar. – Apr. 2008, pp. 180-189) has indicated that the shear that can be resisted by concrete prior to diagonal cracking due to punching shear, decreases as the depth increases. To allow for this, the factor k_{ds} has been introduced into the equations for v_c prior to cracking. This factor decreases from 1.0 for slabs with an average effective depth of 200 mm, to 0.5 for slabs with an average effective depth of 800 mm. The value of v_c, when shear reinforcement is required is given by 12.7.3.5.

C12.7.3.5 *Shear to be resisted by shear reinforcement for punching shear*

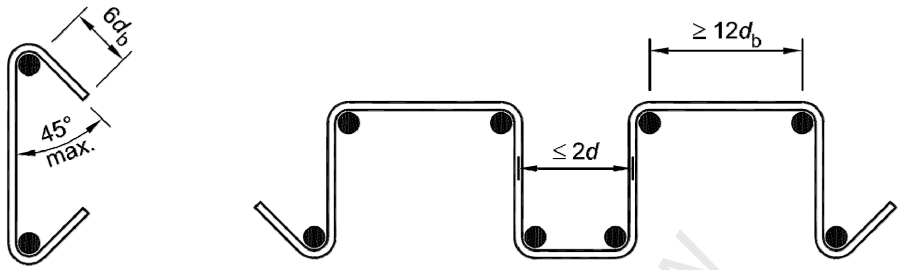
It should be noted that for punching shear the resistance provided by the concrete decreases markedly when diagonal cracking occurs. The shear force for each side in the perimeter needs to be calculated separately.

C12.7.4 *Shear reinforcement consisting of bars or wires or stirrups*

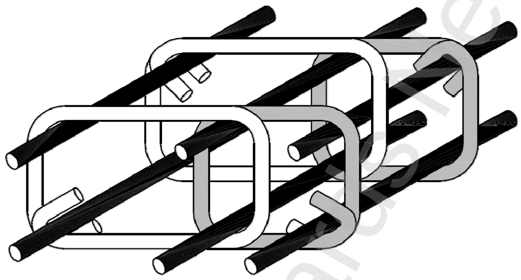
Research [12.29,12.30,12.31,12.32] has shown that shear reinforcement consisting of properly anchored bars or wires and single-or multiple-leg stirrups, or closed stirrups, can increase the punching shear resistance of slabs. The spacing limits given in 12.7.4.4 correspond to slab shear reinforcement details that have been shown to be effective. Clause 12.7.4.5 gives anchorage requirements for stirrup-type shear reinforcement that should also be applied for bars or wires used as slab shear reinforcement. It is essential that this shear reinforcement engages longitudinal reinforcement at both the top and bottom of the slab, as shown for typical details in Figure C12.3(a) to (c). Anchorage of shear reinforcement according to the requirements of 12.7.4.4 is difficult in slabs thinner than 250 mm. Shear reinforcement consisting of vertical bars mechanically anchored at each end by a plate or head capable of developing the yield strength of the bars have been used successfully[12.33].

In a slab-column connection for which the moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section Figure C12.3(d). Spacing limits defined in

12.7.4.4 are also shown in Figure C12.3(d) and (e). At edge columns or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible. Although the average shear stresses on faces AD and BC of the exterior column in Figure C12.3 (e) are lower than on face AB, the stirrups extending from faces AD and BC provide some torsional capacity along the edge of the slab.



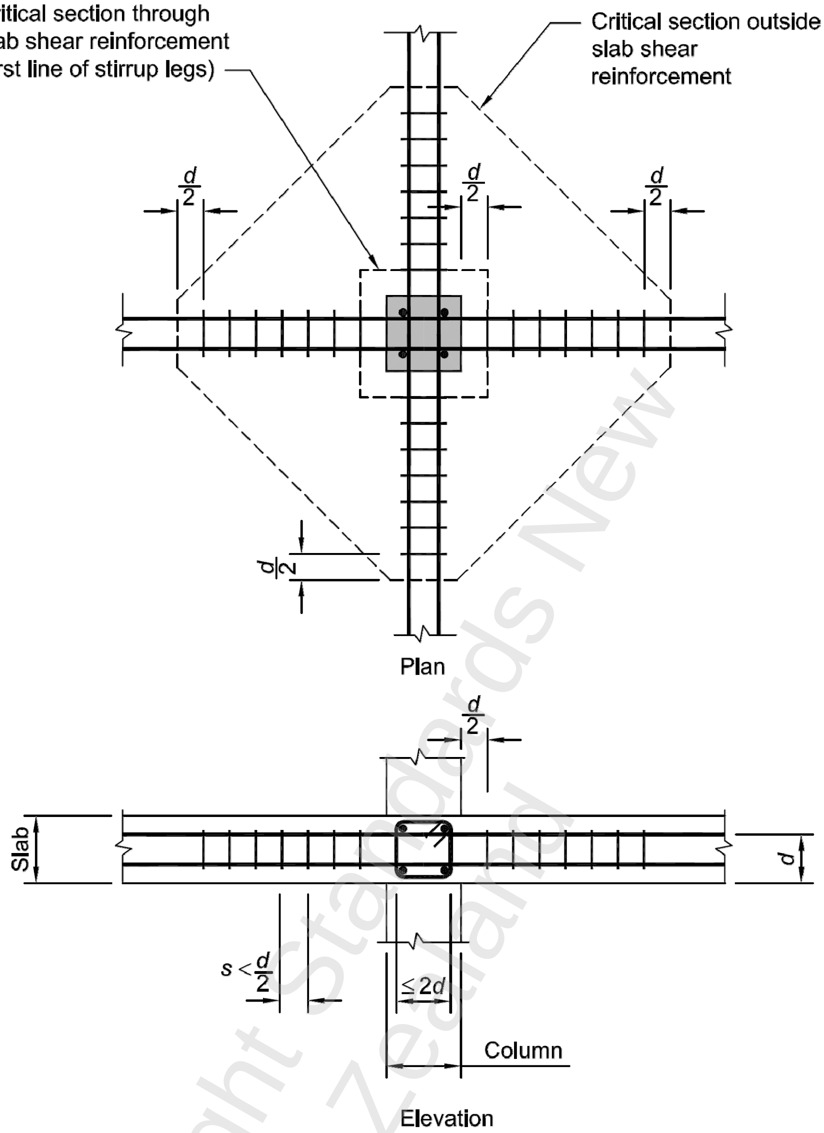
(a) and (b) Single and multiple-leg stirrup type slab shear reinforcement



(c) Single or multiple-leg stirrup type slab shear reinforcement

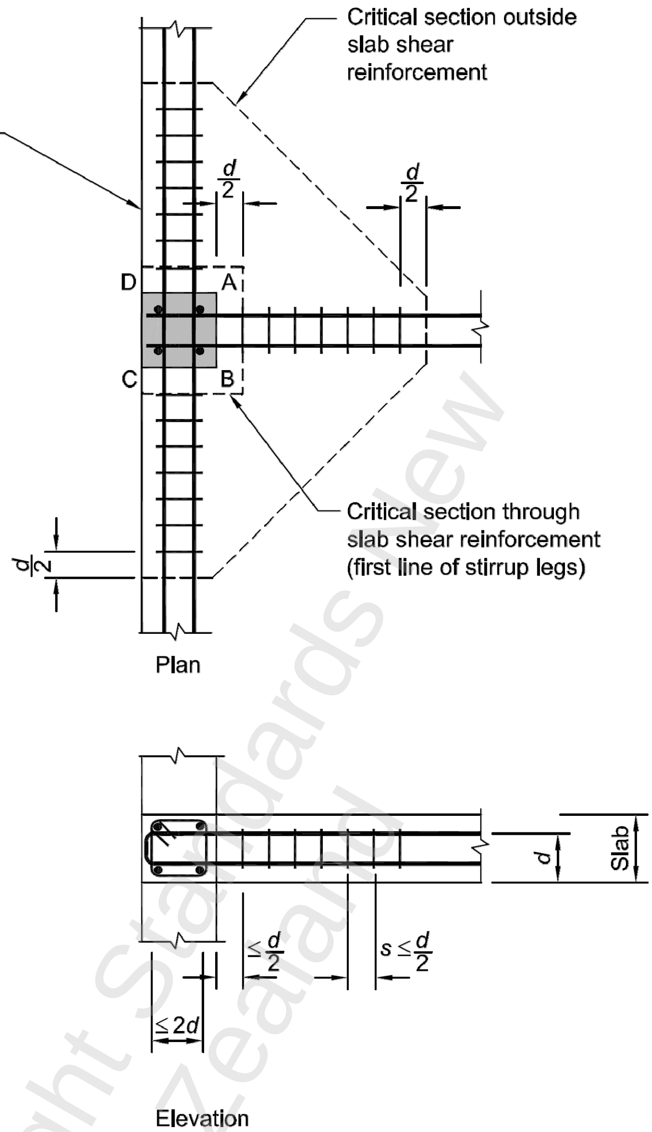
Figure C12.3 – Shear reinforcement for slabs

(Continued on next page)



(d) Arrangement of stirrup shear reinforcement, interior column

Figure C12.3 – Shear reinforcement for slabs (continued)



(e) Arrangement of stirrup shear reinforcement, edge column

Figure C12.3 – Shear reinforcement for slabs (continued)

C12.7.5 *Shear reinforcement consisting of structural steel I or channel-shaped sections and other equivalent devices*

C12.7.5.1 *General*

Based on reported test data[12.34], design procedures are presented for shearhead reinforcement consisting of structural steel shapes. For a column connection transferring moment, the design of shearheads is given in 12.7.7.3.

Three basic criteria should be considered for the design of shearhead reinforcement for connections transferring shear due to gravity load. First, a minimum flexural strength must be provided to ensure that the required shear strength of the slab is reached before the flexural strength of the shearhead is exceeded. Second, the shear stress in the slab at the end of the shearhead reinforcement must be limited. Third, after these two requirements are satisfied, the designer can reduce the negative slab reinforcement in proportion to the moment contribution of the shearhead at the design section.

C12.7.5.2 *Details of shearheads*

The assumed idealised shear distribution along an arm of a shearhead at an interior column is shown in Figure C12.4. The shear along each of the arms is taken as $\alpha_v V_c / \eta$, where α_v and η are defined in 12.7.5.2 (a) and (f), and V_c is defined in 12.7.3.2. However, the peak shear at the face of the column is taken as total shear considered per arm $V^*/\phi\eta$ minus the shear considered carried to the column by the concrete compression zone of the slab. The latter term is expressed as (V_c / η) $(1 - \alpha_v)$, so that it approaches zero for a heavy shearhead and approaches $V^*/\phi\eta$ when a light shearhead is used. Equation 12–15 then follows from the assumption that ϕV_c is about one-half the design shear force V^*. In this equation M_p is the required plastic moment strength of each shearhead arm necessary to ensure that design shear force V^* is attained as the moment strength of the shearhead is reached. The quantity L_v is the length from the centre of the column to the point at which the shearhead is no longer required, and the distance $c_1/2$, is one-half the dimension of the column in the direction considered.

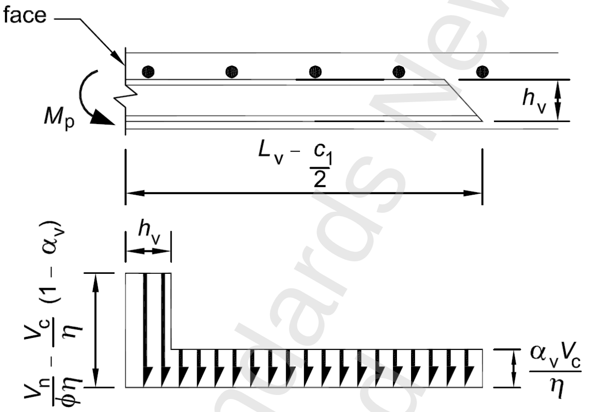
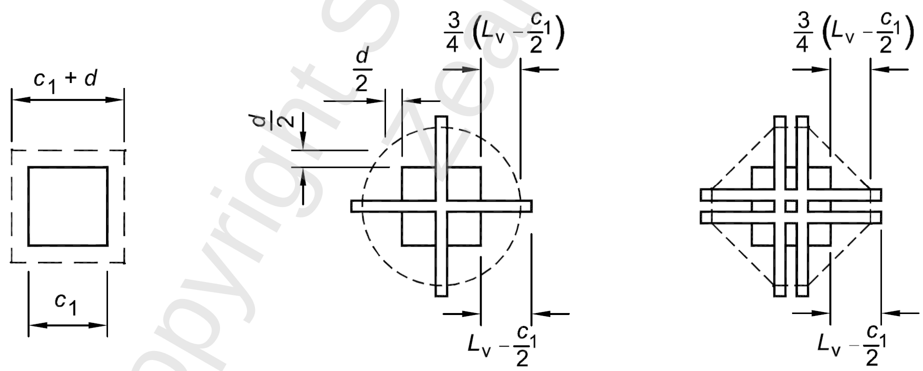
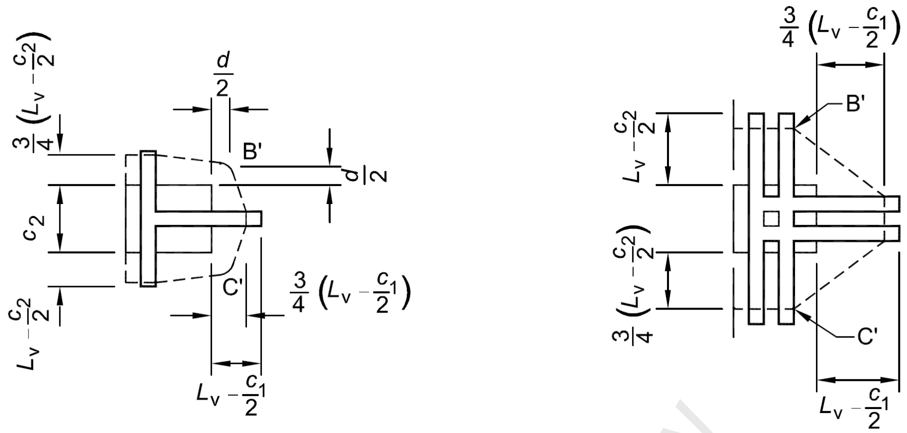


Figure C12.4 – Idealised shear force acting on shearhead



(a) No shearhead (b) Small interior shearhead ($\eta = 4$) (c) Large interior shearhead ($\eta = 4$)

Figure C12.5 – Location of critical section defined in 12.7.5.3



(d) Small edge shearhead (η = 3)

(e) Large edge shearhead (η = 3)

Figure C12.5 – Location of critical section defined in 12.7.5.3 (continued)

C12.7.5.3 *Critical slab section for shear and* **C12.7.5.4** *limit on nominal shear strength*

The test results [12.34] indicated that slabs containing under reinforcing shearheads failed at a shear stress on a critical section at the end of the shearhead reinforcement less than $(^1/_3)\sqrt{f'_c}$. Although the use of over-reinforced shearheads brought the shear strength back to about the equivalent of $(^1/_3)\sqrt{f'_c}\, b_o d$, the limited test data suggest that a conservative design is desirable. Therefore, the shear strength is calculated as $(^1/_3)\sqrt{f'_c}\, b_o d$ on an assumed critical section located inside the end of the shear head reinforcement.

The critical section is taken through the shearhead arms three-quarters of the distance $[L_v - (c_1/2)]$ from the face of the column to the end of the shearhead. However, this assumed critical section need not be taken closer than $d/2$ to the column. See Figure C12.5.

C12.7.5.5 *Moment of resistance contributed by shearhead*

If the peak shear at the face of the column is neglected, and ϕV_c is again assumed to be about one-half of V^*, the moment contribution of the shearhead M_v can be conservatively computed from Equation 12–16, in which ϕ is the strength reduction factor for flexure.

C12.7.6 *Openings in slabs*

Provisions for design of openings in slabs (and footings) were developed in Reference 12.26. The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in Figure C12.6. Additional research [12.27] has confirmed that these provisions are conservative.

C12.7.7 *Transfer of moment and shear in slab column connections*

In Reference 12.35 it was found that where moment is transferred between a column and a slab, 60 % of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 12.7.1, and 40 % by eccentricity of the shear about the centroid of the critical section. For rectangular columns, the portion of the moment transferred by flexure increases as the width of the face of the critical section resisting the moment increases, as given by Equation 12–16.

Most of the data in Reference 12.35 were obtained from tests of square columns, and little information is available for round columns. These can be approximated as square columns. Figure C12.7 shows square supports having the same areas as some non-rectangular members.

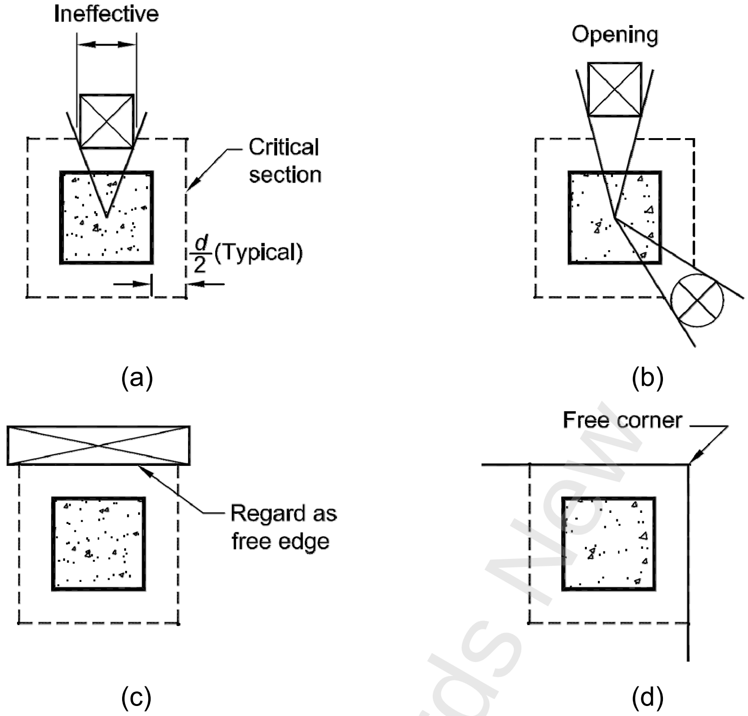


Figure C12.6 – Effect of openings and free edges (effective perimeter shown with dashed lines)

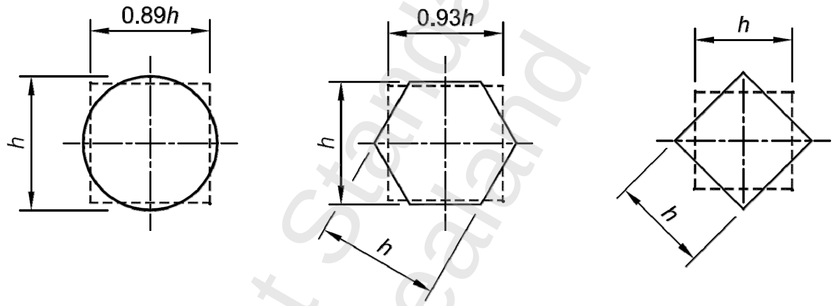


Figure C12.7 – Equivalent square supporting sections

C12.7.7.3 *Unbalanced moment transferred by eccentricity of shear*

The stress distribution is assumed as illustrated in Figure C12.8 for an interior or exterior column. The perimeter of the critical section, *ABCD*, is determined in accordance with 12.7.1. The design shear force V^* and unbalanced moment M^* are determined at the centroidal axis *c-c* of the critical section. The maximum design shear stress may be calculated from:

$$v_{AB} = \frac{V^*}{A_c} + \frac{\gamma_v M^* c_{AB}}{J_c} \quad \text{(Eq. C12–9)}$$

or

$$v_{CD} = \frac{V^*}{A_c} - \frac{\gamma_v M^* c_{CD}}{J_C} \quad \text{(Eq. C12–10)}$$

where γ_v is given by Equation 12–17. For an interior column, A_c and J_c may be calculated by:

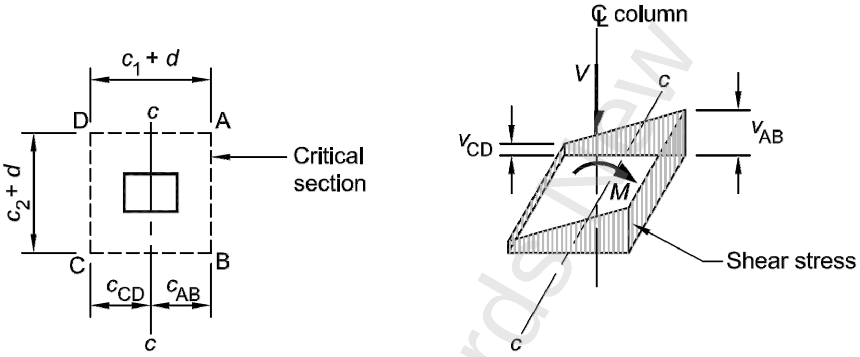
A_c = area of concrete of assumed critical section

$= 2d\,(c_1 + c_2 + 2d)$

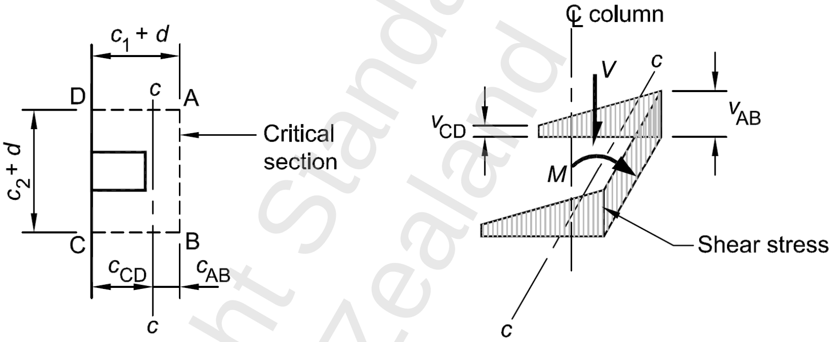
J_c = property of assumed critical section analogous to polar moment of inertia

$$= \frac{d\left(c_1 + d\right)^3}{6} + \frac{\left(c_1 + d\right)d^3}{6} + \frac{d\left(c_2 + d\right)\left(c_1 + d\right)^2}{2} \quad \text{(Eq. C12–11)}$$ A3

Similar equations may be developed for A_c and J_c for columns located at the edge or corner of a slab.



(a) Interior column



(b) Edge column

Figure C12.8 – Assumed distribution of shear stress

The fraction of the unbalanced moment between slab and column not transferred by eccentricity of the shear should be transferred by flexure in accordance with 12.7.7.2. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 12.7.7.2. Often designers concentrate column strip reinforcement near the column to accommodate this unbalanced moment. Available test data[12.35] seems to indicate that this practice does not increase shear strength but may be desirable to increase the stiffness of the slab-column junction.

Where shear reinforcement has been used, the critical section beyond the shear reinforcement generally has a polygonal shape as in Figure C12.3(d) and (e). Equations for calculating shear stresses on such sections are given in Reference 12.32.

Tests [12.36] indicate that the critical sections defined in 12.7.1(b) are appropriate for calculations of shear stresses caused by transfer of moments even when shearheads are used. Then, even though the critical sections for direct shear and shear due to moment transfer differ, they coincide or are in close proximity at the column corners where the failures initiate. Because a shearhead attracts most of the shear as it funnels toward the column, it is conservative to take the maximum shear stress as the sum of the two components.

Note that 12.7.5.5 requires the moment M_p to be transferred to the column in shearhead connections transferring unbalanced moments. This may be done by bearing within the column or by mechanical anchorage.

C12.8 Design of reinforced concrete bridge decks

A2

C12.8.2 *Empirical design based on assumed membrane action*

C12.8.2.1 *General*

Elastic plate bending has been found to be conservative in situations where the boundary conditions of the slab restrict lateral movement. Extensive research into the behaviour of deck slabs has discovered that the primary structural action by which these slabs resist concentrated wheel loads is not flexure, as traditionally believed, but a complex internal membrane stress state referred to as internal arching. This action is made possible by the cracking of the concrete in the positive moment region of the design slab and the resulting upward shift of the neutral axis in that portion of the slab. The action is sustained by in-plane membrane forces that develop as a result of lateral confinement provided by the surrounding concrete slab, rigid appurtenances, and supporting components acting compositely with the slab.

The arching creates what can best be described as an internal compressive dome, the failure of which usually occurs as a result of overstraining around the perimeter of the wheel footprint. The resulting failure mode is that of punching shear, although the inclination of the fracture surface is much less than 45°, due to the presence of large in-plane compressive forces associated with arching. The arching action, however, cannot resist the full wheel load. There remains a small flexural component for which the specified minimum amount of isotropic reinforcement is more than adequate. The steel has a dual purpose; it provides for both local flexural resistance and global confinement required to develop arching effects.[12.37, 12.38]

The provisions are based on research studies[12.39], the findings from which have been verified by full-scale field studies[12.40, 12.41, 12.42]. It has been concluded that if the conditions specified in this clause are met, composite deck slabs can be expected to perform satisfactorily under the wheel loads specified by the

A3 New Zealand Transport Agency's Bridge Manual.

A2

C12.8.2.2 *Conditions*

All tests carried out so far have been restricted to specimens of uniform depth.

Spans with the minimum specified reinforcement have demonstrated nearly complete insensitivity to differential displacement among their supports. Intermediate cross-frames are not needed in order to use the empirical deck design method for cross sections involving torsionally weak open shapes such as T- or I-shaped girders.

Use of separated, torsionally stiff girders without intermediate diaphragms, can give rise to the situation, shown in Figure C12.9 in which there is a relative displacement between the girders and in which the girders do not rotate sufficiently to relieve the moments over the webs. This moment may or may not require more reinforcing than is provided by the empirical deck design.

A2

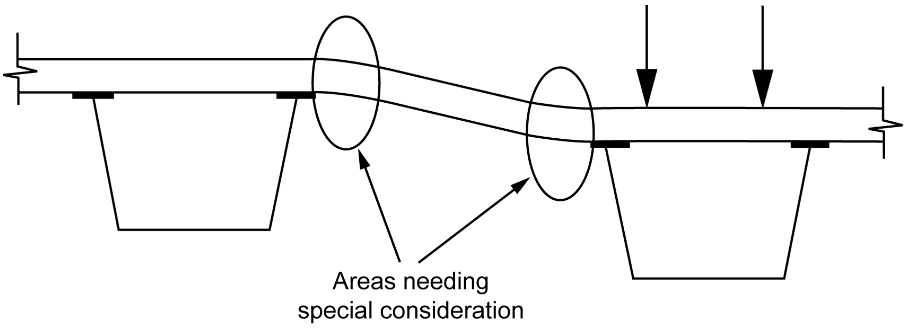


Figure C12.9 Schematic of effect of relative displacements in torsionally stiff cross section

Physical tests and analytical investigations indicate that the most important parameter concerning the resistance of concrete slabs to wheel loads is the ratio between the span length and the depth of the slab. The span length to depth ratio limit of 18.0 is based on the results of experiments.[12.43]

No experience exists for span lengths exceeding 4.100 m. The 175 mm depth is considered an absolute minimum by both the AASHTO and Canadian Bridge Design Codes.

The intention of the overhang provision is to ensure confinement of the slab between the first and the second girders.

30 MPa is the concrete strength generally adopted for *in situ* deck construction in New Zealand bridges. None of the tests included concrete with less than 28 MPa strength at 28 days. On the other hand, tests indicate that the resistance is not sensitive to the compressive strength.

C12.8.2.3 *Reinforcement*

Prototype tests have indicated that 0.2 % reinforcement in each direction in both the top and bottom layers, placed at the minimum required cover, satisfies strength requirements. However, the conservative value of 0.3 % of the gross area, which corresponds to 570 mm$^{2/}$m in a 190 mm thick slab, is specified for better crack control in the positive moment area. Field measurements show very low stresses in the negative moment steel; this is reflected by the 380 mm^2/m requirement, which is about 0.2 % reinforcement steel.

Lap welded splices are not permitted due to fatigue considerations. Tested and pre-approved mechanical splices may be permitted when lapping of reinforcement is not possible or desirable, as often occurs in staged construction or widenings.

Beam and slab bridges with a skew exceeding 25° have shown a tendency to develop torsional cracks due to differential deflections in the end zone, and therefore the provision of additional reinforcement is required in the end zones to counter this.

C12.8.2.4 *Longitudinal negative moments in continuous structures*

The additional longitudinal reinforcement provided in the slab in the negative moment region of continuous beams and girder-type bridges beyond that required for isotropic reinforcement, according to the provisions of 12.8.2.3, need not be matched in the perpendicular direction. Theoretically, this portion of deck will be orthotropically reinforced, but this does not weaken the deck.

REFERENCES

12.1 Pan, A. and Moehle, J.P., "Lateral Displacement Ductility of Reinforced Concrete Flat Plates", ACI Structural Journal, Vol. 86, No. 3, 1989, pp. 250-258.

12..2 Hawkins, N.M. and Michell, D., "Progressive Collapse of Flat Plate Structures", ACI Journal, 1979, No. 7, pp. 775-808.

12.3 Park, R. and Gamble, W.L., "Reinforced Concrete Slabs", John Wiley & Sons, New York, 1980 (2nd edition 2000).

12.4 Hewitt, B.E. and Batchelor, B. de V., "Punching Shear Strength of Restrained Slabs" Journal of the Structural Division, ASCE, Vol. 101, No. ST9, 1975, pp. 1837-1853.

12.5 OHBDC. Ontario Highway Bridge Design Code. Ministry of Transportation of Ontario, Downsview, Ontario, 1991.

12.6 Fenwick, R.C. and Dickson, A.R., "Slabs Subjected to Concentrated Loading", ACI Structural Journal, Vol. 86, No. 6, Nov.- Dec. 1989, pp. 672-678.

12.7 Dickson, A.R., "The Response of Reinforced Concrete Slabs to Concentrated Loading", PhD thesis, University of Auckland, School of Engineering Report, No. 408, Feb. 1986, p. 221.

12.8 Timoshenko, S. and Woinowsky-Krieger, S., "Theory of Plates and Shells", 2nd Edition, McGraw-Hill Book Co., New York, 1959.

12.9 Bares, R., "Table for the Analysis of Plates, Slabs and Diaphragms Based on the Elastic Theory", Bauverlag GmbH, Wiesbaden and Berlin, 1969.

12.10 Hahn, J., "Structural Analysis of Beams and Slabs" English translation by C.V. Amerongen, Pitman, London, 1966.

12.11 Pucher, A., "Influence Surfaces of Elastic Plates", Springer-Verlag, Vienna, 1964.

12.12 Homberg, H., "Double Webbed Slabs", Springer-Verlag, Berlin, 1973.

12.13 Homberg, H., "Dalles d'Epaisseur Variable", Danod, Paris, 1972.

12.14 Hillerborg, A., "Reinforcement of Slabs and Shells Designed According to the Theory of Elasticity", Béton, Vol. 38, No. 2, 1953, (Translated by Building Research Station, Watford, England, 1962. Library Communication No. 1081).

12.15 Wood, R.H., "The Reinforcement of Slabs in Accordance With a Pre-determined Field of Moments", Concrete, Vol. 2, No. 2, February, 1968.

12.16 Armer, G.S.T., Discussion of reference 12.15, Concrete, Vol. 2, No. 8, Aug. 1968, pp. 319-320. (Gives equations for design of slabs with skew reinforcement).

12.17 Vecchio, F.J., "Non-linear Finite Element Analysis of Reinforced Concrete Membranes" American Concrete Institute, Structural Journal, Vol. 86, No. 1, 1989, pp. 25-35.

12.18 Johansen, K.W., "Yield-line Formulae for Slabs", Translated by Cement and Concrete Association, London, 1972.

12.19 Wood, R.H. and Jones, L.L., "Yield-line Analysis of Slabs", Thames & Hudson and Chatto & Windus, London, 1967.

12.20 Jones, L.L., "Ultimate Load Analysis of Reinforced and Prestressed Concrete Structures", Chatto and Windus, London, 1962.

12.21 Park, R., "Ultimate Strength Design of Reinforced Concrete Structures", Vol. 2, "Limit Design of Reinforced Concrete Slabs", Department of Civil Engineering and Extension Studies, University of Canterbury, New Zealand.

12.22 Hillerborg, A., "Design of Reinforced Concrete Slabs According to the Strip Method", Translated by Cement and Concrete Association, London, 1975.

12.23 Wood, R.H. and Armer, G.S.T., "The Theory of the Strip Method for the Design of Slabs", Proceedings of the Institution of Civil Engineers, Vol. 41, October 1968.

12.24 Armer, G.S.T., "The Strip Method: A New Approach to the Design of Slabs", Concrete, Vol. 2, No. 9, September 1968.

12.25 Hawkins, N.M., and Mitchell, D., "Progressive collapse of flat plate structures", ACI Journal, No. 7, 1979, pp 775-808.

12.26 ACI-ASCE Committee 326 (now 426) "Shear and Diagonal Tension", ACI Journal Proceedings Vol. 59, No.1 January 1962, pp. 1–30, No.2, February 1962, pp. 277–234 and No. 3 March 1962, pp. 352-396.

12.27 ACI-ASCE Committee 426, "The Shear Strength of Reinforced Concrete Members," Proceedings, ASCE, V. 100, No. ST8, Aug. 1974, pp. 1543–1591.

12.28 Vanderbilt, M.D., "Shear Strength of Continuous Plates," Journal of the Structural Division, ASCE, V.98, No. ST5, May 1972, pp. 961–973.

12.29 Hawkins, N.M., "Shear Strength of Slabs with Shear Reinforcement," Shear in Reinforced Concrete, SP-42, V. 2, American Concrete Institute, Farmington Hills, MI, 1974, pp. 785-815.

12.30 Broms, C.E., "Shear Reinforcement for Deflection Ductility of Flat Plates," ACI Structural Journal, Vol. 87, No. 6, Nov.-Dec. 1990, pp. 696-705.

12.31 Yamada, T., Nanni, A. and Endo, K., "Punching Shear Resistance of Flat Slabs: Influence of Reinforcement Type and Ratio," ACI Structural Journal, Vol. 88, No. 4, July-Aug. 1991, pp. 555-563.

12.32 Hawkins, N.M., Mitchell, D. and Hannah, S.N., "The Effects of Shear Reinforcement on Reversed Cyclic Loading Behaviour of Flat Plate Structures," Canadian Journal of Civil Engineering (Ottawa), Vol. 2, 1975, pp. 572-582.

12.33 ACI-ASCE Committee 421, "Shear Reinforcement for Slabs (ACI 421.1R-99)," American Concrete Institute, Farmington Hills, MI, 1999, p. 15.

12.34 Corley, W.G. and Hawkins, N.M., "Shearhead Reinforcement for Slabs", ACI Journal, Proceedings, Vol. 65, No. 10, Oct. 1968, pp. 811-824.

12.35 Hanson, N.W. and Hanson, J.M., "Shear and Moment Transfer between Concrete Slabs and Columns," Journal, PCA Research and Development Laboratories, Vol. 10, No. 1, Jan. 1968, pp. 2-16.

12.36 Hawkins, N.M. and Corley, W.G., "Moment Transfer to Columns in Slabs with Shearhead Reinforcement," Shear in Reinforced Concrete, SP-42, American Concrete Institute, Farmington Hills, MI, 1974, pp. 847-879.

12.37 Fang, K.I., "Behaviour of Ontario-Type Bridge Deck on Steel Girders", Ph.D. diss., University of Texas, Austin, Texas, USA, December 1985.

12.38 Holowka, M., Dorton, R.A., and Csagoly, P.F., "Punching Shear Strength of Restrained Circular Slabs", Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1980.

12.39 Batchelor, B. de V., "Membrane Enhancement in Top Slabs of Concrete Bridges" – Chapter 6, "Concrete Bridge Engineering: Performance and Advances", Elsevier Applied Science Publishers Ltd., Barking, Essex, England, 1987.

12.40 Dorton, R.A., Holowka, M. and King, J.P.C., "The Connestogo River Bridge – Design and Testing." Can. J. Civ. Eng., Vol. 4, No. 1 1977, pp. 18-39.

12.41 Holowka, M., "Testing of Composite Prestressed Concrete AASHTO Girder Bridge." RR222, Ministry of Transportation, Downsview, Ontario, 1980.

12.42 Holowka, M., "Testing of a Trapezoidal Box Girder Bridge," RR221, Ministry of Transportation, Downsview, Ontario, 1981

12.43 Hays, C.O., Lybas, J.M., and Guevara, J.O., "Test of Punching Shear Strength of Lightly Reinforced Orthotropic Bridge Decks", University of Florida, Gainsville, Florida, USA, 1989.

NOTES

C13 DESIGN OF DIAPHRAGMS

C13.2 Scope and definitions

This section presents design requirements for diaphragms. The primary role of diaphragms is to ensure efficient interaction of all lateral force-resisting elements in a building. Generally two types of diaphragm actions are encountered in buildings [13.1 13.2 13.3]. The first type of action occurs at every floor where the floor system, acting as a horizontal deep beam, transmits forces generated by wind or an earthquake to various lateral force-resisting components, such as frames or structural walls. The second type of action is encountered where, at a particular level, large in-plane shear forces need to be transferred from one vertical lateral force-resisting component such as a shear core, to others, such as peripheral foundation walls, and in dual structural systems. All diaphragms typically have to undertake both roles of force transfer. Depending on geometry of the structure and the interrelationship of the vertical structure elements (walls or frames), one of the two types will dominate[13.4].

Section 13.3 applies to all diaphragms that are not designed to dissipate energy. Beam or wall elongation may cause isolated yielding of the flooring reinforcement, however, this is deemed to not constitute an energy dissipating diaphragm and therefore the requirements of 13.3 apply.

C13.3 General principles and design requirements

C13.3.1 *Functions of diaphragms*

Most diaphragms will simultaneously act as floor slabs and hence will contain some reinforcement in both directions. However, supplementary reinforcement to enable efficient diaphragm action to develop may sometimes be necessary.

C13.3.2 *Analysis procedures*

Differential creep, shrinkage and temperature effects can influence the serviceability of the structure and should be considered together with in-plane stiffness particularly when precast concrete floor systems are used[13.5]. In particular the effects of differential temperature should be considered on diaphragms that are exposed to the sun and contain precise prestressed units. Failure to allow for the rotations induced by differential temperature can cause damage at the supports of precast units.

In general, diaphragms may be modelled using a strut and tie approach. Design forces and corresponding reactions cannot usually be defined with great precision. However, equilibrium conditions must be established and reinforcement provided so as to ensure adequate strength.

C13.3.3 *Openings*

The presence of large openings in the floor systems, possibly interfering with simple diaphragm action, is often inevitable. Rational analysis, clearly identifying in-plane paths of internal force, should be employed to enable in such a situation the appropriate locations and anchorages of reinforcement to be established. To this end preferably strut and tie models[13.6], some details of which are given in Appendix A, should be used. Load paths within a diaphragm for earthquake forces acting from different directions may be different[13.3].

C13.3.4 *Stiffness*

For most buildings, in-plane deformations associated with diaphragm actions will be negligible. Therefore the assumption of infinite rigidity of diaphragms in the lateral force analysis of the entire structural system will be a satisfactory approximation. However, in long and narrow buildings, particularly where dual systems are used, and where large openings are present, diaphragm flexibility may significantly affect the participation of certain lateral force-resisting vertical elements in the resistance of the total lateral force. Diaphragm flexibility should be taken into account in the overall analysis where the maximum lateral deformation of the diaphragm is more than twice the average storey drift in the relevant storey at the ultimate limit state[13.3, 13.7].

C13.3.6 *Changes in depth*

Joints, including construction joints, may reduce the ability of diaphragms to transfer in-plane forces over the full thickness. In evaluating force transfer, only the effective interface area at the section should be considered.

C13.3.7 *Diaphragms incorporating precast concrete elements*

Where precast floor elements are used, in contrast to cast-in-place concrete slabs, numerous joints in both principal directions of the floor plan will be present. It is essential that continuity in the transfer of internal actions over the entire floor, is assured.

C13.3.7.2 *Requirements for toppings transferring diaphragm forces*

Due to the potential weakness of in-plane shear transfer at joints between precast concrete elements, it is preferable to rely entirely on cast-in-place reinforced concrete topping, at least 50 mm thick. To ensure this minimum thickness over elements with camber, or minimum cover over reinforcement where lapped splices occur, 65 mm thickness for the topping should be specified. Analysis may show that the minimum reinforcement specified in 8.8 is not adequate to resist the derived diaphragm action.

For composite action of the precast and cast-in-place parts of the finished floor slab, satisfactory bond between the two components is essential. This is to prevent separation of the topping and hence to ensure its stability in transferring the in-plane compression diaphragm forces.

C13.3.7.4 *Transfer of diaphragm forces across joints in untopped systems*

Where precast elements are used without an effective cast-in-place concrete topping, in-plane force transfer due to diaphragm action must rely on appropriately reinforced joints between precast elements. This may be difficult to achieve unless precast elements are specifically designed and constructed to allow effective dowels or equivalent reinforcement to be placed in joints that are to be subsequently filled with fresh concrete. Examples are precast prestressed hollow-core floors for which a variety of connection details have been developed[13.8]. This clause requires the designer to verify that connections between precast elements, as well as the reinforcement within each element, are such that the diaphragm performance equivalent to that of a cast-in-place concrete slab with at least minimum reinforcement in both principal directions is achieved.

C13.3.7.5 *Connection of diaphragm to primary lateral force-resisting system*

The requirements of this clause are complementary to those of 13.3.7.4 to ensure that diaphragm forces are safely transferred from precast elements to frames or walls that provide the lateral force resistance for the building. Forces at these connections are in general more critical than those to be transferred from one precast panel to another.

Particular attention must be given to adequate anchorage of the reinforcement in the topping within chord members such as beams, bands or walls. Alternatively, adequately anchored starter bars projecting from such chord members may be spliced with the reinforcement in the topping in accordance with 8.7.2 and 8.7.6.

C13.3.10 *Reinforcement near plastic hinges in beams*

A horizontal diaphragm is a part of the floor system. Therefore, it will interact with the supporting beams when these are subjected to gravity loads and seismic actions. Cast-in-place slabs are expected to function also as beam flanges. Therefore during ductile frame response significant inelastic tensile strains parallel to beams may develop, particularly where beam plastic hinges are formed (see Figure C9.13 and Figure C9.1).

Under such conditions the floor system between beams may need to sustain a tension rather than a compression field. Hence adequate reinforcement in the topping must be provided to transfer tension forces across discontinuities caused by inelastic deformations in the supporting beams that may act as compression members. Suitable detailing of this reinforcement should ensure that forces at node points of appropriate strut-and-tie models can be effectively transferred. Figure C10.2 shows typical bars, placed at approximately the mid-depth of the topping slab, at exterior columns. To ensure adequate anchorage, these bars should extend beyond the centre of a column by at least a length equal to one-quarter of the diagonal distance between adjacent columns or the intersection of orthogonally arranged beams around the edges of a slab panel.

C13.4 Additional design requirements for elements designed for ductility in earthquakes

Unless the diaphragm shear strength of a floor slab is significantly reduced, for example by joints or large openings, earthquake induced diaphragm forces will seldom be critical. Transfer diaphragms, however, may be subjected to large in-plane shear forces and these may necessitate the increase of diaphragm thickness over that required by gravity load requirements. Special attention needs to be given to diaphragm action at any level when, instead of cast-in-place reinforced concrete floor slabs, precast concrete elements are used. To ensure the predictable interaction of vertical lateral force-resisting elements, energy dissipation in diaphragms should be suppressed unless special studies are made. As a general rule diaphragms should not be required to dissipate seismic energy[13.3].

C13.4.3 *Diaphragms incorporating precast concrete elements*

Because in-plane diaphragm actions should be within the elastic domain, special studies are required when in exceptional cases ductile diaphragm response needs to be relied on. Where inelastic action is expected in the diaphragm the topping concrete is likely to be cracked and there is potential for delamination of the topping and precast floor unit. The consequences of this shall be determined and if the flooring system relies on composite action to support gravity loads then mechanical connectors shall be provided across the interface.

REFERENCES

13.1 Bertero, V.V., Popov, E.P., Wang, T.Y. and Vallenas, J., “Seismic Design Implications of Hysteretic Behaviour of Reinforced Concrete Structural Walls”, 6th World Conference on Earthquake Engineering, 1977, Preprints 5, pp. 159-165.

13.2 Kolston, D. and Buchanan, B.W., “Diaphragms in Seismic Resistant Buildings”, Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2, 1980, pp.162-170.

13.3 Paulay, T. and Priestley, M.J.N., “Seismic Design of Reinforced Concrete and Masonry Buildings”, John Wiley and Sons, 1992, p. 767.

13.4 Bull, D.K., “Understanding the Complexities of Designing Diaphragms in Buildings for Earthquakes,” Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 37, No. 2, June 2002.

13.5 “Guidelines for the Use of Structural Precast Concrete in Buildings”, Report of the Study Group of the New Zealand Concrete Society and the New Zealand Society for Earthquake Engineering, Centre of Advanced Engineering, University of Canterbury, 2000, 136 pp.

13.6 Schlaich, J., Schaefer, K. and Jennewein, M., “Towards a Consistent Design of Structural Concrete”, Journal of the Prestressed Concrete Institute, Vol. 32, No. 3, May-June 1987, pp. 74-150.

13.7 Seismology Committee, Structural Engineers Association of California, “Recommended Lateral Force Requirements and Tentative Commentary”, 1988, p. 123.

13.8 Federation International de la Précontrainte (FIP) Recommendations, “Precast Prestressed Hollow Core Floors”, Thomas Telford, London, 1988, p. 48.

NOTES

C14 FOOTINGS, PILES AND PILE CAPS

C14.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 14 of the Standard:

b_o perimeter of critical section for slabs and foundations, mm

q_s soil bearing pressure as determined from the ultimate limit state loads, kPa

A3

C14.2 Scope

This section documents provisions which apply to isolated foundations supporting a single column or wall. However, most of the provisions are generally applicable to combined foundation and raft systems supporting several columns or walls or a combination thereof. Reference to piles is generally limited towards establishing ductility requirements, as generally it is important to ensure these elements can sustain intentional and unintentional post-elastic flexural actions at critical locations. Basic pile design philosophy may be extracted from References 14.1, 14.2, 14.3, 14.4 and 14.5.

C14.3 General principles and requirements

C14.3.4 *Shear in footings*

C14.3.4.1 *General and* **C14.3.4.2** *Spread footings and footing supported by piles*

The shear strength of foundations must be determined for the more severe condition of 12.7.1(a) or (b).The critical section for shear is "measured" from the face of the supported member (column, pedestal or wall), except for supported members on steel base plates.

Clause 12.7.1(a) considers the foundation essentially as a wide beam with a critical section (potential diagonal crack) extending in a plane across the entire width. This case is analogous to a conventional beam, and the design proceeds accordingly.

Clause 12.7.1(b) assumes two-way action, with a critical section (potential cracking) along the surface of a truncated cone or pyramid. The critical section of this case is taken at a distance $d/2$ from the perimeter of the column, pier, pile or other concentrated load.

Computation of shear requires that the soil bearing pressure q_s be obtained from ultimate limit state loads and the design be in accordance with the appropriate equation of Section 7.

Where necessary, shear around individual piles may be investigated in accordance with 12.7.1(b). If shear perimeters overlap, the critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeter which will actually resist the critical shear for the group under consideration. One such situation is illustrated in Figure C14.1.

A3

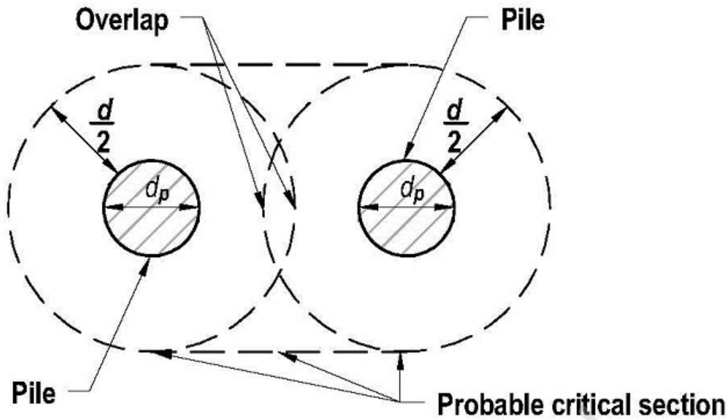


Figure C14.1 – Modified critical section for perimeter shear with overlapping critical perimeters

C14.3.4.3 *Shear in pile caps*

A3 Pile influence on the critical shear section is illustrated for a two-way pile cap in Figure C14.2.

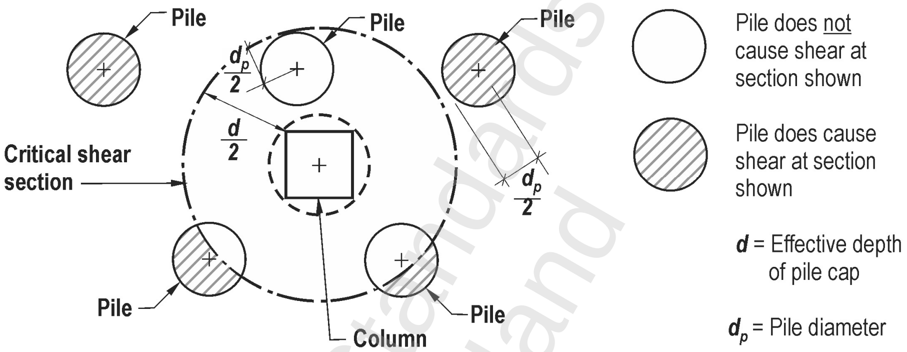


Figure C14.2 – Critical shear section for a two-way pile cap

Where piles are located inside the critical sections d (for one-way shear) or $d/2$ (for two-way shear) from face of column, analysis for shear in deep flexural members in accordance with 9.3.10 needs to be considered.

C14.3.6 *Piled foundations*

C14.3.6.3 *Details for upper ends of piles*

Because of the generally high moments and shears induced at the tops of piles, it is essential to provide adequate confining and shear reinforcement to ensure ductility.

For a cased pile the effect or contribution of the steel shell may be included with respect to confinement for the potential plastic hinge region. However, no such contribution from the shell shall be allowed for in nominal flexural strength calculations because of lack of compatibility of strains between concrete and steel unless special provisions are made to transfer the associated bond forces to the steel.

The presence of a steel pile casing can enhance the flexural capacity of the pile and allowance for this should be made either in the overstrength actions or by isolating the top of the casing so that it does not influence the flexural strength of the pile.

C14.3.6.5 *Minimum longitudinal reinforcement in reinforced concrete piles*

This clause is based on the equivalent requirements for columns and piers as specified in Section 8. However, it was felt that reduction of minimum reinforcement ratios was warranted for piles with a large cross-sectional area. It was considered that one-half of the minimum specified for columns with reinforcement having a lower characteristic strength of 300 MPa was acceptable for piles exceeding

2×10^6 mm^2 in cross-sectional area, with increasing ratios for piles of smaller area. Thus, for reinforcement having a lower characteristic strength of 300 MPa, the minimum reinforcement ratio for piles of cross-sectional area smaller than 0.5×10^6 is that specified for columns (2.4/300 = 0.008). In addition, concessions are made where reinforcement having a lower characteristic strength of 500 MPa is used. Equation 14–1 provides for an interpolation for the required minimum reinforcement ratio when the area of the pile lies between 0.5×10^6 mm^2 ($p_{t,\ min.} = 2.4/f_y$) and 2×10^6 mm^2 ($p_{t,\ min.} = 1.2/f_y$).

Attention should be given to piles that could be subjected to axial tension when under a severe earthquake the overstrength of the superstructure may be developed.

C14.3.6.9 *Piled foundations with permanent casing*

Piles deteriorate due to the action of mechanical, chemical and biological agencies and if an adequate service life is to be obtained from piles in aggressive conditions, a correct choice of material and its treatment are necessary.

The corrosion of steel piles is an electro-chemical phenomenon caused by potential gradients between adjacent areas of the steel surface. The steel corrodes at surfaces that are anodic to the soil and water, but is probably protected by a layer of hydrogen that is released at the cathodic surfaces. Differences in potential are caused by differences in the surface conditions of the steel and by variations in the electrolyte and the amount of oxygen in solution in the water at different points in the length of a pile. The temperature and the time of exposure also determine the amount of corrosion.

The incidence of corrosion of a steel pile that is completely embedded in the ground is largely dependent on the ease with which aerated ground water can reach the pile. Thus, it is small where the permeability of the soil is low, as in a clay, but may be important in a porous soil, such as a sand, where air is present in the pores down to ground water level and dissolved oxygen may be available for some distance below. The rates of corrosion shown by experiments vary from practically nil to about 0.075 mm per year, a commonly used (average) figure being 0.05 mm/year. It is common practice to make an allowance for loss of thickness by corrosion when calculating the thickness of steel required in the wall of a tube pile or in the web and flanges of an H pile. In normal conditions that are not regarded as corrosive, an increase of 1.5 mm in the thickness might be made.

For steel piles that are exposed to sea water and sea air as in the case of a jetty, the loss of steel would be least for that portion of the pile in the soil and greatest for the free standing portion. The corrosion of steel in sea water has been the subject of a number of experiments. In the tests by the Sea-Action Committee of the Institution of Civil Engineers (1920-38) the rate of loss at the surface of steel exposed to the sea water was found to vary from about 0.075 mm per year in temperate waters to about 0.175 mm per year in the tropics.

Provided due allowance has been made for corrosion with respect to the service life of the steel casing, the remaining area of steel shell may be considered as providing a portion of the required longitudinal reinforcing for non-seismic forces. See also C14.3.6.3.

C14.3.6.10 *Transverse reinforcement for confinement and lateral restraint of longitudinal bars*

As with members of superstructures carrying axial forces, there is a need to provide a minimum amount of transverse reinforcement to cater for the loss of cover, to maintain some confinement of the pile core concrete and to prevent buckling of the longitudinal reinforcement.

The length over which the transverse reinforcement to 10.3.10 is required at the upper end of every pile is the region deemed to be most at risk. The pile shall also comply with all the relevant requirements for designing for shear.

As described in C14.3.6.3, the steel shell of a cased pile may be assumed to contribute to transverse reinforcement providing allowance for corrosion has been made in accordance with 14.3.6.9.

C14.4 Additional design requirements for structures designed for earthquake effects

C14.4.1 *Designing for ductility*

C14.4.1.1 *General and* **C14.4.1.2** *Compliance with additional requirements*

The general philosophies of AS/NZS 1170 and NZS 1170.5 include the estimation of design forces acting on the foundation consistent with capacity design principles.

These clauses, are intended to impress upon designers that where energy dissipation is relied on it is essential that yielding be restricted to predictable locations and that such yielding can occur without serious damage. For further information and examples see Reference 14.6.

C14.4.2 *Pile caps*

Where piles caps are expected to absorb moments from a column that is being supported, possibly associated with a column plastic hinge at overstrength, the effect of the large moment gradient along the pile cap should be considered. This requires the treatment of the column-pile cap connection as a beam-column joint in accordance with the requirements of 15.4. This may necessitate the turning of 90° hooks at the bottom end of column bars into the joint region, similar to that shown in Figure C9.18, rather than outward into the pile cap. Joint shear reinforcement within the pile cap may also need to be provided.

The presence of the casing can enhance the flexural capacity of the pile and allowance should be made in the overstrength actions by isolating the top of the case so that it does not bear on the underside of the pile cap of the foundation. The flexural enhancement at overstrength can be ignored.

REFERENCES

14.1 Tomlinson, M.J., "Foundation Design and Construction", Longman Scientific and Industrial, 1986, p. 842.

14.2 Lam, I. Po. and Martin, G.R., "Seismic Design of Highway Bridge Foundations", Volumes 1 to 3, Report FHWA/BD-86/10 June 1986, p. 323.

14.3 Department of the Navy, "Soil Mechanics: Design Manual", NAVFAC DM 7.1, 1982, p. 364.

14.4 Taylor, P.W., "Code Provisions Related to Soils and Foundations", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 9, No. 1, March 1976, pp. 68-73.

14.5 Pender, M.J., "Aseismic Pile Foundation Design and Analysis", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 26, No. 1, Mar. 1993, pp 49-160.

14.6 Taylor, P.W. and Williams, R.L., "Foundations for Capacity Designed Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2, June 1979, pp. 101-108.

C15 DESIGN OF BEAM-COLUMN JOINTS

C15.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 15 of Part 1.

A_{s1}, A_{s2}	area of beam flexural reinforcement, mm^2
A_{sf}	area of reinforcement in effective tension flanges, mm^2
C_c'	concrete compression force in the flexural compression zone of a beam, N
C_s'	compression force in the compression reinforcement of a beam, N
L_1, L_2	span of beam between centre-to-centre of supports, mm
L_{1n}, L_{2n}	length of clear span of beam, measured face-to face of supports, mm
L_c, L_c'	height of column, centre-to-centre of floors or roof, mm
M_{o1}, M_{o2}	flexural overstrength of beam section at faces of column, N mm
p	ratio of non-prestressed tension reinforcement = A_s/bd
T, T'	tension force in tension reinforcement, N
V_c	nominal shear strength provided by concrete, N
V_{jx}^*, V_{jz}^*	design horizontal shear force across a joint in x and z directions, N
V_s	nominal shear strength provided by the shear reinforcement, N
$\phi_{o,fy}$	steel overstrength factor (2.6.5.6) A3
γ	steel compression stress factor

C15.2 Scope

Section 15 covers the design of beam-column joints. Clause 15.3 gives general principles applicable when gravity loads and wind forces are considered or adjacent members contain nominally ductile plastic regions. Clause 15.4 gives design requirements for structures containing limited ductile or ductile plastic regions.

Design of slab/column connections including provisions for shearhead reinforcement is covered in 12.7.5 and 12.7.6.

C15.2.2 *Alternative methods*

Alternative methods may be used providing they are based on rational analysis that has been validated against suitable test results. The approach proposed by Lin and Restrepo[15.9] based on strut and tie models shows much promise. A3

C15.3 General principles and design requirements for beam-column joints

C15.3.1 *Design criteria*

The basic requirements of a beam-column joint are that it must perform satisfactorily under loads at the serviceability limit state, that its strength should not govern the ultimate strength of the structure, and that its behaviour should not impede the development of the full strength of the adjoining members. Other important requirements are ease of construction and access for placing and compacting concrete. A3

The structural demand on joints is greatly dependent on the type of loading, and therefore design procedures appropriate to the severity of each type of loading are necessary. Where static gravity loading governs, strength under monotonic loading without stress reversals will be the design criterion. Seismic forces are more severe, because strength degradation in the joint may occur under repeated reversed actions, and a large amount of joint reinforcement may therefore be required.

C15.3.2 *Design forces*

The joint must be designed to resist the forces considered in designing the members and in those combinations producing the most severe force distribution at the joint. Forces produced by deformations resulting from time-dependent effects such as creep, shrinkage or settlement should be considered.

Forces in the joint should be determined by considering a free body of the joint with forces on the joint-member boundaries properly represented.

A3 Where nominally ductile plastic regions can form in the members connected by the joint, the design shear force should be calculated assuming that the longitudinal reinforcement in the plastic regions is at yield. Therefore, the design horizontal joint shear, if nominally ductile plastic regions were to form either side of the column would be:

$$V^*_{jn} = (A_{s1} + A_{s2})\, f_y - V_{col} \quad \text{(Eq. C15–1)}$$

where

$$V^*_{jh} \le 0.20\, f'_c b_j h_c \text{, or } 10 b_j h_c \quad \text{(Eq. C15–2)}$$

A3 and ϕ = 0.75 as the design forces are not based upon overstrengths. If unidirectional nominally ductile plastic regions form in the span of the connected beams, the actions on the joint should be determined based on actions at the boundaries of the joint that would develop if the reinforcement in the plastic regions was yielding.

C15.3.4 *Maximum horizontal joint shear force*

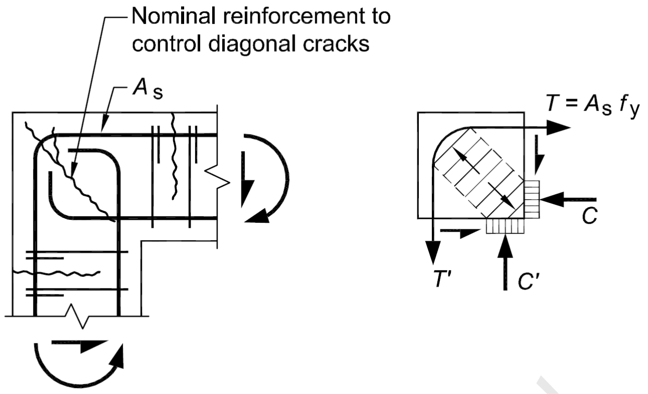
A limit is set on the maximum horizontal design shear force to ensure that diagonal compression failure does not occur.

A3 The effective joint width is illustrated in Figure C15.4.

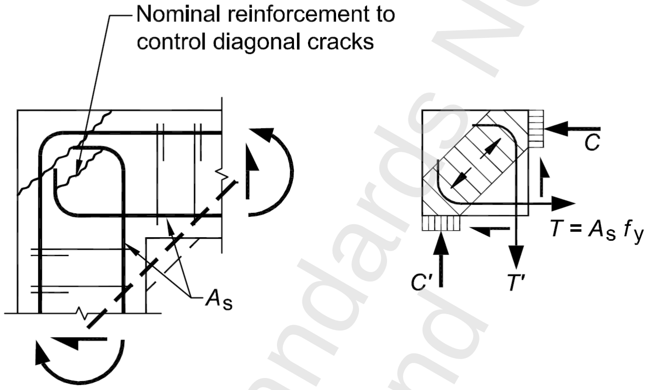
C15.3.5 *Design principles, mechanisms on shear resistance*

Joints subjected to non-seismic loading may be designed using the relevant principles of force equilibrium. A rational analysis may be used to show the extent to which a principal diagonal compression strut can carry a proportion of the joint shear, the remainder being carried by horizontal and vertical or diagonal joint shear reinforcement. Equations 15–1 or 15–2 may be used to evaluate the contributions.

The corner joint of a portal frame is a common example that will not necessarily require other than nominal orthogonal reinforcement. Recommendations for design and detailing are given in Reference 15.1. Design requirements for a knee joint will differ for a moment that tends to close the right angle and for a moment that tends to open it as indicated in Figure C15.1.



(a) Closing loads



(b) Opening corners A2

Figure C15.1– Typical forces at a knee joint of small members

For adequate strength under "closing" moments, knee joints of small members, slabs and walls in particular, are considered to require tension reinforcement continuous around the corner with sufficient radius to prevent bearing or splitting failure, and the amount of tension reinforcement (conservatively) limited to $p < 0.5\sqrt{f'_c}\,/f_y$. When using larger structural members having substantial reinforcing content, secondary reinforcement is required to preserve the integrity of the concrete within the joint by controlling splitting cracks and by providing confinement for the inner corner. A right angle corner joint is more severely affected when the applied moments tend to "open" the angle. Compression forces near the outer corner tend to "push off" the triangular corner portion of the joint. The use of secondary reinforcement to resist diagonal tension cannot be avoided in structural members of major frames, a recommended solution being to provide radial hoops to resist the whole of the diagonal tension across the corner[15.2] . A2

Joints with small members introducing "opening" moments may not develop the full strength of the adjacent members. However, performance can be improved by the addition of a fillet and some diagonal reinforcement, as shown by dashed lines in (b) Opening corners in (b) Opening corners A2

Figure C15.1, to ensure that the critical sections in the adjoining members are sufficiently removed for the joint [15.1].

C15.3.6 *Horizontal joint shear reinforcement*

These provisions apply to the behaviour of joints where significant ductility is not expected to occur adjacent to the joint. Such a joint is therefore not subject to yield incursion along beam bars passing through the joint, nor to degradation under repeated inelastic load cycles. These provisions make due allowance for the considerable contribution of the diagonal compressive strut in the concrete to joint shear transfer. The allowable proportion of the joint shear resisted by joint shear reinforcement reduces with axial load on the column. The factor C_j is introduced to allocate the effect of axial compression to the two A3 principal horizontal directions x and z of a space frame where a joint is required to transfer joint shears V_{jx} and V_{jz} concurrently in each direction. For unidirectional joint loading C_j is unity.

A3 **C15.3.6.2** *Area of horizontal joint shear reinforcement*

When the beam reinforcement in an external beam-column joint terminates short of the vertical column reinforcement on the far side of the joint zone, concentrated shear and tensile stresses can be induced. To balance the joint forces a high shear stress acts between the vertical leg of the hooked bars and the adjacent column bars. These shear stresses may induce principal tensile stresses leading to premature failure. Joint zone steel helps sustain the shear transfer.

C15.3.7 *Vertical joint shear reinforcement*

To sustain a diagonal compression field by a truss mechanism, vertical joint shear reinforcement is also required. This can be computed in the same way as the amount of horizontal joint shear reinforcement.

C15.3.8 *Confinement*

The minimum transverse reinforcement required in the joint is the same as the confinement reinforcement specified for the column ends immediately above or below the joint, except that where the joint is confined by elastic beams on all four sides these requirements may be relaxed.

C15.4 Additional design requirements for beam-column joints with ductile, including limited ductile, members adjacent to the joint

C15.4.1 *General*

Provisions are made for beam-column joints that are subjected to forces consistent with lateral loading on frames causing inelastic displacements. Particularly severe conditions can arise with respect to shear strength and anchorage of the reinforcement passing through or terminating in a joint when plastic regions form at the face of the joint. The basic requirement of the design is that joints must be somewhat stronger than adjacent hinging members, which are normally the beams. Because shear strength and the anchorage of the reinforcement controls joint design, energy dissipation within the joint core is undesirable. It can lead to rapid loss of strength under seismic load conditions and is therefore to be avoided.

Joints, different from those occurring in building frames, may be encountered in bridges, for example where circular piers need to develop continuity with cap beams or pile caps. These will require rational analysis using strut-and-tie models, or equivalent [15.3], to demonstrate the applicability of an admissible load path for internal forces.

C15.4.2 *Design forces*

C15.4.2.1 *Forces acting on beam-column joint*

To ensure that a joint possesses adequate reserve strength, the flexural overstrength of the adjacent beams and the corresponding internal forces must be evaluated. The simultaneous forces in the column that maintain joint equilibrium must also be determined. These must correspond with plastic hinges in the beams that may form either at the column face or at a distance away from the column where the beam overstrengths are developed. In frames where inelastic inter-storey displacements can occur in both principal directions, generally at right angles to each other, development of beam overstrengths from both of those directions should be considered separately[15.4, 15.5]. Where stiff structural systems, such as walls, preclude the possibility of yielding in beams and columns in one or both principal directions of the building, a rational analysis must show that the elastic joint possesses adequate strength.

The same procedure applies to one or two storey frames or the top storey of multi-storey frames where columns may be designed to develop plastic hinges. For the purpose of joint design, the role of beams and columns is simply reversed in such cases and the relevant clauses should be applied in a rational manner.

C15.4.2.2 *Horizontal design shear force*

For the purpose of evaluating the forces within a joint, such as shown in Figure C15.2, the stress resultants in the adjacent beams, normally at the development of overstrengths of the members, may be used. With reference to Figure C15.3 the horizontal design shear force V^*_{jh} across a typical interior joint is:

$$V^*_{jh} = T + C_c' + C_s' - V_{col} \qquad \text{(Eq. C15–3)}$$

For conventionally reinforced concrete members this simplifies to:

$$V^*_{ojh} = \phi_{o,fy}\, f_y\,(A_{s1} + A_{s2}) - V_{col} \qquad \text{(Eq. C15–4)}$$ A3

where $\phi_{o,fy}$ is 1.35 for Grade 300 and 500 Reinforcements.

Similar expressions are obtained for external joints where only one beam frames into a column.

The value of the column shear V_{col} will depend on the column moment gradients above and below the joint. However, from Figure C15.2 and Figure C15.3 its value may be estimated using a mean moment gradient, thus:

$$V_{col} = \frac{2\left(\frac{L_1}{L_{1n}} M_{o1} + \frac{L_2}{L_{2n}} M_{02}\right)}{L_c + L_c'} \qquad \text{(Eq. C15–5)}$$

When necessary the value of the vertical design joint shear force, V^*_{jv}, may be derived from similar considerations. Alternatively, the vertical joint shear force may be approximated as follows:

$$V^*_{jv} = V^*_{vh} \frac{h_b}{h_c} \qquad \text{(Eq. C15–6)}$$

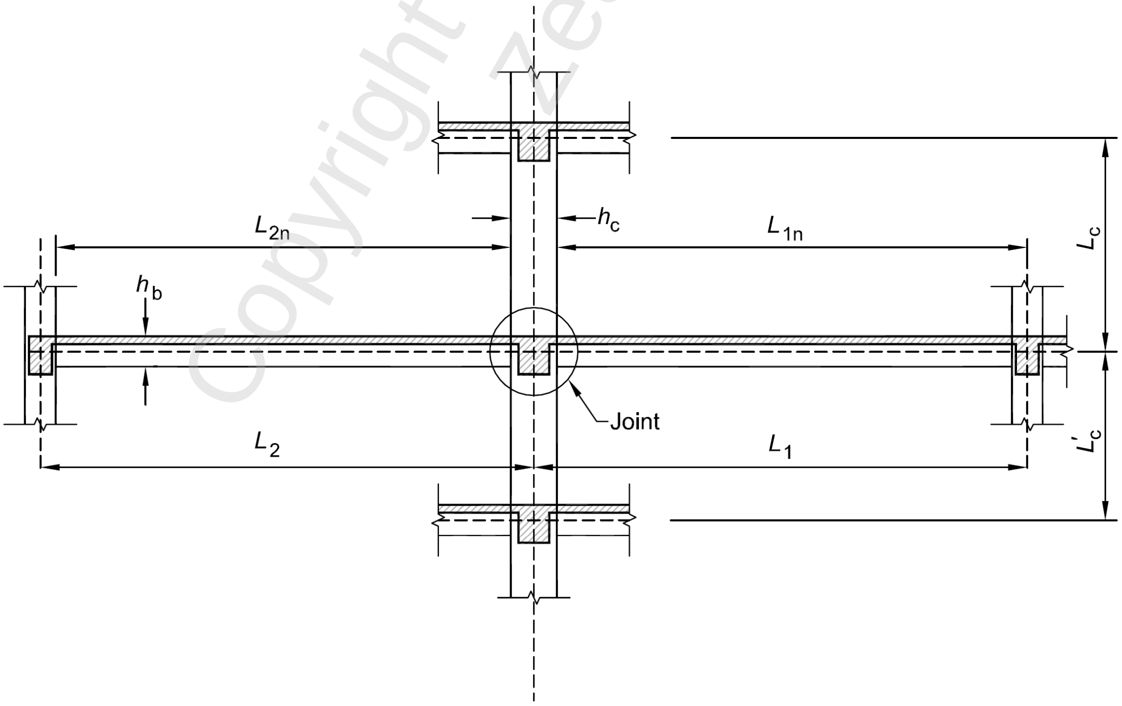


Figure C15.2 – An interior beam-column joint

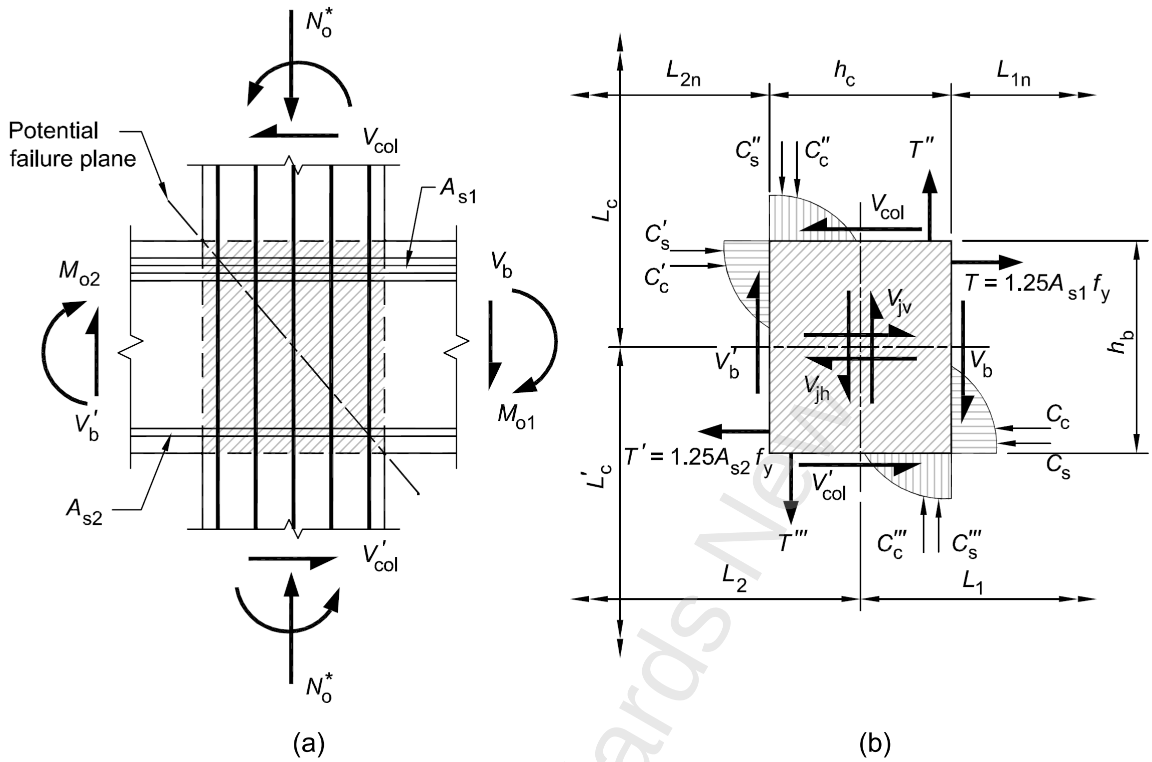


Figure C15.3 – External actions and internal forces of a typical interior beam-column joint

C15.4.3 *Design assumptions*

C15.4.3.1 *The role of shear reinforcement*

The observed failure plane due to shear in joints of one-way frames bisects the joint along a diagonal from one beam-column edge to another. The reinforcement provided must ensure that the shear force responsible for this failure plane is transmitted at most with restricted yielding of the reinforcement[15.6]. In accordance with 2.3.2.2, where joint shear forces are derived from overstrength member input, ϕ may be taken as 1.0. The anchorage of beam or column bars, particularly those that are expected to yield at the joint face, is critical. For this reason average bond stresses at interior joints must correspond to the requirements of 9.4.1.6.

C15.4.3.2 *Maximum horizontal design shear force*

An upper limit on the nominal shear stress across the effective joint area is specified to safeguard the core concrete against excessive diagonal compression stresses. The horizontal nominal shear stress corresponding with the critical horizontal design shear force, is based on the nominal gross horizontal area of the joint, $b_j h_c$, as defined in Figure C15.4 (a) and (b).

The internal joint actions to be considered when calculating the maximum horizontal design shear force, V_{ojh}^*, are associated with the development of plastic hinges in the beams either at or some distance away
A3 from the vertical face of the joint. The factor $\phi_{o,fy}$ in Equation C15–4 indicates that the steel stress corresponding with steel overstrength is considered. When a plastic hinge does not form at this section, the computed tension stress may be used in place of $\phi_{o,fy}$.

C15.4.3.3 *Determination of shear resistance of joint*

The assessment of the shear strength of beam-column joints should be based on the contribution of two generally recognised mechanisms; one consisting of a single diagonal concrete strut assumed to be capable of transferring both horizontal and vertical joint shear forces without the aid of reinforcement, the other a truss mechanism, utilising horizontal and vertical joint shear reinforcement. Shear reinforcement must be adequately anchored at, or beyond, or in the immediate vicinity of the joint core so as to enable a diagonal concrete compression field to be sustained[15.1, 15.5, 15.6]. The estimation of the contribution of a single diagonal concrete strut to, and the beneficial effect of the minimum axial column compression load on the joint shear mechanism, allows the required joint shear reinforcement to be determined.

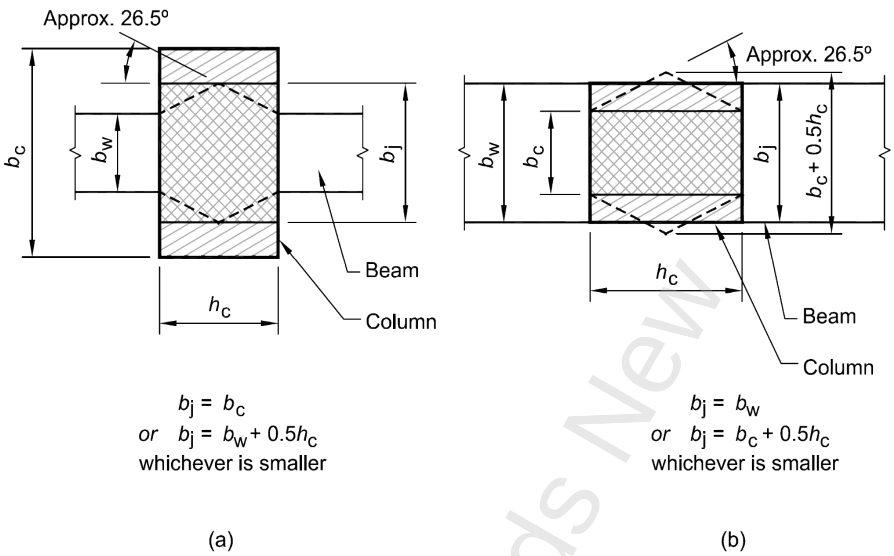


Figure C15.4 – Effective joint areas

A2

Research[15.7, 15.8, 15.9, 15.10] has shown that the amount of horizontal joint shear reinforcement in accordance with NZS 3101:1982 can be reduced. In particular a study[15.7] of the influence of steel and concrete tensile strengths and the ratio β of the compression to tension reinforcement contents in beams on bond strength within a joint core, has indicated that a proportion of the total joint shear force, larger than previously assumed may be assigned to the single diagonal concrete strut. Therefore in NZS 3101:1995 and the current standard less joint shear reinforcement is specified than in NZS 3101:1982. The current Standard is based on NZS 3101:1995, but the design equations have been re-written on a "V_c + V_s" basis (in 15.3.6.1) to make them more understandable. A3

Also other forms of joint reinforcement, when shown to be as effective as horizontal hoops, ties or spirals may be used [15.6, 15.11].

C15.4.3.4 *Horizontal joint shear reinforcement*

The reinforcement required in the joint and the general detailing requirements are provided in 15.4.4 to 15.4.9 for situations where plastic hinges are expected to form in the beams adjacent to the column face. Where the beams are detailed to ensure that plastic hinges are forced away from the column faces, the joint shear forces are calculated based on the overstrength capacity of the beam plastic hinges, but the joint reinforcement is determined in accordance with section 15.3. The reasons for this is that by preventing yielding at the column face, the bond conditions through the joint and the cracking of the joint are likely to be similar to that of joints where nominal ductile plastic regions form on either side of the joint.

C15.4.3.5 *Placement of shear reinforcement*

As required by 15.3.4, for a joint with a wide column, only a part of the column width should be considered as being effective, as shown in Figure C15.4 for a one-way frame. Any horizontal and vertical reinforcement that is present in the column but is placed outside the effective joint area should not be considered to contribute to the joint shear strength.

C15.4.4 *Horizontal joint shear reinforcement*

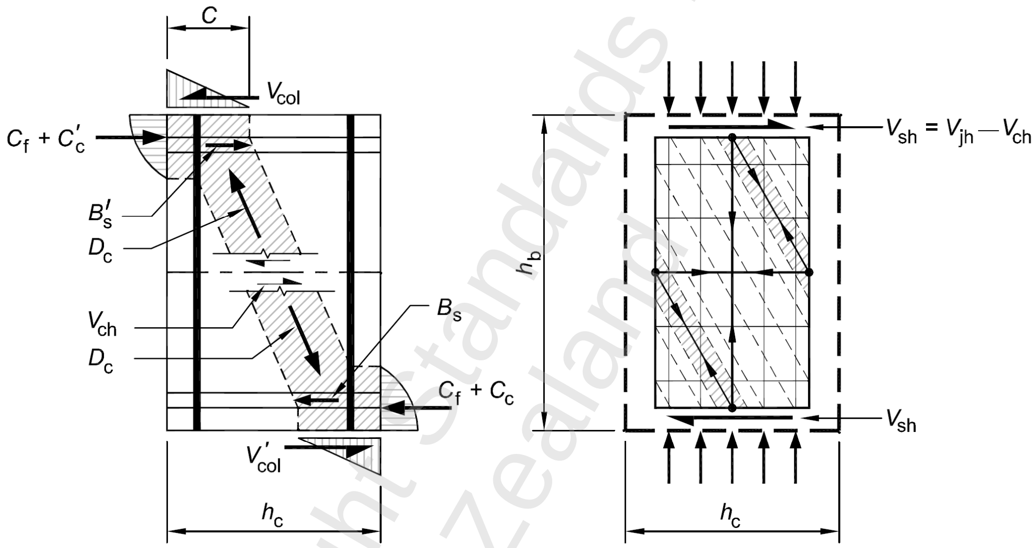
C15.4.4.1 *Area of horizontal joint shear reinforcement*

The models on which the provisions for joint shear resistance are based are shown in Figure C15.5. The horizontal design joint shear force is assigned to a mechanism transferring shear by means of a single diagonal concrete strut, V_{ch}, (Figure C15.5(a)) and a diagonal compression field sustained by horizontal and vertical joint shear reinforcement, that is a truss mechanism, V_{sh} (Figure C15.5(b)), so that:

$$V_{jh} = V_{ch} + V_{sh} \qquad \text{(Eq. C15–7)}$$

Analytical research[15.5, 15.8] in agreement with experimental findings enabled estimations to be made for the contribution of the diagonal strut V_{ch}. Some details of this are given below.

Figure C15.3 shows all the internal beam and column forces which enable the total horizontal design joint shear force, V_{jh}, given by Equation C15–3 to be determined. Subsequently the horizontal design joint shear force is checked to ensure that it does not exceed the smaller of $0.2f'_c\, b_j h_c$ or $10\, b_j h_c$.



(a) Diagonal concrete strut (b) Truss mechanism

Figure C15.5 – Models of the transfer of horizontal joint shear forces

When part of the top beam tension reinforcement, shown as A_{s1} in Figure C15.3(b), is distributed within effective tension flanges of T- or L- beams in accordance with 9.4.1.6, Figure C15.3(b) does not properly represent the forces that are introduced to the beam-column joint. Reinforcement in tension flanges, that is those bars with area A_{sf} placed outside the effective width, b_j, of the joint cannot directly transmit the tension force $T_f = 1.25\, f_y A_{sf}$ to the joint core. Instead, by means of diagonal compression forces from the anchorage regions of the slab bars, as shown in Figure C9.1, concrete compression forces, $C_f = T_f$, are introduced in the relevant flexural compression regions of the beams [15.5, 15.8] as shown in Figure C15.5(a). Thereby a moment, additional to that developed in the rectangular beam sections, being equal to the contribution of tension flanges, is introduced to the joint. With this modification, Equation C15–3 can be rewritten in terms of the forces that are introduced to the beam-column joint thus:

$$V^*_{jh} = (T - T_f) + C_f + C'_c + C'_s - V_{col} \qquad \text{(Eq. C15–8)}$$

A relatively small, but not negligible, fraction of the combined tension and compression forces $T - T_f + C'_s$, introduced by that portion of the top beam reinforcement which is anchored in the joint core by means of bond, is defined as B'_s. By estimating the probable limits of each force component, an expression incorporating all relevant parameters can be derived.

Figure C15.5 (a) illustrates that:

$$V_{ch} = C_f + C_c' + B_s' - V_{col} \quad \text{(Eq. C15–9)}$$

from which V_{ch} can be derived [15.5, 15.8] for interior joints as Equation C15–10 and for exterior joints as Equation C15–11.

$$V_{ch} = V_{jh}^* \left(1 - \frac{6\alpha_i f_y A_s^*}{f_c' \, b_j h_c} \right) \quad \text{(Eq. C15–10)}$$

$$V_{ch} = V_{jh}^* \left(1 - \frac{6\beta_i f_y A_s}{f_c' b_j h_c} \left\{ 0.7 - \frac{C_j N_o^*}{f_c' A_g} \right\} \right) \quad \text{(Eq. C15–11)}$$

The axial load in Equations 15–9 and 15–10 must be derived using capacity design principles. For convenience, particularly when the compression load on the column, N_o^*, is relatively small, at the designer's option, $N_o^* = 0$ may be used in Equations 15–9 and 15–10. In this case $\alpha_i = 1.4$ is applicable, in 15.4.4.1 (a). In rather exceptional cases, when as a result of earthquake actions and gravity loads, the axial load results in net tension, α_i as defined in $\alpha_i = \left(1.4 - 1.6 \frac{C_j N_o^*}{f_c' A_g} \right) \alpha_n$ is applicable . A3

The factor C_j is introduced to proportionally allocate the beneficial effects of axial compression load N_o^* to the two principal directions x and z of the lateral design forces when joint shear forces V_{jx}^* and V_{jz}^* are concurrently developed. For unidirectional joint forces C_j is unity and, for a symmetrical two-way frame $C_j = 0.5$ when the axial load on the column produces compression. For axial tension load, $C_j = 1.0$ must be assumed.

Irrespective of the benefits resulting from the use of various parameters, horizontal joint shear reinforcement must be provided to resist at least 40 % of the horizontal design joint shear force, V_{jh}^*.

Relaxation in the requirements for the design of beam-column joints in frames with limited ductility plastic regions or ductile plastic regions may be applied[15.5, 15.7] because :

(a) Where reduced inelastic steel strains occur, a lesser degree of deterioration within the joint core can be expected;

(b) With increased residual tensile strength of the concrete core, joint shear mechanisms, other than those relying on joint reinforcement, are likely to improve in comparison with those for ductile frames;

(c) With gravity load dominance, often encountered with these types of frames, plastic hinges involving the yielding of bottom beam reinforcement may not occur at column faces. Joint shear forces are therefore reduced;

(d) With the reduction of reversing inelastic strains along bars within a joint core, anchorage conditions can be expected to improve.

Beam-column joint response will be a function of the ductility demand arising in adjacent beams rather than on the structural system. If all members connected to the joint are of low ductility demand then the relaxations from the requirements of 15.4, for joint shear reinforcement, may be utilised.

It should be noted that for frame systems with hinging columns, the requirements for horizontal joint shear reinforcement, need to be interchanged with those of vertical joint shear reinforcement.

In interior joints the reduction of the quantity of horizontal joint reinforcement recognises the less severe demands made upon beam-column joints where LDPR form adjacent to the joint. In selecting α_i , factors such as the variety of forms of structural frames (aspect ratios, gravity loading etc.) were considered. The A2

A2 value of α_i is considered to represent typical or "average" expectations for beam-column joints of frames with limited ductile plastic regions. Such a reduction is supported by recent research[15.6, 15.7, 15.8, 15.14].

A significant relaxation of the requirements for quantities of shear reinforcement may be derived by acknowledging that the stress in the top bars in compression is significantly less than f_y. This is discussed in detail in Reference 15.10. An example is the potential plastic hinge region of a gravity-dominated beam. In this zone, where because of the geometry of the frame and the gravity load distribution, the resulting area of reinforcement in the top of the beam may be twice that of the reinforcement in the bottom of the beam. Because of gravity load dominance it is possible that the bottom reinforcement will not yield during seismic attack. Hence, the compressive stress in the top reinforcement with area A_s^* may be less than 0.5 f_y, i.e. $\gamma \leq 0.5$. This will result in a reduction in the required area of horizontal joint shear reinforcement, A_{jh}. The designer may investigate specific beam-column joints seeking a reduction in A_{jh} when congestion of reinforcement or other factors require it.

In exterior joints the development of the horizontal joint shear involves the transmission of tension forces in the reinforcement by bond and bearing on the inside of the standard hooks. In order for the joint shear mechanism to form, the longitudinal beam tension reinforcement and its associated hooks need to be appropriately sized and located within the joint[15.6, 15.12]. Research [15.8, 15.13, 15.14] indicates that even for low member ductility demands that considerable yield penetration and slippage within the joint, of beam longitudinal reinforcement could occur.

C15.4.4.2 *Prestressed beams*

Where prestressing is used with the anchorages placed outside the joint core, the horizontal joint shear reinforcement required in 15.4.4.1 may be reduced by an amount corresponding to a horizontal confining force of 0.7 P_{cs}. Prestressing steel that is present near the extreme fibres of the section must be assumed to have sustained permanent set strains and therefore to have lost its prestress after the formation of plastic hinges. However, prestressed steel at the central third of the beam depth may be considered to remain effective and the prestress force, P_{cs}, after all losses may be considered to replace an equivalent quantity of horizontal joint shear reinforcement.

A1 **C15.4.4.3** *Distribution of horizontal joint shear reinforcement*

Horizontal stirrup ties anchored around column bars that pass through the joint, may cross the potential diagonal joint shear failure plane at different angles depending on the shape of these ties. Therefore the direction of each tie relative to the direction of the horizontal joint shear force needs to be considered. Sets of horizontal ties placed in the close vicinity of beam bars contribute to joint shear strength with reduced efficiency. The space between the innermost layer of beam bars and the adjacent set of ties within the joint core should preferably not be less than one-half of the vertical spacing between sets of joint ties. Only those ties which are placed within the joint core defined in 15.3.4(a) and (b) and shown in Figure C15.4(b) should be considered to be effective in shear resistance.

A1 **C15.4.4.4** *Minimum horizontal joint reinforcement*

Because column bars passing through joint cores are expected to remain elastic when plastic hinges in beams develop, provision is made for some relaxation in the spacing requirements of ties or hoops within the joint core. Attention must be paid to column bars that are in the same plane as beams in one-way frames. These requirements also apply to circular columns. The distance between horizontal sets of ties and hoops placed immediately below and above the beam bars that enter the joint, must also comply with these requirements, unless a cast-in-place floor slab precludes the possibility of column bar bucking in such a region.

To enable column shear forces to be more efficiently introduced to the core of exterior beam-column joints, it is preferable to place the horizontal transverse reinforcement, both in the column and within the joint, as close to the beam reinforcement anchored in the joint as practicable.

C15.4.5 *Vertical joint shear reinforcement*

A2

C15.4.5.1 *Area of vertical joint reinforcement*

To sustain a diagonal compression field by a truss mechanism, vertical joint shear reinforcement is also required[15.1]. The design vertical joint shear force can be approximated, thus $V^*_{jv} = (h_b / h_c)\ V^*_{jh}$, and this is incorporated in Equation 15–12. The principal role of the vertical reinforcement in a joint, such as shown in Figure C15.3, is to enable a significant portion of the horizontal bond forces introduced by the beam reinforcement to the truss mechanism, shown as V_{sh} in Figure C15.5 to be resolved also into a diagonal compression force. The relevant fraction of the force $V_{sh} = A_{jh}\ f_{yh}$, as well as the average inclination of the diagonal struts were found[15.14] to depend on the stress ratio $V^*_{jh}/f'_c\ b_j h_c$. Because the required amount of vertical joint reinforcement is seldom critical, conservative assumptions were made in the development of Equations 15–12 and 15–13 in order to make it simple.

A joint at which hinging in the column rather than in the beam is expected, is an exception. In this case the vertical joint shear reinforcement should be designed on the same basis as the horizontal joint shear reinforcement for hinging beams. However, for these cases some judgement is required in the interpretation of 15.4.4.1. No experimental studies were available to provide guidance for the design of such joints.

C15.4.5.2 *Vertical joint shear reinforcement*

Intermediate vertical column bars with total area equal to or greater than A_{jv}, placed between the corner bars, as shown in Figure C15.3 (a), need to be provided[15.15]. These need not extend over the full length of a column but they must be adequately anchored in the column above and below the joint.

C15.4.5.3 *Spacing of vertical joint reinforcement*

The most expedient solution for the vertical joint reinforcement is to use existing column bars within the joint core. Such intermediate bars are not expected to be fully stressed due to column load alone. Equation 15–12 is based on the assumption that intermediate bars in columns designed in accordance with capacity design may be stressed at a joint to $0.25f_y$ in tension when no axial load is present. The effect of axial compression or tension on the usable strength of these bars within the joint is allowed for in Equation 15–12 by the parameter α_v. It is important that at least one bar, but for larger columns two or more intermediate vertical column bars, situated between corner bars, should pass through the joint, as shown in Figure C15.3(a). Therefore the column bar spacing in the relevant column faces should not exceed 200 mm. Generally it will be found that where intermediate column bars correspond to at least one-third of the total vertical reinforcement in the column, no additional vertical joint shear reinforcement is required.

C15.4.6 *Joints with wide columns and narrow beams*

Where, due to seismic actions, a narrow beam transmits moments to a wide column, it may be unsafe to assume that the longitudinal column reinforcement located away from the joint area will effectively participate in transferring moments between column and beam. Therefore the longitudinal column reinforcement which is required to interact at a particular level with a narrow beam should be placed within the effective joint width. The cross-shaded area of the column section, shown in Figure C15.4(a), should accommodate such column bars. To resist loads from floors above, or from beams framing into the column from the other direction, and to satisfy minimum requirements for the distribution of longitudinal reinforcement, in accordance with 10.3.8.1, longitudinal bars must also be placed outside the effective joint area, $b_j h_c$, such as shown in Figure C15.6(a).

An example of relevant reinforcing details is shown in Figure C15.6. The longitudinal and transverse reinforcement outside the effective joint area, $b_j h_c$, are to provide for torsional resistance where required and confinement.

C15.4.7 *Eccentric beam-column joints*

To avoid the necessity of having to estimate torsional effects in a column or a joint as a result of eccentric location of a beam which transfers earthquake induced moments, the effective joint width is artificially reduced and, as a concession, the normal design procedure for the joint and the column, as specified in the previous clauses, is allowed. It is considered that this restriction will allow sufficient reserve strength from outside the specified effective joint area of the column, to safely absorb torsional effects. However, some conservatism in design is warranted because the behaviour of eccentric joints is as yet not fully understood.

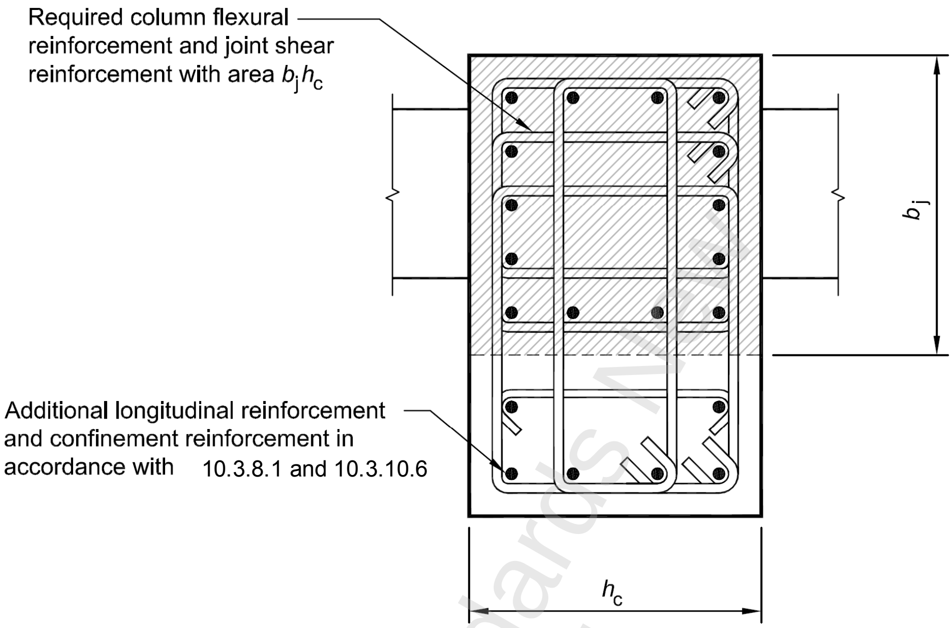


Figure C15.6 – Reinforcing details for joints with wide columns and narrow beams

REFERENCES

15.1 Fenwick, R. and Deam, B., “ Design of Opening in Walls and Slabs,” SESOC Journal, Vol. 16, No.1, April 2003, pp. 28-34.

15.2 Megget, L.M., “The Strength of Small Reinforced Concrete Beam – Column Knee Joints Under Opening Moments”, Proc. Australasian Structural Engineering Conference, Sydney, September 1994.

15.3 Priestley, M.J.N., “Assessment and Design of Joints for Single-level Bridges with Circular Columns”, Report No. SSRP-93/02, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, Feb. 1993, p. 69.

15.4 Blakeley, R.W.G., “Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames”, Section J, Design of Beam-column Joints, Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 4, Dec. 1977, pp. 226-237.

15.5 Paulay, T. and Priestley, M.J.N., “Seismic Design of Reinforced Concrete and Masonry Buildings”, John Wiley and Sons, 1992, p. 767.

15.6 Paulay, T., Park R. and Priestley, M.J.N., “Reinforced Concrete Beam-column Joints Under Seismic Actions”, Journal of ACI, Proceedings Vol. 75, No. 11, November, 1978, pp. 585-593.

15.7 Dai Ruitong and Park, R., “A Comparison of the Behaviour of Reinforced Concrete Beam-column Joints Designed for Ductility and Limited Ductility”, Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 21, No. 4, December 1988, pp. 255-278.

15.8 Cheung, P.C., Paulay, T. and Park, R., “Interior and Exterior Reinforced Concrete Beam-column Joints of A Prototype Two-way Frame with Floor Slab Designed for Earthquake Resistance”, Research Report 89/2, Department of Civil Engineering, University of Canterbury, March 1989, p. 75.

15.9 Lin Chen-Ming and Restrepo, J.I., “Seismic Behaviour and Design of Reinforced Concrete Interior Beams – Column Joints”, Bulletin of NZSEE, Vol. 36, No. 2, June 2002, pp. 108-128.

15.10 Park, R., "Some Considerations in the Design of Reinforced Concrete Interior Beam-column Joints of Moment Resisting Frames", Journal of Structural Engineering Society of New Zealand, Vol. 15, No. 2, September 2002, pp. 53-64.

15.11 Fenwick R.C. and Nguyen, H.T., "Reinforced Concrete Beam-column Joints for Seismic Loading", Report No. 220, Department of Civil Engineering, University of Auckland, 1981, p. 62.

15.12 Megget, L. and Fenwick, R., "Seismic Performance of External Reinforced Concrete Beam-column Joints," Bulletin of the NZ Society for Earthquake Engineering, Vol. 34, No. 4, December 2003, pp. 223-232.

15.13 Paulay, T. and Scarpas, A., "The Behaviour of Exterior Beam-column Joints", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 14, No. 3, September 1981, pp. 131-144.

15.14 Restrepo-Posada, J.I., Park, R. and Buchanan A.H., "Seismic Behaviour of Connections Between Precast Concrete Elements", Research Report No. 93-3, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, April 1993, p. 384.

15.15 Park, R. and Yeoh Sik Keong., "Tests on Structural Concrete Beam-column Joints with Intermediate Column Bars", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 3, September 1978, pp. 198-203.

Table C15.1– Design of reinforced beam-column joints

Table C15.1 Part A – Elastic and nominally ductile and ductile frames with beams forming plastic hinges at column face

	Elastic and nominally ductile		**Ductile frames with beams forming plastic hinges at column face**	
	Interior	**Exterior**	**Interior**	**Exterior**
Design forces	Refer 15.3.2	Same as interior joints	Refer 15.4.2.1	Same as interior joints
Maximum horizontal joint shear, V^*_{jh}	$0.20f'_c\, b_j h_c$ or or 10 $b_j h_c$ (15.3.4)	Same as interior joints	$0.20f'_c\, b_j h_c$ or or 10 $b_j h_c$ (15.3.4))	Same as interior joints
Horizontal concrete shear	$\phi V_{ch} = V^*_{jh}\left(0.5 + \frac{C_j N^*}{A_g f'_c}\right)$ (15.3.6.2)	Same as interior joints	$V_{ch} = V^*_{ojh}\left(1 - \frac{6\alpha_i f_y A^*_s}{f'_c b_j h_c}\right)$ but with the limitation that $0.85 \le \left[\frac{6V^*_{ojh}}{f'_c b_j h_c}\right] \le 1.20$ (15.4.4.1(a))	$V_{ch} = V^*_{ojh}\left(1 - \frac{6\beta f_y A_s}{f'_c b_j h_c}\left(0.7 - \frac{C_j N^*_o}{f'_c A_g}\right)\right)$ but with the limitation that $0.85 \le \left[\frac{6V^*_{ojh}}{f'_c b_j h_c}\right] \le 1.20$ (15.4.4.1(b))
Vertical concrete shear	$\phi V_{cv} = 0.6\, V^*_{jh}\frac{h_b}{h_c} + C_j N^*$ (15.3.7.2)	Same as interior joints	$V_{cv} = \frac{h_b}{h_c}\left(V^*_{jh} - \alpha_v A_{jh} f_{yh}\right)$ (15.4.5.1)	Same as interior joints
Minimum horizontal joint reinforcement	Greater of that required for confinement or restraint of bars in the adjacent column. Where beams frame into all four faces of the joint the required reinforcement may be halved (15.3.8)	Same as interior joints A1	Greater of that required for confinement or restraint of bars in the adjacent column (15.4.4.4) but equal to, or greater than $0.4V^*_{jh}/f_{yh}$ (15.4.4.1(c))	Same as interior joints
Maximum spacing of horizontal reinforcement	Lesser of, 10 times the smallest column bar diameter or 200 mm (15.3.8)	Same as interior joints A1	Lesser of, 10 times the smallest column bar diameter of 200 mm (15.4.4.4)	Same as interior joints
Spacing of vertical joint shear reinforcement	No requirement	Same as interior joints	Spacing shall not exceed $h_c/4$ or 200 mm (15.4.5.3)	Same as interior joints

Table C15.1– Design of reinforced beam-column joints (Continued)

	Elastic and nominally ductile		Ductile frames with beams forming plastic hinges at column face	
	Interior	**Exterior**	**Interior**	**Exterior**
Maximum beam bar diameters passing through column	Refer 9.3.8.4	Not applicable	Refer 9.4.3.5	Not applicable
Anchorage of hooked beam bars in columns considered to commence at-	Not applicable	The face of the column	Not applicable	Anchorage is deemed to commence lesser of half column depth or $8d_b$ (9.4.3.2.1)
Maximum column bar diameters passing through beam			$\frac{d_b}{h_b} \leq 3.2\frac{\sqrt{f_c'}}{f_y}$ (10.4.6.6(a)) or where there is a high degree of protection against formation of column plastic hinges $\frac{d_b}{h_b} \leq 4.0\frac{\sqrt{f_c'}}{f_y}$ (10.4.6.6(b))	Same as interior joints

Table C15.1 Part B – Ductile frames with beams forming plastic hinges away from the column face and with columns forming plastic hinges at the beam face

	Ductile frames with beams forming plastic hinges away from column face		Ductile frames with column forming plastic hinges at the beam face	
	Interior	**Exterior**	**Interior**	**Exterior**
Design forces	Refer 15.4.2.1	Same as interior joints	Refer 15.4.2.1	Same as interior joints
Maximum horizontal joint shear, V_{jh}^*	$0.20\, f_c'\, b_j h_c$ (15.4.3.2)	Same as interior joints	$0.20\, f_c'\, b_j h_c$ or $10\, b_j h_c$ (15.4.3.2)	Same as interior joints
Horizontal concrete shear	$\phi V_{ch} = V_{ojh}^* \left(0.5 + \frac{C_j N_o^*}{A_g f_c'}\right)$ where $\phi = 1.0$ as based on overstrengths (15.4.3.4)	Same as interior joints	$V_{cv} = \frac{h_b}{h_c}\left(V_{jh}^* - \alpha_v A_{jn} f_{yh}\right)$ Developed by interchanging 15–9 and 15–12 as per 15.4.3.3	Same as interior joints

Table C15.1– Design of reinforced beam-column joints (Continued)

	Ductile frames with beams forming plastic hinges away from column face		**Ductile frames with column forming plastic hinges at the beam face**	
	Interior	**Exterior**	**Interior**	**Exterior**
Vertical concrete shear	$V_{cv} = \frac{h_b}{h_c}\left(V_{jh}^{*} - \alpha_v A_{jh} f_{yh}\right)$ (15.4.5.1)	Same as interior joints	$V_{cv} = V_{jv}^{*}\left(1 - \frac{6\alpha_i f_y A_s^{*}}{f_c' b_j h_c}\right)$ Developed by interchanging 15–9 and 15–12 as per 15.4.3.3	Same as interior joints
Minimum horizontal joint reinforcement	Greater of that required for confinement or restraint of bars in the adjacent column, but can be halved when beam frames in on all four forces of joint (15.3.8)	Same as interior joints A1	Greater of that required for confinement or restraint of bars in the adjacent column (15.4.4.4)	Same as interior joints
Maximum spacing of horizontal reinforcement	Lesser of, 10 times the smallest column bar diameter of 200 mm (15.3.8)	Same as interior joints A1	Lesser of, 10 times the smallest column bar diameter of 200 mm (15.4.4.4)	Same as interior joints
Spacing of vertical joint shear reinforcement	Spacing shall not exceed $h_c/4$ or 200 mm (15.4.5.3)	Same as interior joints	Spacing shall not exceed $h_c/4$ or 200 mm (15.4.5.3)	Same as interior joints
Maximum beam bar diameters passing through column	9.3.8.4	Not applicable	9.3.8.4	Not applicable
Anchorage of hooked beam bars in columns considered to commence at:	Not applicable	The face of the column (9.4.3.2)	Not applicable	The face of the column
Maximum column bar diameters passing through beam	$\frac{d_b}{h_b} \le 3.2\frac{\sqrt{f_c'}}{f_y}$ (10.4.6.6(a)) or where there is a high degree of protection against formation of column plastic hinges $\frac{d_b}{h_b} \le 4.0\frac{\sqrt{f_c'}}{f_y}$ (10.4.6.6(b))	Same as interior joints	$\frac{d_b}{h_b} \le 3.2\frac{\sqrt{f_c'}}{f_y}$ (10.4.6.6(a))	Same as interior joints

C16 BEARING STRENGTH, BRACKETS AND CORBELS

C16.3 Bearing strength

C16.3.1 *General*

This section deals with bearing strength on concrete supports. The strength reduction factor used previously has been modified from 0.65 and 0.85 to 0.75 for both confined and unconfined concrete and a corresponding reduction has been made in bearing stress levels so that the design strengths are not changed. This change reduces potential complications in the design of bearing support in checking equilibrium in that all forces now have the same strength reduction factor. The permissible bearing stresses are based on tests reported in References 16.1 and 16.2. A3

When the supporting area is wider than the loaded area on all sides, the surrounding concrete combined with the stress dispersion confines the bearing area, A_1, resulting in an increase in bearing strength[16.1, 16.2, 16.6]. The minimum depth of support will be controlled by the shear requirements of 7.7.

When the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle. Figure C16.1 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support. Such a load path would have steeper sides. However, the frustum described has somewhat flat side slopes to ensure that there is concrete immediately surrounding the zone of high stress at the bearing. A_1 is the loaded area but not greater than the bearing plate or bearing cross-sectional area.

C16.3.2 *Exclusions*

Where confinement is provided by reinforcement, or some other means, there is a significant increase in bearing strength [16.1, 16.2]. In determining the location of confinement reinforcement allowance should be A3
made for the loss of confined area due to arching between hoops or spirals, or between longitudinal bars when rectangular ties are used [16.3, 16.4]. Generally post-tension systems have standard spirals provided A3
with the anchors to provide the required confinement, which have been proved through tests and use to meet the requirements of 16.3.2(a).

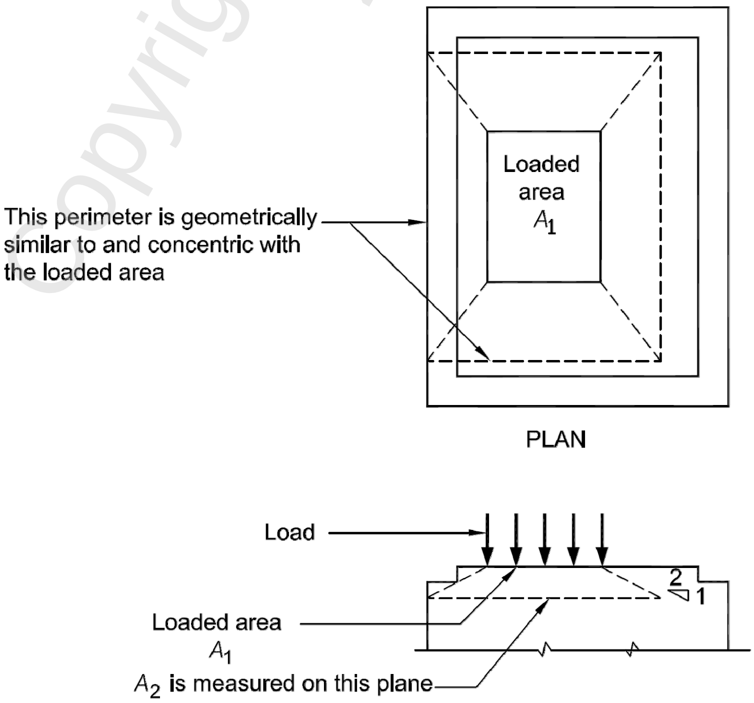


Figure C16.1 – Application of frustum to find A_2 in stepped or sloped supports

C16.4 Design of brackets and corbels

C16.4.1 *Critical section for flexure*

A3

Where a corbel or bracket is supported by a column or wall the critical section may be taken at the face of the wall of column, see Figure C16.2. Where a corbel springs from a beam the load on the corbel induces flexure, shear and torsion in the beam, and as illustrated in Figure C16.3 the critical section moves to the centre-line of reinforcement (vertical stirrups and/or ties, typically) closest to the springing of the corbel. This reinforcement has to resist the vertical load from the corbel as well as additional tension force due to the torsion applied to the beam.

C16.4.3 *Bearing area and bearing stresses*

C16.4.3.1 *Bearing stresses*

The restrictions on the location of the bearing area are necessary, first to ensure development of the yield strength of the reinforcement can occur near the load point, and second to allow for loss of concrete due to spalling from the back face of supported concrete members and the front face of the corbel[16.10 and 16.11], see Figure C16.2. This can arise due to the relative rotation between a supported member and a corbel, which can induce high localised indeterminate reactions, particularly where there is reinforced topping concrete above precast components that can restrain the vertical movement caused by the rotation, see Figure C16.4A. Allowance for potential spalling is provided in 16.3.4.

C16.4.3.2 *Development of primary reinforcement*

The vertical loading on the corbel or bracket is resisted by an inclined compression force (see Figure C16.3). To satisfy equilibrium the tension force over the length of the shear span is constant. Consequently it is necessary that the primary reinforcement is capable of sustaining its tension force between the critical section and the point below the centroid of the vertical force, V^*.

C16.4.4 *Method of design*

A3

Brackets and corbels are cantilevers having shear span-to-depth ratios equal to or less than unity, may be designed following the steps set out in 16.5 provided the lateral force, N_c^* is less than V^*. Where span to depth ratios are less than 1.5 the design may be based on strut-and-tie models. With greater span to depth ratios the members should be designed as for a beam following the appropriate requirements of Sections 7 and 9. The limits on the empirical method are due to the limited range of tests on which this approach is based.

The corbel shown in Figure C16.2 may fail by yielding of the primary tension reinforcement, crushing or splitting of the compression strut, localised bearing between the supported member and corbel, or by a sliding shearing failure at the critical section or the interface with the beam or wall. These failure modes are illustrated and are discussed more fully in Reference 16.5. The notation used in 16.4 and 16.5 is illustrated in Figures C16.2 and C16.3.

A3

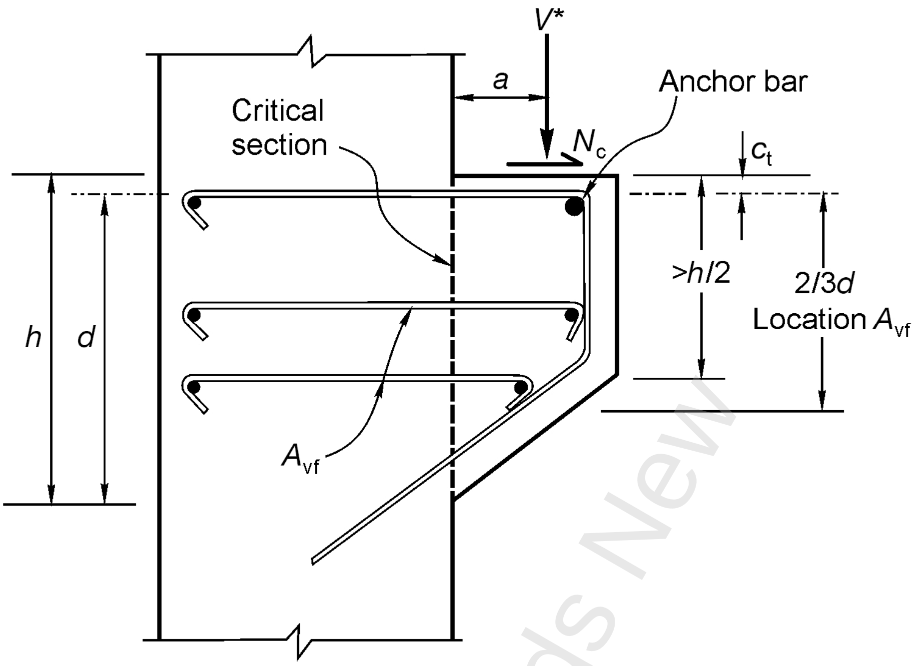


Figure C16.2 – Actions in a corbel

A3

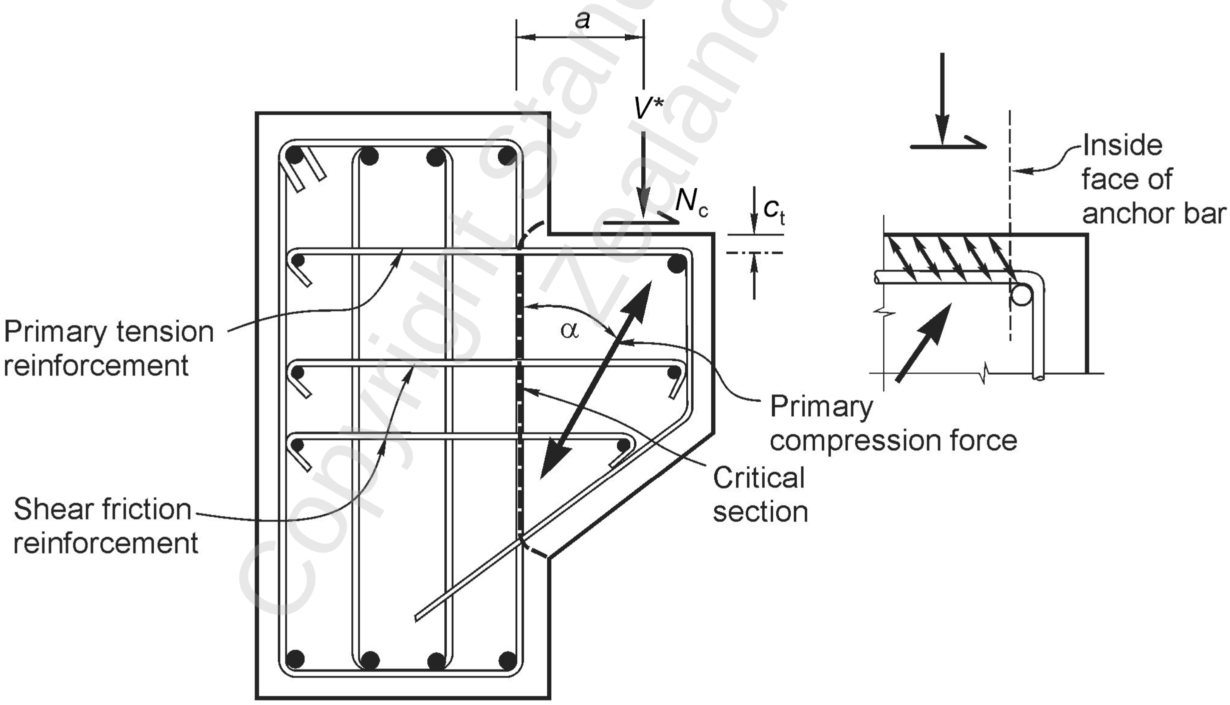


Figure C16.3 – Notation used in 16.5

A3

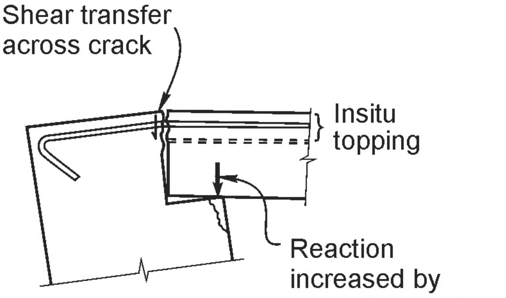


Figure C16.4A – Relative rotation between corbel and supported member

An upper limit of 1.0 for *a/d* is imposed method (b), for two reasons. First, for shear span-to-depth ratios exceeding unity, the diagonal tension cracks are less steeply inclined and the use of horizontal stirrups alone as specified in 16.5.8 is not appropriate. Second, this method of design has only been validated experimentally for *a/d* of unity, or less. An upper limit is provided for N_c^* because this method of design has only been validated experimentally for N_c^* less than, or equal to V^* including N_c^* equal to zero.

The *a/d* limit of 1.5 for use of the strut and tie method is to prevent the inclination of the flexural compression force dropping below limiting value of 1 in 2 and for the influence of the axial tension force on behaviour. With inclinations of diagonal compression struts in excess of 1 to 1.4 wide cracks may form in the serviceability limit state, and for values of this ratio close to or in excess of 1.5 the shear strength in the ultimate limit state may be reduced. Where the limiting *a/d* ratio of 1.5 is exceeded standard design provisions given in chapters 7 and 9 should be followed.

The previous limit of 1.8 for the strut and tie method has been reduced to 1.5 due to the increased significance of lateral forces on the seating. The previous lower limit of $0.2V^*$ for N_c^* has been increased to a more realistic value of $0.7V^*$.

C16.5 Empirical design of corbels or brackets

A3 Figure C16.2 and C16.3 illustrate the notation used in 16.5.

C16.5.1 *Depth at outside edge*

A minimum depth is required at the outside edge of the bearing area so that a premature failure will not occur due to a major diagonal tension crack propagating from below the bearing area to the outer sloping face of the corbel or bracket. Failures of this type have been observed [16.5] in corbels having depths at the outside edge of the bearing area less than required in this section of the code.

C16.5.2 *Design actions at the critical section*

The axial tension force is applied at the level of the flexural tension reinforcement; consequently the moment is taken about the critical section at the level of this reinforcement. Proportions of the area of reinforcement required to resist this moment (A_t) the axial tension force (A_n) and the area found from shear friction (A_{vf}) are combined to determine the total area required.

C16.5.3 *Shear-friction reinforcement*

The area of reinforcement required to resist sliding at the springing of the support. A_{vf}, is found from 7.7. Where the corbel and supporting member are cast monolithically the area of reinforcement is given by:

$$A_{vf} = \frac{V^*}{1.4\lambda\phi f_y} \quad \text{(Eq. C16–1)}$$

Where the concrete is not cast monolithically the factor 1.4 is replaced by the appropriate coefficient of friction, μ_f, given in 7.7.4.3. λ is 1.0 for normal weight concrete and is reduced for lightweight concrete (see 5.2) and f_y is the yield stress of the reinforcement. A3

C16.5.4 *Maximum shear stress*

Tests have shown [16.6] that the maximum shear strength of lightweight concrete corbels or brackets is a function of both concrete strength and *a*/*d* ratio. No data are available for corbels or brackets made from sand-lightweight concrete and the same limits have been placed on both sand lightweight all-lightweight concrete corbels and brackets. A3

C16.5.5 *Reinforcement for flexure*

Reinforcement required to resist moment can be calculated using flexural theory. This is only a component of the required reinforcement, as an additional areas is required to provide resistance to the axial tension force. The design moment is calculated by summing moments about the flexural reinforcement at the critical section. A3

C16.5.6 *Reinforcement for axial tension force*

Because the magnitude of horizontal forces acting on corbels or brackets cannot usually be determined with great accuracy, it is required that N_c^* be regarded as a live load.

C16.5.7 *Primary tension reinforcement*

Tests [16.7] suggest that the total amount of reinforcement (A_s + A_h) required to cross the face of support should be the greater of: A3

(a) The sum of A_{vf} calculated according to 16.5.3 and A_n calculated according to 16.5.6;

(b) The sum of 1.5 times A_f calculated according to 16.5.5 and A_n calculated according to 16.5.6.

If (a) controls, A_s = (2 A_{vf}/3 + A_n) is required as primary tensile reinforcement, and the remaining A_{vf}/3 should be provided as closed stirrups parallel to A_s and distributed within 2*d*/3, adjacent to A_s. Clause 16.5.8 satisfies this by requiring A_h = 0.5(2A_{vf}/3).

If (b) controls, A_s = (A_f + A_n) is required as primary tension reinforcement, and the remaining A_f/2 should be provided as closed stirrups parallel to A_s and distributed within 2*d*/3, adjacent to A_s. Again 16.5.8 satisfies this requirement.

C16.5.8 *Closed stirrups or ties*

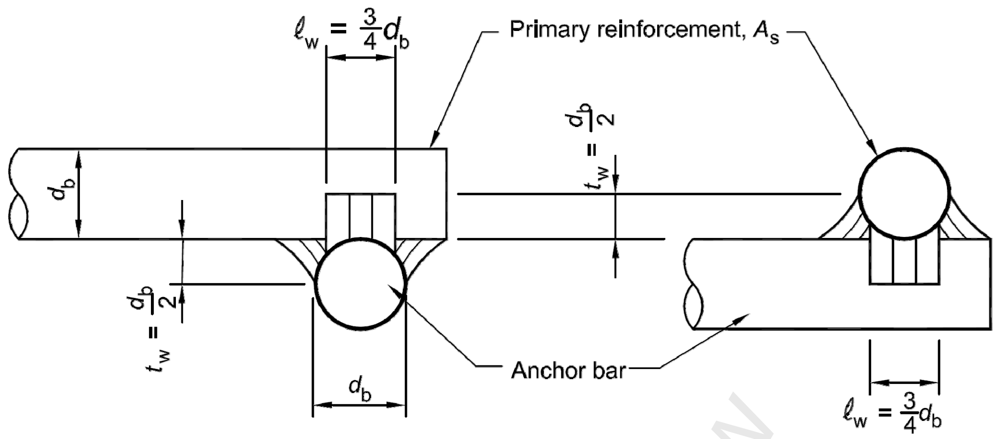
Closed stirrups parallel to the primary tension reinforcement are necessary to prevent a premature diagonal tension failure of the corbel or bracket. The required area of closed stirrups A_h = 0.5(A_s – A_n) automatically yields the appropriate amounts, as discussed in C16.5.7 above.

C16.5.9 *Ratio p*

A minimum amount of reinforcement is required to prevent the possibility of sudden failure should the bracket or corbel crack under the action of flexural moment and outward tensile force N_c^*.

C16.5.10 *Reinforcement A_s*

Because the horizontal component of the inclined concrete compression strut (see Figure C16.3) is transferred to the primary tension reinforcement at the location of the vertical load, the reinforcement A_s is essentially uniformly stressed from the face of the support to the point where the vertical load is applied. It should, therefore, be anchored at its outer end and in the supporting column, so as to be able to develop its yield from the face of support to the vertical load. Satisfactory anchorage at the outer end can be obtained by bending the A_s bars in a horizontal loop as specified in (b), or by welding a bar of equal diameter or a suitably sized angle across the ends of the A_s bars. The welds should be designed to develop the yield strength of the reinforcement A_s. The weld detail used successfully in the corbel tests reported in Reference 16.4 is shown in Figure C16.4. The reinforcement A_s should be anchored within the supporting column in accordance with the requirements of Section 8. A3 A3



A3 **Figure C16.4 – Weld details used in tests of Reference 16.5**

A3 C16.6 Design requirement by strut and tie method

C16.6.1 *Principal reinforcement A_s*

The internal lever-arm used to determine the area of flexural reinforcement required is limited to not greater than 0.87*d* for two reasons. First it is important that the reinforcement does not yield as this could lead to a wide crack reducing the sliding shear strength, and second high-compression strains in the concrete would also reduce the sliding shear resistance. An internal level arm of 0.87*d* is similar to that in a beam responding elastically to the loading**.**

C16.6.2 *Shear friction reinforcement*

Where the angle between the principal compression force and the critical section is small, the ratio of shear stress to normal compression stress at the section in the compression zone is such that principal tensile stresses can be induced and sliding shear might occur. The sliding shear provision would limit the angle to 35° but the limit has been set at 25° as some protection has been added to prevent high compression strains in the compression zone. However, when the 25° limit is exceeded the shear friction requirements reverts to those in 7.7.4. The angle of 25° meets the strut and tie limit in A4.5.

C16.7 Design requirements for beams supporting corbels of brackets

When plastic hinge zones sustain bending moments that involve yielding of flexural reinforcement, the torsional strength essentially decreases to zero[16.8], see 7.6.5. Provided the plastic hinge region contains adequate torsional reinforcement, it behaves in a ductile manner allowing relative rotation to occur between the beam and the column supporting the beam. Without the level of reinforcement given in 7.6.2 being provided there is a danger of a brittle torsional failure occurring, possibly resulting in collapse of the beam. In assessing the need for torsional reinforcement the exclusion given in 7.6.1.2 should not be used unless it can be shown that the rotation associated with peak drift does not cause the threshold torsion to be reached.

C16.8 Design requirements for ledges supporting precast units

C16.8.1 *The ledge support of precast floor units*

In the case of beams, the ledge can simply be the flat surface that makes up the cover concrete, or a typical corbel in profile (see Figure C16.2).

C16.8.2 *Design of ledges*

The distribution of transverse reinforcement required by the empirical method should ensure that the reinforcement necessary to support the floor system is distributed under the bearing area, pertaining to each type of precast floor unit.

For ledges designed according to the strut and tie method, the distribution of transverse reinforcement should meet the dimensions of nodes, and the associated permitted stress levels and anchorage details, as described in Appendix A. In doing so, the need to concentrate reinforcement within the width of the webs of tees and ribs will be satisfied. A3

REFERENCES

16.1 Hawkins, N.M. "The bearing strength of concrete loaded through rigid plates", Magazine of Concrete Research (London), Vol. 20, No. 62, Mar. 68, pp.31-40.

16.2 Richart, F.E., Brandtzaeg, A. and Brown, R.L., "A study of the fracture of concrete under combined compressive stresses", University of Illinois, Engineering Experimental Station, Bulletin 185, 1928.

16.3 Paulay, T., and Priestley, M.J.N., "Seismic design of reinforced concrete and masonry buildings", John Wiley and Sons, New York, pp. 98-103.

16.4 Booth, E. (editor), "Concrete Structures in earthquake regions", Longman Group, London, pp. 79-86.

16.5 Kriz, L.B. and Raths, C.H., "Connections in Precast Concrete Structures – Strength of Corbels," Journal of the Prestressed Concrete Institute, Vol. 10, No. 1, Feb. 1965, pp. 16 47.

16.6 Mattock, A.H., Chen, K.C. and Soongswang, K., "The Behaviour of Reinforced Concrete Corbel," Journal of the Prestressed Concrete Institute, Vol. 21, No. 2, Mar. Apr. 1976, pp. 52 77.

16.7 MacPherson, C., Seismic Performance and Forensic analysis precast hollow-core floor super-assemblage, METhesis, Civil Engineering, University of Canterbury

16.8 Collins, M. P., and Mitchell, D., "Prestressed Concrete Basics", Canadian Prestressed Concrete Institute, Ottawa Canada, 1987. A3

16.9 Fenwick, R., Bull, D. and Gardiner, D., "Assessment of hollow-core floors for seismic performance", Research report, 2010-02, Civil and Natural Resources Engineering, University of Canterbury, 2010.

16.10 Jenson, J., "Experimental investigation of existing hollow-core seating connections pre and post retrofit", ME Thesis, Civil and natural Resource Engineering, Univeristy of Canterbury, 2006

NOTES

C17 EMBEDDED ITEMS, ANCHORS AND SECONDARY STRUCTURAL ELEMENTS

A3

C17.1 Notation

The following symbols which appear in this Section of the Commentary, are additional to those used in Section 17 of Part 1.

A3

b_w effective web width, mm
d distance from extreme compression fibre to centroid of tension reinforcement, mm
e_h distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt
h thickness of member in which an anchor is anchored, measured parallel to anchor axis, mm
h'_{ef} reduced effective anchor embedment depth, mm
μ_p ductility of building part

C17.5 Anchors

A3

Anchors selected for a specific application should be evaluated on their ability to:

(a) Resist all applied forces and accommodate imposed deformations;
(b) Accommodate anticipated structural damage (e.g. spalling of cover concrete in primary seismic force-resisting elements) without loss of strength below an acceptable level [17.1].

A single anchor may contain separate components which act in tension and others which act in shear.

A3

C17.5.3 *Inserts for lifting*

The Department of Labour's Approved Code of Practice for Safe Handling, Transportation and Erection of Precast Concrete[17.2] is available on its website.

C17.5.5 *Strength of anchors by calculation*

A3

Clause 17.5.6 provides design rules based upon ACI 318, but only covering cast-in-place ductile steel headed studs, headed bolts, hooked bolts and hooked steel plates with diameters less than 50 mm and embedment lengths shorter than 635 mm.

C17.5.6 *Strength of cast-in anchors*

C17.5.6.1 *Scope*

This section is restricted in scope to structural anchors that transmit structural loads. The levels of safety defined by combinations of load factors and ϕ factors are appropriate for structural applications. Other standards may require more stringent safety levels during temporary handling.

The wide variety of shapes and configuration of speciality inserts makes it difficult to prescribe generalised design equations for many inserts. The scope of 17.5.6 is therefore limited to cast-in-place anchors.

The addition of supplementary reinforcement in the direction of the load or confining reinforcement, or both, can greatly enhance the strength and ductility of the anchor connection. Such enhancement is practical with cast-in-place anchors. References 17.3, 17.4 and 17.5 provide substantial information on the design of such reinforcement.

A3

The exclusion from the scope of load applications producing high cycle fatigue or extremely short duration impact (such as blast or shock wave) is not meant to exclude seismic load effects. Clause 17.6 presents additional requirements for design when considering seismic actions.

C17.5.6.3 *Strength requirements*

The ϕ factors for steel strength are based upon using f_{ut} to determine the lower characteristic strength of the anchor rather than f_y used in the design of reinforced concrete members. This approach is consistent with ACI 318. Although the ϕ factors for the use of f_{ut} appear low, they result in a level of safety consistent with the use of higher ϕ factors applied to f_y. The smaller ϕ values for shear than for tension do not reflect basic material differences, but rather account for the possibility of a non uniform distribution of shear in connections with multiple anchors.

C17.5.6.6 *Interaction of tension and shear – simplified procedures*

The shear tension interaction expression has traditionally been expressed as:

$$\left(\frac{N^*}{N_n}\right)^{\alpha} + \left(\frac{V^*}{V_n}\right)^{\alpha} \leq 1.0 \quad \text{(Eq. C17–1)}$$

where α varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where α = $^5/_3$ (Figure C17.1). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the second force are present. Any other interaction above expression that is verified by test data however, can be used to satisfy 17.5.6.5.

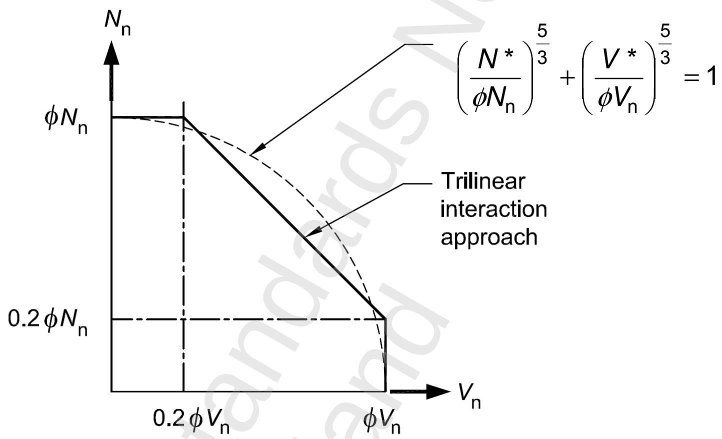


Figure C17.1 – Shear and tensile load interaction equation

C17.5.7.1 *Steel strength of anchor in tension*

The lower characteristic tension strength of anchors is best represented by $A_{se}f_{ut}$ rather than $A_{se}f_y$ because the large majority of anchor materials do not exhibit a well defined yield point.

The limitation of $1.9f_y$ on f_{ut} is to ensure that under service load conditions the anchor does not exceed f_y.

C17.5.7.2 *Strength of concrete breakout of anchor*

The effects of multiple anchors, spacing of anchors and edge distance on the nominal concrete break-out strength in tension are included by applying the modification factors A_n/A_{no} and Ψ_2 in Equation 17–7.

Figure C17.2(a) shows A_{no} and the development of Equation C17–2. A_{no} is the maximum projected area for a single anchor. Figure C17.2 (b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_n is the total projected area for a group of anchors, and A_{no} is the area for a single anchor, there is no need to include n, the number of anchors, in Equation 17–7. If anchor groups are positioned in such a way that their projected areas overlap, the value of A_n is required to be reduced accordingly.

$$A_{no} = 9h_{ef}^2 \quad \text{(Eq. C17–2)}$$

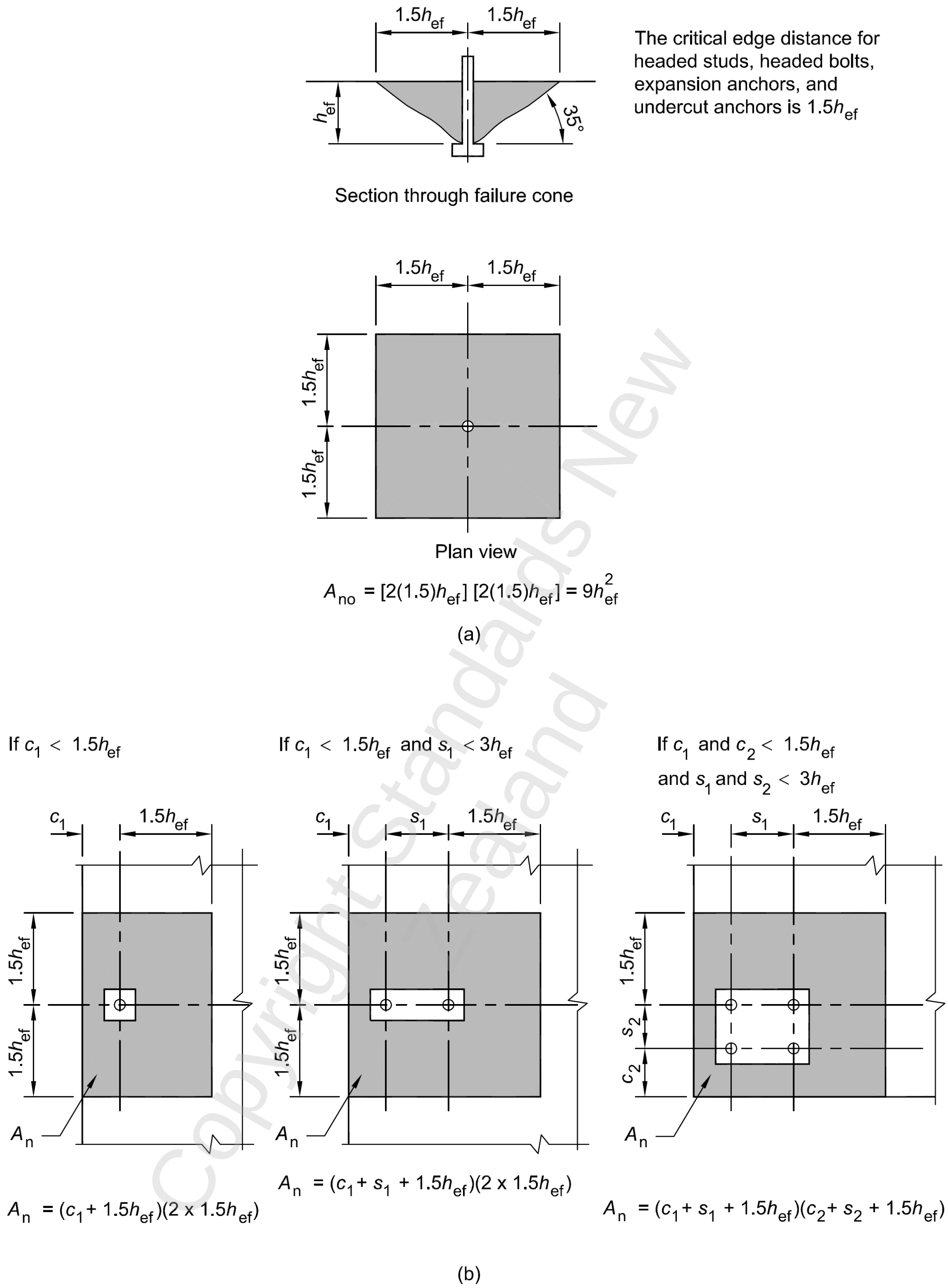


Figure C17.2 – (a) Calculation of A_{no} and (b) Projected areas for single anchors and groups of anchors and calculation of A_n

The basic equation for anchor capacity was derived [17.3, 17.6, 17.7, 17.8] assuming a concrete failure prism with an angle of about 35°, considering fracture mechanics concepts.

The value of k in Equation 17–9 was determined from a large database of test results in uncracked concrete[17.6] at the 5 % fractile. The value was adjusted to a corresponding k value for cracked concrete [17.7, 17.9]. A3

For anchors influenced by three or more edges where any edge distance is less than $1.5h_{ef}$, the tensile breakout strength computed by the ordinary CCD Method, which is the basis for Equation 17–9, gives misleading results. This occurs because the ordinary definitions of A_n/A_{no} do not correctly reflect the edge effects. If the value of h_{ef} is limited to $\frac{c_{max}}{1.5}$, however, where c_{max} is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$, this problem is corrected. As shown by Lutz [17.10], this limiting value of h_{ef} is to be used in Equations 17–8 and 17–9 and in determining Ψ_2. This approach is best understood when applied to an actual case. Figure C17.3 shows how the failure surface has the same area for any embedment beyond the proposed limit on h_{ef} (taken as h'_{ef} in the figure). In this example, the proposed limit on the value of h_{ef} to be used in the computations where $h_{ef} = \frac{c_{max}}{1.5}$, results in $h_{ef} = h'_{ef} = \frac{100\text{ mm}}{1.5} = 67\text{ mm}$. For this example, this would be the proper value to be used for h_{ef} in computing the resistance even if the actual embedment depth is larger.

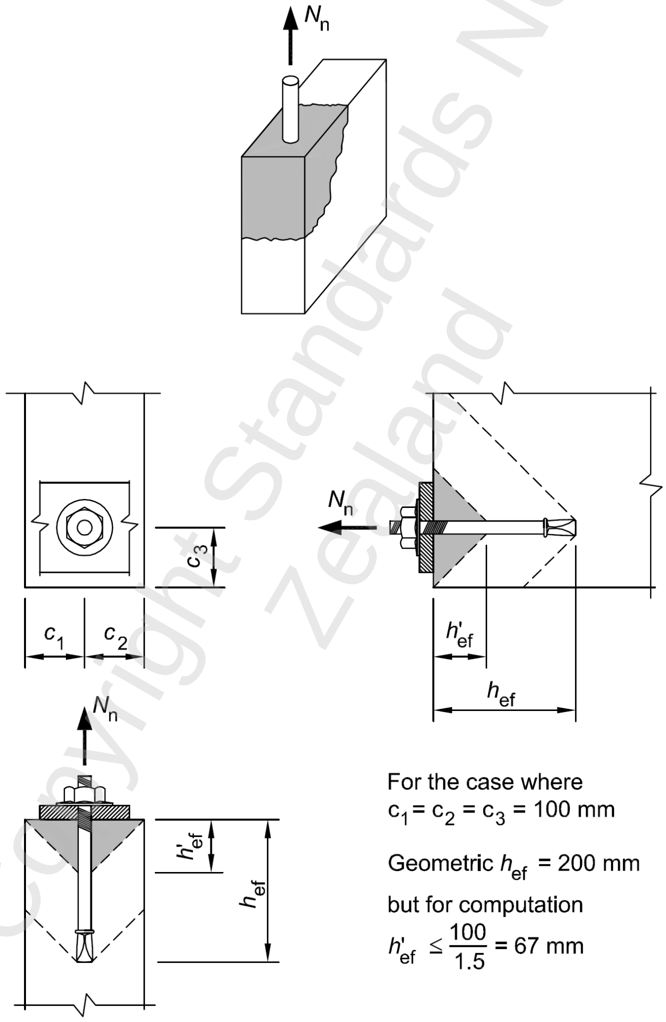


Figure C17.3 – Failure surfaces in narrow members for different embedment depths

Figure C17.4(a) shows dimension $e'_n = e_n$ for a group of anchors that are all in tension but that have a resultant force eccentric with respect to the centroid of the anchor group. Groups of anchors can be loaded in such a way that only some of the anchors are in tension Figure C17.4(b). In this case, only the anchors in tension are to be considered in the determination of e'_n. The anchor loading has to be determined as the resultant anchor tension at an eccentricity with respect to the center of gravity of the anchors in tension. Equation 17–8 is limited to cases where $e'_n \leq \frac{s}{2}$ to alert the designer that all anchors may not be in tension.

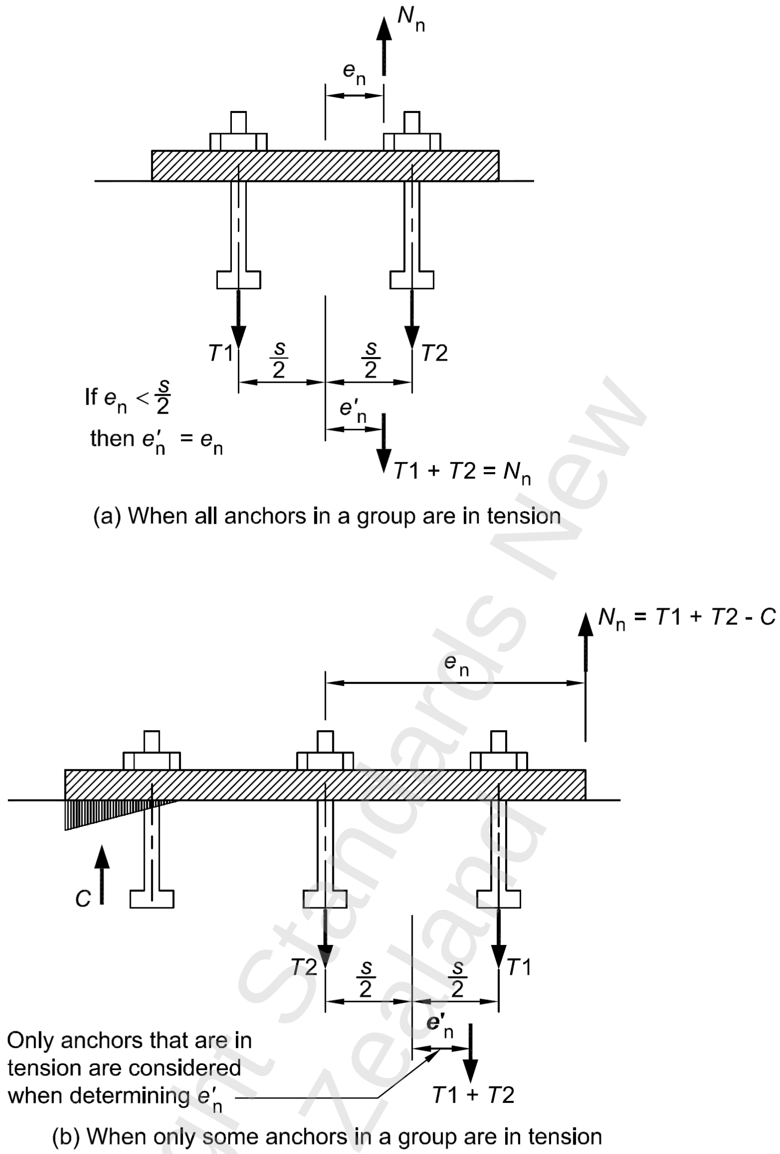


Figure C17.4 – Definition of dimension e'_n for group anchors

If anchors are located close to an edge so that there is not enough space for a complete breakout prism to develop, the load-bearing capacity of the anchor is further reduced beyond that reflected in $\frac{A_n}{A_{no}}$. If the smallest side cover distance is greater than $1.5h_{ef}$, a complete prism can form and there is no reduction ($\Psi_2 = 1$). If the side cover is less than $1.5h_{ef}$, the factor Ψ_2 is required to adjust for the edge effect[17.6].

Cast-in anchors that have not met the requirements for use in cracked concrete according to ACI 355.2 should be used in uncracked regions only. The analysis for the determination of crack formation should include the effects of restrained shrinkage. The anchor qualification tests of ACI 355.2 require that anchors in cracked concrete zones perform well in a crack that is 0.3 mm wide. If wider cracks are expected, confining reinforcement to control the crack width to about 0.3 mm wide is required. A3

C17.5.7.3 *Lower characteristic tension pullout strength of anchor*

The pullout strength equations given in 17.5.7.3 are only applicable to cast-in headed and hooked anchors[17.4, 17.11]. They are not applicable to expansion and undercut anchors that use various mechanisms for end anchorage unless the validity of the pullout strength equations are verified by tests.

The pullout strength in tension of headed studs or headed bolts can be increased by providing confining reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests.

Equation C17–3 corresponds to the load at which the concrete under the anchor head begins to crush [17.4, 17.12]. It is not the load required to pull the anchor completely out of the concrete, so the equation contains no term relating to embedment depth. The designer should be aware that local crushing under the head will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure.

$N_p = A_{brg} 8 f'_c$ .... (Eq. C17–3)

Equation C17–4 for hooked bolts was developed by Lutz based on the results of Reference 17.11. Reliance is placed on the bearing component only, neglecting any frictional component because crushing inside the hook will greatly reduce the stiffness of the connection, and generally will be the beginning of pullout failure. The limits on e_h are based on the range of variables used in the three test programmes reported in Reference 17.11.

$N_p = 0.9\, f'_c\, e_h d_o$ .... (Eq. C17–4)

C17.5.7.4 *Lower characteristic concrete side face blowout strength*

The design requirements for side face blowout are based on the recommendations of Reference 17.13. These requirements are applicable to headed anchors that usually are cast-in anchors. Splitting during installation rather than side face blowout generally governs post-installed anchors, and is evaluated by the ACI 355.2 requirements.

C17.5.8.1 *Lower characteristic shear strength of steel of anchor*

The nominal shear strength of anchors is best represented by $A_{se} f_{ut}$ for headed stud anchors and $0.6 A_{se} f_{ut}$ for other anchors rather than a function of $A_{se} f_y$ because typical anchor materials do not exhibit a well-defined yield point. The use of Equations 17–14 and 17–15 with load factors of AS/NZS 1170 or other referenced loading standard and the ϕ factors of 17.5.6.4 give design strengths consistent with the AISC Load and Resistance Factor Design (LRFD) Specifications[17.14].

The limitation of $1.9 f_y$ on f_{ut} is to ensure that under service load conditions the anchor stress does not exceed f_y. The limit on f_{ut} of $1.9 f_y$ was determined by converting the LRFD provisions to corresponding service level conditions.

C17.5.8.2 *Lower characteristic concrete breakout strength of the anchor in shear perpendicular to edge*

The shear strength equations were developed from the concrete capacity design (CCD)[17.6] method. They assume a breakout cone angle of approximately 35°, and consider fracture mechanics theory. The effects of multiple anchors, spacing of anchors, edge distance, and thickness of the concrete member on nominal concrete breakout strength in shear are included by applying the reduction factors A_v/A_{vo} in Equations 17–16 and 17–21, and Ψ_5 in Equation 17–21. For anchors far from the edge, 17.5.8.2 usually will not govern. For these cases, 17.5.8.1 and 17.5.8.3 often govern.

$A_{vo} = 4.5\,(c_1)^2$ .... (Eq. C17–5)

Figure 17.2(a) shows A_{vo} and the development of Equation C17–5. A_{vo} is the maximum projected area for a single anchor that approximates the surface area of the full breakout prism or cone for an anchor unaffected by edge distance, spacing or depth of member. Figure 17.2(b) shows examples of the projected areas for various single anchor and multiple anchor arrangements. A_v approximates the full surface area of the breakout cone for the particular arrangement of anchors. Because A_v is the total projected area for a group of anchors, and A_{vo} is the area for a single anchor, there is no need to include the number of anchors in the equation.

The assumption shown in the upper right example of Figure 17.2(b), with the case for two anchors perpendicular to the edge, is a conservative interpretation of the distribution of the shear force on an elastic basis. If the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear anchor. The PCI Design Handbook approach[17.15] suggests in Section 6.5.2.2 that the increased capacity of the anchors away from the edge be considered. Because this is a reasonable approach, assume that the anchors are spaced far enough apart so that the shear failure surfaces do no intersect[17.3]. Clauses 17.5.8.2 and 17.5.8.3 allow such a procedure. If the failure surfaces do not intersect, as would generally occur if the anchor spacing s is equal to or greater than $1.5c_1$, then after formation of the near-edge failure surface, the higher capacity of the farther anchor would resist most of the load. As shown in the bottom right example in Figure 17.2(b), it would be appropriate to consider the full shear capacity to be provided by this anchor with its much larger resisting failure surface. No contribution of the anchor near the edge is then considered. Checking the near-edge anchor condition is advisable to preclude undesirable cracking at service load conditions. Further discussion of design for multiple anchors is given in Reference 17.4.

For the case of anchors near a corner subject to a shear force with components normal to edge, a satisfactory solution is to check independently the connection for each component of the shear force. Other specialised cases, such as the shear resistance of anchor groups where all anchors do not have the same edge distance, are treated in Reference 17.3.

The detailed provisions of 17.5.8.2 apply to the case of shear force directed towards and edge. When the shear force is directed away from the edge, the strength will usually be governed by 17.5.8.1 or 17.5.8.4. The case of shear force parallel to an edge is shown in Figure C17.5.

A special case can arise with shear force parallel to the edge near a corner. In the example of a single anchor near a corner (see Figure C17.6), where the edge to the side c_2 is 40 % or more of the distance c_1 in the direction of the load, the shear strength parallel to that edge can be compared directly from Equations 17–21 and 17–16 using c_1 in the direction of the load.

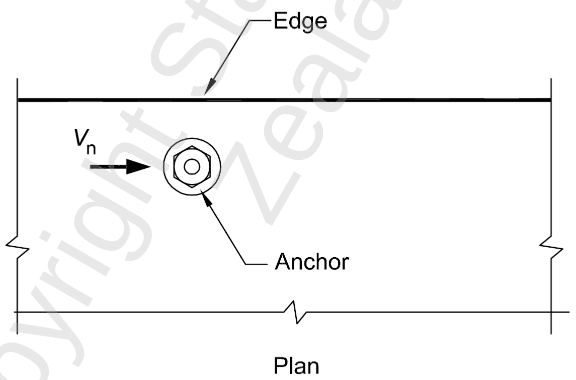


Figure C17.5 – Shear force parallel to an edge

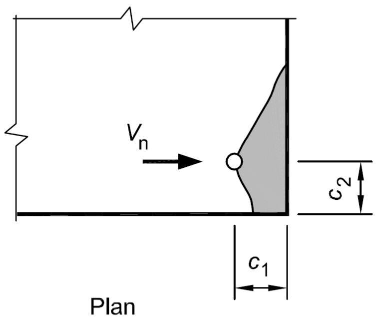


Figure C17.6 – Shear force near a corner

Like the concrete breakout tensile capacity, the breakout shear capacity does not increase with the failure surface, which is proportional to $(c_1)^2$. Instead the capacity increases proportionally to $(c_1)^{1.5}$ due to size effect. The capacity is also influenced by the anchor stiffness and the anchor diameter[17.6, 17.7, 17.3, 17.8].

The constant, k_2 in the shear strength Equation 17–17 was determined from test data reported in Reference 17.6 at the 5 % fractile adjusted for cracking.

For anchors influenced by three or more edges where any edge distance is less than $1.5c_1$, the shear breakout strength computed by the basic CCD Method, which is the basis for Equation 17–17, gives safe but misleading results. These special cases were studied for the κ Method[17.8] and the problem was pointed out by Lutz[17.10]. Similar to the approach used for tensile breakouts in 17.5.7.2, a correct evaluation of the capacity is determined if the value of c_1 to be used in Equations C17–5 and 17–17 to

A3 17–20 is limited to no greater than $\dfrac{h}{1.5}$.

Equation 17–18 for Ψ_5 provides a modification factor for an eccentric shear force towards an edge on a group of anchors. If the shear load originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, with a moment that may or may not also cause tension in the anchors, depending on the normal force. Figure C17.7 defines the term e_v' for calculating the Ψ_5 modification factor that accounts for the fact that more shear is applied on one anchor than the other, tending to split the concrete near an edge. If $e_v' \leq \dfrac{s}{2}$, the CCD[17.6] procedure is not applicable.

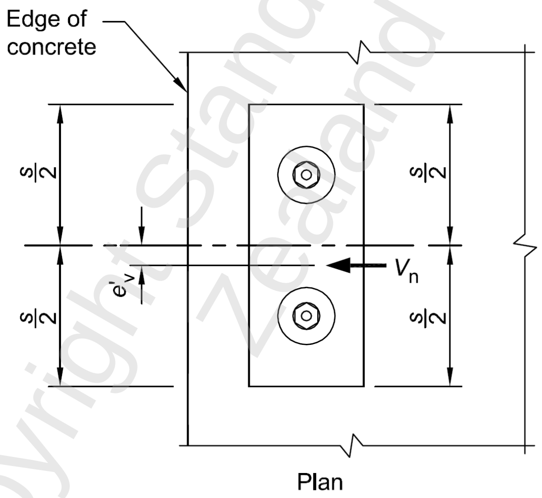


Figure C17.7 – Definition of dimensions e_v'

C17.5.8.3 *Lower characteristic concrete breakout strength of the anchor in shear parallel to edge*
Reference 17.6 indicates that the pry-out shear resistance can be approximated as one to two times the anchor tensile resistance with the lower value appropriate for h_{ef} less than 65 mm.

C17.5.9 ***Durability and fire resistance***

Steel embedments are often vulnerable to corrosion where they pass through cover concrete, and are normally specified either to be hot-dipped galvanised, or to have stainless steel components. Designers should specify embedment with a corrosion resistance appropriate to the physical situation of the embedment, the design life of the structure and the envisaged frequency of maintenance inspections.

The following points should be noted when specifying corrosion protection:

(a) Hot-dipped galvanising of mild steel can engender embrittlement in cold-worked sections, reducing ductility.

(b) Stainless steel, where attached to mild steel by welding, can promote crevice corrosion, or galvanic corrosion, in the contact areas.

Both of these phenomena are discussed in Reference 17.1.

C17.6 Additional design requirements for anchors designed for earthquake effects A3

C17.6.1 *Anchor design philosophy*

Four design philosophies are described with further information provided in 17.6.2 to 17.6.5. The aim of these philosophies is to ensure appropriate anchor performance in an earthquake. A3

C17.6.2 *Anchors designed for seismic separation* A3

A number of problems relevant to the anchor and separation of and damage to non-structural components in buildings are discussed in References 17.1 and 17.12. A common method of accommodating seismic movement is to provide slotted holes in anchor plates. When determining the size of separation required, A3 the design shall consider the impact that creep, shrinkage, and tolerances have on the available separation. The designer should also consider the impact on the magnitude of the required separation resulting from member dilatancy due to geometric effects and/or the formation of plastic hinges.

C17.6.3 *Anchors stronger than the overstrength capacity of the attachment* A3

Where the attached element is designed to form ductile plastic hinges, for example face loading of cladding or wall panels, the design actions acting on the connections may be determined by the use of capacity design principles. In determining the design actions forces in the connections due to shrinkage, creep, temperature or plastic hinge elongation should be considered.

C17.6.4 *Anchors designed to remain elastic* A3

NZS 1170.5 Section 8 provides design recommendations for the design of connections using a parts and portions approach. For non-ductile connections a ductility factor of μ_p = 1.25 is used to assess seismic actions. The requirement to use 0.75 times the strength reduction factor specified in 17.5.6.4, is to ensure that the fastenings have capacity to accommodate earthquakes larger than the design earthquake.

The shear and tension interaction equations for this design philosophy are:

(a) Where $V^* \leq 0.2\,\phi V_n$ then $N^* \leq 0.75\phi N_n$ ....(Eq. C17–6)

(b) Where $N^* \leq 0.2\,\phi N_n$ then $V^* \leq 0.75\phi N_n$ ....(Eq. C17–7)

(c) Where $V^* \leq 0.2\,\phi V_n$ and $N^* > 0.2\phi N_n$ then $\dfrac{N^*}{0.75\phi N_n} + \dfrac{V^*}{0.75\phi V_n} \leq 1.2$ ....(Eq. C17–8)

C17.6.5 *Anchors designed for ductility* A3

Anchors and connections should be designed and detailed to suppress a brittle concrete pull-out failure, or any other failure mode that gives little warning when approaching the ultimate limit state.

Detailing of anchors and connection hardware must be based on an assessment of the forces and the total displacement that may occur due to: A3

(a) Seismic actions such as:
   (i) Inertia forces;
   (ii) Deformation due to member dilatancy, inter-storey drift, plastic hinging and diaphragm actions.
(b) Creep, shrinkage and temperature effects that are likely to occur over the life of the structure;
(c) Gravity, wind loads and applied forces.

While bolts are a quick and simple means of fastening structural, architectural and mechanical elements, A3 ductility in bolts and threaded bars can be difficult to achieve under the types of deformations that anchors are often required to accommodate. The ductility required to meet the intent of this clause can, however, be provided by the use of bolts or threaded bars in conjunction with well detailed steel angles, partially de-bonded anchorage bars, friction connections, or similar force limiting details. If this approach is chosen as A3 a means of achieving overall anchor ductility, the non-ductile cast-in component of the anchor should have a nominal strength equal to, or greater than twice the ultimate limit state force that can be applied to it through the ductile (or limited slip) connecting hardware.

A3

C17.6.6 *Anchors in plastic hinge regions*

In zones of potential plastic hinging the contribution of the cover concrete to the anchorage of anchors should be disregarded. Anchors anchored in zones of flexural cracking should have the failure cone modified to account for the presence of flexural cracks.

REFERENCES

17.1 "Guidelines for the Use of Structural Precast Concrete in Buildings", Report of the Study Group of the New Zealand Group Society and the New Zealand National Society for Earthquake Engineering, Centre of Advanced Engineering, University of Canterbury, Second edition, 1999, p. 144.

17.2 "The Safe Handling, Transportation and Erection of Precast Concrete", Occupational Safety and Health Service, Department of Labour, Wellington, May 2002. http://www.osh.govt.nz.

17.3 "Fastenings to Concrete and Masonry Structures, State of the Art Report," Comite Euro-International du Beton, (CEB), Bulletin No. 216, Thomas Telford Services Ltd., London, 1994..

17.4 "Design of Fastenings in Concrete", Comite Euro-International du Beton (CEB), Thomas Telford Services Ltd., London, Jan. 1997.

17.5 Klingner, R., Mendonca, J. and Malik, J., "Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts under Reversed Cyclic Loading," ACI Journal, Proceedings V. 79, No. 1, Jan.-Feb. 1982, pp. 3-12.

17.6 Fuchs, W., Eligehausen, R. and Breen, J., "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," ACI Structrual Journal, V. 92, No. 1, Jan.-Feb. 1995, pp. 73-93. Also discussion, ACI Structural Journal, V. 92, No. 6, Nov.-Dec. 1995, pp. 787-802.

17.7 Eligehausen, R. and Balogh, T., "Behavour of Fasteners Loaded in Tension in Cracked Reinforced Concrete," ACI Structural Journal, V. 92, No. 3, May-June 1995, pp. 365-379.

17.8 Eligehausen, R. and Fuchs, W., "Load Bearing Behaviour of Anchor Fastenings under Shear, Combined Tension and Shear or Flexural Loadings," Betonwerk + Fertigteiltechnik, 2/1988, pp. 29 35.

17.9 Zhang, Y., "Dynamic Behaviour of Multiple Anchor Connections in Cracked Concrete," PhD dissertation, The University of Texas at Austin, Aug. 1997.

17.10 Lutz, L., "Discussion to Concrete Capacity Design (CCD) Approach for Fastening to Concrete," ACI Structural Journal, Nov.-Dec. 1995, pp. 791-792. Also authors' closure, pp. 798-799.

17.11 Kuhn, D, and Shaikh, F., "Slip –Pullout Strength of Hooked Anchors," Research Report, University of Wisconsin-Milwaukee, submitted to the National Codes and Standards Council, 1996

17.12 ACI Committee 349, "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-85),"see also ACI Manual of Concrete Practice, Part 4, 1987.

17.13 Furche, J. and Eligehausen, R., "Lateral Blowout Failure of Headed Studs Near a Free Edge," Anchors in Concrete-Design and Behaviour, SP-130, American Concrete Institute, Farmington Hills, MI, 1991, pp. 235-252.

17.14 "AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings," Dec. 1999, 327 pp.

17.15 PCI Design Handbook, 5th Edition, Precast/Prestressed Concrete Institute, Chicago, 1999.

17.16 McKenzie, G.H.F., "Problem of Damage to Non-structural Components and Equipment – Walls and Stairs", Proceedings of the Workshop on Earthquake Resistant Reinforced Concrete Building Construction, University of California, Berkeley, 1977, Vol. 3, pp. 1128-1139.

C18 PRECAST CONCRETE AND COMPOSITE CONCRETE FLEXURAL MEMBERS

C18.1 Notation

The following symbols, which appear in this section of the Commentary, are additional to those used in Section 18 of Part 1.

b	width of compression face of a member, mm
b_w	effective web width, mm
f'_c	specified compressive strength of concrete, MPa
L_d	development length, mm
L_n	clear span of member measured from face of supports, mm
V_c	nominal shear strength provided by concrete, N
α	factor for determining reinforcing steel overstrength

A3

A3

C18.2 Scope

Design and construction requirements for precast concrete structural members differ in some respects from those for cast-in-place concrete structural members and these differences are addressed in this section. Where provisions for cast-in-place concrete apply also to precast concrete, they have not been repeated. Similarly, items related to prestressed concrete in Section 19 that apply also to precast concrete are not restated. A3

A key reference document for precast concrete building structures is Reference 18.1. Detailed recommendations concerning precast concrete are also given in References 18.2, 18.3, 18.4, 18.5, 18.6, 18.7, 18.8 and 18.9. Tilt-up concrete construction is a form of precast concrete. It is recommended that Reference 18.10 be reviewed for tilt-up structures.

C18.2.2 *Composite concrete flexural members defined*

The scope of this section is intended to include all types of composite concrete flexural members including composite single -T or double -T members, composite hollow-core, box sections, folded plates, lift slabs, and other structural elements, all of which should conform to the provisions of this section. In some cases with fully cast-in-place concrete, it may be necessary to design the interface of consecutive placements of concrete as required for composite members.

C18.2.3 *Composite concrete and structural steel not covered*

Composite structural steel-concrete members are not covered in this section as such sections are fully covered in Section 10 and NZS 3404.

C18.2.4 *Section 18 in addition to other provisions of this Standard*

This Standard in its entirety applies to composite concrete flexural members except as specifically modified in Section 18. For instance, deep composite beams shall be designed in accordance with 9.3.10.

C18.3 General

C18.3.1 *Design to consider all loading and restraint conditions*

Stresses developed in precast members during the period from casting to final connection may be greater than the actual service load stresses. Care is therefore required to ensure that performance at both the serviceability and ultimate limit states is adequate to meet the requirements of this Standard.

Minor cracking or spalling need not be grounds for rejection provided that member strength and durability are not adversely affected. Guidance on assessing cracks in precast members is given in References 18.11 and 18.12.

C18.3.2 *Include forces and deformations at connections*

The structural behaviour of precast members may differ substantially from that of similar members that are cast-in-place. Design of connections should allow for forces and movements caused by changes in dimensions and orientations from elastic and inelastic deformations including, but not limited to:

A3

(a) Reduction in bearing length due to elongation of beams caused by formation of plastic hinges;
(b) Support rotations due to peak inter-storey drift; and
(c) Changes in the relative position or orientation of supports, due to shrinkage, creep, temperature changes and differential settlement affecting lengths and rotations.S upports and connections should be detailed to minimise the forces resulting from service load deflections or end rotation, or should be designed to accommodate these effects while meeting the serviceability criteria set out in 2.4.4.

C18.3.4 *Tolerances*

Guidance on precast concrete product tolerances is given in Reference 18.13, while guidance on erection tolerances is given in Reference 18.1. In order to prevent misunderstanding, the tolerances assumed in design should be specified in the contract documents.

A3

Design of precast members and connections can be particularly sensitive to tolerances on the dimensions of individual members and on their location in the structure.

C18.3.5 *Long-term effects*

Precast member supports and connections should be detailed to minimize forces due to creep, shrinkage and temperature effects. Alternatively they should be designed to resist the displacement resulting from such actions in a ductile manner.

There is a particular requirement to check camber effects of long span precast concrete members exposed to differential temperature gradients such as occur on the upper levels of parking buildings.

Secondary effects can have a significant influence on long span structures, large plan area structures and buildings such as parking structures exposed to the weather[18.14].

C18.4 Distribution of forces among members

A3 C18.4.1 *Forces perpendicular to the axis of members*

Concentrated point and line loads can be distributed among members provided they have sufficient torsional stiffness and that shear can be transferred across joints. Torsionally stiff members such as hollow-core or solid slabs have more favourable load distribution properties than do torsionally flexible members such as double tees with thin flanges. The actual distribution of the load depends on many factors. Large openings can cause significant changes in distribution of forces.

C18.4.2 *In-plane forces*

In-plane forces result primarily from diaphragm action in floors and roofs, causing tension or compression in the chords and shear in the body of the diaphragm. A continuous path of steel, steel reinforcement, or both, using lap splices, mechanical or welded splices, or mechanical connectors, should be provided to carry the tension, whereas compression may be carried by the net concrete section. A continuous path of steel through a connection may include bolts, weld plates, headed studs, or other steel devices. Tension forces in the connections are to be transferred to the primary reinforcement in the members.

In-plane forces in precast wall systems result primarily from diaphragm reactions and external loads. A3

Connection details should provide for the forces and deformations due to shrinkage, creep, and thermal effects. Connection details may be selected to accommodate volume changes and rotations caused by temperature gradients and long-term deflections. When these effects are restrained, connections and members should be designed to provide adequate strength and ductility.

C18.5 Member design

C18.5.1 *Prestressed slabs and wall panels*

For prestressed concrete members not wider than 2.4 m, such as hollow-core slabs, solid slabs, or slabs with closely spaced ribs, there is usually no need to provide transverse reinforcement to withstand shrinkage and temperature stresses in the short direction. This is generally true also for non-prestressed slabs. The 2.4 m width is less than that in which shrinkage and temperature stresses can build up to a magnitude requiring transverse reinforcement. In addition, much of the shrinkage occurs before the members are tied into the structure. Once in the final structure, the members are usually not as rigidly connected transversely as monolithic concrete, thus the transverse restraint stresses due to both shrinkage and temperature change are significantly reduced. A3

The waiver does not apply to members such as single and double tees with thin, wide flanges.

Where the form of construction described in 18.5.1 is used there is no restraint preventing all the displacements due to temperature change, creep, shrinkage and elongation etc. from occurring at one support. Consequently seating lengths need to be increased to allow for this possibility, see 18.7.4.3 and 18.8.1. A3

C18.5.2 *Composite concrete flexural members*

C18.5.2.1 *Shored and unshored members*

Tests to destruction indicate no difference in flexural strength of members that were constructed as either shored or unshored.

The provisions of 6.8.5 must be considered with regard to deflections of shored and unshored members. Before shoring is removed it should be ascertained that the strength and serviceability characteristics of the structure will not be impaired.

C18.5.2.2 *Design of constituent elements*

The premature loading of precast elements can cause excessive deflections as the result of creep and shrinkage. This is especially so at early ages when moisture content is high and strength is low.

C18.5.2.3 *Reinforcement for composite members*

The extent of cracking permitted is dependent on such factors as exposure and durability. In addition, composite action must not be impaired.

C18.5.4 *Longitudinal shear in composite members*

C18.5.4.1 *Transfer of interface shear between precast and in situ concrete* A3

The maximum longitudinal shear stresses, v_ℓ, apply when the design is based on the ultimate limit state requirements and a strength reduction factor $\phi = 0.75$.

Two different cases are considered for shear transfer between *in situ* concrete and precast components or to structural members that have been constructed previously. The first case applies to shear transfer across the horizontal interface between *in situ* concrete and precast components such as floor units, and to the various interfaces between *in situ* concrete and precast units such as shell beams that confine the *in situ* concrete when it is cast. The second case is where *in situ* concrete is cast against vertical interfaces of members, such as beams, walls and columns that have been constructed before the *in situ* concrete was placed. In this case any required shear transfer should be based on the shear friction provisions.

A3 In reviewing composite concrete flexural members at the serviceability limit state, including actions resulting from handling and construction loads, the resulting longitudinal shear stress should be compared with the maximum stresses considered for the ultimate limit state (allowed in 18.5.4.1).

The full transfer of longitudinal shear between segments is to be achieved by either contact stresses or properly anchored ties, or both.

Tests[18.15] indicate that longitudinal shear does not present a problem in T-beams when the portion below the flange is designed to resist the vertical shear, the interfaces of the components are rough and minimum ties are provided according to 18.5.5. These considerations may also be used with other segmental shapes.

The top surfaces of precast hollow-core slabs produced by a dry-mix extrusion process are difficult to roughen to the requirements of 18.5.4.1 without causing damage to the integrity of the slab. Tests[18.16, 18.17] have shown that surfaces produced by processes that do not leave accumulated surface laitance, although not artificially roughened, can develop adequate bond with the cast-in-place topping concrete and provide adequate shear transfer for diaphragm action. Shear keys at 1200 mm centres further improve composite action.

A3

C18.5.4.2 *Interface shear stress*

A2 The shear stress acting along a plane in a region of a member, which does not contain flexural or shear cracks, and the stresses in the reinforcement and concrete remain within their elastic ranges, can be calculated directly from Equation 18–1. In this equation, Q is the first moment of area beyond the shear plane being considered about the axis of bending and I is the transformed second moment of area. If this equation is used in a region where flexural cracks exist, the section properties I and Q should be calculated neglecting concrete in tension and relating the properties to the axis of bending, which is not the same as the zero stress line, if axial load or prestress acts on the section.

In a reinforced concrete beam with zero axial load, the shear stress in the flexural tension zone can be calculated from Equation 18–2. This equation can be derived from consideration of equilibrium in a segment of a beam, as illustrated in Figure C18.1. This figure shows a segment of beam of length $\Delta\ell$, subject to a shear force of V, which is equal to V^*/ϕ. The shear force in the concrete immediately above the reinforcement balances the change in flexural tension force, ΔT. The average horizontal shear stress, $v_{d\ell}$, times the width, b_w, and the length, $\Delta\ell$, for equilibrium must equal ΔT. However, ΔT is equal to the change in bending moment over the length, $\Delta\ell$, divided by the internal level-arm, jd, that is $\Delta T = \Delta M/jd$. The change in bending moment, ΔM, is equal to the shear force times the distance $\Delta\ell$. Using these relationships gives the expression:

$$\Delta T = \frac{\Delta M}{jd} = \frac{V^* \Delta\ell}{\phi\, jd} = v_{d\ell}\, b_v\, \Delta\ell \quad \text{(Eq. C18–1)}$$

that can be simplified to:

$$v_{d\ell} = \frac{V^*}{\phi\, jd\, b_v} \quad \text{(Eq. C18–2)}$$

which is identical to Equation 18–2.

A2

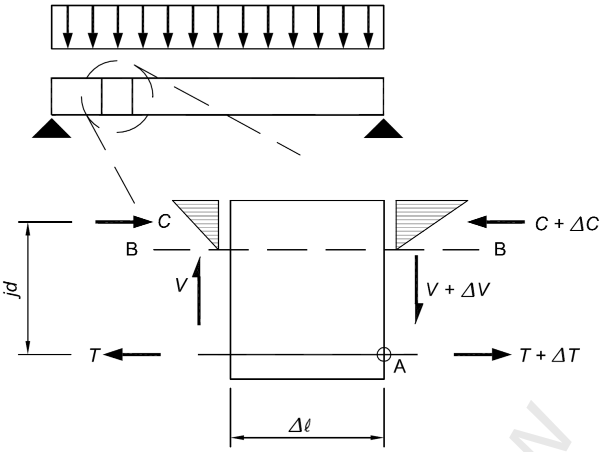


Figure C18.1 – Derivation of shear stress

The longitudinal shear stress in a member sustaining axial load, or in a prestressed member, may be found by carrying out section analyses at 2 locations, not more that *h*/2 apart. The difference in stresses and associated with the internal forces at these two sections can be used to find the longitudinal shear stresses as explained in text books on structural mechanics.

A3

C18.5.4.3 *VOID*

C18.5.4.4 *Transfer of shear where tension exists*

Proper anchorage of bars extending across joints is required to ensure that contact of the interfaces is maintained.

C18.5.4.5 *Requirements for bridge superstructures*

The provisions of this clause are necessary in bridges to safeguard against tendency for progressive breakdown of horizontal shear strength under traffic vibrations. Compared with buildings, bridges have the following characteristics which influence the provisions of this clause:

(a) Greater possibility of overloads;

(b) Dynamic effects associated with highway loads;

(c) Potentially longer life;

(d) Greater probability of the presence of an aggressive environment.

C18.5.4.6 *Bridge deck overlays*

Bridge deck overlays may be required for a variety of reasons. The deck may require rehabilitation. There may be a lack of cover to the deck reinforcement, or overlays may also be used to improve rideability.

Research presented in the References 18.18, 18.19, and 18.20, determined that shear dowels through the interface were generally not required when the old concrete face is adequately rough and clean. The report and the papers provide a basis for determining when interface dowels through the overlay area are required.

Perimeter dowel reinforcement along the free edges of the overlay should, however, be provided where the potential for curl of the overlay exists due to environmental effects and differential shrinkage. The following formula can be used to determine the reinforcement required to prevent edge curl:

$A_{dp} = 25.3 h_o^{3/2} / f_{dy}$ (mm²/m)

Where: A_{dp} = area of perimeter dowel reinforcement
h_o = thickness of the overlay
f_{dy} = yield stress of the dowel reinforcement

C18.5.5 *Ties for longitudinal shear*

C18.5.5.1 *Minimum anchorage into composite topping*

Anchorage of large diameter stirrups is not possible in thin topping concrete[18.21].

C18.5.5.2 *Minimum area and spacing of ties*

The minimum areas and maximum spacings are based on test data given in References 18.15 to 18.22 inclusive.

C18.5.6 *Precast shell beam construction*

C18.5.6.2 *Requirements for fully-composite action*

Section and material properties such as *d*, the distance from the extreme compression fibre to the centroid of the tension reinforcement, b_w, the web width, f'_c, the specified compressive strength of the concrete and the area of reinforcement are sensitive to the load sense of the critical section and whether the assumption of composite behaviour is appropriate or not.

Sectional properties *d* and b_w for beams incorporating precast shells may be taken as described in Figure C18.2 [18.23, 18.24]. These properties may be used for determining longitudinal reinforcement ratios, shear stresses and other design parameters as appropriate.

The width of the compression face of a member *b* may be larger than indicated in Figure C18.2 when, in accordance with 9.3.1.2, the composite beam has flanges on one or both sides or when the topping of any supported precast floor system is considered to act as flanges.

In Figure C18.2 a "positive" moment causes tension in the bottom fibre of the precast shell while a "negative" moment causes tension in the top fibre of the cast-in-place core.

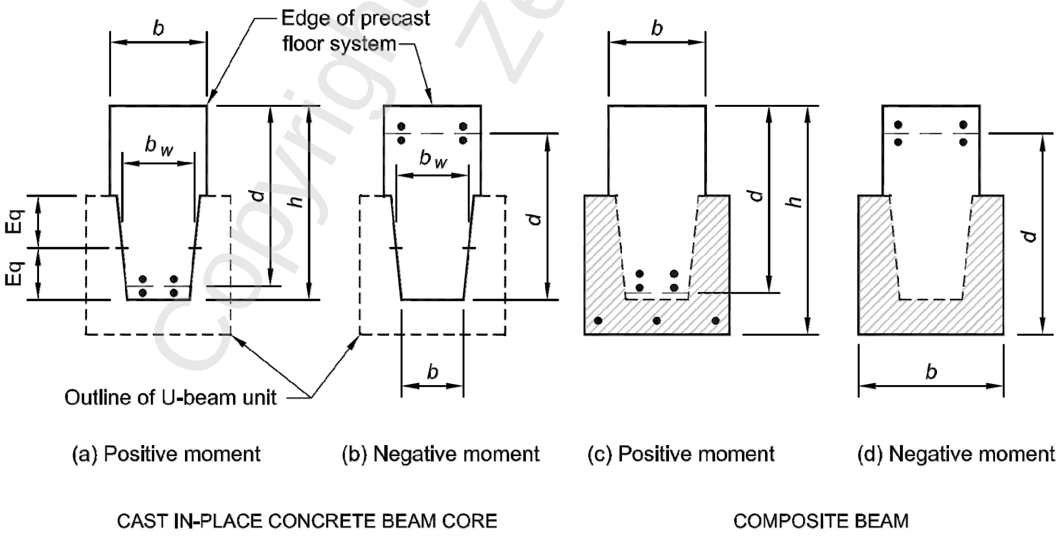


Figure C18.2 – Properties of beam sections

The resultant shear stresses on the interface between the precast shell and the cast-in-place core are very complex and arise from a number of sources. One source is the imposed shear stresses along the interface that arise from the transfer of tension forces from the longitudinal reinforcement to the compression zones in the concrete. Other sources arise from the support of gravity loads.

A2

A2

It is possible that support to the ends of the shell may not be provided in service or is lost (e.g. it is expected that the cover concrete of the column will be damaged during a major seismic event and support from the cover concrete for the ends of the precast units cannot be relied upon). When no support for the ends of the precast shells occurs, vertical shear stresses on the interface will be due to the self weight of the precast shell, the floor system and the cast-in-place beam core and floor topping, as well as superimposed dead loads and live loads.

A rational method for establishing the effect of shear stresses on the interface between the precast shell and the cast-in-place core should be employed. One possible method is discussed in References 18.23 and 18.24.

C18.5.6.3 *Design of precast shell*

As described in 18.3, actions in addition to the primary structural actions can be applied to the precast shell.

If the bond between the shell and the cast-in-place core cannot be relied upon then the shell must span between, or to zones of support such as columns or regions of the beam where full composite action is assured. The unsupported lengths of shell shall be designed, in a rational manner, to carry any applied forces [18.23, 18.24.]

If, due to strategic placement of temporary propping, the precast shells are designed to carry the self weight and imposed construction loads, then stability (and in particular torsional stability) during construction should be carefully assessed. A2

Where composite action between the shell and *in situ* concrete is relied upon in design, the shear stresses between the shell and the *in situ* concrete should be checked to ensure they comply with the permissible shear stress levels in 18.5.4.

C18.5.6.4 *Shear strength of composite beam*

Reference 18.23 demonstrates that if fully composite action is assumed for flexure then the minimum amount of transverse reinforcement for the shell is in proportion to the contribution to the composite flexural strength made by the shell. However, special consideration needs to be given to the spacing and development relationships between the transverse reinforcement of the shell and the core and the position of the diagonal strut in the assumed truss mechanism for resisting shear.

Conservatively the designer may choose to disregard the contribution to flexure of the longitudinal reinforcement in the precast shell and design for flexure and shear considering the beam core alone.

C18.6 Structural integrity and robustness

These provisions for precast concrete structures broadly follow those recommended for adoption by ACI 318. The overall integrity of a precast structural system, which is inherently discontinuous, can be substantially enhanced by providing continuity in tension at connections in both horizontal directions, as well as vertically by means of relatively simple detailing of the reinforcement. The aim is to ensure that all precast elements making up a floor system can effectively interact to transmit diaphragm forces. Moreover, should vertical supports become displaced due to unexpected actions, sufficient continuity should remain to enable catenary action to be mobilised, thereby minimising the risk of total collapse of precast systems.

C18.6.1 *Load path to lateral force-resisting systems*

Rational concepts satisfying equilibrium criteria must be used to ensure that effective load paths can be also utilised for the disposition of unexpected or non-quantifiable horizontal forces.

C18.6.2 *Diaphragm action*

Where precast elements form part of diaphragms, design earthquake forces are likely to govern desired diaphragm performance and hence the requirements of 13.3.7.4 must be followed.

Individual members may be connected in different ways when forming part of the clearly identified load path. For example spandrel beams may be connected intermittently or continuously to the floor system. Alternatively, spandrels may be connected to the columns only, which in turn must then be connected to the diaphragm system (see Figure C10.2). Figure C18.3 shows locations where peripheral reinforcement for diaphragm action is required.

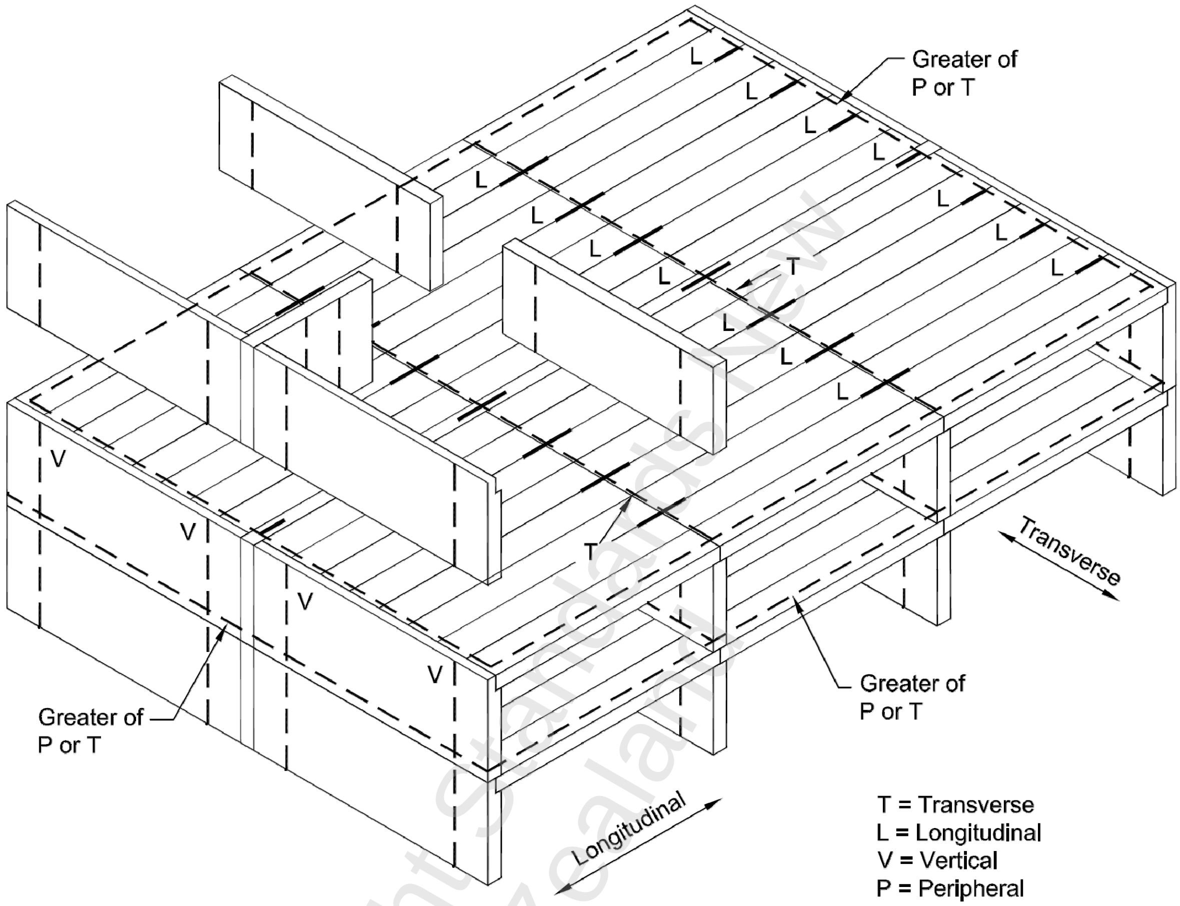


Figure C18.3 – Typical locations for tying reinforcement in a large panel structure

A3 C18.6.3 *Wall structures*

To preserve structural integrity, minimum provisions for the tying of bearing wall construction, often called large panel construction, are intended to provide catenary hanger support for floor slabs in case of loss of
A3 bearing wall support. The effectiveness of these provisions have been shown by tests[18.25]. These requirements are based on the recommendations of the Precast/Prestressed Concrete Institute for the design of precast concrete buildings with bearing walls [18.26].

When designing this tying reinforcement:

(a) Longitudinal tension reinforcement at locations L in Figure C18.3 may project from panels and be lapped or mechanically spliced, or they may be embedded in grouted joints, with sufficient length and cover to develop the required design strength [18.27].

(b) Tying reinforcement may be uniformly spaced either encased in panels or in a topping, or they may be concentrated at transverse bearing walls with spacing not exceeding 3 m.

(c) The requirements for peripheral reinforcement are not additive to those for the transverse or longitudinal tying reinforcement.

C18.6.4 *Joints between vertical members*

The recommendations for connections by vertical reinforcement at a column base or at horizontal joints, including that for wall panels, are approximate and should be considered as minima. When earthquake forces are to be considered, simple analysis may show that joint reinforcement in excess of that derived from the estimates of this clause may be required.

The requirements for minimum tension strength in 18.6.4(a) should be applied only to minor vertical members which, because of small gravity induced compression load, may be subjected to net axial tension where lateral forces are introduced to the structure. For columns, the integrity of which is essential in sustaining gravity loads, the requirements of 10.3.8.1 apply.

C18.6.5 *Connections*

Horizontal force transfer at member supports, based on friction only, particularly under seismic actions, should not be relied on. A3

C18.6.7 *Deformation compatibility of precast flooring systems*

Floors constructed using precast units should be designed and detailed to enable the deformations and displacements imposed on them by the supporting structure to be accommodated, without losing their strength to sustain gravity loading and diaphragm actions. A3

Hollow-core units are brittle in character and as such, care is required to ensure satisfactory performance can be obtained in situations where either high diaphragm shear stresses are induced or relative displacements may be imposed between the hollow-core units and the supporting structure. A2

Research into the seismic performance of hollow-core flooring systems has been conducted and is summarised in Reference 18.29. The detail described in this clause is considered best practice, based upon the information to hand during the preparation of the Standard. Improved details may be developed as more research data becomes available. A3 A2

A3

Design should be based on rational calculations or on methods proved through testing. The following issues should be addressed: A2

(a) The precast units and composite concrete topping (if any) should be designed to sustain the maximum negative moments, or moment, and axial tension that can be applied to the units at the supports simultaneously with the seismic actions associated with vertical ground motion. The vertical seismic actions should be based on a structural ductility factor not exceeding 2; A3

(b) Where hollow-core units are used and the cores are not filled at the supports, calculations should demonstrate that shear stresses normal to the axis of the unit, together with the associated flexural stresses in the webs, will not lead to failure of the webs; A2

A3

(e) The use of filled cells and reinforcement. Testing of floors incorporating hollow-core units has shown that the seismic performance specified under 18.6.7 may be achieved by using the detailing described below:

  (i) Cells shall be filled at not more than 600 mm centres with a maximum of 50 % of cells filled per hollow-core unit. The cells shall be filled and reinforced in accordance with (iii) below for a minimum distance from the support of the greater of 800 mm or 3 times the hollow-core unit depth; and A2

A2 (ii) Each of these cells shall be filled with the same concrete at the same time that the topping concrete is cast; and

(iii) Each filled cell shall contain Grade 300 plain reinforcement near the bottom of the cell, with an ultimate strength of 60 kN. This reinforcement shall be anchored by standard hooks at each end into the concrete core and the support beam; and

(iv) Reinforcement in the topping passing over the end of the hollow-core unit shall:

(A) Comprise Grade 300 reinforcing; and

(B) Be anchored in the supporting beam or the adjacent span: and

(C) Bar diameter shall not be greater than one-fifth the topping thickness; and

(D) Reinforcement up to a maximum ultimate strength of 120 kN/m shall extend a minimum distance into the span beyond the end of the hollow-core unit by the greater of (L_d + 400) mm, or 0.2 times the hollow-core span;

(E) Where the minimal strength ($A_s f_y$) of the reinforcement exceeds 113 kN/m, that portion in excess of this limit shall extend the entire span of the hollow-core.

The proposed detail for the support of hollow-core flooring units is shown in Figure C18.4.

Elongation of plastic hinge regions in beams and/or relative rotation between supporting structure and the precast floor units, can lead to the formation of wide cracks at the support zones. This cracking can induce high strains in any reinforcement that ties the precast units and their *in situ* topping concrete to the supports. As a result, high axial forces and negative bending moments can be induced into the end of the units which are not designed to sustain these actions.

To prevent brittle failure, the capacity at the floor/beam interface should have a lower capacity than the composite hollow-core and topping. This requires a limitation on the area of the reinforcement crossing the critical section (refer Figure C18.5), and the termination point of any reinforcement crossing this point.

The yield capacity limitations provided in (a) assume a probable lower limit on the capacity of the hollow-core floor and overstrength of the reinforcement. The minimum development length beyond the critical section is increased by 400 mm to accommodate the possibility of diagonal cracking.

Figure C18.4 requires that within the core, plain round bars are placed only at the bottoms of the filled cells. The provision of multiple layers of reinforcement in the filled cores can result in the filled core effectively behaving as a short cantilever that can pry apart the top and bottom of the hollow-core units. To ensure that hinging does not occur at the ends of the filled cores under negative moment, the topping
A3 reinforcement crossing the critical section is to extend beyond the end of the unit by a development length plus 400 mm.

A2 Where the recommended solution is not adopted, capacity design principles are to be applied to the hollow-core unit and its supports. This process is undertaken to ensure that in the event of relative displacement between the support and units (due to rotation and/or elongation) cracking will be confined to the critical section between the end of the hollow-core unit and the supporting beam. The structural overstrength actions transmitted across the critical section into the hollow-core unit are to be calculated assuming that the reinforcement sustains a stress of αf_y, where α is 1.6 for Grade 300 and 1.5 for Grade 500 reinforcement. The area of added reinforcement is calculated to ensure the nominal bending moment and axial load capacity of the hollow-core unit is greater than the maximum structural actions induced by the overstrength actions at the critical section, together with gravity load and vertical seismic actions, and that the shear stresses induced in the negative moment flexurally cracked tension zone are not sufficient to cause a diagonal tension failure in the zone containing the filled cores and beyond this zone.[18.28], see 19.3.11.2.4.

A3

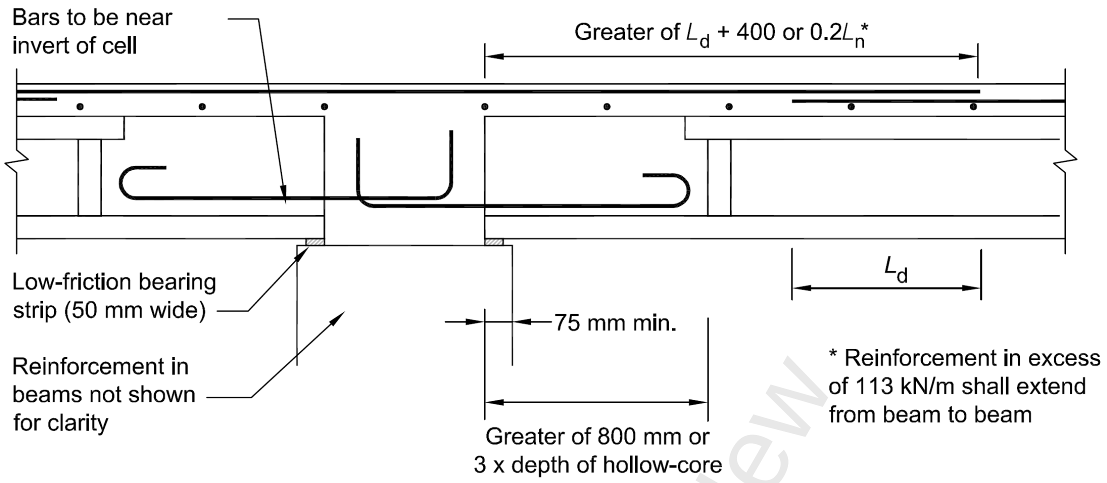


Figure C18.4 – Hollow-core reinforcing in cells on low friction bearing strips

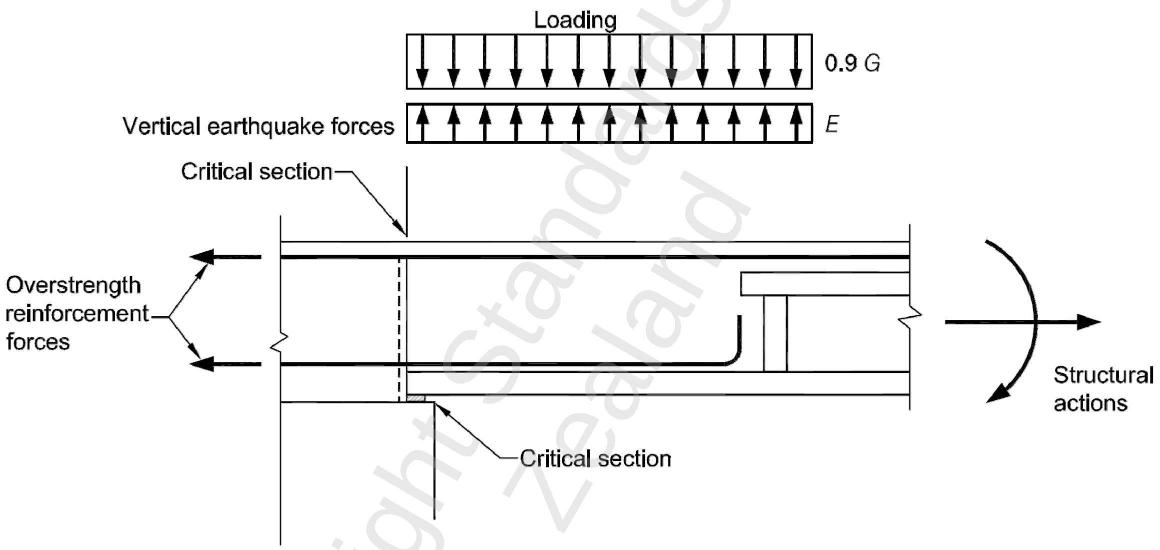


Figure C18.5 – Capacity design actions in hollow-core

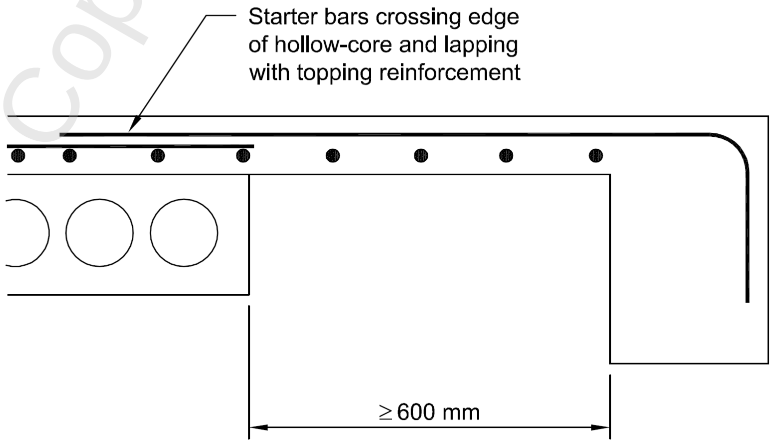


Figure C18.6 – *In situ* edge slab reinforcement

A3 **C18.6.7.2** *Precast flooring parallel to beams, walls and other structural elements*

Where a hollow-core unit runs parallel and adjacent to a beam, wall or other structural element, the potential exists for damage to occur to the brittle hollow-core unit due to a need for deformation compatibility at the interface. The local high curvatures generated in a plastic hinge generate incompatible displacements between the hollow-core unit and the beam. The local forces and stresses induced in the hollow-core unit by this relative displacement have been found to generate extensive horizontal cracking in the webs, which separate the top and bottom flanges of the unit leading to the danger of collapse[18.29]. The placing of the hollow-core unit a distance equal to the greater of 600 mm or six times the thickness of the linking slab away from the beam, as shown in Figure C18.6, is to provide a more flexible link between the two components.

Particular care should be taken with all forms of precast unit where appreciable differential vertical displacements can arise between precast units and concrete beams or structural steel beams. In particular, high localised differential displacements can arise between precast floor units and eccentrically braced steel beams of floor units running parallel to in-line adjacent structural walls.

C18.7 Connection and bearing design

C18.7.1 *Transfer of forces between members*

A3 A variety of methods are permitted for connecting precast members. These connections are intended to transfer forces both in-plane and perpendicular to the plane of the members.

C18.7.3 *Connections using different materials*

Various components in a connection (such as bolts, welds, plates, and inserts) have different properties that can affect the overall behaviour of the connection.

A3 C18.7.4 *Floor or roof members supported by bearing on a seating*

The requirements of 18.7.4 apply to all cases where load transfer from the precast unit to its support relies on seating. It may be shown by analysis or test that alternative details are acceptable. Tests should comply with the protocols outlined in AS/NZS 1170.0 Appendix A and B.

The requirements of 18.8.1 apply to ductile and limited ductile structures and to regions of nominally ductile structures which contain ductile or limited ductile potential plastic hinge regions.

Clause 18.7.4.1 requires the designer to ensure that damaging actions are not induced in supported members. Two cases are identified. The first of these relates to pretensioned units, where reinforcement in topping concrete is anchored in a supporting beam. Tension induced in this reinforcement applies an axial tension force and a negative moment to the end of the supported unit. These actions can in certain cases greatly reduce the shear strength, potentially causing a premature failure condition.

The second case relates to a rib or a web which is embedded in the supporting structure. With this detail the torsional stiffness and strength of the support may induce positive moments in the rib in the zone where the reinforcement is not developed. This may lead to a flexural crack through an un-reinforced section at low drift levels and possible structural failure of the rib. This form of failure is identified in Figure C18.7 and two ways in which this can be avoided are illustrated in Figure C18.8.

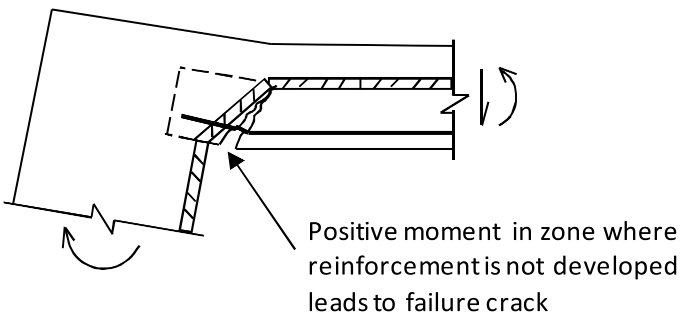
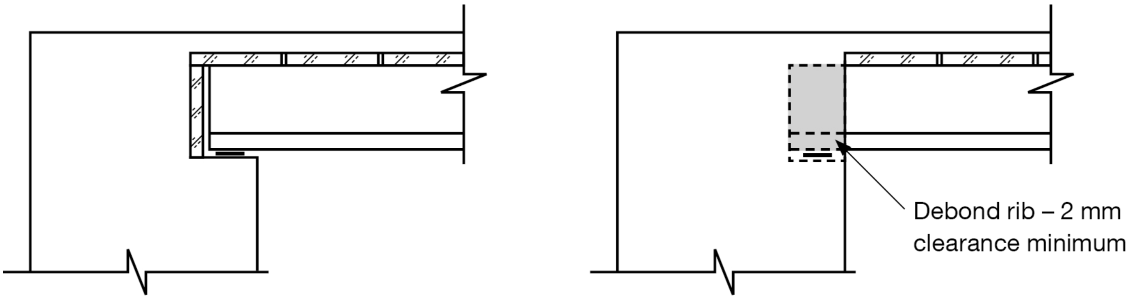


Figure C18.7 – Precast rib positive moment failure

A3



(a) Rib not encased

(b) Rib encased, debonded with a minimum of 2 mm clearance to the sides of the rib

Figure C18.8 – Precast rib not restrained by support member

In the design of the support details for these units it is essential to ensure that structurally damaging actions cannot be induced into the end of the unit or the supporting ledge where reinforcement is not effectively developed and the concrete may be unconfined.

Spalling from the face of the supporting ledge where supports were subject to relative rotation from inter-storey drift was the most commonly observed damage to the seating of precast units following the Canterbury earthquakes.

Spalling at the support of a precast unit will normally occur because there is a high point causing high localised bearing stress at an edge, or because of horizontal sliding between the precast unit and its support pulling off the unreinforced edge, or from a combination of these factors. Horizontal sliding at the support interface can be caused by shrinkage or relative rotation between the precast unit and its support. Rotation can result from seismic effects, thermal gradients, load variations etc. Transfer of high loads increases the risk of spalling and increases the horizontal friction forces. Figure C18.9 shows possible spalling from the end of the precast unit and the edge of the seating.

Relative rotation between a supported unit and the support may cause spalling to occur at both the back face of the unit and the edge of the support. Loss of support due to spalling reduces the safety of the support of the unit. The use of a low-friction bearing material between the support ledge and the precast units reduces the risk of spalling from horizontal sliding and the magnitude of the forces that act on the support. The bearing material should be set back from the edge of the support and from the end of the precast unit to reduce the likelihood of spalling at these locations.

In locating where bearing stresses may be assumed to act on a supported unit, allowance needs to be made for the potential loss of concrete by spalling from the back face of the unit and front face of the support ledge. Where the back face of the unit is not armoured an allowance of 30 mm should be made for slabs, hollowcore units, and for units supported by ribs or webs. Where the front face of the supporting member is not armoured, the allowance for spalling from the front face of a corbel or bracket is defined in 16.3 and 16.4. For support members which are not corbels or brackets, the allowance for spalling is to be taken as the greater of the cover to the longitudinal bars, or 30 mm. In this instance, the longitudinal bar is the first horizontal bar employed in the reinforcement assembly within the supporting member, which may be a trimmer bar.

The limitations on bearing stresses in Section 16 may require precast tee units with high support reactions to have armouring to the end of the tee or the edge of the support.

Consideration should be given to armouring for tee units with long spans or high loads before these limits are reached, as at these locations the concrete may be unconfined, unreinforced, or the reinforcing may not have developed. Reduction of seating as a result of spalling can have serious consequences.

A3

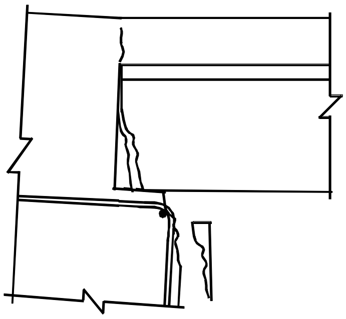


Figure C18.9 – Damage to support ledge and precast unit due to friction and rotation

Flange-supported precast tee units may have metal components cast in to transfer the load to the support. In these cases the bearing area may be less than the nominal width of the tee web, resulting in higher bearing stresses and an increased risk of spalling from the face of the support.

Where a floor is exposed to the sun, such as upper levels of carparking buildings, thermal gradients through the floor can induce significant daily rotations at the support. Particular care should be taken in these situations. The potential for damage is increased by the large number of cycles over the life of the structure.

As well as design for seismic actions such as support rotation and seating reduction caused by beam elongation, seating of precast units needs to accommodate forces and movements from a variety of other sources including longitudinal movements from thermal variations, longitudinal movement from creep and shrinkage, rotational movements from thermal gradients particularly for exposed members, and rotational movements from variations in loading. The effect of these actions becomes more significant as span lengths increase.

The amount of shrinkage occurring in concrete is affected by a number of variables, particularly the quantity of excess water in the concrete mix. Unrestrained concrete shrinkage is typically within the range of 0.5 to 1.0 mm per metre. The higher value relates to high water content and low modulus aggregates.

Precast units may have undergone some shrinkage prior to being installed. In some cases where the precast units are incorporated into a cast *in situ* structure, the shrinkage of the structure may exceed the residual shrinkage of the precast units. In these cases it may be possible to ignore shrinkage and creep for spans up to 12 m.

Precast floor and roof units that are notched at their ends to reduce the overall structural depth require care to ensure reinforcement can be developed, a verifiable load path is designed, and to provide resistance to cracking from the notch.

C18.7.5 Development of positive moment reinforcement

It is unnecessary to develop positive bending moment reinforcement beyond the ends of the precast element if the system is statically determinate.

A3

C18.7.6 Precast stairs and ramps

Stairs and ramps are to be capable of sustaining the inter-storey drifts associated with a maximum considered earthquake. Stairs should typically be detailed with a fixed top connection and sliding base connection. The friction forces at the sliding connection should be evaluated and the stair detailed to accommodate both the forces (tension/compression) and the lateral displacements (transverse movement). If the sliding end is located in a recess, steps should be taken to ensure that movement is not able to be inadvertently restricted at any stage of the building's life. Stair strings have collapsed in earthquakes due to compression forces causing bending failure at intermediate landings.

Split scissor stairs may be fixed at the floor levels and free to slide on their mid-height supporting beam. However, the horizontal friction forces should be considered in the design of the supporting beam.

Detailing should be such that maintenance contractors cannot easily fill the sliding joint. It is therefore recommended that the lower step be left to slide freely on top of the landing, as shown in Figure C18.10.

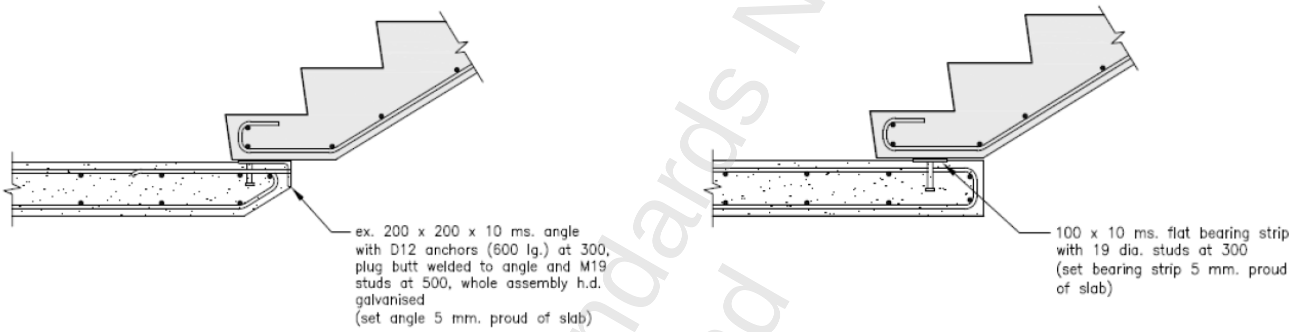


Figure C18.10 – Typical stair details

C18.8 Additional requirements for ductile structures designed for earthquake effects

C18.8.1 *Seating requirements for ductile structures*

This clause deals with the additional requirements for the support of members on corbels, brackets or walls seats that arise from potential deformation associated with ductile and limited ductile plastic regions.

An example of a calculation for a required seating length on a corbel is given below.

Referring to Figure C18.11, a rib and infill floor with a depth of 225 mm in a moment resisting ductile frame building is spanning between supports at 8 m centres. The moment resisting frame consists of 700 mm deep beams spanning between 800 mm wide columns. The seating of the floor units is 125 mm above the centre of the 700 mm deep beams forming the frame.

The details are shown in Figure C18.11 together with the critical dimensions required for the calculations.

A3

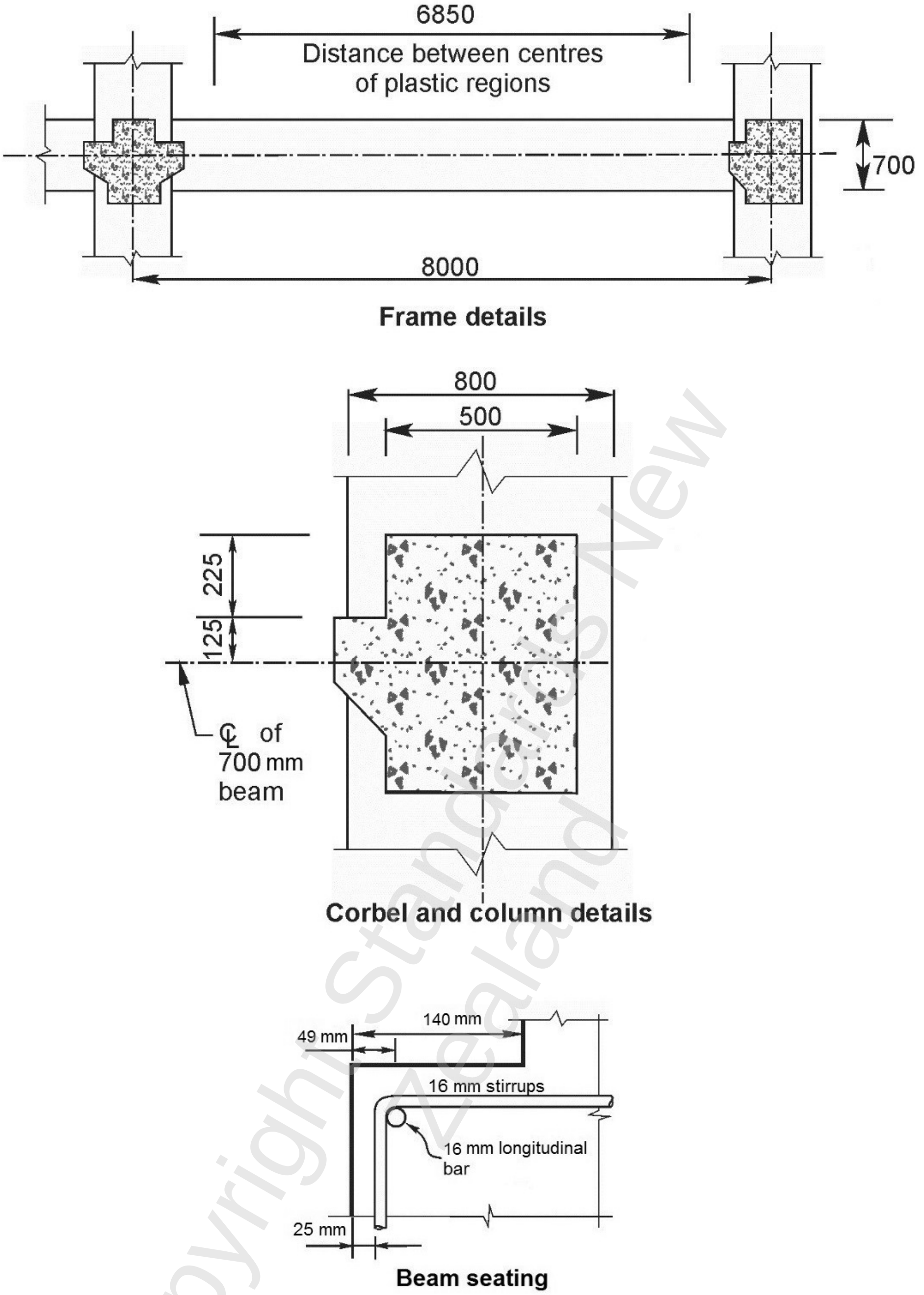


Figure C18.11 – Design seating length for support of a member in a ductile seismic frame

An analysis for ultimate limit state demands indicates that plastic hinges form in the 700 mm deep beams. The centre of rotation in a plastic hinge is taken at half the effective plastic hinge length, ℓ_p (2.6.1.3.3). In this case $\ell_p \approx 0.5h$, giving a distance between centres of the plastic hinge zones of: $8000 - 2\times\frac{800}{2} - 2\times\frac{0.5\times700}{2} = 6850$ mm. The peak plastic hinge rotation, corresponding to the ultimate limite state rotation divided by the S_p factor, is 0.0255 radians. From geometry the corresponding peak drift of the columns is 6850/8000 × 0.0255 = 0.0218 radians.

A3

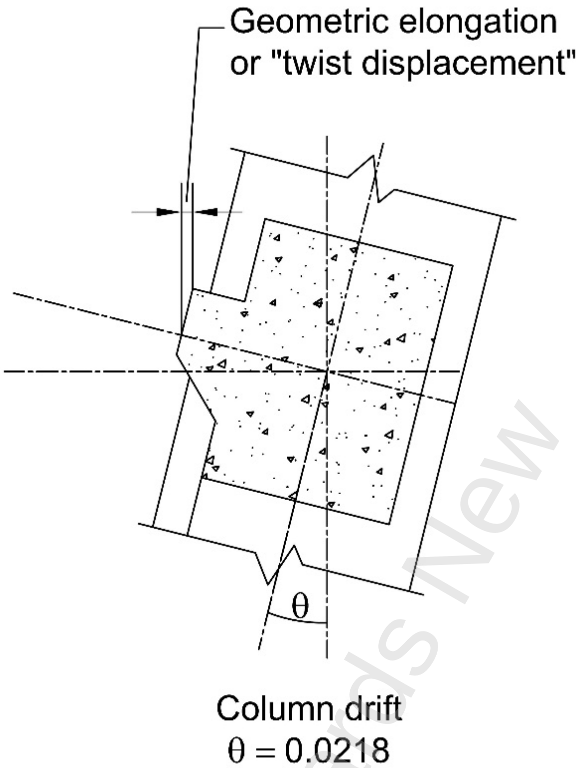


Figure C18.12 – Geometric displacement of the support

(a) For this example it is assumed that the bearing length required by 18.8.1(a) is 10 mm;

(b) In the ductile detailing length, assuming the support ledge is not armoured, the cover to stirrups is likely to be 25 mm for an interior environment (exposure classification, see A1, 3.4.2). With 16 mm stirrups, the cover to the longitudinal reinforcement of the support beam is 41 mm. Assuming 16 mm longitudinal reinforcing bars, the distance to the centreline of the bars is 25 + 16 + 16/2 = 49 mm. (Outside the ductile detailing length, the loss of support to the cover to the longitudinal bars is assumed to be 41 mm);

(c) Consider loss of support due to elongation of beams parallel to the floor that are pushing the supports further apart. Elongation at mid-depth of the 700 mm beam (calculated from Equation 7–15(b)) is 19.9 mm at peak ultimate limit state (design/S_p) assuming that (d-d') is 600 mm ($\sigma_{e\ell}$ = 2.6 × (0.0255/2) × 600 = 19.9 mm). Allowing for the maximum considered earthquake level event the elongation would be 1.5 × 19.9 ≈ 30 mm, however the maximum value is limited to 0.036h_b which is 25 mm;

(d) The elongation of 25 mm is at the mid-height of the beams, but the seating of the floor units is 125 mm above that. The peak drift of the columns is 0.0218 radians, and allowing for the maximum considered earthquake level event (drift × 1.5), the drift causes the seating to move (“twist displacement”) 0.0218 × 125 × 1.5 = 4 mm. See Figure C18.12;

(e) The end of the precast unit is not armoured so allow 30 mm loss of seating for spalling of the precast unit;

(f) Shrinkage and creep: the length of unit is 7500 mm, and 0.5 mm/m of shrinkage gives 0.0005 × 7500 = 3.75 mm. Therefore allow 4 mm;

(g) Construction tolerances are specified in NZS 3109. The distance between supports will have a tolerance of ± 10 mm and the length of the precast unit may be ± 5 mm giving a possible construction tolerance allowance of 15 mm for the bearing allowance.

Hence, 10 + (25 + 16 + 8) + 25 + 4 + 30 + 4 +15 = 137 mm (140 mm).

The total required seating length would be taken as 140 mm.

This could be reduced by 79 mm (49 + 30) if the corbel and back face of the precast unit were armoured meaning that the allowance for spalling can be discounted. However the minimum value of 75 mm from 18.7.4.3 would govern as the required seating length with the armoured unit and support.

A3 **C18.8.3.2** *Frame dilatancy*

Adequate support is considered to be provided where precast floor units meet the seating requirements of 18.8 or have hanger bars tied into the support that are sufficiently ductile to prevent collapse of the floor units at the maximum anticipated inelastic beam elongation.

Elongation (beam and geometric) due to plastic hinging in earthquake shaking can cause the columns in a reinforced concrete frame to move apart. While the frames may have recentred after the earthquake shaking has stopped there may still be a residual increase in beam length due to the accumulation of elongation, and this phenomenon is termed “frame dilatancy”.

In reinforced concrete frames, a single span of flooring (particularly precast) can span two or more bays of frames, particularly where there are intermediate columns, as is often the case in perimeter frames. Because elongation occurs at the ends of beams where plastic hinges form, there are two plastic hinges for each bay of framing. A span of floor will be subjected to elongation typically two times the number of bays that the floor spans over. It is particularly important to recognise the number of plastic hinges and the aggregating effect of beam elongation on the overall frame. Figure C18.13 shows a single span of precast flooring spanning over two bays of framing where there are four components of beam elongation. Hence a precast flooring unit in this instance needs to have sufficient seating to accommodate four components of beam elongation.

Additionally, if both ends of the precast unit are tied into the support beam (for example with continuity reinforcement) then the frame dilation is shared between each support or end. However, where one end of the unit is left free to slide the frame dilation is concentrated at one end of the unit, and consequently the reduction of seating at one end can be considerably greater than if the elongation is shared evenly between the two ends.

A3

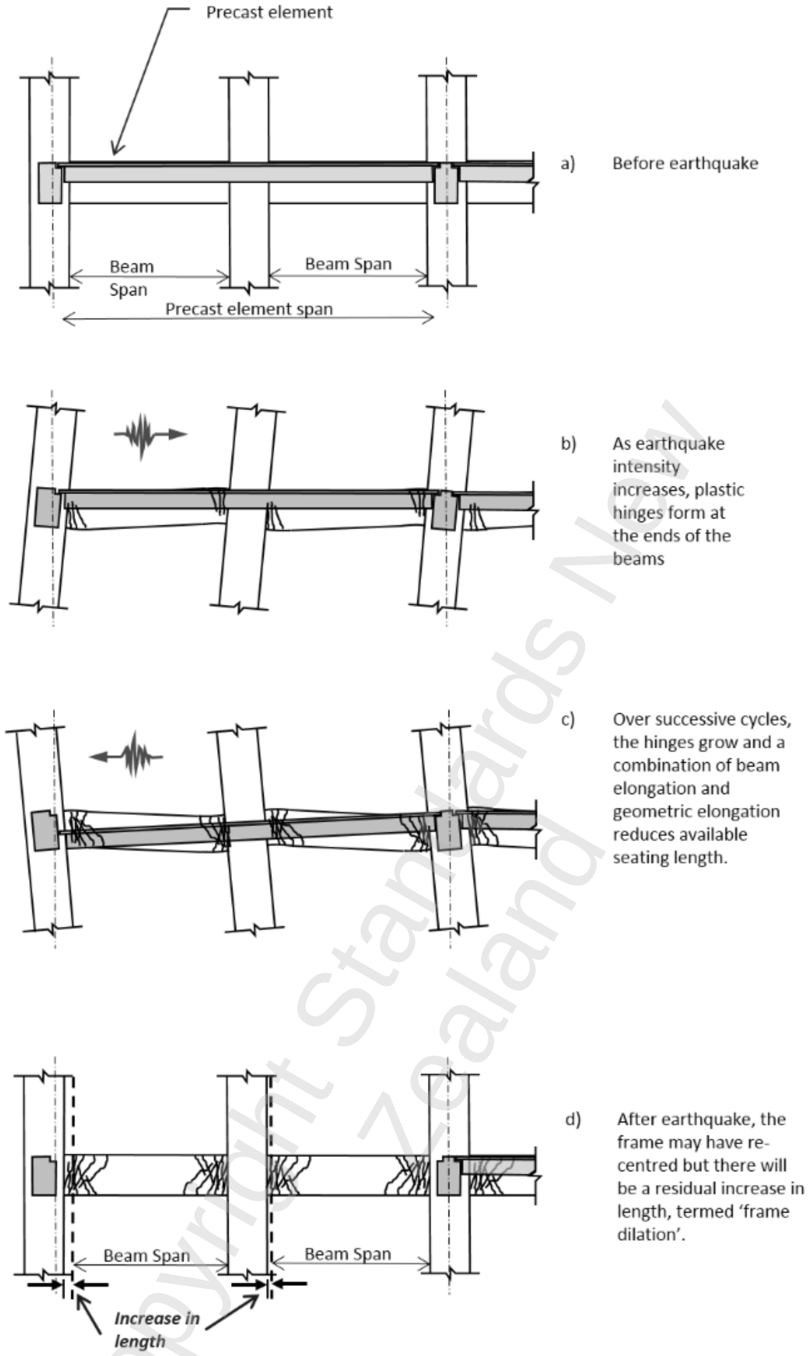


Figure C18.13 – Frame dilatancy

C18.8.3.3 *Precast shell beams construction*

Tests show that precast concrete, pretensioned beam shells, when detailed correctly, can be used in frames that need to exhibit some ductility during seismic attack[18.23, 18.24].

C18.8.3.3.1 *Length of potential plastic hinge regions in moment resisting frames*

The degradation of the bond between the cast-in-place core and the precast shell in the plastic hinge regions, during a severe earthquake, means that fully composite action should not be relied on for designing the beam flexural reinforcement in the plastic hinge regions (nominal and design flexural strengths).

When critical sections of plastic hinge regions (with tension in the bottom fibres) have been designed to occur at a distance greater than the depth of the core away from the support faces the prestressed and non-prestressed reinforcement of the precast shell can increase the flexural strength of the beam. In determining the overstrength capacity of the beam, the development lengths of the longitudinal

A3 prestressed and non-prestressed reinforcement are to be considered in the calculation of the forces in the reinforcement of the shell.

When the top fibres of the beam are in tension, it has been shown that some compression can be developed in the bottom of the shell and this has resulted in a flexural overstrength approaching that of a fully composite section[18.23]. Therefore it is considered prudent to use the fully composite actions for overstrength considerations when the top fibres of the beam are in tension.

C18.8.3.3.2 *Flexural strength in potential plastic regions*

Tests[18.23] have shown that the shell can detach from the cast-in-place core in the plastic hinge regions during severe seismic attack. Therefore the applied forces to the shell (such as gravity loads), at the plastic hinge region, need to be carried by the shell alone unless adequate reinforcing ties from the shell anchoring into the cast-in-place core are provided. Further, at plastic hinge regions in beams next to columns, the support of the shell from the columns cannot be relied on and therefore the shell may be required to cantilever from the region of the beam where composite action is provided.

C18.8.4 *Broad categories of precast concrete seismic systems*

Provisions of Section 18 are based predominantly on field and laboratory experience with monolithic reinforced concrete building structures and precast concrete building structures designed and detailed to behave like monolithic building structures. Extrapolation of these requirements to other types of cast-in-place or precast concrete structures should be based on evidence provided by field experience, tests, or analysis. ACI TI.1-01, can be used in conjunction with Section 18 to demonstrate that the strength and toughness of a proposed frame system equals or exceeds that provided by a comparable monolithic concrete system.

C18.8.4.2 *Equivalent monolithic systems*

The toughness requirements in 18.8.4.2 refer to the concern for the structural integrity of the entire lateral-force-resisting system at lateral displacements anticipated for ground motions corresponding to the design earthquake. Depending on the energy-dissipation characteristics of the structural system used, such displacements may be larger than for a monolithic reinforced concrete structure.

REFERENCES

18.1 "Guidelines for the Use of Structural Precast Concrete in Buildings", Report of a Study Group of the New Zealand Concrete Society and the New Zealand National Society for Earthquake Engineering, University of Canterbury Printery, Christchurch, New Zealand, Second edition 1999, p. 144.

18.2 "Industrialisation in Concrete Building Construction", SP-48, American Concrete Institute, Farmington Hills, MI, 1975, p. 240.

18.3 Waddell, J.J., "Precast Concrete: Handling and Erection," Monograph No. 8, American Concrete Institute, Farmington Hills, MI, 1974, p. 146.

18.4 "Design and Typical Details of Connections for Precast and Prestressed Concrete," MNL-123-88, 2nd Edition, Precast/Prestressed Concrete Institute, Chicago, 1988, p. 270.

18.5 PCI Design Handbook – "Precast and Prestressed Concrete", MNL-120-92, 4th Edition, Precast/Prestressed Concrete Institute, Chicago, 1992, p. 580.

18.6 "Design of Prefabricated Concrete Buildings for Earthquake Loads," Proceedings of Workshop, Apr. 27-29, 1981, ATC-8, Applied Technology Council, Redwood City, CA, p. 717.

18.7 PCI Committee on Building Code and PCI Technical Activities Committee, "Proposed Design Requirements for Precast Concrete," PCI Journal, Vol. 31, No. 6, Nov.-Dec. 1986, pp. 32-47.

18.8 ACI-ASCE Committee 550, "Design Recommendations for Precast Concrete Structures (ACI 550R-93)," ACI Structural Journal, Vol. 90, No. 1, Jan.-Feb. 1993, pp. 115-121. Also ACI Manual of Concrete Practice.

18.9 International Federation for Structural Concrete "State-of-the-art Report on the Seismic Design of Precast Concrete Structures" Bulletin 27, Lausanne, 2003, p. 254.

18.10 ACI Committee 551, "Tilt-up Concrete Structures (ACI 551R-92)," American Concrete Institute, Farmington Hills, MI, 1992, p. 46. Also ACI Manual of Concrete Practice.

18.11 PCI Committee on Quality Control Performance Criteria, "Fabrication and Shipment Cracks in Prestressed Hollow-core Slabs and Double Tees", PCI Journal, Vol. 28, No. 1, Jan.-Feb. 1983. pp. 18-39.

18.12 PCI Committee on Quality Control Performance Criteria, "Fabrication and Shipment Cracks in Precast or Prestressed Beams and Columns", PCI Journal, Vol. 30, No. 3, May-June 1985, pp. 24-49.

18.13 NZS 3109 – Concrete Construction.

18.14 "Recommended Practice for Erection of Precast Concrete", Prestressed Concrete Institute, Chicago, 1985, p. 87.

18.15 Saemann J.C. and Washa, George W., "Horizontal Shear Connections Between Precast Beams and Cast-in-place Slabs" ACI Journal, Proceedings Vol. 61, No. 11, Nov. 1964, pp. 1383–1409. See also discussion ACI Journal June 1965.

18.16 Concrete Technology Associates, Technical Report 74 B6 "Composite Systems Without Roughness", Tacoma, Washington.

18.17 Scott, Norman L., "Performance of Precast Prestressed Hollow Core Slab with Composite Concrete Topping", PCI Journal, March-April 1973, pp. 64-77-2728.

18.18 Seible F., Latham C., and Krishnan K., "Structural Concrete Overlays in Bridge Deck Rehabilitation – Summary of Experimental Results, Analytical Studies and Design Recommendations", University of California, San Diego, Department of Structural Engineering Report No. SSPR-88/04, June 1988, 83 pp.

18.19 Seible F. and Latham C.T., "Horizontal Load Transfer in Structural Concrete Bridge Deck Overlays", ASCE Journal of Structural Engineering, Vol. 116, No. 10, October 1990, pp. 2691-2710.

18.20 Seible F. and Latham C.T., "Analysis and Design Models for Structural Concrete Bridge Deck Overlays, ASCE Journal of Structural Engineering, Vol. 116, No. 10, October 1990, pp. 2711-2728.

18.21 Mattock, A.H., "Anchorage of Stirrups in Thin Cast-in-place Topping", PCI Journal, Nov.-Dec. 1987, pp. 70-85.

18.22 Mast, Robert F., "Auxiliary Reinforcement in Concrete Connections," Proceedings, ASCE Vol. 94, No. ST6, June 1968, pp. 1485-1504.

18.23 Bull, D.K. and Park, R., "Behaviour of Structural Concrete Frames with Precast Concrete Beam Shells Subjected to Seismic Loading". Dept. of Civil Eng. University of Canterbury, 1984.

18.24 Park, R. and Bull, D.K., "Seismic Resistance of Frames Incorporating Precast Prestressed Concrete Beam Shells", PCI Journal, July-August 1986, pp. 54-93.

18.25 "Design of Concrete Structures for Buildings", CAN3-A23.3-M84, and "Precast Concrete Materials and Construction", CAN3-A23.4-M84, Canadian Standards Association, Rexdale, Ontario.

18.26 PCI Committee on Precast Concrete Bearing Wall Buildings, "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads", PCI Journal, Vol. 21, No. 2, March-April 1976, pp. 18-51.

18.27 Salmons, J.R. and McCrate, Timothy E., "Bond Characteristics of Untensioned Prestressing Strand", PCI Journal, Vol. 22, No. 1, January-February 1977, pp. 52-65.

18.28 Fenwick, R., Deam, B and Bull D, "Failure modes for hollowcore flooring units" SESOC Journal, Vol.17, No. 1, April 2004, pp 52-70.

18.29 Fenwick, R.C., Bull, D.K., Gardiner, D.R., "Assessment of Hollow-Core Floors for Seismic Performance", Research Report 2010-02, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand, 2010. A3

NOTES

C19 PRESTRESSED CONCRETE

C19.1 Notation

The following symbols which appear in this section of the Commentary, are additional to those used in Section 19 of Part 1.

B	bursting force in a prestressed anchorage zone, N
B_o	overall bursting force due to a group of post-tension anchorages, N
B_ℓ	local bursting force to post-tension anchorage, N
d'	distance from extreme compression fibre to centroid of compression reinforcement, mm
e	eccentricity of post-tensioned cable from centroid, mm
ℓ	span length, mm
ℓ_a	lever-arm used for calculation of bursting force in an anchorage zone for a post-tensioned cable, mm
M_n	nominal bending strength at the ultimate limit state, N mm
M_o	bending moment resisted at decompression of extreme tension fibre, mm
P	prestressing force in a tendon or tendons, N
S_b	spalling force in anchorage zone of post-tensioned cables, N
S_c	spalling force in anchorage zone due to compatibility, N
S_v	spalling force due to inclination of post-tensioned cable in anchorage zone, N
V_o	shear force resisted at decompression of extreme tension fibre, N
x	the distance from the section being investigated to the support, mm

C19.2 Scope

C19.2.1 *General*

The provisions of Section 19 were developed primarily for structural members such as slabs, beams and columns that are commonly used in buildings. Many of the provisions may be applied to other types of construction, such as bridges, pressure vessels, pipes, etc. Application of the provisions is left to the judgement of the engineer in cases not specifically cited in the code.

C19.2.2 *Other provisions for prestressed concrete*

Some sections of the Standard are excluded from use in the design of prestressed concrete for specific reasons. The following discussion provides explanation for such exclusions.

Clause 8.3.5 of the Standard is excluded from application to prestressed concrete because the requirements for bonded reinforcement and unbonded tendons for cast-in-place members are provided in 19.3.6.6 and 19.3.6.7.

The empirical provisions of 9.3.1.2, 9.3.1.3 and 9.3.1.4 for T-beams were developed for non-prestressed reinforced concrete, and if applied to prestressed concrete would exclude many standard prestressed products in satisfactory use today. Hence, proof by experience is considered to permit variations. By excluding these clauses there are no special requirements for prestressed concrete T-beams in the Standard. Instead, the determination of an effective width of flange is left to the experience and judgement of the engineer. Where possible, the flange widths in 9.3.1.2(a) and (b) should be used unless experience has proved that variations are safe and satisfactory. It is not necessarily conservative in elastic analysis and design considerations to use the maximum flange width as permitted in 9.3.1.2(a). Nevertheless, the requirement that the flange and web be built integrally or otherwise effectively bonded together is applicable to prestressed concrete T-beams.

For prestressed concrete, the limitations on reinforcement given in 9.3.8.2 and 10.3.8.1 are replaced by those in 19.3.6.6, 19.3.6.7 and 19.3.10.

Clause 9.3.6 does not apply to prestressed members in its entirety. However, 19.3.3.5.3 is used to control cracking in Class C prestressed flexural members.

In the design of continuous prestressed concrete slabs secondary moments need to be recognised and allowance made for them. Also, volume changes due to creep and shrinkage can create additional loads on a structure that are not adequately covered in Section 12. Because of these unique properties associated with prestressing, many of the design procedures of Section 12 are not appropriate for prestressed concrete structures and are replaced by the provisions of 19.3.10.

Some of the requirements in 11.3 and 11.4 for wall design, are largely empirical, utilising considerations not intended to apply to prestressed concrete.

C19.3 General principles and requirements

C19.3.1.1 Design requirements

The design investigation should include all stages that may be significant. The three major stages are:

(a) Jacking stage, or prestress transfer stage – when the tensile force in the prestressing steel is transferred to the concrete and stress levels may be high relative to concrete strength;

(b) The serviceability limit state stage when service load is applied – after long-term volume changes have occurred; and

(c) The ultimate limit state stage when the ultimate limit state load is applied – when the strength of the member is checked. There may be other stages that require investigation. For example, if the cracking load is significant, this load stage may require study, or the handling and transporting stage may be critical.

From the standpoint of satisfactory behaviour, the two stages of most importance are the serviceability limit state and the ultimate limit state.

C19.3.1.3 *Secondary prestressing moments*

When an indeterminate prestressed concrete structure is prestressed, bending moments are induced by reactions resulting from the application of the prestress forces. These bending moments are generally referred to as secondary prestress moments. The magnitude of these moments depends upon the tendon profile and the member stiffness. They are important at the serviceability limit state and must be included in calculations. However, at the ultimate limit state the stiffness is greatly reduced, due to the flexural cracking and the non-linear behaviour of the concrete and reinforcement. As a consequence the secondary moments are reduced and, provided the ultimate flexural strength is limited by a ductile failure mechanism, they generally have a negligible effect on strength requirements. However, as rotation occurs in the plastic regions with the reduction of secondary moments, they may in some cases limit the permitted magnitude of redistributed bending moment (see 19.3.9). As shear failure may occur in a brittle manner and it can occur before any moments are redistributed, critical shear force moment combinations should consider any adverse load cases that may arise both with and without redistribution of moments, including secondary moments.

C19.3.1.5 *Possibility of buckling*

This refers to the type of post-tensioning where the tendon makes contact with the prestressed concrete member intermittently. Precautions should be taken to prevent buckling of such members. In particular, if thin webs or flanges are under high pre-compression, buckling is possible between supports of slender members. If the tendon is in complete contact with the member being prestressed, or is an unbonded tendon in a duct not excessively larger than the tendon, buckling the member as a whole when the prestressing force is introduced is not possible, but thin flanges should be checked for local buckling.

C19.3.1.6 *Section properties*

In considering the area of the open ducts, the critical sections should include those that have coupler sheaths that may be of a larger size than the duct containing the prestressing steel. Also, in some instances, the trumpet or transition piece from the conduit to the anchorage may be of such a size as to create a critical section. If the effect of the open duct area on design is deemed negligible, section properties may be based on total area.

In post-tensioned members after grouting and in pretensioned members, section properties may be based on effective sections using transformed areas of bonded prestressing steel and non-prestressed reinforcement in the section. Alternatively, gross sectional areas may be used but it should be noted that the use of gross section properties can, in some cases, lead to significant errors arising in stress and deflection calculations (see Appendix CE).

C19.3.1.7 *Tendons deviating from straight lines*

The deviation of cables from a straight line causes forces, which may result in damage if there is inadequate cover or resistance.

C19.3.1.8 *Reinforcement for shrinkage and temperature stresses*

In large prestressed concrete members, such as box girders, and where prestressing is remote from the faces of the member, supplementary reinforcement should be provided at the faces in the direction of the prestressing to control random cracking. Such cracking may be initiated by differential thermal conditions, and different creep and shrinkage characteristics in the different elements making a section (see Reference 19.23).

C19.3.1.9 *Stress concentrations*

Stress concentrations, which can lead to cracking, can arise where inserts or ducts are formed in prestressed members. Stress concentrations also arise in anchorage zones of prestressing tendons (see 19.3.13).

C19.3.1.10 Unbonded tendons

Unbonded tendons may be used providing that they are adequately protected against corrosion, that the exposure conditions are not inappropriately harsh, that cracking is controlled by bonded reinforcement, and that the serviceability and ultimate limit state requirements are met.

In seismic design [19.1] there are advantages in using unbonded tendons with non-prestressed reinforcing steel since such structures are self-centering after an earthquake (that is, the residual displacement is negligible) and the structure remains mainly undamaged. An important requirement in seismic design using unbonded tendons is that the anchorages must be able to withstand the fluctuations in tendon stress that will occur during an earthquake.

C19.3.2 *Classification of prestressed members and sections*

This clause defines three classes of behaviour of prestressed flexural members. Class U members are assumed to behave as uncracked members. Class C members are assumed to behave as cracked members. The behaviour of Class T members is assumed to be in transition between uncracked and cracked. The serviceability requirements for each class are summarised in Table C19.1. For comparison, Table C19.1 also shows corresponding requirements for non-prestressed members.

These classes apply to both bonded and unbonded prestressed flexural members. Two-way slab systems in buildings, in which uniformly distributed loading is critical, should be designed as class U. A2

The precompressed tensile zone is that portion of the member cross section in which flexural tension occurs under dead and live loads. Prestressed concrete is usually designed so that the prestress force introduces compression into this zone, thus effectively reducing the magnitude of the tensile stress.

C19.3.3 *Serviceability limit state requirements – flexural members*

C19.3.3.1 *General*

A method for computing stresses in a member containing flexural cracks is given in Reference 19.2.

Reference 19.3 provides information on computing deflections of cracked members.

C19.3.3.3 *Section properties*

Using gross section properties to calculate stresses and deflections is generally acceptable. However, it should be noted that appreciable errors can arise, particularly where high reinforcement contents are used, or the reinforcement (prestressed and non-prestressed) is highly eccentric in the section. For class C members transformed section properties must be used.

For class C members transformed cracked section properties must be used for calculating stresses but allowance may be made for tension stiffening in assessing deflections.

C19.3.3.5 *Permissible stresses in concrete*

A2 Permissible stresses in concrete address serviceability. Adherence to these does not ensure adequate structural strength, which should be checked in accordance with other requirements in the standard.

The permissible stresses immediately after transfer are calculated allowing for losses due to elastic shortening, relaxation of prestressing steel which may have occurred before transfer, seating at transfer and stresses due to self weight of the member. Generally shrinkage and creep effects are not included at this stage, but they may need to be considered in post-tensioned concrete if an appreciable amount of shrinkage can occur before pre-stressing is applied.

For class U and T members, the permissible stress limit of $0.45 f_c'$ for long-term loading was set to prevent excessive deformation due to creep in the concrete and to provide protection against fatigue failure. For short-term loading, traffic over load for bridges or total service live load for buildings, the allowable compression stress is increased by 33 % to $0.6 f_c'$, as short-term loading has little influence on creep and the occasional overload has little influence on fatigue of concrete.

Differential temperature conditions induce high local compression stresses in the top exposed surface of a member. However, these stresses decrease rapidly with distance from the surface and any local inelastic deformation in these fibres would have no significant influence on either the strength or stiffness of the member. Consequently, under these loading conditions a higher local compression stress in acceptable.

For class C members the compression stresses in the extreme fibres are increased by a few percent to give a comparable performance to T members. The increased compression stress levels correspond to the stress levels that could be expected in T members, which are stressed to their maximum permissible level based uncracked section properties when flexural cracks form.

Part (b) in 19.3.3.5.2 limits the maximum tensile stress sustained in concrete immediately after transfer to the stated values, unless additional reinforcement is provided as specified in (c). This reinforcement is required to control any potential cracks. Part (c) indicates that this reinforcement is required to be proportioned to sustain a tension force equal to the tension force resisted by the concrete if no cracking occurs. The allowable stress level in this reinforcement is limited to the values given in part (c) of the clause.

Table C19.1 – Summary of serviceability limit state design requirements

A2

	Prestressed			**Non-prestressed**
	Class U	Class T	Class C	
Assumed behaviour	Uncracked	Transition between cracked and uncracked	Cracked	Cracked
Section properties for stress calculation at service loads	Gross or transformed section 19.3.3.3	Gross or transformed section 19.3.3.3	Section cracked transformed 19.3.3.3	Cracked section transformed
Allowable stress at transfer	19.3.3.5.1 19.3.3.5.2	19.3.3.5.1 19.3.3.5.2	19.3.3.5.1 19.3.3.5.2	No requirement
Allowable compression stress service loads	19.3.3.5.1	19.3.3.5.1	19.3.3.5.1	No requirement
Allowable tensile stress service loads	19.3.2	19.3.2	No requirement	No requirement
Deflection calculation basis	Gross or transformed section 6.8.4	Gross or transformed section 6.8.4	Cracked transformed section and effective inertia, 6.8.4 and 6.8.3	Cracked transformed section and effective inertia, 6.8.3
Crack control	No requirement	No requirement	19.3.3.5.3	2.4.4
Computation of Δf_s or f_s for crack control	No requirement	No requirement	Cracked transformed section	Cracked transformed section , or $0.6f_y$
Side skin reinforcement	19.3.3.7, with 2.4.4	19.3.3.7, with 2.4.4	19.3.3.7, with 2.4.4	2.4.4

C19.3.3.5.3 Crack widths for Class C members

Spacing requirements in 19.3.3.5.3(b) have been adapted from the 2002 edition of the ACI Building Code and reference 19.4. It should be noted crack widths will increase in cases where there is repetitive loading. In such situations a conservative approach should be taken to the spacing of reinforcement given by Equations 19–1 and 19–2.

For conditions of corrosive environments, defined as an environment in which chemical attack (such as seawater, corrosive industrial atmosphere, or sewer gas) is encountered, cover greater than that required by 3.11 should be used, and tension stresses in the concrete reduced to eliminate possible cracking at service loads. The engineer should use judgement to determine the amount of increased cover and whether reduced tension stresses are required.

For post-tensioned members designed as cracked members it is usually advantageous to provide crack A2 control by the use of deformed reinforcement, for which the provisions of 19.3.3.5.3(b) may be used directly. Bonded reinforcement required by other provisions of this standard may also be used as crack control reinforcement.

A2 In checking the expression in (b) and (c) only tension steel nearest the tension face needs to be considered in selecting the value of c_c, or g_s to be used in computing spacing requirements. To account for prestressing steel, such as strand, having bond characteristics less effective than deformed reinforcement, the k_b factor is introduced.

A2 In using the method in (c) to assess crack widths it should be noted that an accurate determination of crack widths is not possible. In particular, repetitive loading is likely to increase crack widths to values greater than those indicated by the equations.

A2 The method in (c) assumes that the approach for reinforced concrete given in 2.4.4.6 can also be adapted to prestressed concrete. However, in deriving the coefficients for the equations in 2.4.4.6 no allowance was made for initial compression stresses induced in reinforcement due to shrinkage of the concrete. Shrinkage and creep strains in prestressed concrete have a greater effect than in reinforced concrete. However, to use the reinforced concrete crack width equation for prestressed concrete it was felt that some adjustment should be made for shrinkage effects in the test beams. Based on judgement, with an assumed shrinkage strain of the order of 300 x 10^{-6} in the test beams, an allowance of 50 MPa initial compression has been assumed, hence the value of (Δf_s – 50) is used in place of f_s with prestressed concrete.

The value of Δf_s is the change in stress in the reinforcement, either prestressed or non-stressed, from the stress sustained when the surrounding concrete is decompressed (zero stress), f_{dc}, to the stress sustained under the serviceability load stage being considered. Creep and shrinkage strains in concrete reduce tensile stresses in prestressed reinforcement (as in loss of prestress) and induce compressive stresses in non-prestressed reinforcement. Consequently the value of f_{dc} for non-prestressed reinforcement can be found from the strain profile in the section prior to cracking and the strain change in prestressed (or other) reinforcement due to creep and shrinkage. For example, if the loss of prestress due to shrinkage is 70 MPa at a prestressed tendon, and due to creep it is 60 MPa, the value of the stress in the reinforcement at the same level at decompression, f_{dc} is 130 MPa in compression for reinforcement at the same level as the prestressed reinforcement. If the reinforcement is to work at a stress of 150 MPa in tension then (Δf_s – 50) is equal to 230 MPa, and this value is used for assessing the crack width at the level of the tendon.

The maximum limitation of 250 MPa for Δf_s is imposed as high changes in strain in the reinforcement A2 may break down bond and increase crack widths. With Δf_s values of 150 MPa or less only fine cracks are induced and consequently no further check on crack widths is required.

A2

C19.3.3.6.2 Permissible service load stress ranges in prestressed and non-prestressed reinforcement

The Standard does not distinguish between temporary and effective prestressing steel stresses. Only one limit on prestressing steel stress is provided because the initial prestressing steel stress (immediately after transfer) can prevail for a considerable time, even after the structure has been put into service. This stress, therefore, should have an adequate safety factor under service conditions and cannot be considered as a temporary stress. Any subsequent decrease in prestressing steel stress due to losses can only improve conditions and no limit on such stress decrease is provided in the Standard.

Maximum stresses in tendons recognise the higher yield strength of low-relaxation wire and strand meeting the requirements of the standards listed in 19.2.1. For such tendons, it is more appropriate to specify permissible stresses in terms of specified minimum yield strength. For the low-relaxation wire and strands, with f_{py} equal to 0.90 f_{pu}, the 0.94 f_{py} and 0.82 f_{py} limits are equivalent to 0.85 f_{pu} and 0.74 f_{pu}, respectively.

The higher yield strength of the low-relaxation tendons does not change the effectiveness of tendon anchorages; thus, the maximum stress at post-tensioning anchorages (and couplers) is not increased above the value of 0.70f_{pu}, For ordinary tendons (wire, strands, and bars) with f_{py} equal to 0.85f_{pu}, the 0.94f_{py} and 0.82f_{py} limits are equivalent to 0.80f_{pu} and 0.70f_{pu}, respectively. For bar tendons with f_{py} equal to 0.80 f_{pu}, the same limits are equivalent to 0.75 f_{pu} and 0.66 f_{pu}, respectively.

Designers should be concerned with setting a limit on final stress when the structure is subject to corrosive conditions or repeated loadings.

C19.3.3.7 *Reinforcement on sides of beams* A2

Longitudinal skin reinforcement is required in beams with a depth of 1 metre or more in the flexural tension zone (for ultimate limit state) to control crack widths and prevent an excessive loss of shear strength, see C9.3.9.3.4.

C19.3.4 *Loss of prestress in tendons*

C19.3.4.1 *General*

Prestress losses may be expected to vary substantially for different applications. Although the actual loss will have little effect on the design strength of the member, it will affect serviceability limit state stresses and behaviour, such as deflection, camber and cracking load. These aspects can control the design. Methods of computing losses are given in References 19.5, 19.6, 19.7 and 19.8 and in Appendix CE.

To determine effective prestress f_{se}, allowance for the following sources of loss of prestress shall be considered:

(a) Immediate loss of prestress resulting from:
- (i) Elastic shortening of concrete;
- (ii) Friction loss due to intended or unintended curvature in post-tensioning tendons;
- (iii) Prestressing steel seating at transfer during anchoring;

(b) Time dependent loss of prestress resulting from:
- (i) Shrinkage of concrete;
- (ii) Creep of concrete;
- (iii) Relaxation of prestressing steel stress.

C19.3.4.2 *Loss of prestress due to creep and shrinkage*

The loss of prestress due to creep and shrinkage and elastic shortening of concrete may be calculated from the modified effective modulus method, see Appendix CE.

C19.3.4.2.3 and **C19.3.4.2.4** *Loss of prestress due to friction* and *Determination of losses*

Loss due to friction along post-tensioned tendons during prestressing occurs due to curvature friction and wobble friction. Curvature friction results from bends or curves in the specified tendon profile. Wobble friction results from unintended deviation of prestressing sheath or duct from its specified profile. The friction curvature coefficients and wobble coefficients recommended for Equation 19–3 give a range that generally can be expected [19.9] for ducts and sheaths. Plastic ducts will lead to smaller values for these coefficients. Friction loss should be based on experimentally determined curvature and wobble friction coefficients and should be verified during tendon stressing. Values of curvature and wobble friction coefficients used in design should be shown on the design drawings.

When safety or serviceability of the structure may be involved, the acceptable range of prestressing steel jacking forces or other limiting requirements should either be given or approved by the structural engineer in conformance with the permissible stresses of 19.3.3.5 and 19.3.3.6.

C19.3.4.3.2 and **C19.3.4.3.3** *Loss of prestress due to shrinkage and creep of the concrete*

Texts on prestressed concrete, or on creep and shrinkage in concrete, give many different ways of calculating prestress loss. The method illustrated in Appendix CE is one of the simpler methods, which appears to give realistic predictions.

For the restricted case where reinforcement is distributed throughout the member so that its effect on shrinkage is mainly axial, the loss of prestress in the tendons due to shrinkage of concrete may be assessed as $(E_p \varepsilon_{cs}) / (1 + 15 A_s/A_g)$.

C19.3.4.3.4 Loss of prestress due to tendon relaxation

For the purposes of preliminary design prior to the selection of a specific system, the following values of R_b may be used:

(a) Normal relaxation prestressing strand .... ≤ 7 %
(b) Low relaxation prestressing strand and wire .... ≤ 2.5 %
(c) Prestressing bar.. ≤ 4 %.

The loss of stress due to relaxation of prestressing tendon is equal to $R_{sc}f_{pi}$.

C19.3.6 ***Flexural strength of beams and slabs***

C19.3.6.2 *Nominal flexural strength*

Design moment strength of prestressed flexural members may be computed using strength equations similar to those for non-prestressed concrete members. When part of the prestressing steel is in the compression zone, a method based on applicable conditions of equilibrium and compatibility of strains at a factored load condition should be used.

For other cross sections, the design moment strength ϕM_n is computed by an analysis based on stress and strain compatibility using the stress-strain properties of the prestressing steel and the assumptions given in 7.4.

C19.3.6.3 *Strain compatibility analysis*

A2 A number of appropriate stress strain relationships for concrete and reinforcement may be found in the literature. In particular stress strain relationships for concrete may be found in Reference 19.10 and for the prestressed reinforcement in Reference 19.11

C19.3.6.4 *Alternative method*

Equation 19–7 may underestimate the flexural strength of beams with high percentages of reinforcement and, for more accurate evaluations of their strength, the strain compatibility and equilibrium method should be used. Use of Equation 19–7 is appropriate when all of the prestressed reinforcement is in the tension zone. When part of the prestressed reinforcement is in the compression zone, a strain compatibility and equilibrium method should be used.

By inclusion of the ω' term, Equation 19–7 reflects the increased value of f_{ps} obtained when compression reinforcement is provided in a beam with a large reinforcement index. When the term $[p_p f_{pu}/f'_c + (d/d_p)(\omega - \omega')]$ in Equation 19–7 is small, the neutral axis depth is small, the compressive reinforcement does not develop its yield strength, and Equation 19–7 becomes unconservative. This is the reason why the term $[p_p f_{pu}/f'_c + (d/d_p)(\omega - \omega')]$ in Equation 19–7 may not be taken as less than 0.17 if compression reinforcement is taken into account when computing f_{ps}. If the compression reinforcement is neglected when using Equation 19–7, ω' is taken as zero, then the term $[p_p f_{pu}/f'_c + (d/d_p)(\omega)$ may be less than 0.17 and an increased and correct value of f_{ps} is obtained.

When d' is large, the strain in compression reinforcement can be considerably less than its yield strain. In such a case, the compression reinforcement Equation 19–7 does not influence f_{ps} as favourably as implied by Equation 19–7. For this reason, the applicability of Equation 19–7 is limited to beams in which d' is less than or equal to $0.15d_p$.

The term $[p_p f_{pu}/f'_c + (d/d_p)(\omega - \omega')]$ in Equation 19–7 may also be written $[p_p f_{pu}/f'_c + A_s f_y/(bd_p f'_c)]$. This form may be more convenient, for instance when there is no non-prestressed tension reinforcement.

Equation 19–9 reflects results of tests on members with unbonded tendons and span-to-depth ratios greater than 35 (one-way slabs, flat plates, and flat slabs) [19.12, 19.13]. These tests also indicate that Equation 19–8 overestimates the amount of stress increase in such members. Although these same tests indicate that the moment strength of those shallow members designed using Equation 19–8 meets the ultimate limit state load strength requirements, this reflects the effect of the requirements for minimum bonded reinforcement, as well as the limitation on concrete tensile stress that often controls the amount of prestressing force provided.

C19.3.6.5 *Non-prestressed reinforcement*

As well as deformed reinforcing bars the use of lengths of unstressed strand or wire offcuts is permitted to increase the flexural tensile strength providing it can be developed in accordance with Section 8. Where reinforcement is used to carry compression it must be adequately restrained against buckling and strands should not be used for this purpose.

C19.3.6.6 *Limits for longitudinal reinforcement*

C19.3.6.6.2 *Limiting neutral axis depth*

The limiting neutral axis depth is to ensure that sections have some ductility at the flexural strength using limiting strains of 0.003 and 0.0044 is consistent with the corresponding values of beams designed with Grade 500 reinforcement.

C19.3.6.6.3 *Minimum cracking moment*

This provision is a precaution against abrupt flexural failure developing immediately after cracking. A flexural member designed according to standard provisions requires considerable additional deflection beyond cracking to reach its flexural strength. This considerable deflection would warn that the member strength is approaching. If the flexural strength were reached shortly after cracking, the warning deflection would not occur.

C19.3.6.6.4 *Placement of bonded reinforcement*

Bonded steel when required should be placed near the tension face of prestressed flexural members. The purpose of this bonded steel is to control cracking under full service loads or overloads.

C19.3.6.7 *Minimum bonded reinforcement*

C19.3.6.7.1 *Minimum bonded reinforcement in members with unbonded tendons*

Some bonded reinforcement is required by the standard in members prestressed with unbonded tendons to ensure adequate flexural performance at ultimate member strength, rather than performance as a tied arch, and to limit crack width and spacing at service load when the flexural tensile strength of the concrete is exceeded. Providing the minimum bonded reinforcement as stipulated in 19.3.6.7.1 and 19.3.6.7.2 helps to ensure adequate performance.

Research has shown that unbonded post-tensioned members do not inherently provide large capacity for energy dissipation under severe earthquake loadings because the member response is primarily elastic. For this reason, unbonded post-tensioned structural elements reinforced in accordance with the provisions of this section should be assumed to carry only vertical loads, or to act as horizontal diaphragms between energy dissipating elements under earthquake load forces. The minimum bonded reinforcement areas required by Equation 19–10 are absolute minimum areas independent of grade of steel or design yield strength. Where seismic actions may arise, including vertical seismic forces, the area of reinforcement should be increased to ensure adequate ductility.

The minimum amount of bonded reinforcement for members other than two-way flat slab systems is based on research comparing the behaviour of bonded and unbonded post-tensioned beams [19.14]. Based on this research it is advisable to apply the provisions of 19.3.6.7.1 also to one-way slab systems.

C19.3.6.7.2 *Minimum bonded reinforcement in two-way flat slab systems with unbonded tendons*

The minimum amount of bonded reinforcement in two-way flat slab systems is based on reports by ACI-ASCE Committee 423[19.5, 19.12]. Limited research available for two-way flat slabs with drop panels [19.15] indicates that behaviour of these particular systems is similar to the behaviour of flat plates. Reference 19.16 was revised by Committee 423 in 1983 to clarify that 19.3.7.3 applies to two-way flat slab systems in the following cases:

(a) For usual loads and span lengths, flat plate tests summarised in the Committee 423 report [19.5] and experience since the 1963 ACI Building Code [19.17] was adopted indicate satisfactory performance without bonded reinforcement in the areas described in 19.3.6.7.2(a);

(b) In positive moment areas, where the concrete tensile stresses are between $0.17\sqrt{f'_c}$ and $0.5\sqrt{f'_c}$ a minimum bonded reinforcement area proportioned according to Equation 19–11 is required. The tensile force N_c is computed at service load on the basis of an uncracked, homogeneous section;

(c) Research on unbonded post-tensioned two-way flat slab systems evaluated by ACI-ASCE Committee 423 [19.2, 19.5, 19.12, 19.18] shows that bonded reinforcement in negative moment regions, proportioned on the basis of 0.075 % of the cross-sectional area of the slab-beam strip, provides sufficient ductility and reduces crack width and spacing. To account for different adjacent tributary spans, Equation 19–12 is given on the basis of an equivalent frame. For rectangular slab panels, Equation 19–12 is conservatively based upon the larger of the cross-sectional areas of the two intersecting equivalent frame slab-beam strips at the column. This ensures that the minimum percentage of steel recommended by research is provided in both directions. Concentration of this reinforcement in the top of the slab directly over and immediately adjacent to the column is important. Research also shows that where low tensile stresses occur at service loads, satisfactory behaviour has been achieved at ultimate loads without bonded reinforcement. However, the standard requires minimum bonded reinforcement regardless of service load stress levels to help ensure flexural continuity and ductility, and to limit crack widths and spacing due to overload, temperature, or shrinkage. Research on post-tensioned flat plate-to-column connections is reported in References 19.16, 19.19, 19.20, 19.21 and 19.22.

C19.3.6.7.3 *Lengths of bonded reinforcement*

Bonded reinforcement should be adequately anchored to develop ultimate load forces. The requirements of Section 8 will ensure that bonded reinforcement required for flexural strength under ultimate loads, will be adequately anchored to develop tension or compression forces. The minimum lengths apply for bonded reinforcement required by 19.3.6.7.1 or 19.3.6.7.2, but not required for flexural strength in accordance with 19.3.6.5. Research [19.2] on continuous spans shows that these minimum lengths provide adequate behaviour under service load and factored load conditions.

C19.3.7.2 *Axial load limit*

The tensile strains in the prestressing tendons will generally be greater than the limiting compression strain of 0.003 in the concrete. Consequently the remaining tensile stress in the tendons will generally reduce the value of $N_{n,max}$.

C19.3.8.1 *General*

With statically indeterminate structures, bending moments may be induced by reactions arising from prestressing forces. These moments are referred to as secondary moments. Along with other self strain actions, such as differential temperature forces in bridge structures and roofs of buildings due to solar radiation, they are important in the serviceability limit state. However, the magnitude of self strain actions decreases as the member stiffness reduces. Consequently for ductile structures, where the stiffness tends to zero at collapse, the secondary moments are of little consequence. However, as secondary moments reduce, inelastic deformation is induced in the plastic zones. With structures which have limited capacity to sustain inelastic deformation this inelastic deformation may reduce the ability of the member to sustain additional deformation associated with redistribution of moments.

C19.3.8.2 *Serviceability limit state*

In the serviceability limit state secondary prestressed actions should be considered together with actions arising from differential temperature conditions and redistribution of bending moments due to creep of concrete in structures, which are built in a stage by stage process, such that the structural form is changed after part of the prestress or load has been applied (see Reference 19.23).

C19.3.8.3 *Ultimate limit state*

To prevent shear failure, or other non-ductile failure modes from developing prior to a ductile flexural failure mode, possible adverse effects of self-strain actions should be considered. In particular it should be noted that differential temperature due to solar radiation on bridge structures or roofs, can induce significant tensile stresses on sections, which may in turn reduce the shear sustained by the concrete (V_{ci}, V_{cw}), thus requiring an increase in the amount of shear reinforcement. In addition secondary moments can increase shears and reduce flexural cracking moments. Consequently these actions can in some cases increase the shear to be resisted and reduce the shear resistance of the concrete (V_{ci}).

C19.3.9 *Redistribution of design moments for ultimate limit state*

The general provisions for redistribution of negative moments given in 6.3.7 apply equally to prestressed members but with the tighter limits specified in 19.3.9.

For the moment redistribution principles of 19.3.9 to be applicable to beams with unbonded tendons, it is necessary that such beams contain sufficient bonded reinforcement to ensure that, after cracking, any inelastic deformation is spread over a region of the beam and is not all concentrated at a section. The minimum bonded reinforcement requirements of 19.3.6.7.2 will service this purpose.

Determining secondary moments, or the rotations arising from these in a structure, which contains flexural cracking and or inelastic deformation, is a complex matter. The approximation inherent in adding the secondary moment found from the precracked state to the redistributed moment in Equation 19–13, leads to acceptable values of redistribution.

C19.3.10 *Slab systems*

C19.3.10.1 *Design actions*

Use of analysis procedures is required for determination of both service and ultimate limit state moments and shears. The equivalent frame method of analysis has been shown by tests on large structural models to satisfactorily predict ultimate moments and shears in prestressed slab systems (see References 19.20, 19.21, 19.24, 19.25, 19.26.) The references show also that using prismatic section or other approximation of stiffness may provide erroneous results on the unsafe side.

Simplified methods of analysis using average coefficients do not apply to prestressed concrete slab systems.

C19.3.10.2 *Design strengths*

Tests indicate that the moment and shear strengths of prestressed slabs are controlled by total prestressing steel strength and by the amount and location of non-prestressed reinforcement, rather than by tendon distribution. (See References 19.19. 19.20, 19.21, 19.24, 19.25 and 19.26).

C19.3.10.3 *Service load conditions*

For prestressed flat slabs continuous over two or more spans in each direction, the span-thickness ratio generally should not exceed 42 for floors and 48 for roofs; these limits may be increased to 48 and 52, respectively, if calculations verify that both short- and long-term deflection, camber, and vibration frequency and amplitude are not objectionable.

Short- and long-term deflection and camber should be computed and checked against the requirements of serviceability of the structure.

The maximum length of a slab between construction joints is generally limited to between 30 m and 45 m to minimize the effects of slab shortening, and to avoid excessive loss of prestress due to friction.

C19.3.10.4 *Tendon layout*

This clause provides specific guidance concerning tendon distribution that will permit the use of banded tendon distributions in one direction. This method of tendon distribution has been shown to provide satisfactory performance by structural research.

These restrictions do not apply to slabs that are designed to resist concentrated loading, such as occurs in bridge decks. Some bridges are constructed using hollow precast members, which are placed side by side and nominally stressed together with a stress level below that required to sustain transverse moments. In such structures lateral distribution of structural actions is based on shear transfer between units and the torsional stiffness of the units. It should be noted that in these structures rotation occurs between the units and calculations should be made to ensure that this rotation will not damage the surfacing.

C19.3.11 ***Shear strength***

A2 **C19.3.11.1** *Beams and one-way slabs*

Where *in situ* concrete is added to precast prestressed units or beams, the section properties are changed. However, unless the precast units are propped it is the precast unit which resists the dead load, while subsequent actions are resisted by the composite section. In calculating the shear stresses and other actions, such as the flexural decompression moment in Equation 19–15 or stress levels in Equations 19–16 and 19–17, it is important to keep track of the appropriate section properties. Creep and shrinkage in the concrete can cause long-term loading to partially redistribute actions from the initial section to the final section. Where appropriate, allowance should be made for the redistribution of long-term actions due to creep and shrinkage effects (see APPENDIX CE)

C19.3.11.2.1 *Simplified method for determining nominal shear strength of concrete in beams and one-way slabs*

This clause gives a simple method of finding the nominal shear strength provided by concrete in prestressed beams and one-way slabs[19.27]. It may be applied to beams having prestressed reinforcement only, or to members reinforced with a combination of prestressed reinforcement and non-prestressed deformed bars. Equation 19–14 is most applicable to members subject to uniform loading and may give conservative results when applied to composite girders.

In applying Equation 19–14 to simply supported members subject to uniform loads V^*d/M^* can be expressed as:

$$\frac{V^*d}{M^*} = \frac{d(\ell - 2x)}{x(\ell - x)} \quad \text{(Eq. C19–1)}$$

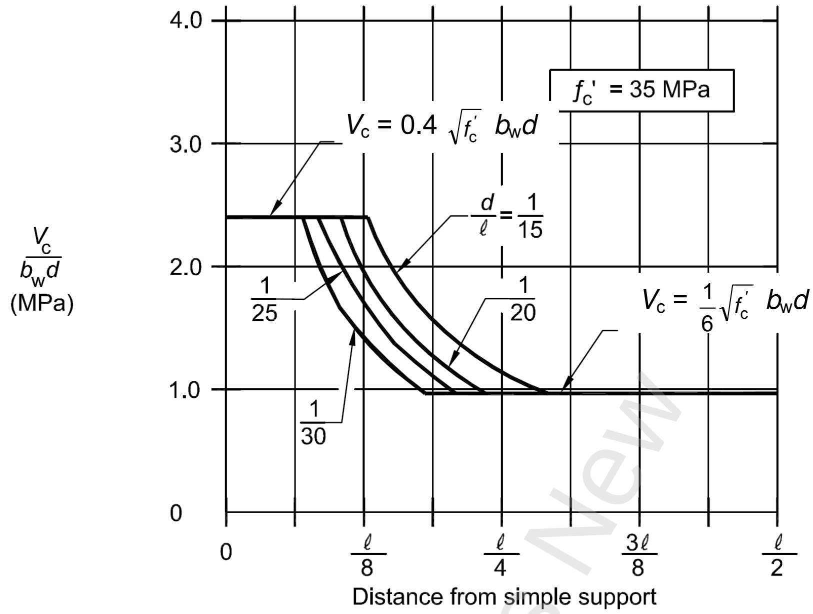


Figure C19.1 – Application of Equation 19–14 to uniformly loaded prestressed members

where ℓ is the span length and x is the distance from the section being investigated to the support. For concrete with f_c' equal to 35 MPa, V_c from 19.3.11.2.1 varies as shown in Figure C19.1. Design aids based on this equation are given in Reference 19.28.

Self strain actions, such as arise with differential temperature conditions, can induce high tensile stresses in both the extreme fibres and the webs of beams. A consequence of this is that flexural shear and web-shear cracking shear forces can be reduced. As Equations 19–14 and C19–1 do not allow for these effects, this method should not be used where self strain effects may be significant.

C19.3.11.2.2 *General method for determining V_c beams and one-way slabs*

Two types of inclined cracking occur in concrete beams; web-shear cracking and flexure-shear cracking. Two types of inclined cracking are illustrated in Figure C19.2.

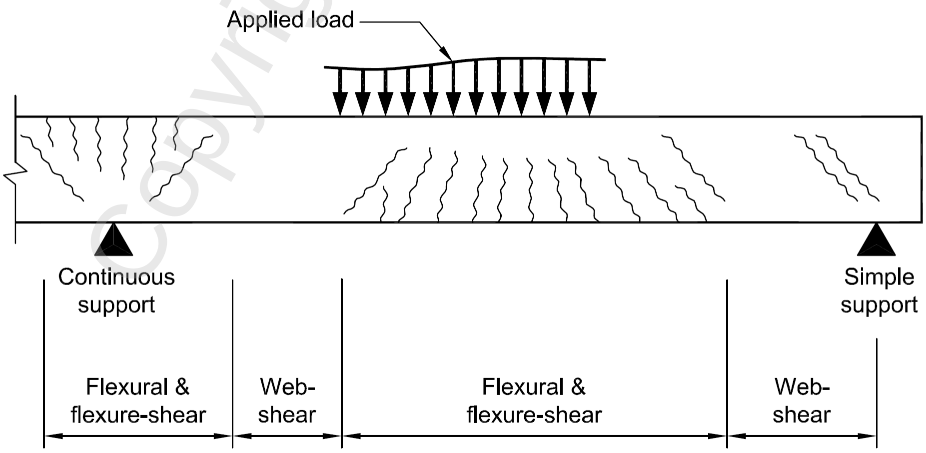


Figure C19.2 – Types of cracking in concrete beams

Web-shear cracking occurs near the mid-height of a member in regions where the bending moment to shear ratio is low, as illustrated in Figure C19.2. This cracking occurs when the principal tensile stress exceeds the tensile strength of the concrete. Flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, shear stresses are induced in the flexural tension zone. When these shear stresses reach a magnitude similar to that acting in an equivalent reinforced concrete beam, diagonal cracking occurs. It should be noted that the shear stresses are not distributed uniformly over an area A_{cv}, (this is a simplifying assumption made in the Standard) but instead in prestressed members they tend to be concentrated in the compression zone. The nominal shear strength provided by the concrete can be found by adding the shear resisted by the beam at decompression of the extreme tension fibre to the shear resistance of an equivalent reinforced concrete beam.

The shear resisted at decompression of the extreme tension fibre, V_o, is given by:

$$V_o = \frac{V^* M_o}{M^*} \qquad \text{(Eq. C19–2)}$$

where M_o is the bending moment sustained when the extreme tension fibre is decompressed (reaches zero stress). The shear resisted by the concrete in an equivalent reinforced concrete beam is equal to the value V_b. Hence the sum of these two gives the value of V_{ci} as indicated in Equation 19–15.

Equations 19–15 and 19–17 may be used to determine the shear forces causing flexure-shear and web-shear cracking, respectively. The shear strength provided by the concrete V_c is assumed equal to the lesser of V_{ci} and V_{cw}. Reference 19.29 gives some background to the calculation of the shear resistance provided by the concrete, though the approach given in that reference has been modified to give a smooth transition between prestressed and reinforced beams and allow the effects of redistribution of actions due to creep to be incorporated.

For a composite member, where part of the load is resisted by only a part of the section, the appropriate section properties should be used with each part of the load to determine the value of M_o, in the calculation of V_{ci}. Likewise in the determination of V_{cw} the appropriate section properties should be used with each component of the shear. It should be noted that creep redistribution results in redistribution of prestress and dead loading that initially acts on part of a composite member, to the full section. The extent of this redistribution depends on the creep characteristics of the concrete and the age when the structural form is modified (see Reference 19.23).

Equation 19–17 is based on the assumption that web-shear cracking occurs due to the shear causing a principal tensile stress of approximately $0.33\sqrt{f_c'}$ at the centroidal axis of the cross section. V_p is calculated from the effective prestress force without load factors.

It should be noted that self strain actions, such as differential temperature, can reduce the decompression moment, M_o, and also reduce the longitudinal compressive stress at the level of the neutral axis. For this reason self strain stresses can significantly reduce the values of V_{ci} and V_{cw}. Hence the general method of 19.3.11.2.2 should be used where these actions are significant.

Web-shear cracking occurs when the principal tensile stresses in the web reach the direct tensile strength of the concrete, which is taken as $0.33\sqrt{f_c'}$. Equation 19–17 predicts the web-shear cracking shear that corresponds approximately to the diagonal tensile stress of $0.33\sqrt{f_c'}$. In Equation 19–17 the value of V_p is the shear force carried by the inclination of the prestressing tendons relative to the axis of the member. It is based on the prestressing force after all losses have occurred and it is applied without a load factor.

The shear resistance provided by the concrete is taken as the lesser of V_{ci} and V_{cw}. This is an assumption which appears to be on the unconservative side. However, the expression for the shear resistance provided by web reinforcement, as specified by 19.3.11.3, is conservative, as the diagonal cracks develop at a smaller angle than is implied by the equations for V_s. This angle is typically about 30° in prestressed beams with a significant prestress level rather than the implied $\tan^{-1} j$ (typically 42°). However, when the V_c and V_s components are added the errors tend to cancel out and a safe design criterion is achieved.

C19.3.11.2.3 *Shear strength in transfer length*

The effect of the reduced prestress near the ends of pretensioned beams on the shear strength should be taken into account. Clause 19.3.11.2.3 (a) relates to the shear strength at sections within the transfer length of prestressing steel when bonding of prestressing steel extends to the end of the member.

Clause 19.3.11.2.3 (b) relates to the shear strength of sections within which some of the prestressing steel is not bonded to the concrete, or within the transfer length of the prestressing steel for which bonding does not extend to the end of the beam. A2

C19.3.11.2.4 Shear strength of pretensioned floor units near supports A2

Precast pretensioned floor units are designed to resist gravity loading, acting as simply supported units. As such, the moments close to the support are small and the shear strength provided by the concrete is limited by web shear cracking (Equation 19–17). However, generally reinforcement in the concrete topping is extended into the supporting structure. This is required to enable the floor to act as a diaphragm and distribute lateral forces to the lateral force resisting elements. It also has the advantage of stiffening the floor for live load.

Under seismic actions (or wind) sway occurs, and this can lead to rotation between the supports and the precast units. This rotation can induce negative moments close to the supports. In addition, elongation of beams due to the formation of plastic hinges can induce axial tension in the floor. In precast units without top pretension strands the negative moments and axial tension can lead to flexural cracking. The formation of these cracks greatly reduces the shear strength provided by the concrete, as this is now controlled by the flexural shear strength, as given by Equation 19–15. Consequently, shear strength determined from test on simply supported units considerably over-estimates the shear strength when negative moments are acting. It may be noted that without pretension reinforcement near the top of the unit the value of M_o in Equation 19–15 is essentially zero and the shear strength is close to that of a reinforced concrete beam.

In hollow-core units with near circular ducts the web width is close to a minimum at the mid-depth of the unit. In this location the crack widths are small, while the wider crack widths occur where the web width is wider. It has been shown that shear strength decreases as crack widths increase (see Reference 9.10). Hence, for this form of precast unit the allowable shear stress based on the minimum web width can be increased as indicated in (b)(ii).

The relative magnitudes of rotation at the supports and elongation of the beams determines whether a pure negative moment acts at a support, or a lesser moment and axial tension acts. Analyses by Woods, L., Fenwick, R and Bull D., "Seismic Performance of Hollow-core Flooring; the Significance of Negative Moments," Paper 5.3, NZSEE Conference, Wairakei, April 2008, indicated that in either case the critical shear stresses in the negative moment zone were of similar magnitude. Consequently, for design it is recommended that the critical shear force is determined assuming the over-strength moment acts at one of the supports and no moment acts at the other support.

C19.3.11.2.5 *Shear strength in two-way prestressed concrete slabs*

For prestressed slabs and footings, a modified form of the reinforced concrete equations may be used for punching shear calculations at slab-column junction, provided the region contains sufficient bonded reinforcement. Research [19.20, 19.30] indicates that the shear strength of two-way prestressed slabs around interior columns is conservatively predicted by Equation 19–18. V_c from Equation 19–18 corresponds to a diagonal tension failure of the concrete initiating at the critical section defined in 12.7.1 (b). The mode of failure differs from a punching shear failure of the concrete compression zone around the perimeter of the loaded area predicted by Equation 12–6. Consequently, the term β_c does not enter into Equation 19–18. Design values for f_c' and f_{pc} are restricted due to limited test data available for higher values. When computing f_{pc}, loss of prestress due to restraint of the slab by shear walls and other structural elements should be taken into account.

In a prestressed slab with distributed tendons, the V_p term in Equation 19–18 contributes only a small amount to the shear strength; therefore, it may be conservatively taken as zero. If V_p is to be included, the tendon profile assumed in the calculations should be noted.

For an exterior column support where the distance from the outside of the column to the edge of the slab is less than four times the slab thickness, the prestress is not fully effective around the total perimeter b_o of the critical section. Shear strength in this case is therefore conservatively taken to be the same as for a non-prestressed slab.

C19.3.11.3.1 Details of shear reinforcement in slabs

Shear reinforcement is ineffective in thin slabs due to the difficulty of effectively anchoring stirrups in the compression zone. The problem becomes more acute as the concrete cover is increased.

C19.3.11.3.4 Modification of design of shear reinforcement in beams and one-way slabs due to prestress

The design for shear reinforcement in prestressed concrete beams and slabs is very similar to that for reinforced concrete beams and slabs. The required modifications are indicated in this clause.

As indicated in 19.3.11.3.4 some increase in spacing of stirrups, beyond that required for reinforced concrete beams (9.3.9.4.12), is permitted for prestressed concrete members. Tests [19.31] of prestressed beams with minimum web reinforcement based on Equations 9–10 and 19–19 indicated that the smaller A_v from these two equations was sufficient to develop ductile behaviour.

Equation 19–19 may be used only for prestressed members meeting the minimum prestress force requirements given in 19.3.11.3.4. This equation is discussed in Reference 19.31

Even when the design shear force V^* is less than one-half of the shear strength provided by the concrete ϕV_c, the use of some web reinforcement is recommended in all thin-web post-tensioned prestressed concrete members to reinforce against tensile forces in webs resulting from local deviations from the design tendon profile, and to provide a means of supporting the tendons in the design profile during construction. If sufficient support is not provided, lateral wobble and local deviations from the smooth parabolic tendon profile assumed in design may result during placement of the concrete. In such cases, the deviations in the tendons tend to straighten out when the tendons are stressed. This process may impose large tensile stresses in webs, and severe cracking may develop if no web reinforcement is provided. Unintended curvature of the tendons, and the resulting tensile stresses in webs, may be minimised by securely tying tendons to stirrups that are rigidly held in place by other elements of the reinforcing cage and held down in the forms. The maximum spacing of stirrups used for this purpose should not exceed the smaller of 1.5h or 1.2 m. When applicable, the shear reinforcement provisions will require closer stirrup spacings.

For repeated loading of flexural members, the possibility of inclined diagonal tension cracks forming at stresses appreciably smaller than under static loading should be taken into account in the design. In these instances, it would be prudent to use at least the minimum shear reinforcement expressed by Equations 9–10 and 19–19, even though tests or calculations based on static loads show that shear reinforcement is not required.

C19.3.13 ***Anchorage zones for post-tensioned tendons***

C19.3.13.1.1 Definition of anchorage zone

Based on the principle of Saint-Venant, the extent of the anchorage zone may be estimated as approximately equal to the largest dimension of the cross section measured from the anchorage. In complex sections, such as box girders, which contain a number of elements in the section, the critical dimension tends to be the dimension of the element (slab or web) which contains the anchor. When anchorage devices located away from the end of the member are tensioned, a large tensile force, or if unreinforced a wide crack, may be induced locally behind the anchor. This tensile force, or wide crack, arises from the incompatibility of deformations ahead of and behind the anchorage device, as is illustrated in Figure C19.3. In this situation the anchorage zone extends to include the region immediately behind the anchor.

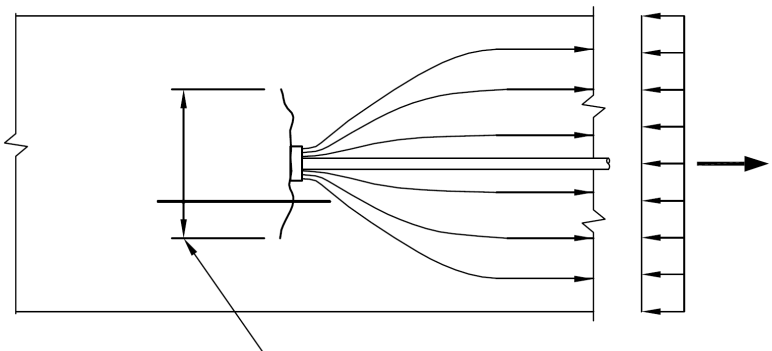


Figure C19.3 – Splitting crack at anchor located away from end of member

C19.3.13.1.2 Design of anchorage zones

In an anchorage zone tensile stresses are induced in the concrete due to the dispersion of the concentrated force or forces into the member. The principal sources of these stresses are illustrated Figure C19.3, Figure C19.4, Figure C19.5 and Figure C19.6, and they are listed in 19.3.13.4.3. In determining these forces allowance must be made for the three dimensional dispersion of stress, which results in tension forces being induced in two planes. In addition it is necessary to ensure that the concrete does not fail in compression due to the high bearing stresses against the anchor. This aspect is considered in 19.3.13.3.2.

Additional information on the design of anchorage zones for specific types of anchors may be obtained from References 19.32, 19.33, 19.34 and 19.35.

C19.3.13.3.2 Bearing stress against anchors A2

The zone, which resists the very high local stresses introduced by the anchorage device, cannot be completely designed until the specific characteristics of the anchorage device are selected, and hence these details cannot be finalised until the shop drawing stage. The behaviour of the local zones subjected to high compression stresses at the anchors is not strongly influenced by the geometry and loading of the overall structure. Consequently most standard anchors are supplied with standard confining reinforcement. When special anchorage devices are used, the anchorage device supplier should furnish information to show the device and associated confinement reinforcement meets the requirements of 16.3. The main consideration in the design for the local high compression stresses is to ensure and the adequacy of any confining reinforcement that is provided to increase the bearing capacity of the concrete.

C19.3.13.3.3 Tensile strength of concrete

No allowance should be made for the tensile strength of concrete in assessing the spalling and bursting stresses. The concrete in the anchorage zone may be already stressed in tension, or cracked, due to actions such as differential temperature, differential shrinkage or tensile stresses resulting from restraint against thermal contraction from the heat of hydration, or other causes generally not considered in detail in design.

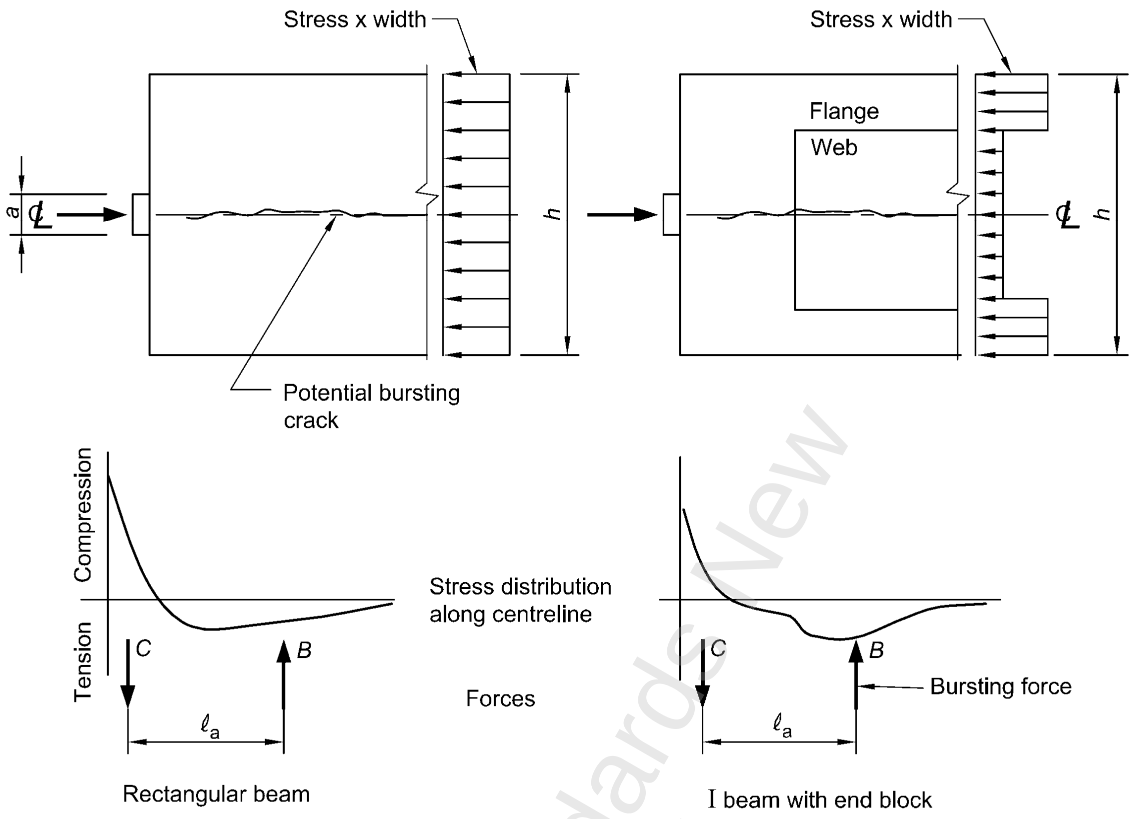
C19.3.13.4.1 and **C19.3.13.4.2** Permitted methods *and simplified and linear elastic methods*

Linear elastic methods, such as the finite element method, usually assume that the concrete has equal stiffness in all directions. However, before reinforcement can act to resist tension that is associated with bursting forces, the concrete must crack so that the reinforcement can be strained and hence sustain stress. With the formation of the cracks the stiffness normal to the direction of the crack decreases compared to the stiffness in the longitudinal direction. This leads to stress redistribution. A consequence of this is that the bursting force reduces and its centre of action is located further away from the anchorage than is predicted from the analysis. This should be recognised and reinforcement found in such cases should be extended further along the member than is indicated by an analysis in which this change of stiffness is not recognised[19.36, 19.37].

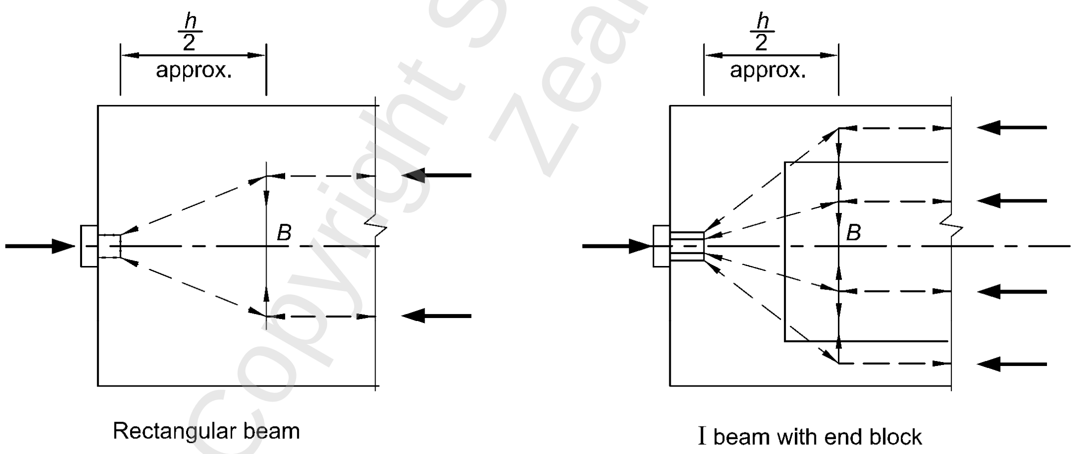
A number of texts contain charts, which show the distribution of bursting stresses in rectangular members subjected to concentrated loads, such as occurs with prestressing anchors. These charts should only be used with rectangular members. Simplified methods may be used provided they allow in a rational manner for the section shape and any change in shape of the section over the anchorage zone.

C19.3.13.4.3 Reinforcement required for tension forces in anchorage zones

Bursting forces are illustrated in Figure C19.4 for the simple case of a single anchorage located at the end of a member. The end zone may be considered to act as a deep beam, which is loaded by a concentrated force at one end and supported by a linearly distributed stress at the other end. For the case of a rectangular section and a central point load the bending moment is equal to 0.125 $P(h - a)$ where h is the depth of the member and a is the dimension of the bearing. For design purposes the internal lever-arm may be taken as $h/2$, giving a bursting force of 0.25 $P(1 – a/h)$. This approximation gives a conservative estimate of the bursting force as it ignores stress redistribution associated with the formation of the bursting crack. This illustrates the “deep beam analogy”, as shown in Figure C19.4(a), which can be extended to cover the case where the section shape is not rectangular and where the shape changes in the anchorage zone, as is illustrated on the right hand side of Figure C19.4(a). The approach can also be applied where the prestressed anchorage force is eccentric to the member. However, in this case the magnitude of the internal lever-arm, ℓ_a, in Figure C19.4(a), reduces from $h/2$ to $(h-e)/2$ where e is the distance of the centroid of the anchorage forces from the mid-section. Figure C19.4(b) illustrates the same problem, but this time using strut and tie models.



(a) Deep beam analogy and bursting forces



(b) Strut and tie analogy and bursting forces

Figure C19.4 – Bursting forces in anchorage zone with single prestress anchor

Where there is a group of prestressed anchors two different cases have to be considered. First of all there is an immediate local bursting force, B_ℓ, associated with each anchor, and secondly there is an overall bursting force, B_o, associated with all the group of anchorage forces. The situation is illustrated in Figure C19.5. As indicated a strut and tie analysis may be used to assess the forces, or alternatively the local forces may be assessed from an equivalent prism, as indicated in Figure C19.5(c), using either a strut and tie model or the deep beam analogy. The dimension of the prism, h_e, is equal to the centre-to-centre distance between the anchors, or for the outside anchor the centre-to-centre distance or twice the distance to the free edge. To find the overall bursting force, B_o, the anchors, provided they are relatively close, are combined to one equivalent anchorage force, as illustrated in Figure C19.5(d).

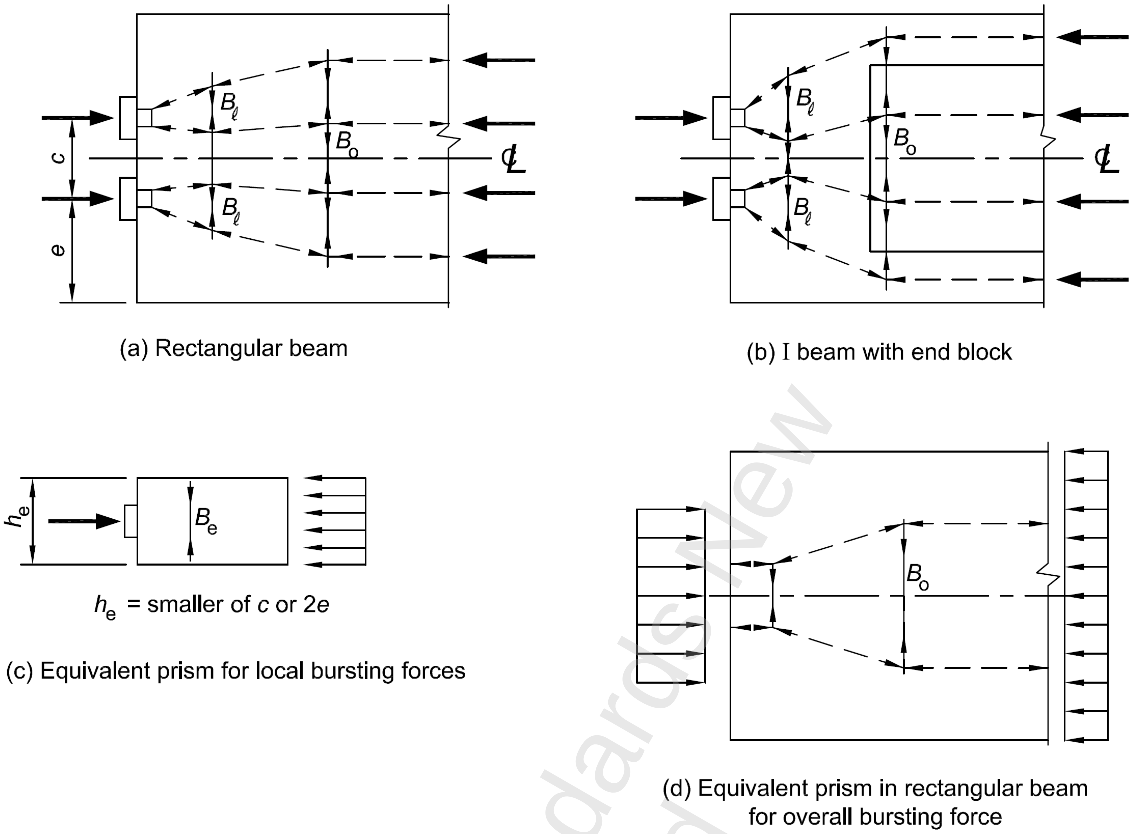


Figure C19.5 – Bursting forces with multiple anchors

Where the prestressing anchorage forces are highly eccentric, as illustrated in Figure C19.6(a), high tension forces may be generated in the concrete close to the surface of the end face of the member. These forces may be determined from the deep beam analogy, or from a strut and tie model as illustrated in Figure C19.6(a). Tension forces, which develop close to the end face of a member, are known as spalling forces. In this case these arise from the requirements of equilibrium, and consequently it is important that they are recognised and adequate reinforcement is detailed to sustain them.

Figure C19.6(b) shows another source of spalling tension force, which in this case arises from the cable being inclined to the axis of beam where it is anchored. The vertical component of the cable force, $P\tan\theta$, is resisted by the vertical component of the compression force below the anchor and a tension force, S_v, above the anchor. This force can be conservatively estimated from the magnitudes of the longitudinal forces above and below the anchor. Generally reactions which support reactions acting near the end of the beam reduce this spalling tension force.

Figure C19.6(c) shows a further cause of spalling tension. In this case the tension in the concrete arises from compatibility rather than a requirement for equilibrium. The high local compression stresses at the anchor cause the member in this location to distort to form a concave shape. On each side of this zone there are convex profiles, and associated with each of these are high local tensile strains. The resultant tensile stresses disappear when cracks form as this allows the deformation to occur freely. However, reinforcement is required to control these spalling cracks. For this reason 19.3.13.4.5 requires a minimum area of reinforcement to be placed on the back face of end anchorage zones. In assessing the total area of spalling tension reinforcement the area required to control compatibility induced spalling cracks does not need to be added to other sources of spalling tension force.

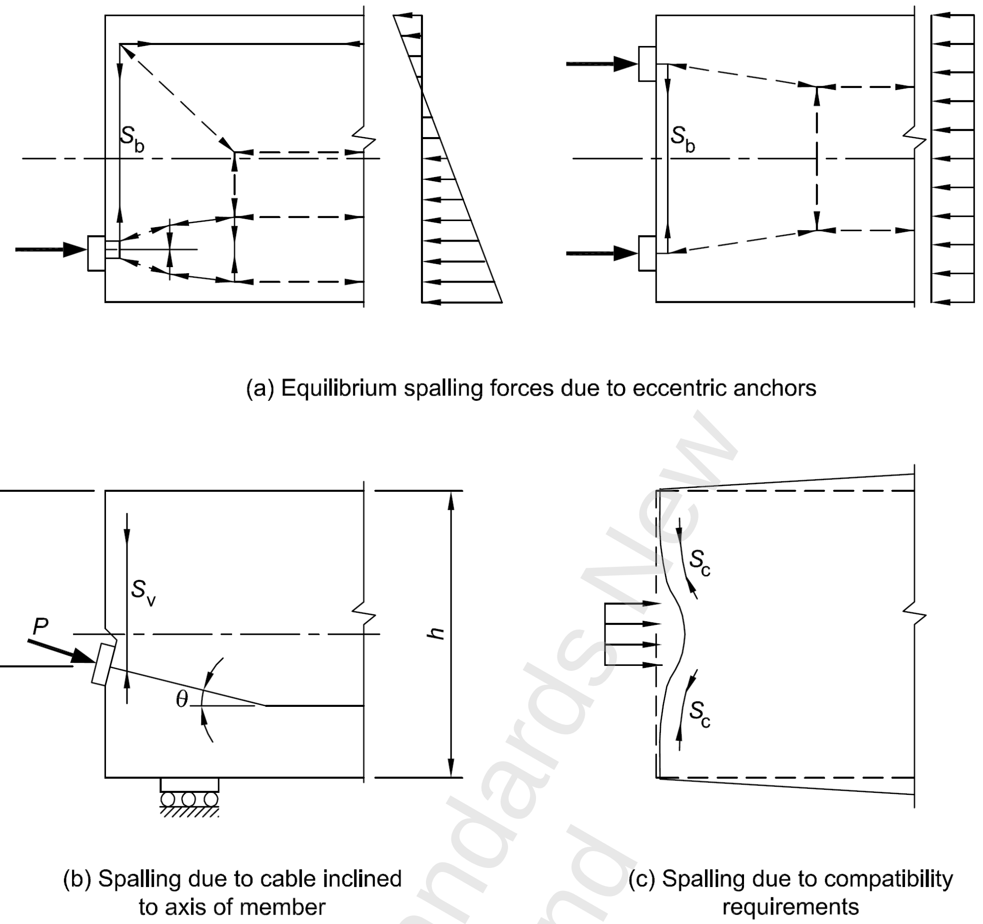


Figure C19.6 – Spalling forces in anchorage zones

C19.3.13.4.4 Anchorage devices away from end of members

Reinforcement is required to be placed close to an anchorage device, that is located away from the end of a member, to control the splitting crack that forms immediately behind the anchor, as illustrated in Figure C19.3. The area of reinforcement, which is specified in 19.3.13.4.4, to control this crack, is based on the assumption that this reinforcement, when stressed to $0.6f_y$, can resist $^1/_4$ of the maximum force in the cable at the anchor. The reinforcement should be extended so that it can be fully developed on both sides of the crack.

C19.3.14 *Curved tendons*

Where cables are curved in either a vertical profile or a horizontal profile, the bearing forces that are induced in the concrete need to be considered in the design. Figure C19.7 illustrates a case where splitting cracks are induced in a web of a beam due to the curvature of cables over a support. The stress distribution in the concrete is very similar to that which occurs in an anchorage zone, with bursting forces being induced in the concrete. The presence of an ungrouted duct on the compression side of the cable that is being stressed, increases the bursting forces and stresses and the likelihood of failure of the web. To prevent this type of failure, reinforcement should be placed as indicated on the right hand side of the figure.

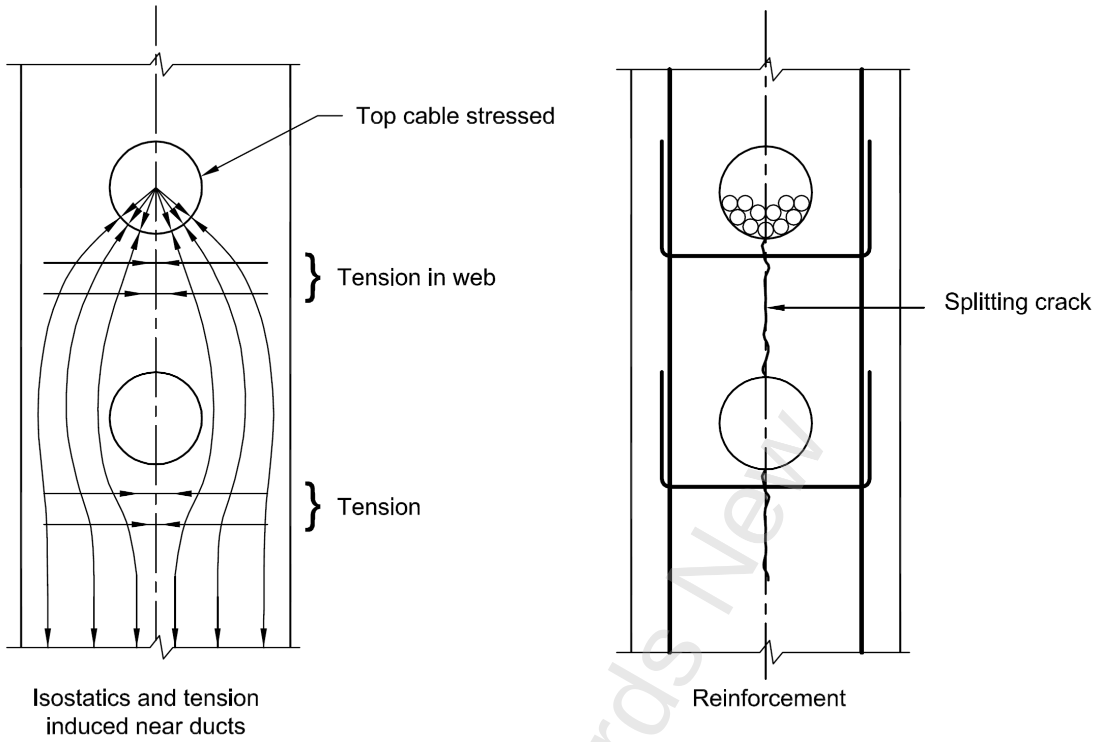


Figure C19.7 – Splitting failure in web due to bearing associated with vertical curvature of cable

If a cable is curved in plan, as in the case of a curved bridge or the wall of a circular tank, the lateral force the cable applies to the concrete is balanced by an equal and opposite lateral force in the concrete. However, as illustrated in Figure C19.8, the lateral force in the concrete is spread over the whole section, and as a result local bending moments and shear forces are induced in the web or wall. In addition the lateral force from the cable tries to punch out the concrete cover as shown on the left-hand side of the figure. The tensile resistance of the concrete to this punching force may be considerably reduced below the direct tensile strength of the concrete due to the presence of flexural cracks, which may be caused by the local bending moment and the shear forces acting on the web. To prevent possible failure of the web, reinforcement is required to resist both the local moments and the punching action of a cable or cables, and the combined effects of shear in the web with local moments also need to be considered.

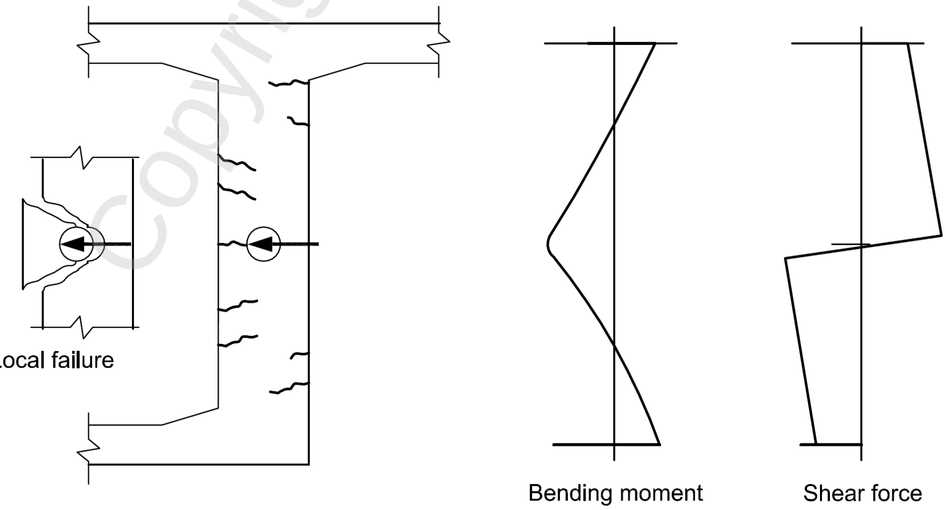


Figure C19.8 – Local bending moments and shear force in web with horizontal curvature

C19.3.15 *Corrosion protection for unbonded tendons*

C19.3.15.1 *General*

Suitable material for corrosion protection of unbonded prestressing steel should have the properties identified in Section 5.1 of Reference 19.34

C19.3.15.2 *Watertightness*

Typically, sheathing is a continuous, seamless, high-density polyethylene material that is extruded directly onto the coated prestressing steel.

C19.3.17 *Post-tensioning anchorages and couplers*

C19.3.17.1 *Strength of anchorages and couplers*

The required strength of the tendon-anchorage or tendon-coupler assemblies for both unbonded and bonded tendons, when tested in an unbonded state, is based on 95 % of the specified breaking strength of the prestressing steel in the test. The prestressing steel material should comply with the minimum provisions of the applicable specifications. The specified strength of anchorages and couplers exceeds the maximum design strength of the prestressing steel by a substantial margin, and, at the same time, recognises the stress riser effects associated with most available post-tensioning anchorages and couplers. Anchorage and coupler strength should be attained with a minimum amount of permanent deformation and successive set, recognising that some deformation and set will occur when testing to failure. Tendon assemblies should conform to the 2 % elongation requirements in Reference 19.38 and industry recommendations [19.22]. Anchorages and couplers for bonded tendons that develop less than 100 % of the specified breaking strength of the prestressing steel should be used only where the bond transfer length between the anchorage or coupler and critical sections equals or exceeds that required to develop the prestressing steel strength. This bond length may be calculated by the results of tests of bond characteristics of untensioned prestressing strand[19.39], or by bond tests on other prestressing steel materials, as appropriate.

C19.3.17.3 *Fatigue of anchorages and couplers*

For discussion on fatigue loading, see Reference 19.40.

For detailed recommendations on tests for static and cyclic loading conditions for tendons and anchorage fittings of unbonded sections, see Clause 4.1.3 of Reference 19.41, and Clause 14.2.2 of Reference 19.38.

For recommendations regarding protection see Clauses 4.2 and 4.3 of Reference 19.30, and Clauses 3.4, 3.6, 5.6, and 8.3 of Reference 19.34.

C19.3.18 *External post-tensioning*

External attachment of tendons is a versatile method of providing additional strength, or improving serviceability, or both, in existing structures. It is well suited to repair or upgrade existing structures and permits a wide variety of tendon arrangements.

Additional information on external post-tensioning is given in Reference 19.42.

C19.3.18.3 *Attachment to member*

External tendons are often attached to the concrete member at various locations between anchorages (such as mid-span, quarter points, or third points) for desired load balancing effects, for tendon alignment, or to address tendon vibration concerns. Consideration should be given to the effects caused by the tendon profile shifting in relationship to the concrete centroid as the member deforms under effects of post-tensioning and applied load.

C19.3.18.4 *Protection against corrosion*

Permanent corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable to the environment in which the tendons are located. Some conditions will require that the prestressing steel be protected by concrete cover or by cement grout in polyethylene or metal tubing, other conditions will permit the protection provided by coatings such as paint or grease. Corrosion protection methods should meet the fire protection requirements, unless the installation of external post-tensioning is to only improve serviceability.

C19.4 Additional design requirements for earthquake actions

The design requirements for prestressed and partially prestressed members are similar in principle to those for non-prestressed members and many provisions in relevant sections apply to prestressed as well as to reinforced members. The provisions are based mainly on the recommendations of the Seismic Committee of NZPCI [19.43] and more recent research on hybrid jointed frames[19.1].

C19.4.2 *Materials*

C19.4.2.1 *Prestressing steel*

It is of particular importance that the prestressing steel complies with the specified requirements for percentage elongation at rupture to ensure adequate ductility. Where possible, ultimate flexural strength calculations should be based on the actual prestressing steel stress-strain relationships. The strain in reinforcement, calculated from the required curvature and using the effective plastic hinge length, is over-estimated, as yield penetration into joint zones and spreading of yield along a beam due to diagonal cracking is ignored, see C2.6.1.3.3. Consequently, prestressed reinforcement satisfying this criterion should have sufficient strain capacity to sustain strains in the maximum creditable earthquake without rupture.

C19.4.2.2 *Concrete*

The slope of the falling branch of the concrete stress strain curve increases, and the ultimate compressive strain reduces, with increasing concrete strength. Consequently, unless special transverse reinforcement is provided to increase the ultimate compressive strain, very high strength concrete should not be used in plastic hinge regions.

C19.4.2.3 *Grouting of tendons*

When unbonded post-tensioned tendons are used it must be ensured that the anchorages are capable of withstanding the fluctuation of tendon force that occurs in an earthquake. Unbonded tendons should be used with non-prestressed steel reinforcement in accordance with 19.4.5.2, or other means of energy dissipation. Partially debonded tendons passing through a beam-column joint may be used if designed in accordance with Reference 19.44.

A2 ### C19.4.3 *Beams and floor slabs*

C19.4.3.2 *Redistribution of moments*

Design for ductile or limited ductile behaviour implies substantial capacity for moment redistribution. Therefore provided the appropriate criteria given in 19.4.3.3 (b) or (c) are satisfied moments resulting from elastic analysis may be redistributed in accordance with 19.3.9.3, 19.3.9.4 and 19.3.9.5 to gain a more advantageous seismic resistance, and thus a more efficient design.

C19.4.3.3 *Nominally ductile, limited ductile and ductile plastic regions*

There is limited experimental work available to assess curvature limits that can be sustained under cyclic loading of prestressed members. The requirements in this clause have been based on judgement.

With nominally ductile T- or L- beams high curvature ductility can generally be sustained when the flange is in compression. However, when the stem sustains the flexural compression force the curvature is likely to be limited.

With limited ductile plastic regions some longitudinal reinforcement is required in the compression zone, except where there are large flanges, as the strength of the concrete degrades with cyclic loading. The corresponding area of compression reinforcement specified for reinforced concrete beams is not practical for most prestressed beams, consequently to prevent a premature failure with the lower level of compression reinforcement the confinement requirements have been increased.

Where ductile plastic regions are to be used an analysis using stress strain relationships, which allow inelastic deformation of concrete and cyclic loading, are required to demonstrate that the region has sufficient inelastic deformation capacity to sustain on average 1.5 times the curvature required at the ultimate limit state. The 1.5 factor gives the region the capacity to sustain the curvature demand associated with the maximum creditable earthquake without collapse.

References 19.45, 19.46, 19.47 and 19.48 give information on the ductility of prestressed and partially prestressed concrete members.

C19.4.3.4 *Contribution of reinforcement in flanges to strength of beams*

The effect of slab reinforcement in contributing to both the design ultimate flexural strength and the beam overstrength, and must be considered when calculating the moments introduced to columns when plastic hinges form in beams. Where a significant portion of the flexural tensile strength of a beam arises from reinforcement in the flanges a strut and tie analysis is required of the joint zone (see clause 15.2.2).

C19.4.3.5 *Transverse reinforcement*

Closed stirrups are required to be present in potential plastic hinge regions to provide confinement to the concrete, to prevent buckling of non-prestressed compression reinforcement and to provide shear strength. The shear strength provided by the concrete in potential plastic hinge regions shall be taken to be zero due to the degradation caused by cyclic loading.

C19.4.3.6 *Floors with precast pretensioned units* A2

Under severe seismic actions, relative rotation between supporting structure and precast floor units can cause wide cracks to open up between the supporting structure and the end of the precast units. Reinforcement, which is generally in topping concrete, at this location can be stressed close to its ultimate stress, inducing either negative moment, or negative moment and axial tension into the unit. Analyses indicate that the shear stresses induced in the units in the negative moment region are of similar magnitude regardless of whether the support sustains its maximum moment or a lesser moment and axial tension. In design for this situation shear forces should be based on the assumption that the maximum overstrength negative moment acts at one end of the unit and zero moment acts at the other end. In the analysis actions associated with vertical ground, motion should not be overlooked.

It is important that capacity design is applied to ensure that the actions transmitted to the floor by the negative moments and axial tension acting at the ends of the unit, together with gravity loads and vertical seismic actions, cannot result in shear or flexural failure of the floor.

Care in detailing is also required to ensure that the precast units are not cast into the supporting structure in such a way that positive moments can be induced into a precast unit, causing flexural cracks to develop close to end of the precast unit where the pretension strands are not adequately developed. A flexural crack in this location can open up due to elongation associated with plastic deformation in beams. This may lead to a premature flexural shear failure of the unit.

In some situations, sway of a structure can result in the supporting structure for precast floor units being subjected to twist. Where this occurs, consideration should be given to using precast units which can sustain the required level of twist without the danger of inducing a brittle torsional failure. A typical situation where this could occur is where a hollow-core unit is supported by an active link in an eccentrically braced frame, or on a diagonally reinforced coupling beam.

C19.4.4 ***Design of columns and piles***

C19.4.4.1 *Confinement and anti-buckling reinforcement*

General requirements for prestressed concrete columns and ductile regions of piles are similar to those for reinforced concrete columns.

C19.4.4.2 *Minimum reinforcement content*

The minimum reinforcement content ensures that the ultimate flexural strength of the column is greater than the flexural cracking moment. This ensures that the column will not fail in a brittle mode at one section with the formation of a single crack. The criterion is based on the most critical case of a column with minimal axial load and assuming the flexural tensile strength is approximately 50 % greater than the average value for deflection calculations. This corresponds approximately to an upper characteristic modulus of rupture strength.

C19.4.4.3 *Spacing of longitudinal reinforcement*

The longitudinal reinforcement should be distributed reasonably uniformly around the perimeter of the section in order to assist the confinement of concrete in potential plastic hinge regions.

C19.4.4.4 *Transverse reinforcement in potential plastic regions*

Transverse reinforcement is necessary in potential plastic hinge regions to provide confinement to the concrete, to prevent buckling of non-prestressed compressive reinforcement and to provide shear strength. The shear strength provided by the concrete shall be taken to be zero.

C19.4.5 *Prestressed moment resisting frames*

C19.4.5.1 *Beam tendons at beam-column joints*

Such an arrangement of tendons results in more ductile plastic hinge behaviour of beams under inelastic cyclic actions than where the tendons are all concentrated at mid-depth in the beam. However, in addition to top and bottom tendons, it is very desirable to have at least one tendon located within the middle third of beam depth to resist some of the joint core shear force.

C19.4.5.2 *Partially prestressed beams*

A possible design technique to satisfy this clause would involve prestressing reinforcement designed to balance a portion of the serviceability limit state gravity loads, with the additional required seismic moment capacity and ductility provided by top and bottom layers of non-prestressed reinforcement. Under these circumstances the beam prestressing tendon or tendons at the column faces could be located in the central third of the beam depth to avoid loss of effective prestress force under reversed inelastic cycling, and to improve the shear resistance of the joint core.

C19.4.5.3 *Ducts for grouted tendons*

Corrugated ducts provide the best bond transfer between tendon and concrete and are thus preferred in regions of high bond stress, such as beam-column joint cores.

C19.4.5.4 *Jointing material*

Limited testing has indicated that precast joints at the faces of columns can function effectively with no other connection through the jointing material than the grouted tendons[19.49]. Some form of mechanical interlock is required to hold the jointing material in place. Where possible, the plastic hinge regions should be forced to form away from the jointing faces, by the use of suitable reinforcing details, haunches, or other means.

C19.4.6 *Design of hybrid jointed frames*

The so called hybrid systems combine unbonded tendons with non-prestressed longitudinal mild steel reinforcement or additional devices to provide energy dissipation. The systems have the advantages of being self centring (i.e. almost zero residential deflection) after an earthquake, and should exhibit negligible damage after an earthquake. Hybrid systems have been extensively investigated and developed [19.50, 19.51, 19.52] under the US PRESSS (PREcast Seismic Structural Systems) programme. The systems recognise the advantages of precast concrete construction over cast-in-place methods. The design of hybrid jointed frames is described in Appendix B of Part 1.

REFERENCES

19.1 fib “State-of-the-art Report on the Seismic Design of Precast Concrete Building Structures,” Bulletin 27, International Federation of Structural Concrete, Lausanne, 2004.

19.2 "Analysis of Cracked Prestressed Concrete Sections: A Practical Approach," PCI Journal, Vol. 43, No. 4, Jul.-Aug. 1998.

19.3 PCI Design Handbook – "Precast and Prestressed Concrete", 4th Edition, Precast/Prestressed Concrete Institute, Chicago, 1992, pp. 4-42 through 4-44.

19.4 Frosch, R. J., "Another Look at Cracking and Crack Control in Reinforced Concrete", Structural Journal, American Concrete Institute, Vol. 96, No. 3, May 1999, pp. 437-442.

19.5 ACI-ASCE Committee 423, "Tentative Recommendations for Prestressed Concrete," ACI Journal, Proceedings, Vol. 54, No. 7, Jan. 1958, pp. 545-578.

19.6 ACI Committee 435, "Deflections of Prestressed Concrete Members (ACI 435.1R-63)(Reapproved 1989)," ACI Journal, Proceedings Vol. 60, No. 12, Dec. 1963, pp. 1667-1728. Also ACI Manual of Concrete Practice.

19.7 PCI Committee on Prestress Losses, "Recommendations for Estimating Prestress Losses," Journal of the Prestressed Concrete Institute, Vol. 20, No. 4, July-Aug. 1975, pp. 43-75.

19.8 Zia, P., Preston, H.K., Scott, N.L. and Workman, E.B., "Estimating Prestress Losses," Concrete International: Design & Construction, Vol. 1, No. 6, June 1979, pp. 32-38.

19.9 "PCI Post-tensioning Manual", Prestressed Concrete Institute Chicago, 1972, A-2 "Friction Losses"

19.10 Mander J.B., Priestley M.J.N. and Park R. "Theoretical stress strain model for confined concrete", Journal of Structural Engineering, 1988, Vol. 114, No. 8, pp. 1804-1826.

19.11 Devalapura R.K. and Tadros M.K., "Stress-strain modeling of 270 ksi low relaxation prestressing strand", PCI Journal, Vol. 37., No. 2, Mar. 1992, pp 100-105.

19.12 ACI-ASCE Committee 423, "Recommendations for Concrete Members Prestressed with Unbonded Tendons (ACI 423.3R-89)," ACI Structural Journal, Vol. 86, No. 3, May-June 1989, pp. 301-318. Also ACI Manual of Concrete Practice.

19.13 Mojtahedi, S. and Gamble, W.L., "Ultimate Steel Stresses in Unbonded Prestressed Concrete," Proceedings, ASCE, Vol. 104, ST7, July 1978, pp. 1159-1165.

19.14 Mattock, A.H., Yamazaki, J. and Kattula, B.T., "Comparative Study of Prestressed Concrete Beams, with and without Bond," ACI Journal, Proceedings, Vol. 68, No. 2, Feb. 1971, pp. 116-125.

19.15 Mast, R.F., "Unified Design Provision for Reinforced and Prestressed Concrete Flexural and Compression Members," ACI Structural Journal, Vol. 89, No. 2, Mar.-Apr. 1992, pp. 185-199.

19.16 Foutch, D.A., Gamble, W.L. and Sunidja, H., "Tests of Post-tensioned Concrete Slab-edge Column Connections," ACI Structural Journal, Vol. 87, No. 2, Mar.-Apr. 1990, pp. 167-179.

19.17 ACI Committee 318 "Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-63)", SP-10, American Concrete Institute, Farmington Hills, MI, 1965, pp. 78-84.

19.18 Odello, R.J. and Metha, B.M., "Behaviour of a Continuous Prestressed Concrete Slab with Drop Panels," Report, Division of Structural Engineering and Structural Mechanics, University of California, Berkeley, 1967.

19.19 Smith, S.W. and Burns, N.H., "Post-tensioned Flat Plate to Column Connection Behaviour," Journal of the Prestressed Concrete Institute, Vol. 19, No. 3, May-June 1974, pp. 74-91.

19.20 Burns, N.H. and Hemakom, R., "Test of Scale Model Post-tensioned Flat Plate," Proceedings, ASCE, Vol. 103, ST6, June 1977, pp. 1237-1255.

19.21 Hawkins, N.M., "Lateral Load Resistance of Unbonded Post-tensioned Flat Plate Construction," Journal of the Prestressed Concrete Institute, Vol. 26, No. 1, Jan. – Feb. 1981, pp. 94-116.

19.22 "Guide Specifications for Post-tensioning Materials," Post-tensioning Manual, 5th Edition, Post-tensioning Institute, Phoenix, Ariz., 1990, pp. 208-216.

19.23 Bryant, A.H., Wood, J.A. and Fenwick, R.C., "Creep and Shrinkage in Concrete Bridges", Road Research Unit Bulletin No. 70, National Roads Board, New Zealand, 1984.

19.24 "Design of Post-tensioned Slabs," Post-tensioning Institute, Phoenix, Ariz., 1984, p. 54.

19.25 Gerber, L.L. and Burns, N.H., "Ultimate Strength Tests of Post-tensioned Flat Plates," Journal of the Prestressed Concrete Institute, Vol. 16, No. 6, Nov.-Dec. 1971, pp. 40-58.

19.26 Scordelis, A.C., Lin, T.Y. and Itaya, R., "Behaviour of a Continuous Slab Prestressed in Two Directions," ACI Journal, Proceedings, Vol. 56, No. 6, Dec. 1959, pp. 441-459.

19.27 MacGregor, J.G. and Hanson, J.M., "Proposed Changes in Shear Provisions for Reinforced and Prestressed Concrete Beams," ACI Journal, Proceedings, Vol. 66, No. 4, April 1969, pp. 276-288.

19.28 PCI Design Handbook – Precast and Prestressed Concrete, 4th Edition, Precast/Prestressed Concrete Institute, Chicago, 1992, p. 580.

19.29 ACI Committee 318, "Commentary on Building Code requirements for reinforced concrete," (aci 318-63), SP-10, American Concrete Institute, 1965, pp. 78-84.

19.30 ACI-ASCE Committee 423, "Recommendations for Concrete Members Prestressed with Unbonded Tendons (ACI 423.3R-89)," American Concrete Institute, Farmington Hills, MI, p. 18. Also ACI Manual of Concrete Practice.

19.31 Olesen, S.E., Sozen, M.A. and Siess, C.P., "Investigation of Prestressed Reinforced Concrete for Highway Bridges, Part IV: Strength in Shear of Beams with Web Reinforcement," Bulletin No. 493, University of Illinois, Engineering Experiment Station, Urbana, 1967.

19.32 ACI-318-2002, "Building Code Requirements for Structural Concrete and Commentary", American Concrete Institute, Michigan.

19.33 American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges," 16th Edition, 1996.

19.34 "Specification for Unbonded Single Strand Tendons," Revised 1993, Post-tensioning Institute, Phoenix, Arizona, p. 20.

19.35 Breen, J.E., Burdet, O., Roberts, C., Sanders, D., Wollmann, G. and Falconer, B., "Anchorage Zone Requirements for Post-tensioned Concrete Girders," NCHRP Report 356, Transportation Research Board, National Academy Press, Washington, D.C., 1994.

19.36 Fenwick, R.C. and Lee, S.C., "Anchorage Zones in Prestressed Concrete Beams", Magazine of Concrete Research, Vol. 38, No. 135, June 1986, pp. 77-89.

19.37 Collins, M P. and Mitchell, D., "Prestressed Concrete Basics", Canadian Prestressed Concrete Institute, Ottawa, Ontario, 1987, (Chapter 9, pp 386-430).

19.38 ACI Committee 301, "Standard Specifications for Structural Concrete for Buildings (ACI 301-96)," American Concrete Institute, Farmington Hills, MI, 1996, p. 34. Also, ACI Manual of Concrete Practice.

19.39 Salmons, J.R. and McCrate, T.E., "Bond Characteristics of Untensioned Prestressing Strand," Journal of the Prestressed Concrete Institute, Vol. 22, No. 1, Jan.-Feb. 1977, pp. 52-65.

19.40 ACI Committee 215, "Considerations for Design of Concrete Structures Subjected to Fatigue Loading (ACI 215R-74)(Revised 1992)," American Concrete Institute, Farmington Hills, MI, 1992, p. 24. Also ACI Manual of Concrete Practice.

19.41 ACI-ASCE Committee 423, "Recommendations for Concrete Members Prestressed with Unbonded Tendons," ACI Structural Journal, Vol. 86, No. 3, May-June 1989, p. 312.

19.42 Barth, F., "Unbonded Post-tensioning in Building Construction Concrete Construction Engineering Handbook, CRC Press, 1997, pp. 12.32-12.47.

19.43 Seismic Committee of the New Zealand Prestressed Concrete Institute, "Recommendations for the Design and Detailing of Ductile Prestressed Concrete Frames for Seismic Loading", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 9, No. 2, June 1976, pp. 89-96.

19.44 Priestley, M.J.N. and Jian Ren Toa., "Seismic Response of Precast Prestressed Concrete Frames with Partially Debonded Tendons," PCI Journal, Jan.–Feb. 1993, pp. 58-69.

19.45 Park, R. and Thompson, K.J., "Cyclic Load Tests on Prestressed and Partially Prestressed Concrete Beam-column Joints", Journal of the Prestressed Concrete Institute, Vol. 22, No. 5, September-October 1977, pp. 84-110.

19.46 Thompson, K.J. and Park, R., "Ductility of Prestressed and Partially Prestressed Concrete Beam Sections", Journal of Prestressed Concrete Institute, Vol. 125, No. 2, March-April 1980, pp. 46-70.

19.47 Nishiyama, M., "Seismic Design of Prestressed Concrete Buildings", Bulletin of New Zealand National Society for Earthquake Engineering, Vol. 23, No. 4, December 1990, pp. 288-304.

19.48 FIP, "Flexural Ductility of Prestressed Concrete: Enhancement by Lateral Confinement", Federation Internationale de la Precontrainte, London, 1992, p. 19.

19.49 Blakeley, R.W.G. and Park R., "Seismic Resistance of Prestressed Concrete Beam-column Assemblies", Journal of the American Concrete Institute, Proc. Vol. 68, No. 9, September 1971, pp. 677-692.

19.50 Stanton, J.E., Stone, W.C. and Choek, G.S., "A Hybrid Reinforced Precast Frame for Seismic Regions," PCI Journal, Vol. 42, No. 2, 1997, pp. 20-42.

19.51 Priestley, M.J.N., "Overview of PRESSS Research Programme", PCI Journal, Vol. 36, No. 4, 1991, pp. 50-57.

19.52 Priestley, M.J.N., Sritharun, S., Contey, J.R. and Pampanin, S., "Preliminary Results and Conclusions from the PRESSS Five-storey Precast Concrete Test Building,", PCI Journal, Vol. 44, No. 6, 1999, pp. 42-67.

NOTES

APPENDIX CA – STRUT-AND-TIE MODELS

CA1 Notation

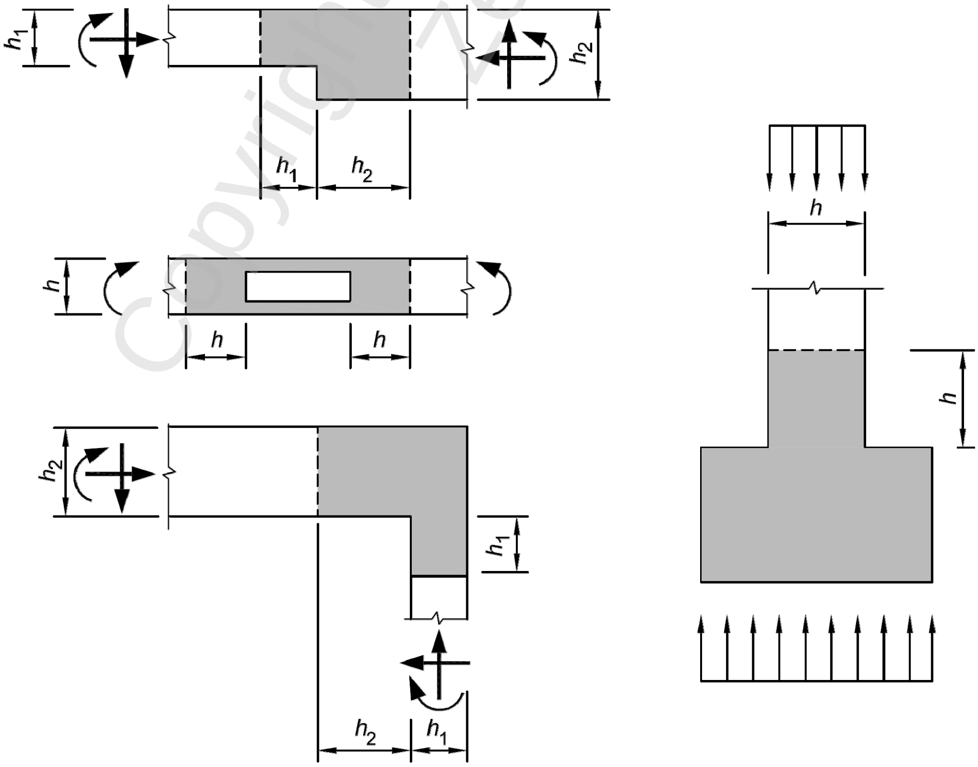
The following symbols, which appear in this Appendix, are additional to those used in Appendix A of Part 1. A3

b	thickness of concrete member forming a strut, mm
C, C_1, C_2, C_3	compression forces acting on a nodal zone, N
f_{si}	the stress in the ith layer of surface reinforcement, MPa
ℓ_a	length in which anchorage of a tie should occur, mm
ℓ_b	width of bearing, mm
R, R_1, R_2	reactions, N
T	tension force acting on a nodal zone, N
w_t	effective width of concrete concentric with a tie, used to dimension nodal zone, mm
$w_{t,max}$	maximum effective width of concrete concentric with a tie, mm
β_1	factor defined in 7.4.2.7(d)
w_{n1}, w_{n2}, w_{n3}	lengths of sides of nodal zones, mm

CA2 Definitions

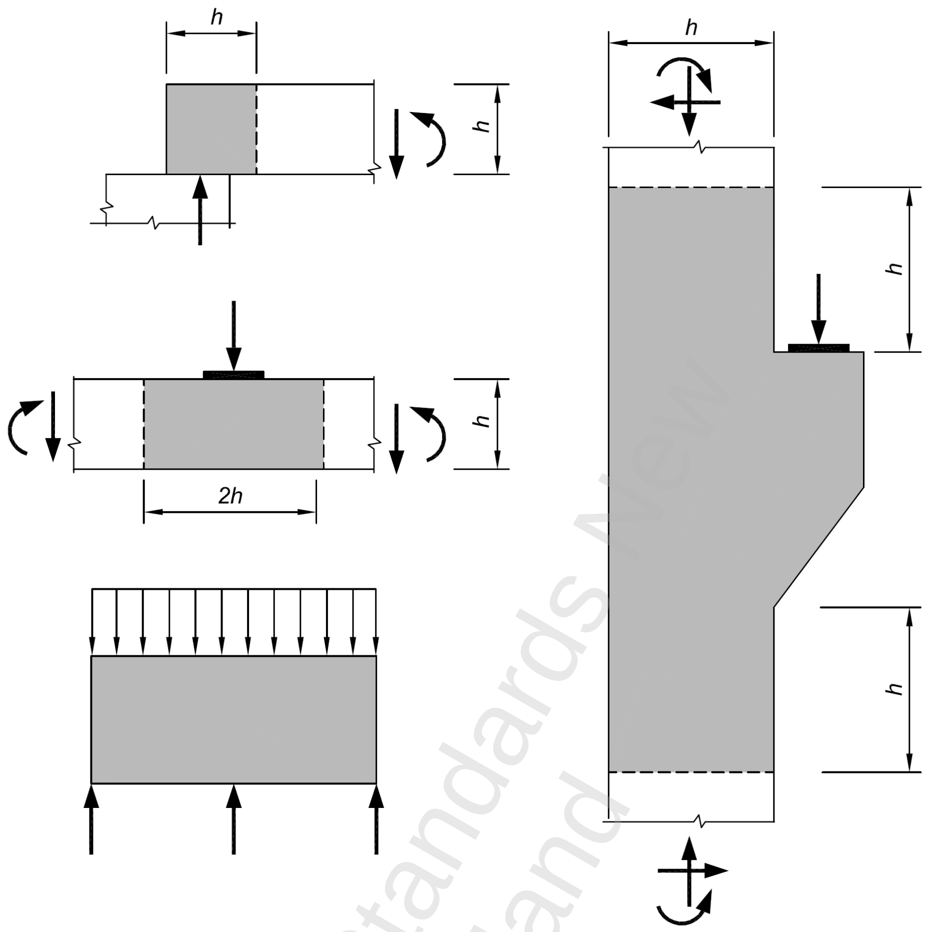
B-REGION. In general, any portion of a member outside of a D-region is a B-region.

DISCONTINUITY. A discontinuity in the stress distribution occurs at a change in the geometry of a structural element or at a concentrated load or reaction. St. Venant's principle indicates that the strains due to axial load and bending approach a linear distribution at a distance approximately equal to the overall height of the member, h, away from the discontinuity. For this reason, discontinuities are assumed to extend a distance h from the section where the load or change in geometry occurs. Figure CA.1(a) shows typical geometric discontinuities, and Figure CA.1(b) shows combined geometrical and loading discontinuities.



(a) Geometric discontinuities

Figure CA.1 – D-regions and discontinuities *(Continued on next page)*



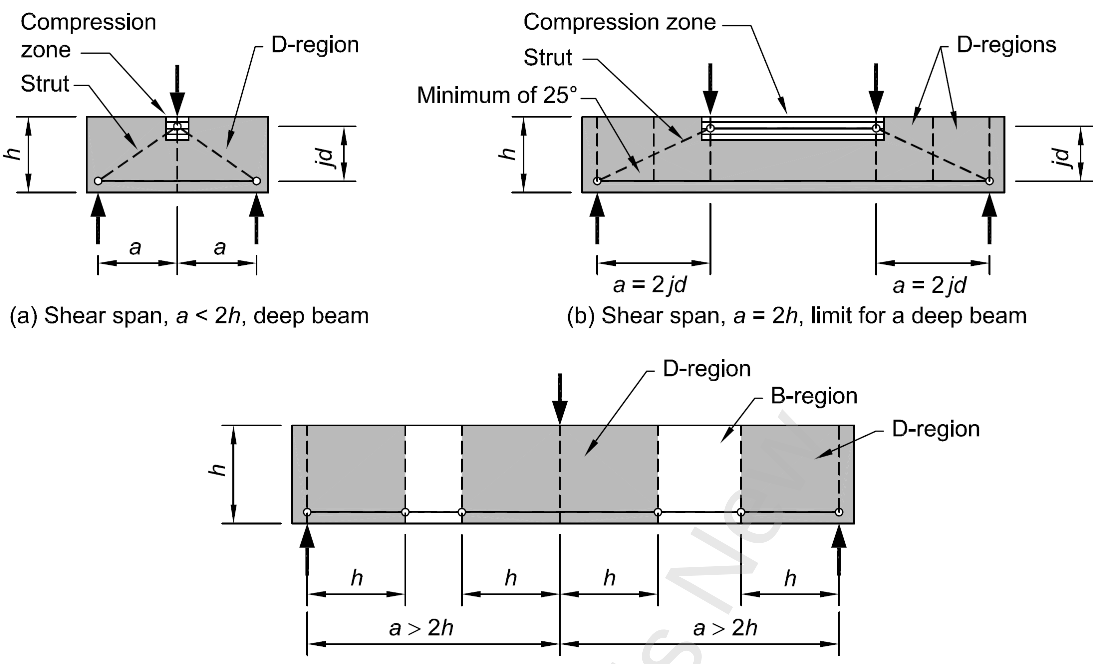
(b) Loading and geometric discontinuities

Figure CA.1 – D-regions and discontinuities (Continued)

D-REGION. The shaded regions of Figure CA.1(a) and (b) show typical D-Regions.[A.1] The plane sections assumption of 7.4.2.2 is not applicable in such regions.

Each shear span of the beam in Figure CA.2(a) is a D-region. If two D-regions overlap or meet as shown in (b), they can be considered as a single D-region for design purposes. The maximum length-to-depth ratio of such a D-region would be approximately two. Thus, the smallest angle between the strut and tie in a D-region is arctan 2 = 26.5°, rounded to 25°.

If there is a B-region between the D-regions in a shear span, loaded as shown in Figure CA.2(c), the strength of the shear span is governed by the strength of the B-region if the B- and D- regions have similar geometry and reinforcement[A.2]. This is because the shear strength of a B-region is less than the shear strength of a comparable D-region. Shear spans containing B-regions, the usual case in beam design, are designed for shear using the traditional shear design procedures from 7.5, ignoring D-regions.



(a) Shear span, $a < 2h$, deep beam

(b) Shear span, $a = 2h$, limit for a deep beam

(c) Shear span, $a > 2h$, slender beam

Figure CA.2 – Description of deep and slender beams

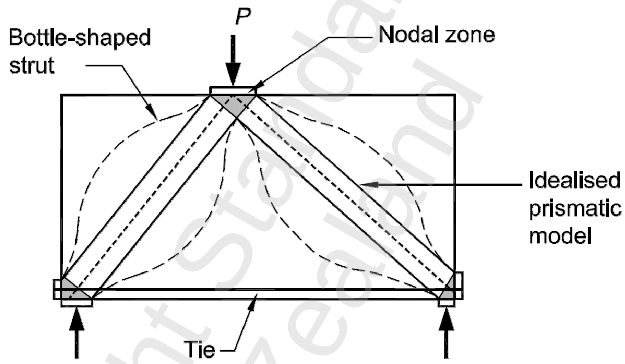
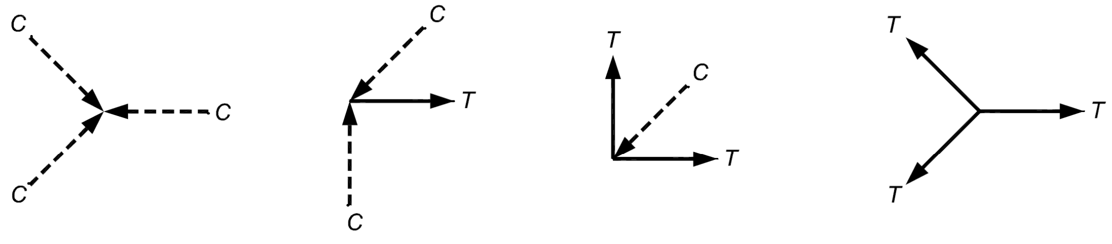


Figure CA.3 – Description of strut-and-tie model

DEEP BEAM. See Figure CA.2(a), Figure CA.2(b), and Figure CA.3 and 9.3.1.6 and 9.3.10.

NODE. For equilibrium, at least three forces should act on a node in a strut-and-tie model, as shown in Figure CA.4. Nodes are classified according to the signs of these forces. A C-C-C node resists three compressive forces, a C-C-T node resists two compressive forces and one tensile force, and so on.

NODAL ZONE. Historically, hydrostatic nodal zones as shown in Figure CA.5 were used. These were largely superseded by what are called extended nodal zones, shown in Figure CA.6.



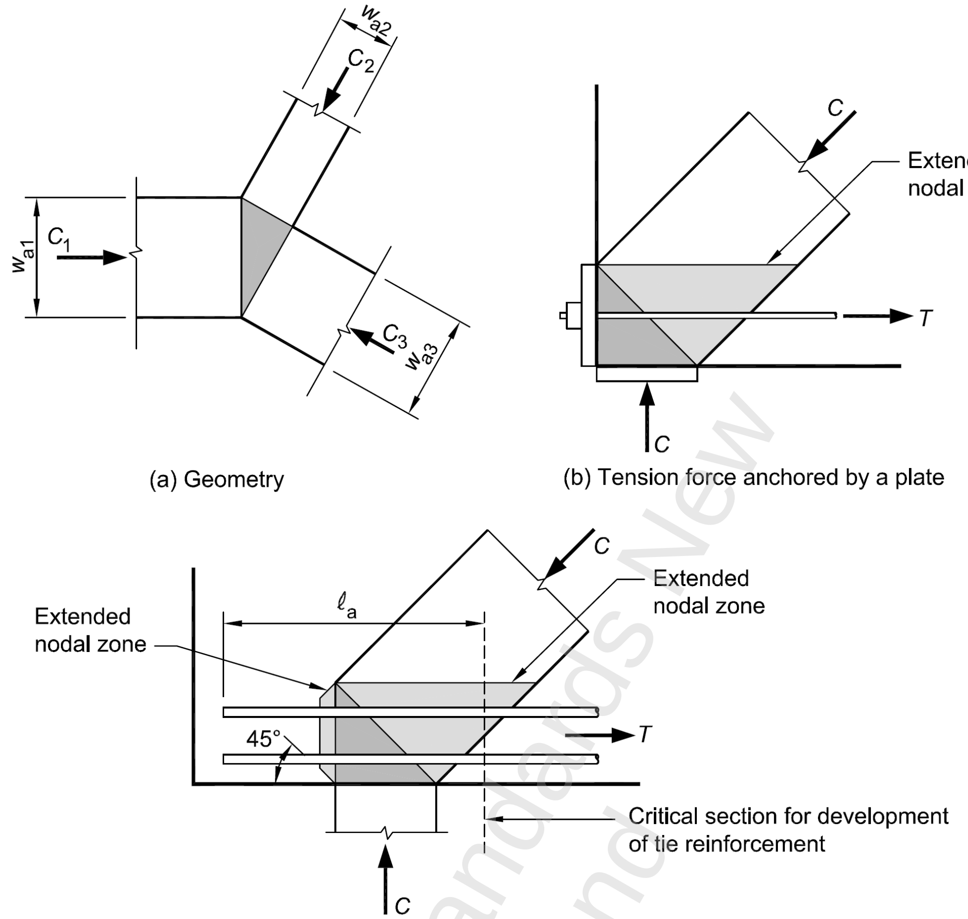
(a) C-C-C Node

(b) C-C-T Node

(c) C-T-T Node

(d) T-T-T Node

Figure CA.4 – Classification of nodes



(a) Geometry

(b) Tension force anchored by a plate

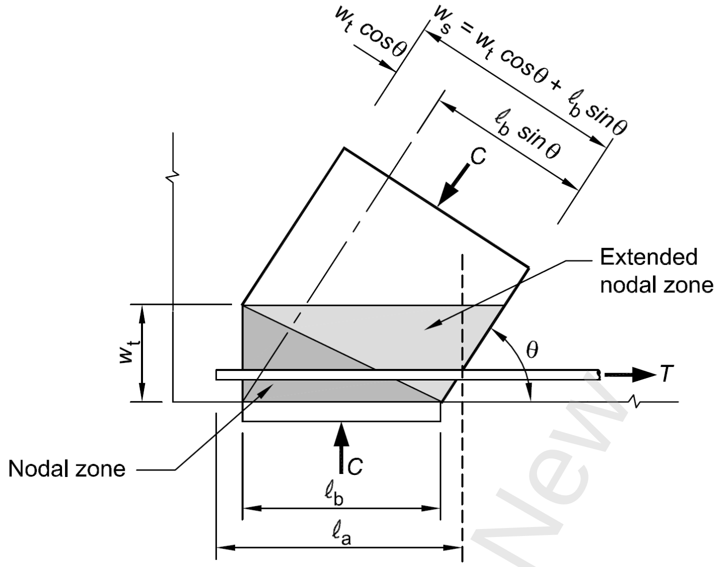
(c) Tension force anchored by bond

Figure CA.5 – Hydrostatic nodes

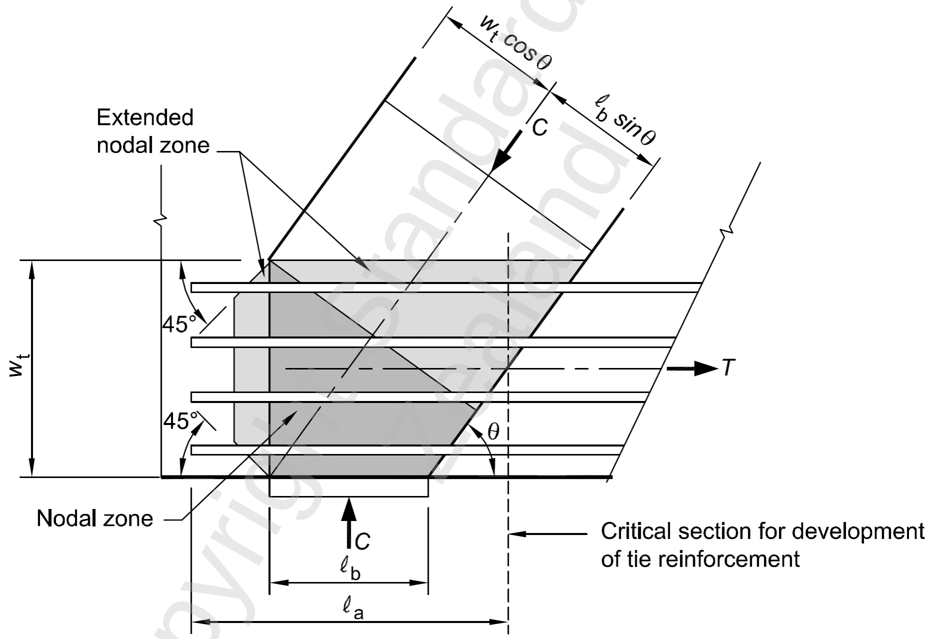
HYDROSTATIC NODAL ZONE. A hydrostatic nodal zone has loaded faces perpendicular to the axes of the struts and ties acting on the node and has equal stresses on the loaded faces. Figure CA.6(a) shows a C-C-C nodal zone. If the stresses on the face of the nodal zone are the same in all three struts, the ratios of the lengths of the sides of the nodal zone, w_{n1}: w_{n2}: w_{n3} are in the same proportions as the three forces C_1: C_2: C_3.

These nodal zones are called hydrostatic nodal zones because the in-plane stresses are the same in all directions.

Strictly speaking, this terminology is incorrect because the in-plane stresses are not equal to the out-of-plane stresses.



(a) One layer of steel



(b) Distributed steel

Figure CA.6 – Extended nodal zone showing the effect of the distribution of the force

A C-C-T nodal zone can be represented as a hydrostatic nodal zone if the tie is assumed to extend through the node to be anchored by a plate on the far side of the node, as shown in Figure CA.5(b), provided that the size of the plate results in bearing stresses that are equal to the stresses in the struts. The bearing plate on the left side of Figure CA.5(b) is used to represent an actual tie anchorage. The tie force can be anchored by a plate, or through development of straight or hooked bars, as shown in Figure CA.5(c).

In the nodal zone shown in Figure CA.7, the reaction R equilibrates the vertical components of the forces C_1 and C_2. Frequently, calculations are easier if the reaction R is divided into R_1, which equilibrates the vertical component of C_1 and R_2, which equilibrates the vertical component of the force C_2, as shown in Figure CA.7.

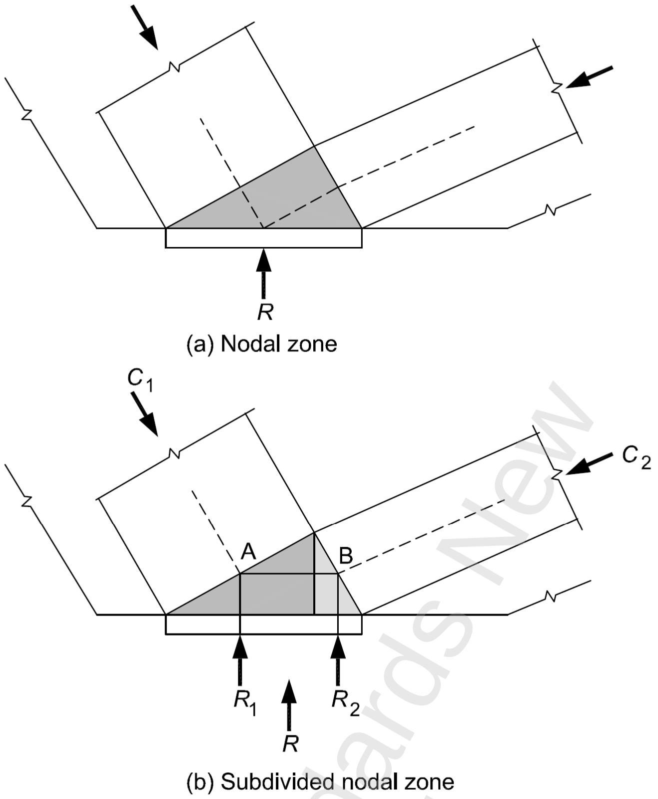


Figure CA.7 – Subdivision of nodal zone

STRUT. In design, struts are usually idealised as prismatic compression members, as shown by the straight line outlines of the struts in Figure CA.3. If the effective compression strength f_{cu} differs at the two ends of a strut, due either to different nodal zone strengths at the two ends, or to different bearing lengths, the strut is idealised as a uniformly tapered compression member.

BOTTLE-SHAPED STRUTS. A bottle-shaped strut is a strut located in a part of a member where the width of the compressed concrete at mid-length of the strut can spread laterally [A.1, A.3].

The curved dashed outlines of the struts in Figure CA.3 and the curved solid outlines in Figure CA.8 approximate the boundaries of bottle-shaped struts. A split cylinder test is an example of a bottle-shaped strut. The internal lateral spread of the applied compression force in such a test leads to a transverse tension that splits the specimen.

To simplify design, bottle-shaped struts are idealised either as prismatic or as uniformly tapered, and crack control reinforcement from A5.3 is provided to resist the transverse tension. The amount of confining transverse reinforcement can be computed using the strut-and-tie model shown in Figure CA.8(b) with the struts that represent the spread of the compression force acting at a slope of 1:2.5 to the axis of the applied compressive force. Alternatively for f'_c not exceeding 40 MPa, Equation A–4 can be used. The cross-sectional area A_c of a bottle-shaped strut, is taken as the smaller of the cross-sectional areas at the two ends of the strut. (See Figure CA.8(a)).

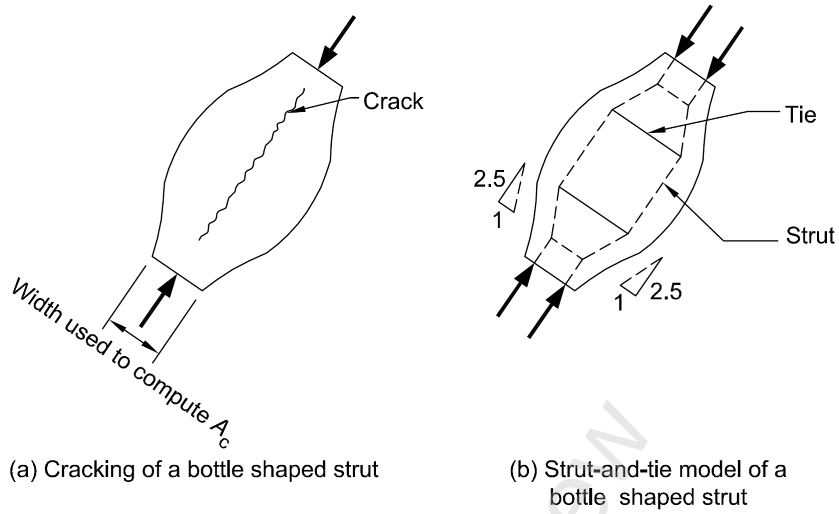


Figure CA.8 – Bottle-shaped strut

STRUT-AND-TIE MODEL. The components of a strut-and-tie model of a single-span deep beam loaded with a concentrated load are identified in Figure CA.3. The cross-sectional dimensions of a strut or tie are designated as thickness and width, both perpendicular to the axis of the strut or tie. Thickness is perpendicular to the plane of the truss model, and width is in the plane of the truss model.

TIE. A tie consists of reinforcement or prestressing steel plus a portion of the surrounding concrete that is concentric with the axis of the tie. The surrounding concrete is included to define the zone in which the forces in the struts and ties are to be anchored. The concrete in a tie is not used to resist the axial force in the tie. Although not considered in design, the surrounding concrete will reduce the elongation of the tie, especially at service loads.

CA3 Scope and limitations

References A.1, A.4 and A.5 provide methodologies for establishing internal forces.

CA4 Strut-and-tie model design procedure

CA4.1 *Truss models*

The truss model described in A4.1 is referred to as a strut-and-tie model. Details of the use of strut-and-tie models are given in References A.1, A.2, A.3, A.6, A.7, A.8 and A.9. The design of a D-region includes the following four steps:

(a) Define and isolate each D-region;

(b) Compute resultant forces on each D-region boundary;

(c) Select a truss model to transfer the resultant forces across the D-region. The axes of the struts and ties, respectively, are chosen to approximately coincide with the axes of the compression and tension fields. The forces in the struts and ties are computed;

(d) The effective widths of the struts and nodal zones are determined considering the forces from Step (c) and the effective concrete strengths defined in A5.2 and reinforcement is provided for the ties considering the steel strengths defined in A6.1. The reinforcement should be anchored in the nodal zones.

Strut-and-tie models represent strength (ultimate) limit states and designers should also comply with the requirements for serviceability in the Standard. Deflections of deep beams or similar members can be estimated using an elastic analysis to analyse the strut-and-tie model. In addition, the crack widths in a tie can be checked using 2.4.4.6 assuming the tie is encased in a prism of concrete corresponding to the area of tie from CA6.2.

A number of different strut and tie models can be used in any situation. The optimum one is that which requires the least strain energy, which generally corresponds to the arrangement with the shortest length of ties. This is a useful guide to selecting an appropriate strut and tie solution.

Strut and tie arrangements based on elastic analyses of elastic models are not always appropriate as such models fail to recognise the redistribution that occurs due to stiffness changes associated with cracking, which must occur before reinforcement can act in tension.

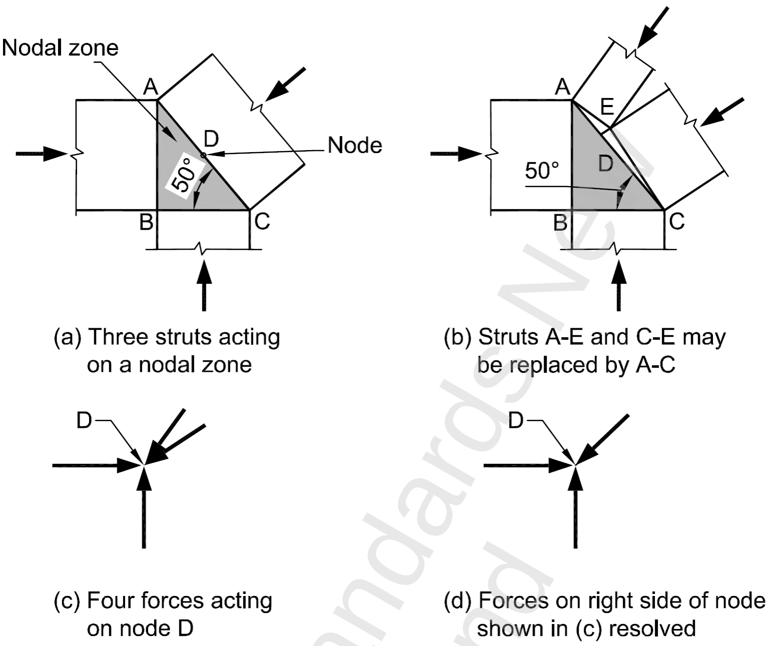


Figure CA.9 – Resolution of forces on a nodal zone

CA4.3 *Geometry of truss*

The struts, ties, and nodal zones making up the strut-and-tie model all have finite widths that should be taken into account in selecting the dimensions of the truss. Figure CA.6 shows a node and the corresponding nodal zone. The vertical and horizontal forces equilibrate the force in the inclined strut. If the stresses are equal in all three struts, a hydrostatic nodal zone can be used and the widths of the struts will be in proportion to the forces in the struts.

If more than three forces act on a nodal zone in a two-dimensional structure, as shown in Figure CA.9(b), it is generally necessary to resolve some of the forces to end up with three intersecting forces. The strut forces acting on faces A-E and C-E in Figure CA.9(b) can be replaced with one force acting on face A-C. This force passes through the node at D.

Alternatively, the strut-and-tie model could be analysed assuming all the strut forces acted through the node at D, as shown in Figure CA.9(c). In this case, the forces in the two struts on the right side of node D can be resolved into a single force acting through point D, as shown in Figure CA.9(d).

If the width of the support in the direction perpendicular to the member is less than the width of the member, transverse reinforcement may be required to restrain vertical splitting in the plane of the node. This can be modelled using a transverse strut-and-tie model. (Figure CA.13)

CA4.5 *Minimum angle between strut and tie*

The angle between the axes of struts and ties acting on a node should be large enough to mitigate cracking and to avoid incompatibilities due to shortening of the struts and lengthening of the ties occurring in almost the same directions. This limitation on the angle prevents modelling the shear spans in slender beams using struts inclined at less than 25° from the longitudinal steel. See Reference A.8.

Using a 25° strut angle, where there is only one strut, can lead to excessive diagonal cracking in the serviceability limit state. To prevent this the strut angle should be limited to 35°.

It should be noted that a strut angle of 25° can result in significant stress redistribution from the initial elastic condition. For example using a single strut to resist the flexure and shear in the shear span of the beam shown in Figure CA.10 on the right hand side can lead to excessive crack widths in the outside diagonal cracks in the serviceability limit state. To avoid this problem, which arises when there is a single diagonal strut that acts, it is recommended that a strut angle of 35° or more should be used. This limit should not apply where a series of diagonal struts are assumed to act, as illustrated in Figure CA.10 on the left hand side. (See reference A.10.)

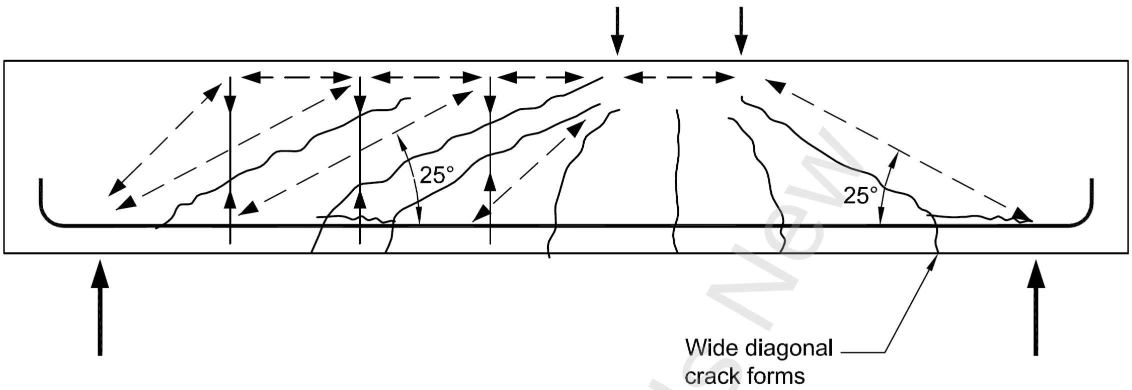


Figure CA.10 – Single and multiple struts

CA4.6 *Design basis*

Factored (ultimate) loads are applied to the strut and tie model, and the forces in all the struts, ties, and nodal zones are computed. If several loading cases exist, each should be investigated. The strut-and-tie model, or models, are analysed for the loading cases and, for a given strut, tie, or nodal zone, F^* is the largest force in that element for all loading cases.

CA5 Strength of struts

CA5.1 *Strength of strut in compression*

The width of strut w_s used to compute A_c is the smaller dimension perpendicular to the axis of the strut at the ends of the strut. This strut width is illustrated in Figure CA.5(a) and Figure CA.6(a) and (b). In two-dimensional structures, such as deep beams, the thickness of the struts may be taken as the width of the member.

CA5.2 *Effective compressive strength of concrete*

The strength coefficient, $\alpha_1 f'_c$, in Equation A–3, represents the effective concrete strength under sustained compression, similar to that used in Equation 10–10.

(a) The value of β_s in A5.2(a) applies to a strut equivalent to the rectangular stress block in a compression zone in a beam or column.

(b) The value of β_s in A5.2(b) applies to bottle-shaped struts as shown in Figure CA.3. The internal lateral spread of the compression forces can lead to splitting parallel to the axis of the strut near the ends of the strut, as shown in Figure CA.8. Reinforcement placed to resist the splitting force restrains crack width, allows the strut to resist more axial load, and permits some redistribution of force.

The value of β_s in A5.2(b) includes the correction factor, γ, for lightweight concrete because the strength of a strut without transverse reinforcement is assumed to be limited to less than the load at which longitudinal cracking develops.

(iii) The value of β_s in A5.2(b)(iii) applies, for example, to compression struts in a strut-and-tie model used to design the longitudinal and transverse reinforcement of the tension flanges of beams and box girders, box girders, and walls. The low value β_s reflects that these struts need to transfer compression across cracks in a tension zone.

(iv) The value of β_s in A5.2(b)(iv) applies to strut applications not included in A5.2(a),(b)(i) and (b)(iii). Examples are struts in a beam web compression field in the web of a beam where parallel diagonal cracks are likely to divide the web into inclined struts, and struts are likely to be crossed by cracks at an angle to the struts (see Figure CA.11(a) and (b)). Clause A5.2(b)(iv) gives a reasonable lower limit on β_s except for struts described in A5.2 (b)(i) and (iii).

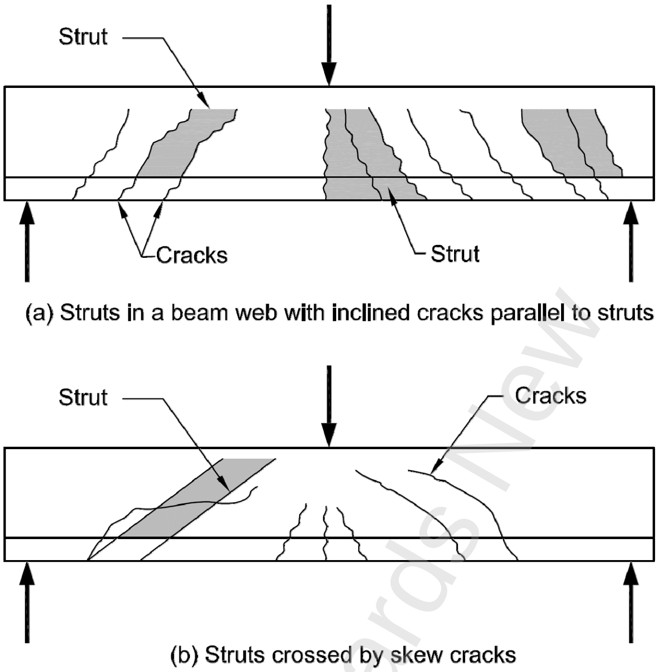


Figure CA.11 – Type of struts

CA5.3 *Reinforcement for transverse tension*

The reinforcement required by A5.3 is related to the tension force in the concrete due to the spreading of the strut, as shown in the strut-and-tie model in Figure CA.8(b). Clause A5.3 allows designers to use local strut-and-tie models to compute the amount of transverse reinforcement needed in a given strut. The compressive forces in the strut may be assumed to spread at a 2.5:1 slope, as shown in Figure CA.8(b). For concrete strengths not exceeding 40 MPa the amount of steel required by Equation A–4 is deemed to satisfy A5.3.

Figure CA.12 shows two layers of reinforcement crossing a cracked strut. If the crack opens without shear slip along the crack, the vertical bars in the figure will cause a stress of perpendicular to the strut, where the subscript 1 refers to the vertical bars in Figure CA.12. Equation A–4 is written in terms of a reinforcement ratio rather than a stress to simplify the calculation. The summation adds the ratio resulting from the perpendicular bars, denoted by the subscript 2 in Figure CA.12.

Often, the confinement reinforcement given in A5.3 is difficult to place in three-dimensional structures such as pile caps. If this reinforcement is not provided, the value of f_{cu} given in Equation A–8 is used.

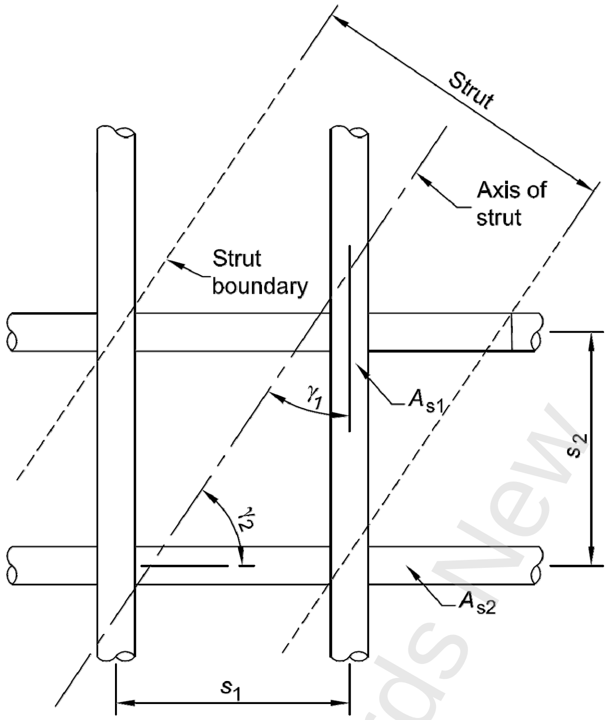


Figure CA.12 – Reinforcement crossing a strut

CA5.3.2 *Placement of reinforcement*

In a corbel with a shear span-to-depth ratio less than 1.0, the confinement reinforcement required to satisfy A5.3 is usually provided in the form of horizontal stirrups crossing the inclined compression strut, as shown in Figure C16..

CA5.4 Increased strength of strut due to confining reinforcement

The design of tendon anchorage zones for prestressed concrete sometimes uses confinement to enhance the compressive strength of the struts in the local zone. Confinement of struts is discussed in References A.6 and A.11.

CA5.5 Increased strength of strut due to compression reinforcement

The strength added by the reinforcement is given by the last term in Equation A–5. The stress f_s' is the reinforcement in a strut at nominal strength can be obtained from the strains in the strut when the strut crushes. For Grade 300 or 500 reinforcement, f_s' can be taken as f_y.

CA6 Strength of ties

CA6.2 Axis and width of tie

The effective tie width assumed in design w_t can vary between the following limits, depending on the distribution of the tie reinforcement.

(a) If the bars in the tie are in one layer, the effective tie width can be taken as the diameter of the bars in the tie plus twice the cover to the surface of the bars, as shown in Figure CA.6(a); and

(b) A practical upper limit of the tie width (upper limit of amount of reinforcement that may be anchored at the node) can be taken as the width corresponding to the width in a hydrostatic nodal zone, calculated as:

$$w_{t,max} = F_{nt}/f_{cu}\, b \quad \text{(Eq. CA-1)}$$ A3

where f_{cu} is the applicable effective compression strength of the nodal zone given in A7.2. If the tie width exceeds the value from (a) above, the tie reinforcement should be distributed approximately uniformly over the width and thickness of the tie, as shown in Figure CA.6(b).

CA6.3 Anchoring of tie reinforcement

Anchorage of ties often requires special attention in nodal zones of corbels or in nodal zones adjacent to exterior supports of deep beams. The reinforcement in a tie should be anchored before it leaves the extended nodal zone at the point defined by the intersection of the centroid of the bars in the tie and the extensions of the outlines of either the strut or the bearing area. In Figure CA.6(a) and (b), this occurs where the outline of the extended nodal zone is crossed by the centroid of the reinforcement in the tie. Some of the anchorage may be achieved by extending the reinforcement through the nodal zone as shown in Figure CA.5(c), and developing it beyond the nodal zone. If the tie is anchored using 90° hooks, the hooks should be confined within the reinforcement extending into the beam from the supporting member to avoid cracking along the outside of the hooks in the support region.

In deep beams, hairpin bars spliced with the tie reinforcement can be used to anchor the tension tie forces at exterior supports, provided the beam width is large enough to accommodate such bars.

Figure CA.13 shows two ties anchored at a nodal zone. Development is required where the centroid of the tie crosses the outline of the extended nodal zone.

The development length of the tie reinforcement can be reduced through hooks, mechanical devices, additional confinement, or by splicing it with several layers of smaller bars.

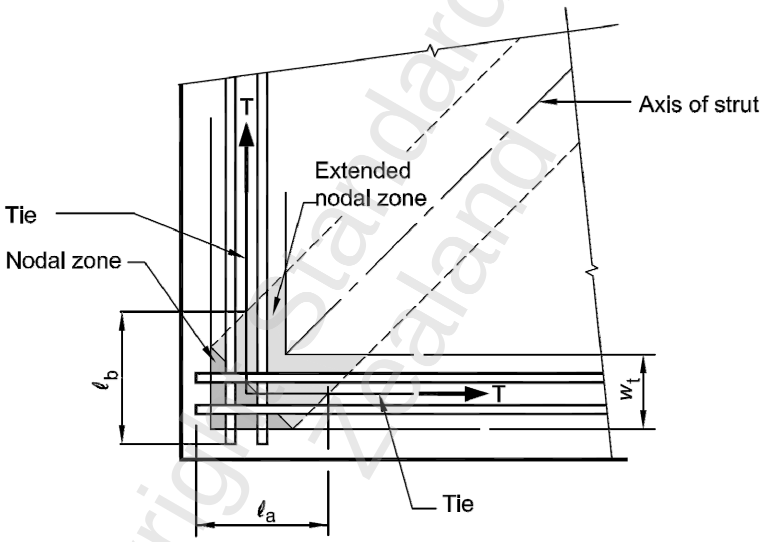


Figure CA.13 – Extended nodal zone anchoring two ties

CA7 Strength of nodal zones

CA7.1 *Nominal compression strength*

If the stresses in all the struts meeting at a node are equal, a hydrostatic nodal zone can be used. The faces of such a nodal zone are perpendicular to the axes of the struts, and the widths of the faces of the nodal zone are proportional to the forces in the struts.

Assuming the principal stresses in the struts and ties act parallel to the axes of the struts and ties, the stresses on faces perpendicular to these axes are principal stresses, and A7.1(a) is used. If, as shown in Figure CA.6(b), the face of a nodal zone is not perpendicular to the axis of the strut, there will be both shear stresses and normal stresses on the face of the nodal zone. Typically, these stresses are replaced by the normal (principal compression) stress acting on the cross-sectional area A_c of the strut, taken perpendicular to the axis of the strut as given in A7.1(a).

In some cases, A7.1(b) requires that the stresses be checked on a section through a subdivided nodal zone. The stresses are checked on the least area section which is perpendicular to a resultant force in the nodal zone. In Figure CA.7(b), the vertical face which divides the nodal zone into two parts is stressed by the resultant force along A-B. The design of the nodal zone is governed by the critical section from A7.1(a) or A7.1(b), whichever gives the highest stress.

CA7.2 *Compressive stress on face of nodal zone*

The nodes in two-dimensional members, such as deep beams, can be classified as C-C-C if all the members intersecting at the node are in compression; as C-C-T nodes if one of the members acting on the node is in tension; and so on, as shown in Figure CA.4. The effective compressive strength of the nodal zone is given by Equation A–8, as modified by A7.2(a) – (c) apply to C-C-C nodes, C-C-T nodes, and C-T-T or T-T-T nodes, respectively.

The β_n values reflect the increasing degree of disruption of the nodal zones due to the incompatibility of tension strains in the ties and compression strains in the struts. The stress on any face of the nodal zone or on any section through the nodal zone should not exceed the value given by equation, as modified by A7.2(a) – (c).

CA7.3 *Nodal zones for three-dimensional strut-and-tie models*

This description of the shape and orientation of the faces of the nodal zones is introduced to simplify the calculations of the geometry of a three-dimensional strut-and-tie model.

CA8.3 *Openings in walls modelled by strut and tie*

Reference A.12 provides a capacity design approach for determining the compression force.

REFERENCES

A.1 Schlaich, J., Schäfer, K. and Jennewien, M., "Toward a Consistent Design of Structural Concrete," PCI Journal, Vol. 32, No. 3, May-June, 1987, pp. 74-150.

A.2 Collins, M.P. and Mitchell, D., "Prestressed Concrete Structures", Prentice Hall Inc., Englewood Cliffs, NJ, 1991, p. 766.

A.3 MacGregor, J.G., "Reinforced Concrete: Mechanics and Design", 3rd Edition., Prentice Hall, Englewood Cliffs, NJ, 1997, p. 939.

A.4 MacGregor, J.G., "Reinforced Concrete – Mechanics and Design", Prentice Hall, New Jersey, 1988, p. 818.

A.5 Marty, P., "Basic Tools of Reinforced Concrete Beam Design", ACI Journal Proceedings, Vol. 82, No. 1, Jan.-Feb. 1983, pp. 46-56.

A.6 FIP Recommendations, "Practical Design of Structural Concrete", FIP-Commission 3, "Practical Design," Sept. 1996, Pub.: SE-TO, London, Sept. 1999.

A.7 Menn, C., "Prestressed Concrete Bridges", Birkhäuster, Basle, p. 535.

A.8 Muttoni, A., Schwartz, J. and Thürlimann, B., "Design of Concrete Structures with Stress Fields," Birkhäuster, Boston, Mass., 1997, p. 143.

A.9 Joint, ACI-ASCE Committee 445, "Recent Approaches to Shear Design of Structural Concrete," ASCE Journal of Structural Engineering, Dec., 1998, pp. 1375-1417.

A.10 Watstein, D and Mathey, R.C., "Strains in Beams Having Diagonal Cracks", ACI Journal Proc., Vol. 55, No. 6, Dec. 1956, pp. 717-728.

A.11 Bergmeister, K., J.E. and Jirsa, J.O., "Dimensioning of the Notes and Development of Reinforcement," IASE Colloquium Stuttgart 1991, International Association for Bridge and Structural Engineering Zurich, 1991, pp. 551-556.

A.12 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley and Sons, New York, 1992, p. 768.

NOTES

APPENDIX CB – SPECIAL PROVISIONS FOR THE SEISMIC DESIGN OF DUCTILE JOINTED PRECAST CONCRETE STRUCTURAL SYSTEMS

CB2 Definitions

Precast concrete structural systems are classified according to the type of connection between the precast concrete elements. The two broad categories of precast concrete structural systems identified are "jointed" and "equivalent monolithic". Definitions are given for jointed systems, hybrid systems and equivalent monolithic systems. It is to be noted that hybrid systems are a special type of jointed system in which in addition to unbonded post-tensioned tendons, non-prestressed reinforcing steel or other means of energy dissipation are provided.

The term hybrid is used to signify a response behaviour intermediate between the non-linear elastic and near elasto-plastic, which maintains the self-centering properties of the former and a variable part of the energy dissipating properties of the latter. The typical hysteresis rule of a hybrid structural system during lateral loading is referred to as "flag-shape", being given by the sum of the self-centering and the energy dissipation contributions (see Figure CB.1).

In the equivalent monolithic approach the connections between precast concrete elements are designed to have equivalent strength and toughness to their cast-in-place counterpart. That is, cast-in-place construction is emulated. Typical arrangements for ductile precast reinforced concrete equivalent monolithic beam-column sub assemblies are shown in Figure CB.2.

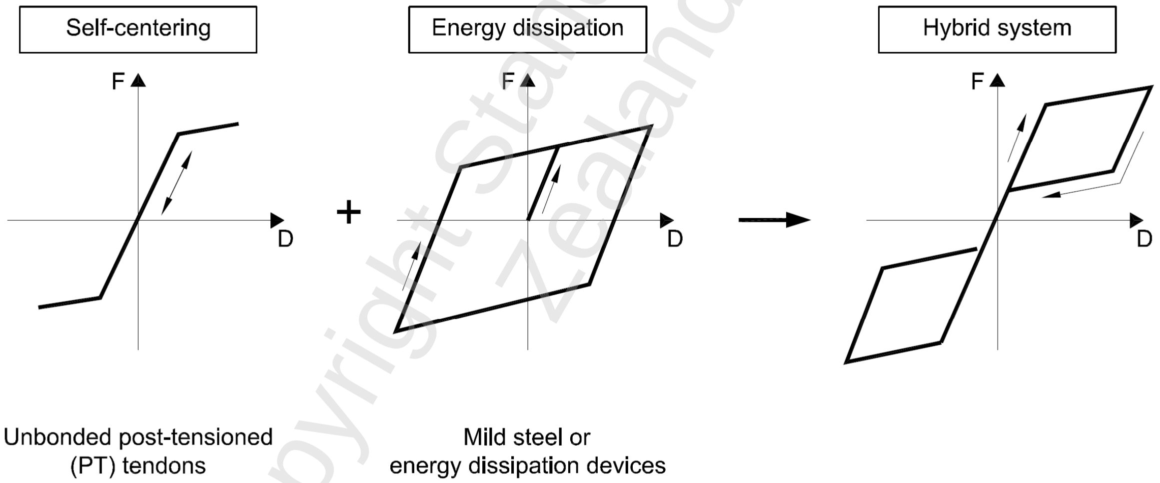


Figure CB.1 – Idealised flag-shape hysteretic rule for a hybrid system B.1

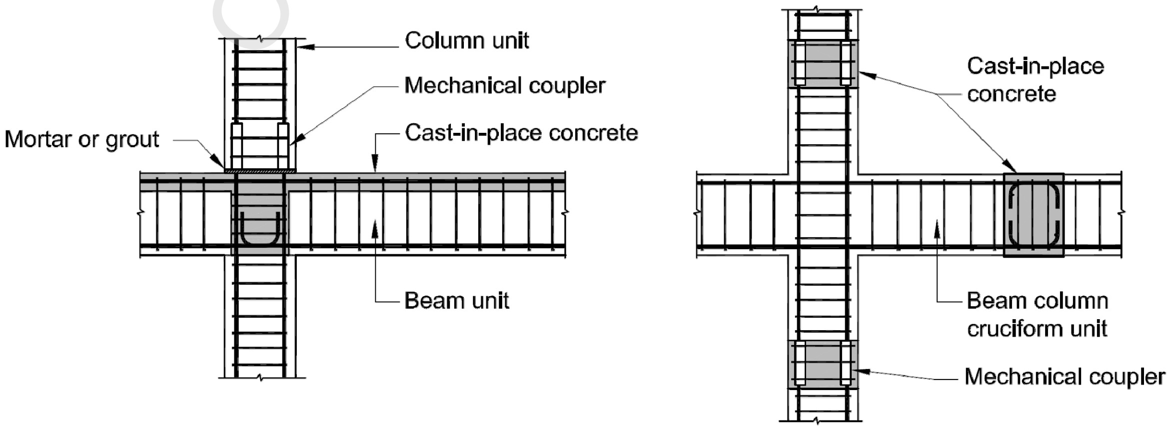


Figure CB.2 – Typical equivalent monolithic arrangements of precast reinforced concrete units and cast-in-place concrete B.1, B.2

CB3 Scope and limitations

This Appendix provides design provisions and commentary for the seismic design of ductile jointed precast concrete systems (moment resisting frames, structural walls or dual systems) consisting of precast concrete elements assembled by post-tensioning techniques with or without energy dissipation being provided by non-prestressed reinforcing steel or other energy dissipating devices.

Research into and development of precast and prestressed concrete structures for seismic areas has resulted in the experimental validation and practical applications of different innovative examples of jointed connections for both moment resisting frames and structural walls (see for example References, B.1, B.3, B.4, B.5, B.6, B.7, B.8, B.9, B.10, B.11 and B.12. A comprehensive state-of-the art report for both jointed and equivalent monolithic structural systems has been published by the *fib*, International Federation for Structural Concrete[B.1].

The commentary on this Appendix provides further explanation of the fundamental issues as well as references to other publications containing analytical or experimental evidence supporting the performance of well designed jointed systems in seismic regions.

Alternative jointed solutions with equivalent behaviour can also be developed and adopted, provided that evidence of satisfactory performance is supplied through both analytical and experimental investigations. In such cases, the ACI document on acceptance criteria based on structural testing [B.13] can be used.

CB4 General design approach

The design may be either force-based or displacement-based (see Reference B.1). Appropriate modifications to the structural parameters assumed in design should be made given the special features of jointed structural systems. Design evidence is given in Reference B.1, B.14 and B.15 and other publications.

A particularly efficient type of jointed ductile system is the hybrid system [B.1, B.5], where unbonded post-tensioning tendons with self-centering properties are adequately combined with longitudinal non-prestressed steel reinforcement or devices that can provide appreciable energy dissipation.

CB4.2 *Drift limits*

Drift limits are typically set recognising a correspondence between observed damage (both structural and non-structural) and displacement demand or, ultimately, strain levels in the materials. The particular gap opening mechanism (shown in Figure CB.3) which is typical of jointed ductile systems can guarantee negligible damage to the structural system components when compared to monolithic solutions. An exception to this is the yielding of the energy absorbing sacrificial fuses such as non-prestressed reinforcement or special dissipation devices which are able to be made replaceable. In addition, the design of non-structural elements linked to the structural elements (i.e. windows, claddings) can take advantage of a modular system where the inelastic demand is accommodated and controlled within defined locations. In general, reduced global (structural and non-structural) damage can be expected to occur, when compared with equivalent monolithic, for example cast-*in situ*, solutions. No difference to the damage of contents not directly connected to the structural skeleton can be anticipated.

In considering the above, pending further analytical and experimental evidence, allowances for higher level of drift limits corresponding to a damage control limit state are given.

CB4.3 *Self-centering and energy dissipation capabilities of hybrid structures*

As shown in Figure CB.1, the hysteresis loop of a jointed hybrid system can ideally be modelled as a combination of a non-linear elastic hysteresis loop to represent the moment contribution of the unbonded tendons, M_{pt} (self-centering characteristic), and of an appropriate dissipating rule M_s (for example elasto-plastic, friction, viscous-elastic or other) to represent the moment contribution of longitudinal non-prestressed steel or of alternative energy dissipation devices (for example in the form of vertical connectors between adjacent rocking wall panels). Hence:

$M(\theta) = M_{pt}(\theta) + M_s(\theta)$ .... (Eq. CB–1)

where (θ) implies the rotation at the particular gap opening.

The moment contribution of unbonded tendons and non-prestressed steel reinforcement or devices are calculated at each level of rotation about the centroid of the concrete compression forces in the section. A method for combining the hysteresis loops of prestressed concrete and non-prestressed concrete was first proposed by Thompson and Park[B.16]. Further details on modelling issues can be found in References B.1, B.17 and B.18.

Typical hybrid solutions adopted by the PRESSS programme in the USA [B.3, B.4, B.6, B.7] are shown in Figure CB.3(a) and (b) for beam-column sub-assemblies and walls, respectively.

When dealing with wall systems, the axial load action provides an additional self-centering contribution which should be either included in, or separately added to, the non-linear elastic behaviour associated with the contribution of the unbonded tendons.

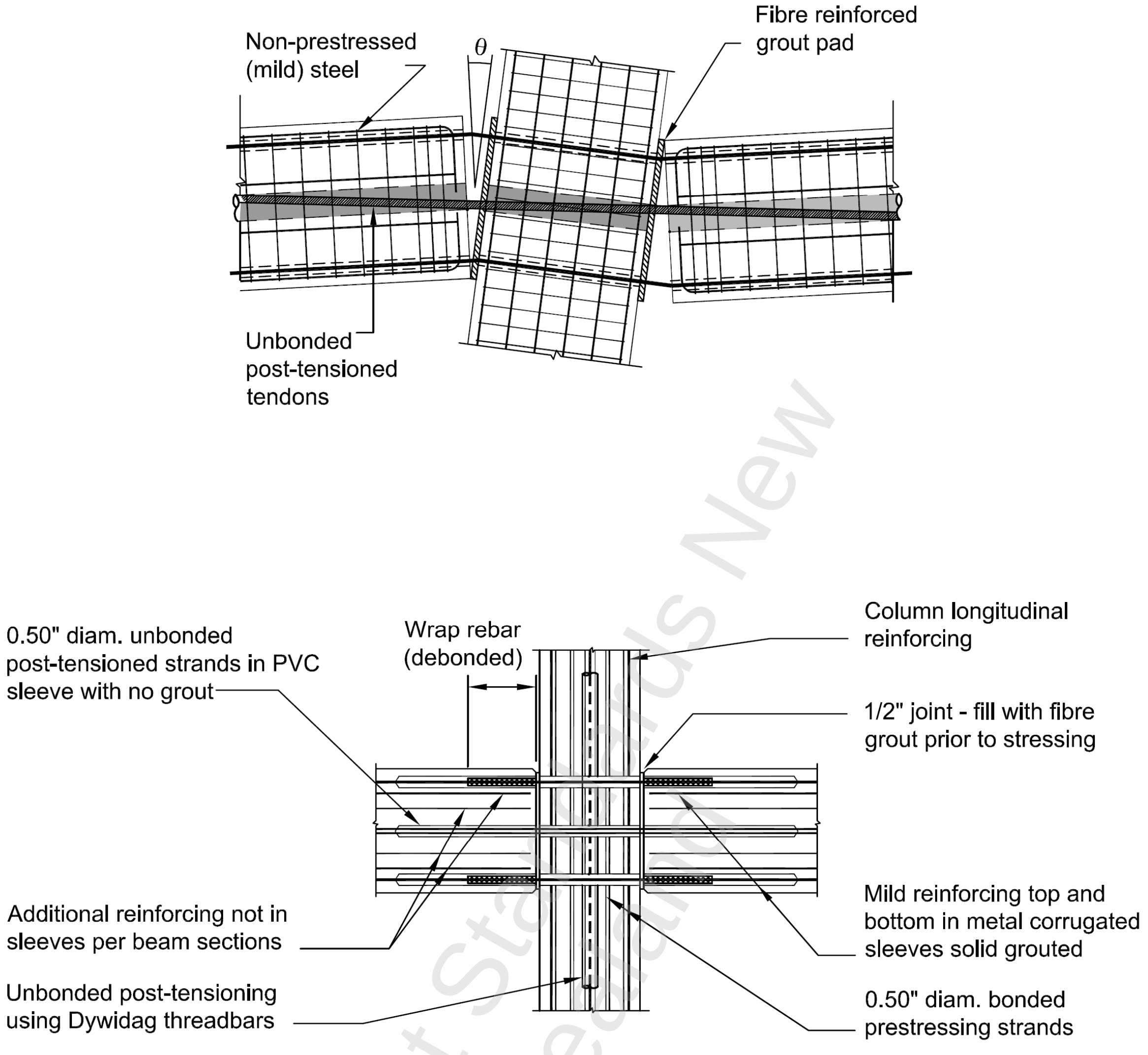
As illustrated in Figure CB.4(a), different combinations of post-tensioned (plus axial load) versus mild steel (non-prestressed steel or dissipating devices) giving the moment ratio, $(M_{pt} + M_N)/M_s$ corresponding to the target rotation/drift level will directly influence the key parameters of the hysteresis loop.

Lower and upper bounds are given respectively by a precast connection/system with unbonded tendons only (maximum self-centering, minimum energy dissipation) and by an equivalent monolithic system with non-prestressed steel reinforcement only (maximum energy dissipation, minimum self-centering).

The feasible residual deformation and equivalent viscous damping of the hysteretic rule can be adopted as the main design or assessment parameter of a connection or a whole structural system. Using simplified charts such as those shown in Figure CB.4(b) and (c) an adequate flexural strength ratio of post-tensioned steel (plus axial load) and non-prestressed steel, M_{pt}/M_s, can be defined in the preliminary design phase in order to satisfy the desired requirements or, vice-versa, the expected influences of this ratio on the overall behaviour can be predicted in an assessment procedure [B.1]. Note that the design charts presented as example in Figure CB.4(b) and (c) refer to a general hybrid hysteresis loop given by the combination of a Non-linear Elastic (NLE) rule and a degrading-stiffness Takeda rule with alternative loading and unloading parameters α and β.

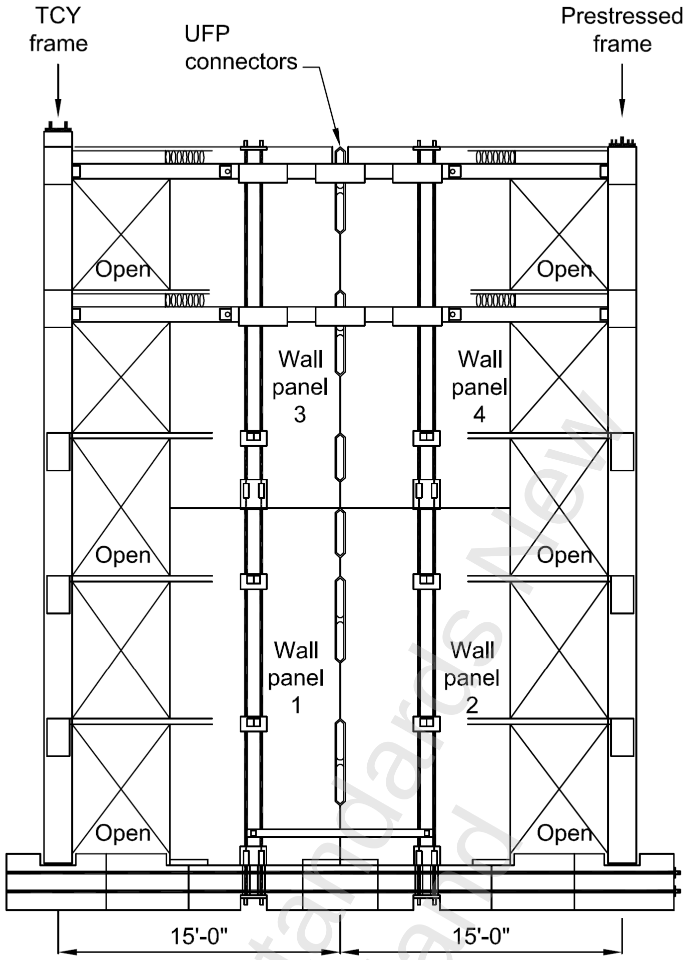
Results of research on the self-centering capabilities and passive energy dissipation of ductile hybrid structural systems include References B.1 and B.18. There has also been increased recognition and emphasis in publications on residual deformation as a performance criterion[B.19, B.20, B.21. B.22, B.23. B.24].

It is worth noting that the aforementioned condition (Equation B–1) for full-recentering refers to the jointed connection itself. Full recentering of the whole seismic resisting system can be still achieved by accounting for the overstrength provided by floor flange effects as well as by other yielding mechanisms occurring in the secondary systems due to the deformation induced by the primary structure.

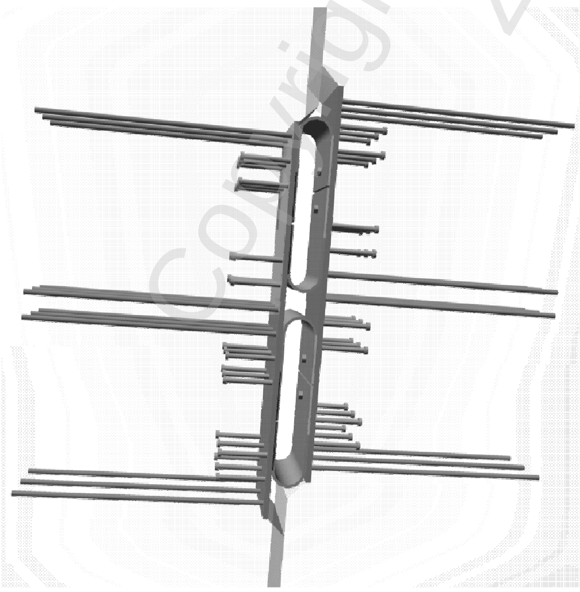


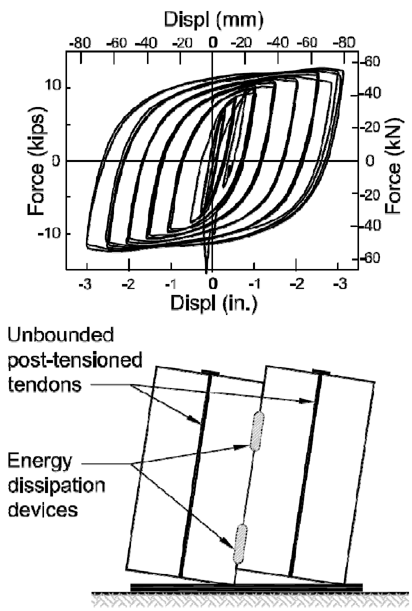
(a) Frame system

A3 **Figure CB.3 – Example of jointed (hybrid) systems and their mechanisms developed under the PRESSS programme** [B.4, B.6, B.7]



(b) Wall system





(c) UFP (U-shape Flexural Plate) energy dissipation devices

Figure CB.3 – Example of jointed (hybrid) systems and their mechanisms developed under the PRESSS programme [B.4, B.6, B.7] (continued)

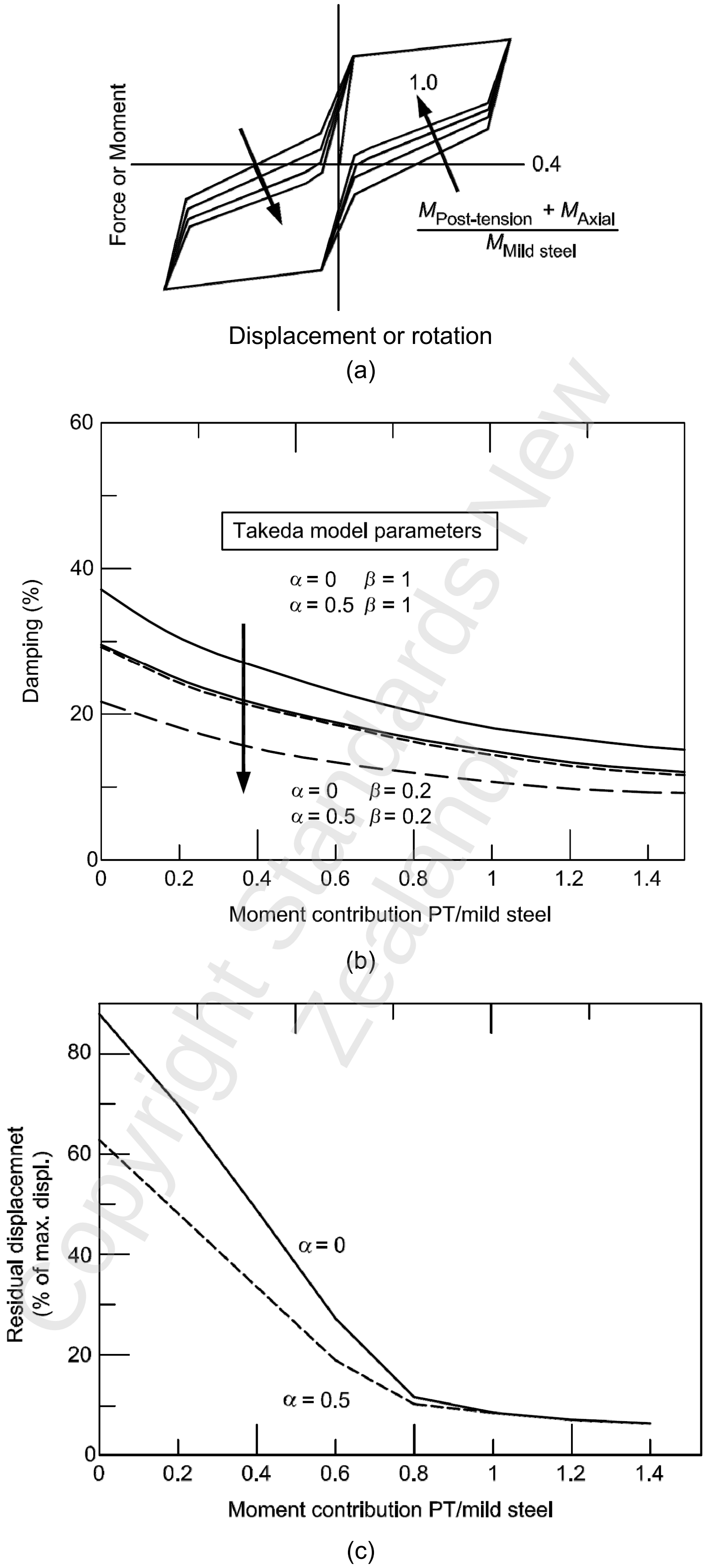
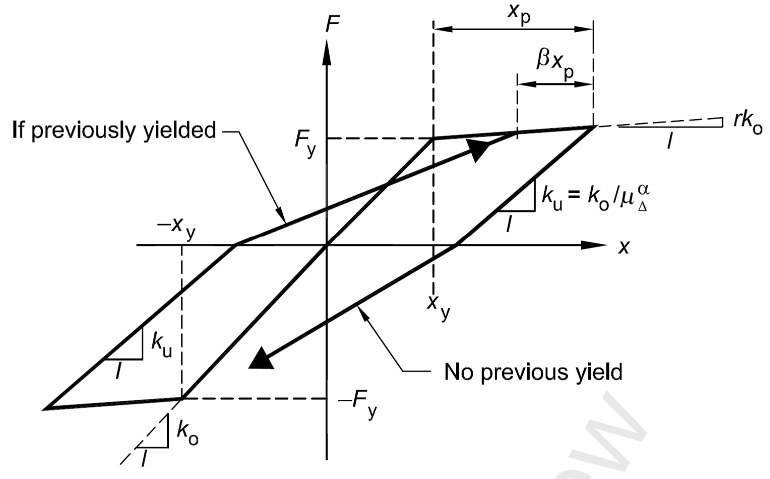


Figure CB.4 – Influence of the prestressing steel/non-prestressed steel moment contribution ratio on the key parameters of hybrid systems (equivalent viscous damping and residual displacement for a given ductility level) [B.1]



(d)

Figure CB.4 – Influence of the prestressing steel/non-prestressed steel moment contribution ratio on the key parameters of hybrid systems (equivalent viscous damping and residual displacement for a given ductility level) [B.1] (continued)

CB5 Behaviour of connections

CB5.1 *Inelastic behaviour of connections*

The critical sections occur at the interfaces of two connecting precast concrete members (beam-to-column, wall-to-wall) or between precast concrete members and the foundation. The opening and closing of the gap (Figure CB.5) is thus expected to result in much reduced or negligible damage in the adjacent structural elements. Since the unbonded tendons are designed to remain in the elastic range there will also be negligible or no residual deformation of the structure after an earthquake. The unbonded tendons will close the joints and provide a self-centering capacity [B.1].

The rocking mechanism of a hybrid beam-column is shown in Figure CB.5.

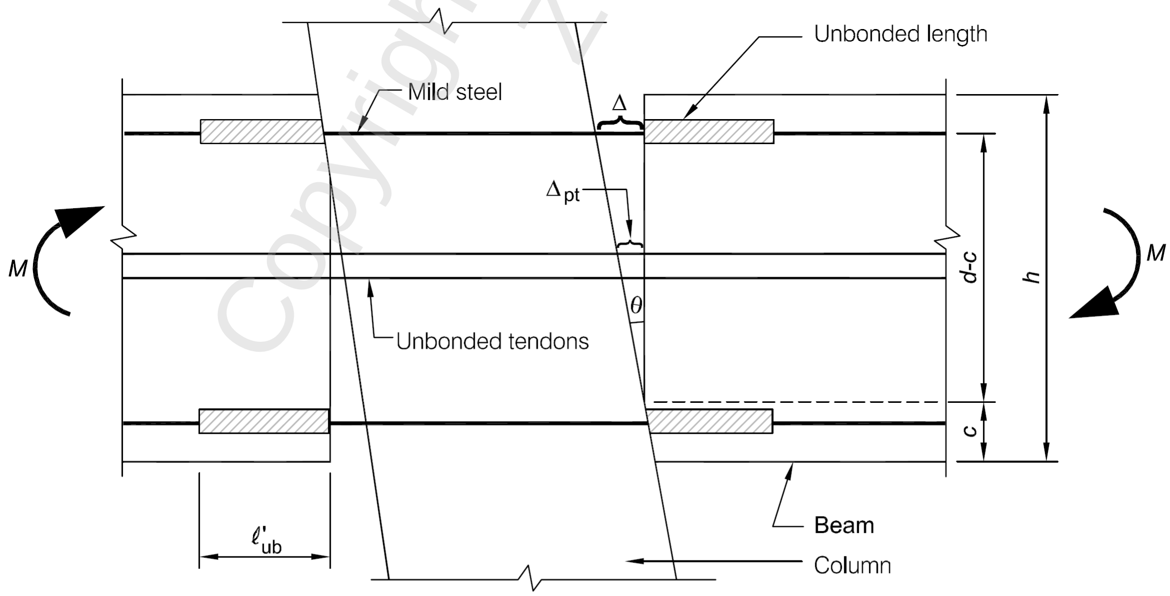


Figure CB.5 – Rocking mechanism of a beam-column hybrid connection [B.1, B.26]

CB5.4 *Shear transfer at critical connections*

The total vertical shear forces cannot be reliably transferred at the interfaces between precast concrete members by friction induced by post-tensioning. Therefore the design vertical shear force due to factored gravity loads should be transferred by other devices such as corbels or alternative solutions, leaving to the friction contribution induced by the post-tensioning action only the shear component due to seismic loads. The shear contribution due to friction induced by the tendons can be evaluated as $\phi\mu C_{c,}$ being ϕ the strength reduction factor corresponding to shear, μ the coefficient of friction at the interface and C_c the resultant compression force in the concrete at the interface.

When interface grout is used to accommodate the tolerances, special attention should be taken to avoid premature crushing of the grout pad at design level of drift: non-shrink grout should be used with a thickness not exceeding 30 mm. The specified compressive strength of the grout shall be equal to, or greater than f'_c in the structural members. Enhanced toughness and performance of the interface grout can be achieved using fibre reinforcement.

CB6 Design of moment resisting frames

CB6.1 *General*

The seismic provisions of Section 19 apply to moment resisting frames.

CB6.2 *Anchorage, location and longitudinal profile of the post-tensioned tendons*

A symmetrical arrangement of prestressing tendons in beams at the beam-column joint is preferred in order to minimize the elongation of the tendons during lateral loading.

CB6.3 *Prestressing force in beams*

Compliance must be with the upper and lower bounds for the initial prestressing force in beams.

CB6.4 *Evaluation of flexural strength at target inter-storey drift levels*

The evaluation of flexural strength at different inter-storey drift levels can be derived following a simplified procedure to determine the complete moment-rotation section behaviour for connections/systems subjected to local strain-incompatibility issues such as:

(a) Partially bonded or unbonded tendons;

(b) Unbonded length of non-prestressed steel reinforcement or energy dissipation devices;

(c) Hybrid connection with combinations of the above.

The conceptual flowchart of the simplified moment-rotation procedure shown in Figure CB.6, as proposed in Reference B.25, is intended to reflect that typically used for section analyses of monolithic systems. For a given rotation, θ the depth of the neutral axis, c, corresponds to a unique solution respecting both the equilibrium equations at a section level and compatibility conditions at a member level.

Referring to the peculiar mechanism (gap opening and closing at the critical interface) of the hybrid beam-column connection of Figure CB.5, the strain levels in the unbonded post-tensioned tendons, $\alpha_{pt}(\theta)$, and in the non-prestressed steel reinforcement (with a short unbonded length at the critical section) fracture, $\varepsilon_s(\theta)$, can be evaluated as follows:

$$\varepsilon_{pt} = \frac{n\Delta_{pt}}{\ell_{ub}} \qquad \text{(Eq. CB-2)}$$

$$\varepsilon_s = \frac{(\Delta - 2\Delta_{sp})}{\ell'_{ub}} \qquad \text{(Eq. CB–3)}$$

where n is the number of total joint openings along the beam (at beam-column interfaces); ℓ_{ub} and ℓ'_{ub} are the unbonded lengths in the tendons and in the non-prestressed steel reinforcement, respectively; Δ_{pt} and

Δ_s, are the elongations at the level of the tendons and of the non-prestressed steel reinforcement respectively; Δ_{sp} is the elongation due to strain penetration of the non-prestressed steel reinforcement (assumed to occur at both ends of the unbonded region).

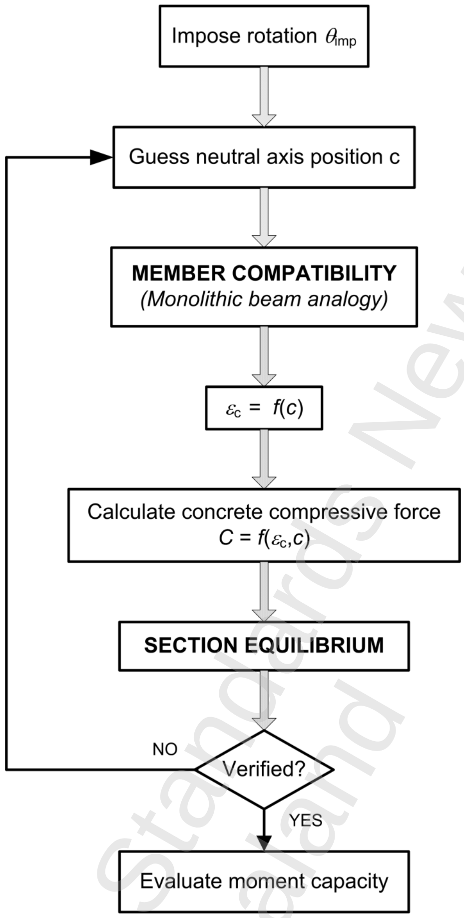


Figure CB.6 – Schematic flow chart of a complete moment-rotation procedure in presence of strain incompatibility [B.25]

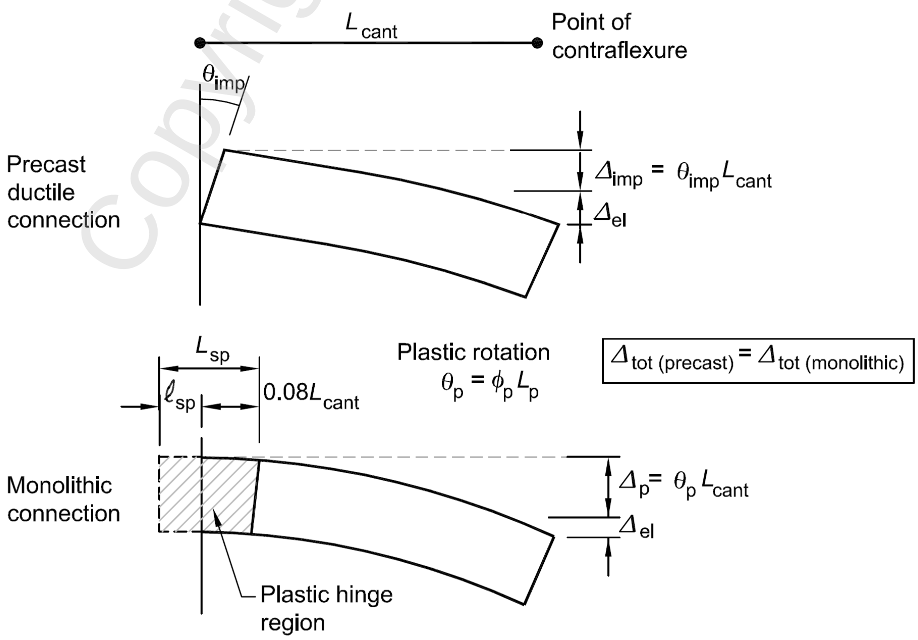


Figure CB.7 – Monolithic beam analogy for member compatibility condition

The member compatibility condition is introduced in the form of an analogy, in terms of global behaviour (beam-edge displacement), between a precast connection and an equivalent monolithic one [B.25]. The main assumption is that two equivalent (same geometry and reinforcement) precast and cast-in-place beams will develop the same total deflection. The elastic contributions are equal. The inelastic contributions should also be equal although resulting from different mechanisms. In the jointed precast case the inelastic deformation is localised at the interface. In the monolithic case it is distributed along a plastic hinge.

By introducing an analogy with an equivalent monolithic solution, the ultimate and yield curvature (ϕ_u and ϕ_y) can be utilised, having assumed an appropriate value for the plastic hinge length (the results are not sensitive to significant variation in these parameters). After a few simple algebraic simplifications, a simple and familiar relationship between concrete strain, ε_c, and neutral axis depth, c, is derived as Equation B–7, which satisfies the member compatibility condition.

CB7 Design of structural wall systems

The proposed section analysis procedure used to develop a moment-rotation relationship for precast jointed beam-column connections in the presence of strain incompatibility (due to the use of any unbonded concept), can be directly extended and modified for the analysis and design of general hybrid wall systems where vertical unbonded post-tensioning is combined with additional sources of energy dissipation in the form of alternative vertical connections between the precast panels or longitudinal non-prestressed steel reinforcement or external steel dissipators at the base (see References B.10, B.12 and B.14).

A simplified model based on a concentrated plasticity approach can consist of a multi-mass elastic mono-dimensional element with a non-linear flag-shape rotation spring at the critical base section where the non-linear behaviour is concentrated through a rocking motion. The self-centering contribution provided by the vertical unbonded tendons as well as the axial load can be modelled with a non-linear elastic rule, while appropriate hysteresis rules should be used to model the alternative sources of energy dissipation (for example, elasto-plastic, rigid-plastic, viscous or other depending on the use of non-prestressed steel or steel flexural plates, friction or viscous devices).

CB8 System displacement compatibility issues

Different issues related to displacement incompatibilities between structural and non-structural components, as well as between lateral load resisting systems and not-seismic systems (i.e. gravity load design frames) should be evaluated. Beam elongation effects as well as floor-to-lateral load resisting systems (either frame, wall or dual systems) should be properly accounted for when designing a proper seismic load path and evaluating the likely performance of overall system.

CB8.4 *Beam elongation*

Beam elongation is a term used to describe an increase in distance between column centrelines at one or more levels of a reinforced concrete or prestressed concrete frame. This occurs in frames in which flexural deformation causes plastic hinging and cracking in a reinforced concrete beam or gap opening at the beam-column interface of a prestressed jointed frame under significant lateral displacements.

Possible effects of this phenomenon on frame response, have been based on analytical and experimental investigation findings [B.26, B.27; B.28; B.29, B.30, B.31] in terms of:

(a) Damage to and interaction with the floor system;
(b) Increase of column curvature and, thus, increase in flexural and shear demand;
(c) Increase of beam moment capacity due to the increase of beam axial force;
(d) Increase of residual local deformations.

Alternative modelling approach can be adopted to account for the effects of beam elongation on the global response of a frame system. Refined models can consist of multi-springs as contact elements at the beam-column interface representing discrete contact region [B.32, B.33]. Such a model represents a viable but relatively complex tool, which follow the variation of the neutral axis position corresponding to the gradual opening of the gap.

A simple modification to the section analysis approach presented in B6.4 and CB6.4, can be alternatively adopted as suggested in Reference B.32. A series of springs can be adopted to capture the effects of beam elongation in restraining a frame system. For example Figure CB.8 shows the proposed model of a two bays, three columns, one storey sub-assembly. The restraint effects coefficient Ω, as indicated in Equation B–10, refers to the same typology. It should be noted that the column restraining effects on beams in a multi-storey frame system will be reduced in the storeys above the first storey. Different values for the equivalent column lateral stiffness coefficient k_c for the first storey and for the storeys above the first need to be estimated.

Within this simplified model, the variation of shear forces induced into the column by the increase in beam axial force can be taken into account by adding, at the maximum drift level, the contribution caused by system elongation, which, according to the assumptions made, is drift-dependent.

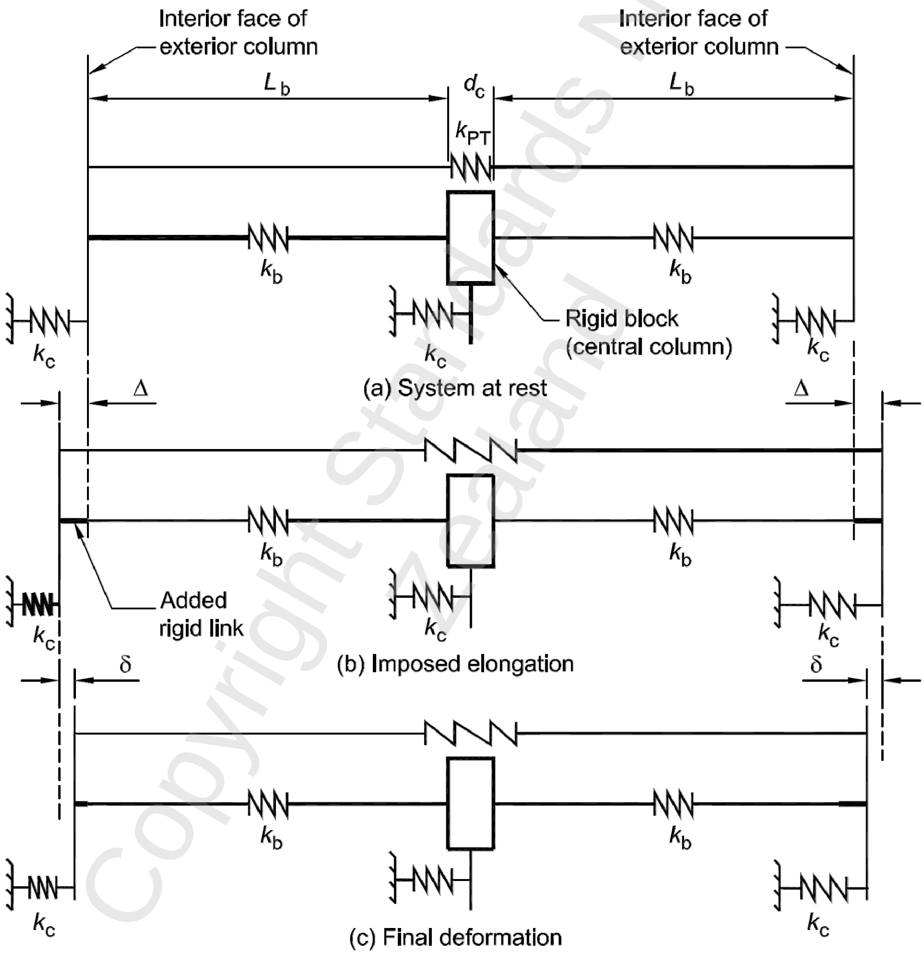


Figure CB.8 – Spring model of assembly elongation

CB8.5 Floor-to-lateral-load resisting system incompatibility

Diaphragm action in the floor system can be significantly impaired due to displacement incompatibility effects between floor and lateral-load resisting systems and can lead to an extensive and unexpected damage. Loss of diaphragm action (i.e. interruption of load path) or full collapse of the floor due to loss of seating (B.32) can occur if inappropriate design measures are considered.

Vertical displacement incompatibility between the lateral-load resisting systems and the floor (see Figure CB.9 for frame-floor action) could be accommodated by properly locating the collectors (either in the form of ordinary reinforcement, or mechanical couplers) in regions of minimum displacement incompatibility and/or by assigning them adequate flexibility in the vertical plane. As a feasible solution, the two "displacement incompatible" systems can be disconnected in the regions of higher incompatibility, maintaining the coupling where more appropriate and needed to transfer the inertia forces.

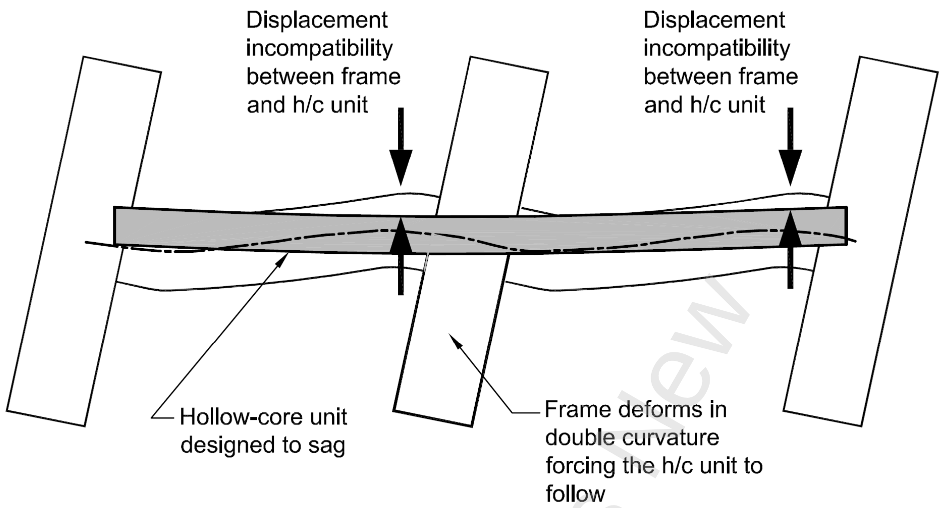


Figure CB.9 – Example of vertical displacement incompatibility between floor and frame systems[B.31]

REFERENCES

B.1 International Federation for Structural Concrete, "Seismic Design of Precast Concrete Building Structures", Bulletin 27, Lausanne, 2003, p. 25.

B.2 Watanabe F., "Seismic Design for Prefabricated and Prestressed Concrete Moment Resisting Frames", Proceedings of the 46th PCI Annual Convention, Orlando, Florida, 2000.

B.3 Priestley, M.J.N. and Tao, J.R., "Seismic Response of Precast Prestressed Concrete Frames with Partially Debonded Tendons", PCI Journal, Vol. 38, No. 1, 1993, pp. 58-67.

B.4 Priestley, M.J.N., "Overview of the PRESSS Research Programme," PCI Journal, Vol. 36, No. 4, 1991, pp. 50-57.

B.5 Stanton, J.F., Stone, W.C. and Cheok, G.S., "A Hybrid Reinforced Precast Frame for Seismic Regions", PCI Journal, Vol. 42, No.2, 1997, pp. 20-32.

B.6 Priestley, M.J.N., Sritharan, S., Conley, J.R. and Pampanin, S., "Preliminary Results and Conclusions from the PRESSS Five-storey Precast Concrete Test-building", PCI Journal, Vol. 44, No. 6, 1999, pp. 42-67.

B.7 Nakaki, S.P., Stanton, J. and Sritharan, S., "An Overview of the PRESSS Five-storey Precast Test Building", PCI Journal, Vol. 44, No. 2, 1999, pp. 26-39.

B.8 Cheok, G.S., Stone, W.C. and Kunnath S.K., "Seismic Response of Precast Concrete Frames with Hybrid Connections", ACI Structural Journal, Vol. 95, No. 5, 1998, pp. 527-539.

B.9 Pampanin, S., Pagani, C. and Zambelli, S., "Cable-stayed and Suspended Post-tensioned Solutions for Precast Concrete Frames: The Brooklyn System", Proceedings of New Zealand Concrete Industry Conference, Queenstown, 2004.

B.10 Kurama, Y., Sause, R., Pessiki, S. and Lu, L-W., "Lateral Load Behaviour and Seismic Design of Unbonded Post-tensioned Precast Concrete Walls", ACI Struct. Journal, Vol. 96, No. 4, 1999, pp. 622-632.

B.11 Rahman, A. and Restrepo, J. I., "Earthquake Resistant Precast Concrete Buildings: Seismic Performance of Cantilever Walls Prestressed using Unbonded Tendons", Research Report 2000-5, Dept. of Civil Engineering, University of Canterbury, 2000.

B.12 Holden T.J., "A Comparison of the Seismic Performance of Precast Wall Construction Emulation and Hybrid Approaches," Research Report 2001-05. Department of Civil Engineering, University of Canterbury, 2001.

B.13 ACI Innovation Task Group 1 and Collaborators, "Acceptance Criteria for Moment Frames Based on Structural Testing (ACI T1.1-01) and Commentary (ACI T1.1R-01)", American Concrete Institute, Farmington Hills, MI, 2001.

B.14 ACI Innovation Task Group 1 and Collaborators, "Special Hybrid Moment Frames Composed of Discreetly Jointed Precast and Post-tensioned Concrete Members" (ACI T1.2-03) 1 and Commentary (ACI T1.2R-03)", American Concrete Institute Farmington Hills, MI, 2003.

B.15 Priestley, M.J.N., "Displacement-based Design of Precast Jointed Ductile Systems", PCI Journal, Vol. 47 No. 6, 2004, pp 66-79.

B.16 Thompson, K.J. and Park R., "Seismic Response of Partially Prestressed Concrete," Journal of Structural Design, ASCE, ST8, 1980, pp. 1755-1775.

B.17 Pampanin S., Nishiyama, M., 2002, "Critical Issues in Modeling the Seismic Behavior of Precast/Prestressed Concrete Connections-Systems", 1st fib Congress, Osaka.

B.18 Pampanin S., Priestley, M.J.N. and Sritharan, S., "Passive Energy Dissipation and Self-centering Capabilities in Precast Ductile Connections", Proceedings of the Second European Conference on Structural Control, ENPC, Champs-sur-Marne, France, 2000.

B.19 MacRae, G.A. & Kawashima, K., 1997. "Post-earthquake Residual Displacements of Bilinear Oscillators", Earthquake Engineering and Structural Dynamics, Vol. 26, pp. 701-716.

B.20 Pampanin, S., Christopoulos, C. and Priestley M.J.N., "Framework for Performance-based Seismic Design and Assessment of Frames Considering Residual Deformations", Proceedings of the 12th European Conference Engineering, Paper n.100, London, 2002.

B.21 Pampanin, S., Christopoulos, C. and Priestley M.J.N., "Residual Deformations in the Performance-based Seismic Assessment of Frame Systems, Research Report ROSE 2002/02, (European School on Advanced Studies on Reduction of Seismic Risk), Pavia, Italy, 2002.

B.22 Christopoulos, C., Pampanin, S. and Priestley, M.J.N., "Performance-based Seismic Response of Frame Structures including Residual Deformations, Part 1: Single-degree-of-freedom Systems," Journal of Earthquake Engineering, Vol. 7, No. 1, 2003, pp. 97-118.

B.23 Pampanin, S., Christopoulos, C. and Priestley, M.J.N., "Performance-based Seismic Responses of Frame Structures Including Residual Deformations, Part II: Multi-degree-of-freedom Systems", Journal of Earthquake Engineering, Vol. 7, No. 1, 2003, pp. 119-147.

B.24 Christopoulos, C. Pampanin S., "Towards Performance-based Design of MDOF Structures with Explicit Consideration of Residual Deformations", ISET Journal of Structural Engineering, Special Issue on "Performance Based Seismic Design" (Guest Editor M.J.N. Priestley) Vol. 41, No. 1, Paper 440, 2004.

B.25 Pampanin, S., Priestley, M.J.N. and Sritharan, S., "Analytical Modelling of the Seismic Behaviour of Precast Concrete Frames Designed with Ductile Connections", Journal of Earthquake Engineering, Vol. 5, No.3, 2001, pp. 329-367.

B.26 Fenwick, R.C. and Fong, A., "The Behaviour of Reinforced Concrete-beams Under Cyclic Loading", Research Report, Department of Civil Engineering, University of Auckland, 1979.

B.27 Douglas, K.T., "Elongation in Reinforced Concrete Frames", PhD Thesis, Department of Civil Engineering, University of Auckland, 1992.

B.28 Fenwick, R. C. and Megget, L. M., "Elongation and Load Deflection Characteristics of Reinforced Concrete Members containing Plastic Hinges", Bulletin of the New Zealand National Society for Earthquake Engineering, 26, 1, Mar. 1993, pp. 28-41.

B.29 Restrepo, J.I., "Seismic Behaviour of Connections between Precast Concrete Elements", PhD. Dissertation, Department of Civil Engineering, University of Canterbury, 1993.

B.30 Christopoulos, C., "Self-centering Post-tensioned Energy Dissipating (PTED) Steel Frames for Seismic Regions", Ph.D. Dissertation, University of California, San Diego, 2002.

B.31 Matthews J., Bull D. and Mander J., "The Performance of Hollow-core Floor Slabs and the Effects They Have on Their Supporting Beams following a Severe Earthquake", Proceedings fib Symposium on Concrete Structures in Seismic Regions, Athens, 2003.

B.32 Kim, J., "Behaviour of Hybrid Frames under Seismic Loading," Ph.D. Dissertation, Department of Civil Engineering, University of Washington, Seattle, 2002.

B.33 Carr, "Ruaumoko Program for Inelastic Dynamic Analysis – Users Manual", Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 2004.

APPENDIX CD – METHODS FOR THE EVALUATION OF ACTIONS IN DUCTILE AND LIMITED DUCTILE MULTI-STOREY FRAMES AND WALLS

CD1 Notation

CD1.1 Standard symbols

$d_{p,i}$ distance from beam centreline at level *i* to point of inflection in storey *i*, mm
E a lateral force representing earthquake forces, N
$E_{E,i}$ lateral force at level *i* found in an equivalent static or first mode analysis of structure, N
$E_{o,I}$ lateral force at a level corresponding to overstrength actions, N
$E_{o,A,i}$ lateral force assigned to column A at level *i*, N
$E_{o,I}$ a lateral earthquake force t a level corresponding to overstrength actions, N
h height of moment resisting frame above base, mm
h_b overall depth of beam, mm
ℓ_w length of wall, mm
M bending moment, N mm
M_n Nominal flexural strength of a section, N mm
M_o overstrength bending moment at a critical section of a primary plastic region, Nmm
M^*_{col} design moment at the critical sections of a column, N mm
M_E a bending moment at the centre of a beam-column joint from an elastic analysis for the lateral earthquake actions, N mm
n the number of floors above the column section considered
n_t total number of storeys in a building
V a shear force, N
V^*_o capacity design shear force in plastic region in a wall, N
$V_{ob,A,i}$ shear induced in column A in storey *i* when beam input moment acts on column at level *i*, N
$V_{obM,A}$ component of shear force induced in column A due to beam overstrength input moment acting at intersection of beam and column centrelines, N
$V_{oE,A,i}$ component of shear force induced in column A in storey *i* due to lateral force $E_{o,A,i}$ acting on column at intersection of beam and column centrelines, N
ω_v dynamic magnification factor for shear in a wall
μ structural ductility factor
ΣM_{Eb} sum of beam moments at intersection of beam and column centrelines found in an equivalent static (or 1st mode analysis), N mm

CD1.2 Subscripts

CD1.2.1 *First subscript*

$_E$ a structural action (moment shear etc.) obtained from an equivalent static or modal analysis of the structure, N or N mm
$_{Eb}$ a structural action in a beam obtained from an equivalent static or modal analysis of the structure, N or N mm
$_{Ec}$ a structural action in a column obtained from an equivalent static or modal analysis of the structure, N or N mm
$_n$ a nominal strength calculated assuming lower characteristic material strengths
$_o$ an action calculated assuming overstrength actions are sustained in plastic regions
$_{ob}$ an action induced when overstrength moments act in beams
$_{obM}$ an action due to a beam input overstrength moment acting on a joint zone, N
$_{oE}$ an action arising from a lateral force corresponding to overstrength actions, N
$_{oc}$ an action induced when overstrength moments act in columns

CD1.2.2 *Second and third subscripts*

A, B, C	identifies a line of columns shown on a figure
above	above intersection of beam and column centrelines
b	relates to a beam
below	below intersection of beam and column centrelines
c	relates to a column
i	level in a frame
ℓ	left hand side of intersection of beam and column centrelines
r	right hand side of intersection of beam and column centrelines
top	upper critical section of a column in a storey
bottom	lower critical section of a column in a storey

CD1.3 Superscript

* a design action

CD2 General

The objective is to ensure that in the event of a major earthquake the majority of plastic deformation is confined to the plastic regions identified in accordance with 2.6.5.2. These plastic regions are referred to as primary plastic hinges regions, which are detailed to sustain plastic deformation. The term secondary plastic (hinge) region refers to regions, which may sustain inelastic deformation due to factors not considered in analysis, such as elongation of members associated with inelastic deformation and plastic deformation which develops as a result of higher mode effects. In this context higher modes refers to both elastic modes of behaviour and modes of behaviour that develop due to the changes in dynamic characteristics associated with the formation of plastic regions.

CD3 Columns multi-storey ductile frames

CD3.1 *General*

A2 With both methods the aim is to ensure that in the event of a major earthquake a ductile failure mechanism will form in preference to other possible non-ductile failure modes. This is achieved in both cases by designing the columns to have a margin of strength above that of the beams. This ensures that a column sway mode as illustrated in Figure CD.1(a) cannot develop, and that there is a margin of safety against the premature formation of a mixed beam-column sway mode as illustrated in Figure CD.1(c). The two methods achieve this in different ways. With Method A the strength of each column at each level is considered individually. Each column is designed to be able to resist with a specified margin of strength the maximum actions that can be induced in it when the beams framing into the column are sustaining their overstrength actions. With Method B the approach is to consider the combined strength of all the columns in each level of a frame. The column sway shear strength of each storey in a frame is designed to have a specified margin of strength above that which can be sustained by the beam sway failure mode. It is the sum of the strengths of all the columns in the level which is important, rather than each individual column as with Method A. Method A was developed for use with reasonably regular frames. Method B can be applied to a wider range of frame structures and there is greater freedom where the potential plastic regions are located. The methods differ in the level of protection they give to the formation of plastic hinges in columns in the zone between the mid-height of the second storey and the top storey of the structure. For this reason each method involves different detailing requirements.

Method A provides a high level of protection against the formation of plastic regions in the columns between the mid-region of the second storey and the top storey. Where a high level of protection exists 10.4.6.8.2 (a) allows for the positioning of splices any where in the column, while 10.4.7.4.3 and 10.4.7.5.3 allows a reduction in the quantity of confining reinforcing required. The advantages of this method are therefore simplifications in the detailing requirements. However, it should be noted that elongation of beams, which is associated with the formation of plastic hinges, displaces columns outwards from the centre of a frame. This action causes secondary plastic regions (hinges) to develop in columns either immediately above or below the first elevated level in the structure. In these locations plastic hinges may form and therefore it is not permitted to relax the amount of confinement provided or the location of longitudinal bar splices in the first storey or the lower region of the second storey. Method A also permits plastic deformation to arise in the event of a major earthquake in the columns of the upper storey of ductile frames. A2

Method B gives a high level of protection against the premature formation of a mixed beam-column sway failure mode, such as is illustrated in Figure CD.1(c) but it gives a lower level of protection against localised plastic deformation in the columns than Method A. Method B gives the designer greater freedom in the locations where primary plastic regions may be located. Some of these regions may be located in the columns away from the base, provided the column sway storey strength is maintained at the required level and the material strain limits are satisfied. With Method B, some plastic deformation due to the formation of secondary plastic regions is to be expected in a design level earthquake. Consequently, confinement complying with 10.4.7.4.1, 10.4.7.4.2, 10.4.7.5.1, and 10.4.7.5.2 is required. Additionally column splices need to be confined to mid-height regions of any storey as required by 10.4.6.8.2. A2

With both methods it is assumed that plastic regions form at the base of the columns and the selected ductile failure mechanism is a beam sway mode, see Figure CD.1. The columns are proportioned to restrict inelastic deformation to the chosen primary plastic regions and to prevent the premature formation of a mixed beam-column sway mode. Both methods are applicable to moment resisting frames where column sidesway mechanisms are not permitted as specified in 2.6.7.2. In selecting the method that is used consideration should be given to the different detailing requirements associated with each of them.

A2

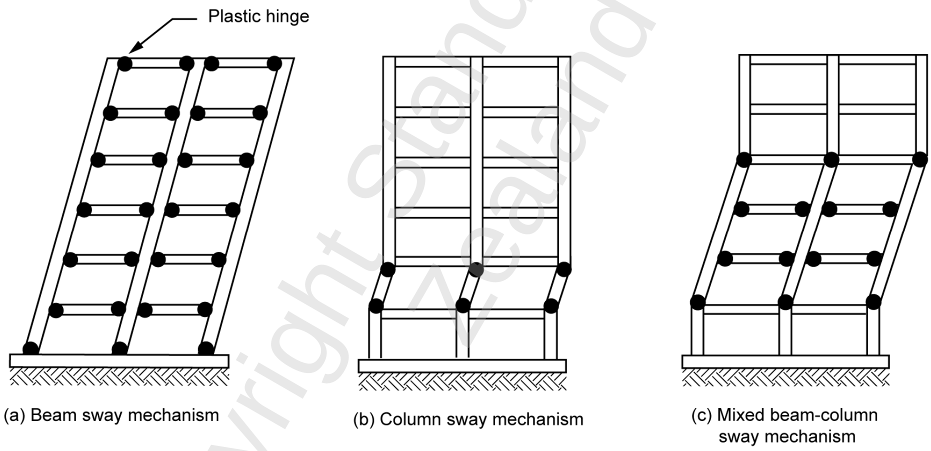


Figure CD.1 – Failure modes for moment resisting frames

In recognition of the high level of protection provided against the formation of secondary plastic hinges in columns designed by Method A, between 3 h_c above the first storey and 3 h_c below the top storey, requirements on the location of lap splices in longitudinal reinforcement are relaxed and the quantity of confinement reinforcement is reduced in this zone.

Method A has been developed from Appendix A in NZS 3101:1995. It has been changed so that it complies with NZS 1170.5 [D.1], which requires columns in two-way frames to be designed for bi-axial actions. References D.2 and D.3 give background information on this method of determining capacity design actions for columns and on the general behaviour of columns in earthquakes. Method B is based on an approach given in NZS 1170.5 [D.1].

Both methods

When gravity load considerations govern the strength of beams, both Methods A and B may predict high design moments for columns, which may be considerably in excess of the values found in the design analyses of the structure. However, the reinforcement need not exceed the amount required for a column designed for the structural actions based on the values found from an analysis assuming the structure is nominally ductile. It should be noted that in determining the area of reinforcement for this case appropriate values of strength reduction factor (ϕ = 0.85), structural performance (S_p, 0.9) and structural ductility factors (μ, 1.25), should be used.

Method A

The procedure is intended to apply for moment resisting frames. Where columns are stiff compared to the beams, that is the columns have wall like characteristics, cantilever action is likely to dominate the moment pattern of columns and an approach applicable to structures with structural walls, or wall frames, may be more appropriate.

Ductile failure mechanism (both methods)

In frames with two or less storeys primary plastic (hinge) regions may be located in both beams and columns and a column sway mechanism is acceptable provided it is shown that the material strains in the plastic regions are less than the maximum permissible values. In frames with more than 2 storeys the strengths of the columns are designed to meet the capacity design requirements. This ensures that a beam sway failure mode will develop in preference to other non-ductile failure modes in the event of a major earthquake.

Column moments due to beam overstrength actions (both methods)

The first step in the capacity design of columns is to find the bending moments, which act in the beams when the potential plastic regions are sustaining overstrength moments. Where the plastic regions do not form at the faces of the columns the required moments should be found from statics using the overstrength moments acting at the critical sections of the plastic regions and the shear force diagram. Once the values at the actions at the column faces have been established the resultant moment that the beams apply to each beam-column joint zone at the intersection of the beam and column centre-lines can be found. This value is the beam input overstrength moment. The process is illustrated in Figure CD.2 (a) and (b). The beam input overstrength moment at each joint zone consists of the sum of the moments from the beams, which lie in the same plane or close to the same plane. This is illustrated in Figure CD.2 (b) where the beam input overstrength moment is shown as ΣM_{ob}. In this figure the subscript "$_{ob}$" stands for an overstrength beam action while "ℓ" and "r " refer to beams on the left and right hand sides of the joint zone respectively.

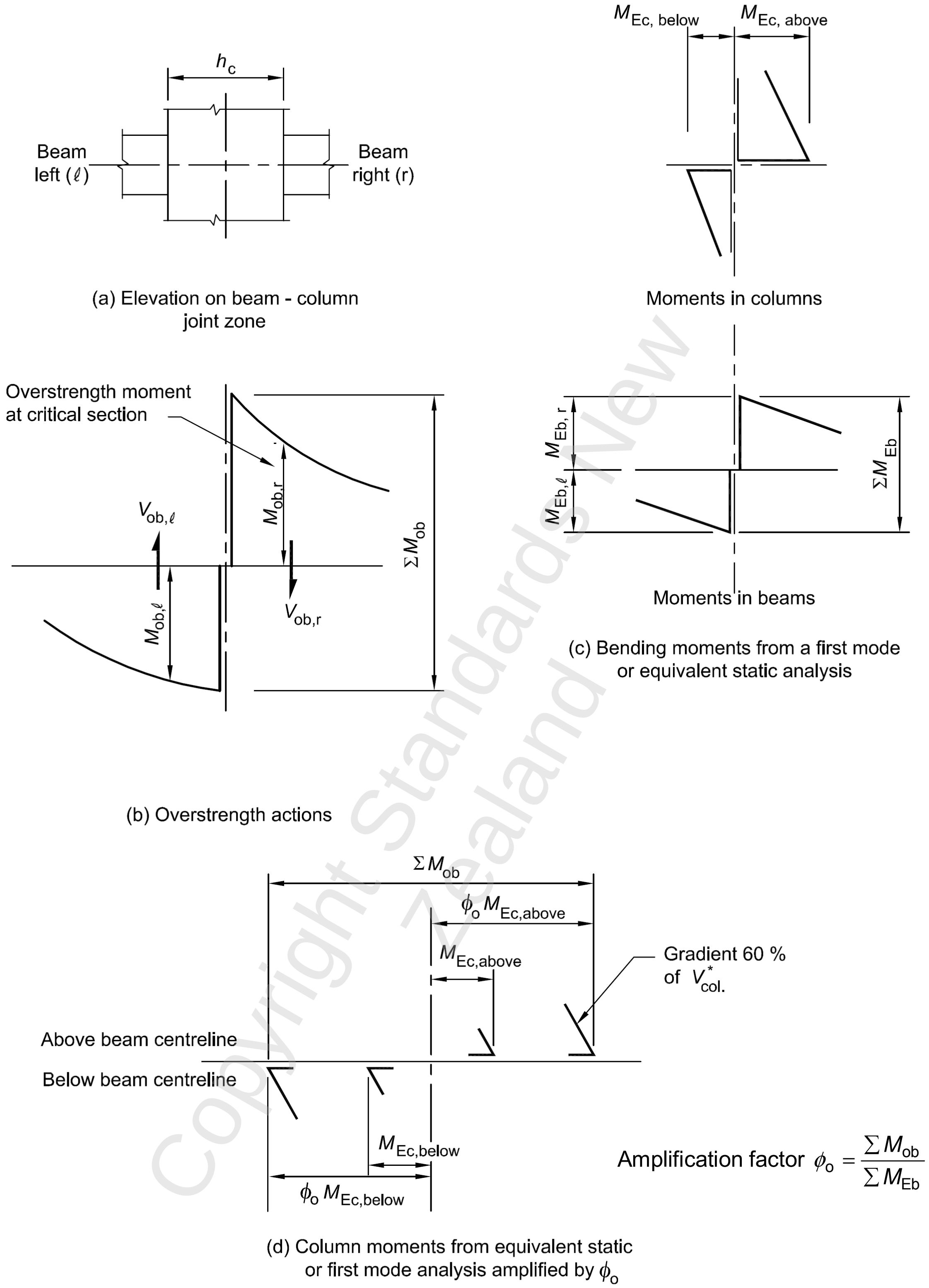


Figure CD.2 – Distribution of input beam overstrength moments into columns – Method A

CD3.2 Design moments and shears in columns by Method A

CD3.2.1 General

This clause sets out the steps required to find the capacity design moments in the columns at the level of the beam centre-lines.

CD3.2.2 Distribution *of beam input moment into columns*

A2 In region 1 the seismic moments in the columns at the beam-column intersection point being considered, which have been found from an analysis for earthquake actions, $M_{Ec,above}$ and $M_{Ec,below}$ in Figure CD.2(d), are scaled so that the sum of these moments is equal to the beam input overstrength moment, ΣM_{ob}. This step establishes equilibrium for the actions acting on the joint zone being considered. The analysis for earthquake actions may be an equivalent static analysis or a first mode analysis of the structure. One-way of achieving this scaling is illustrated in Figure CD.2(c) and (d). The overstrength amplification factor, ϕ_o, is found. This value is equal to the ratio of the beam input overstrength moment, ΣM_{ob}, at the intersection point being considered, to the corresponding sum of the seismic design moments (ΣM_{Eb}). It is given by:

$$\phi_o = \frac{\Sigma M_{ob}}{\Sigma M_{Eb}} \qquad \text{(Eq. CD–1)}$$

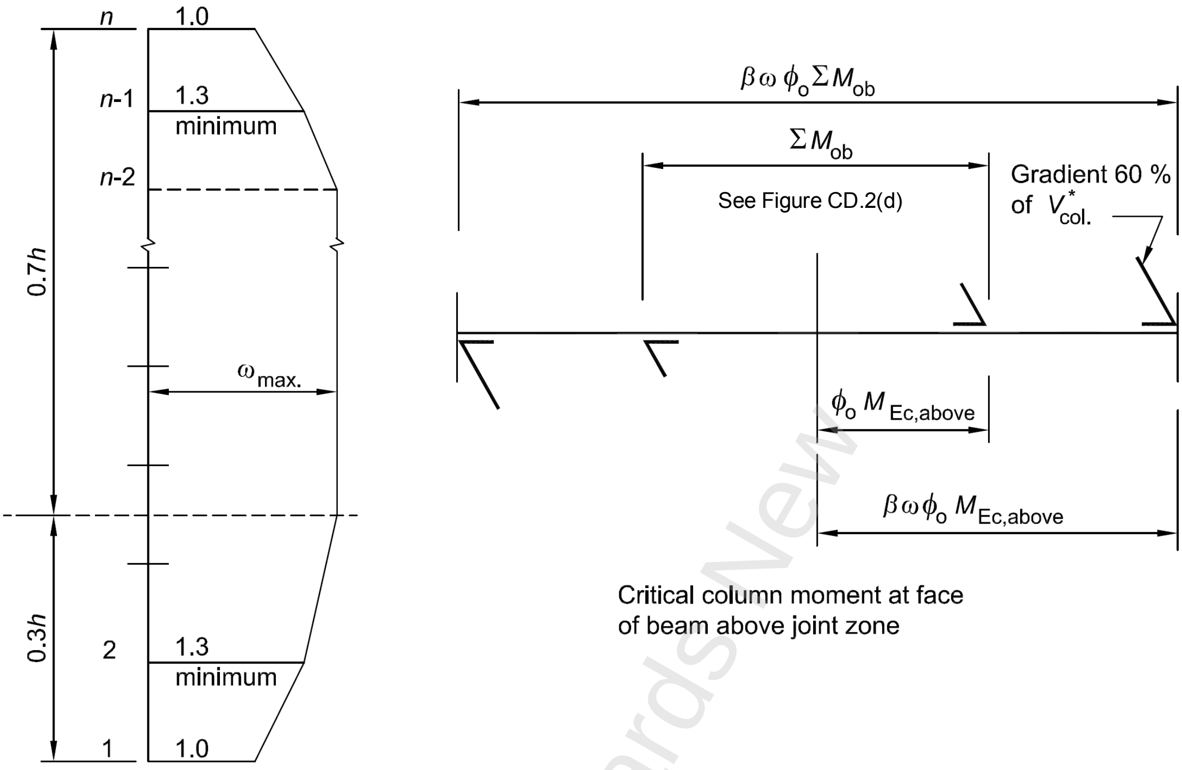
A2 In region 2 the columns are acting like cantilevers. In this case, to ensure that plastic deformation is confined to the primary plastic regions (at the base of the column), the flexural strength of the column in region 2 is related to the overstrength of the primary plastic region[D3] This is achieved by multiplying the column moments above and below each level in the region by $\phi_{o,b}$ instead of ϕ_o used in region 1, where $\phi_{o,b}$ is the ratio of the overstrength moment in the primary plastic region to the corresponding moment arising from the equivalent static or first mode analysis.

In load cases involving earthquake actions, allowance is made for biaxial actions in determining the required design strengths at the critical section of the potential primary plastic hinge in each column and at the section immediately below the uppermost level (see 2.6.7.5(d)).

CD3.2.3 Dynamic *magnification and modification factors*

When plastic hinges form the dynamic characteristics of the frame change. This can lead to a change in the distribution of moments in the columns (see reference D.3, page 215-221). The dynamic magnification factor, ω, allows for this effect. Analyses have shown that the dynamic magnification factor depends on the fundamental period of the building and on the height of the beam-column joint being considered in the structure.

The bending moments found in D3.2.2 are multiplied by the dynamic magnification factor, ω, which is illustrated in Figure CD.3(a) to give the envelope column moments at the level of the centreline of beams, as shown in Figure CD.3(b).

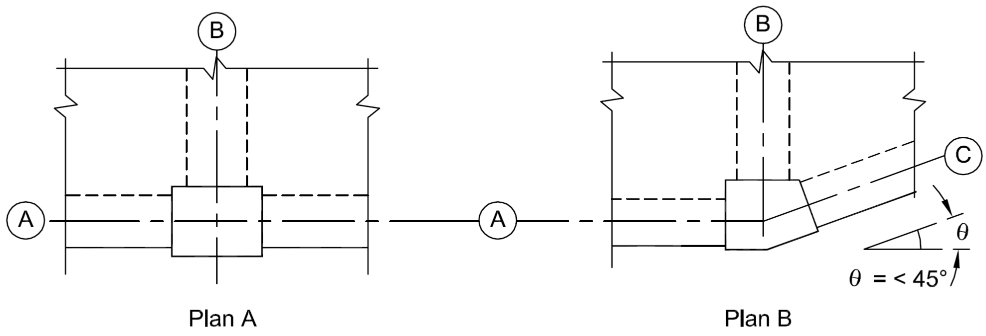


(a) Dynamic magnification factor (b) Design moment at critical section in column

Figure CD.3 – Dynamic magnification factor and design moment for column

In some cases the overstrength moments in a beam may be increased by the presence of reinforcement or prestressed units in a floor slab, which are located some distance from the beam. In such cases the ratio of M_o/M_n can be high. However the maximum moment is typically not sustained until inter-storey drifts are of the order of 2 to 3 percent are reached. Such high displacements should reduce the magnitude of the dynamic magnification of column moments that can occur. To allow for this effect the modification factor β is introduced.

Where a column acts in more than one frame bi-axial actions need to be considered in design. However, simultaneous peak dynamic magnification along two or more axes is unlikely. Consequently, in such cases dynamic magnification and modification factors, $\beta\omega$, need to be applied to the beams of one frame, with the $\beta\omega$ values for the beams in the second or subsequent frames being replaced by 1.0. There is one exception to this, and this occurs when the enclosed angle between the beams in two of the frames is less than 45°. Where this occurs the two frames are likely to act together and hence they should be assigned the same $\beta\omega$ values. This situation is illustrated in Figure CD.4.



$\omega\beta$ values for biaxial moments in columns

Case	Direction of action considered	Plan A Frame		Plan B Frame		
		A	B	A	B	C
1	Actions along Axis A	$\omega\beta$	1.0	$\omega\beta$	1.0	$\omega\beta$
2	Actions Along Axis B	1.0	$\omega\beta$	1.0	$\omega\beta$	1.0

$= \beta\omega\phi_o M_{EC,above} - 0.3\ h_b\ V^*_{col,above}$

Figure CD.4 – Dynamic magnification and modification factors for columns contributing to more than one frame

CD3.2.4 Critical *design moments in columns*

The critical column bending moments are at the faces of the beam. These values can be found by reducing the bending moments found at the intersection of the beam and column centrelines by the product of the shear force in the column times the distance between the beam centreline and the face of the beam. Analyses show that the peak bending moment in the column does not occur simultaneously with the peak shear force in the column. Consequently the moment at the face of the beam is calculated assuming that the column shear force is 60 % of the maximum shear force, V^*_{col}, for the column given in D3.2.6. For example with this value the critical design moment in the column at the face of the beam, above the joint zone, M^*_{col}, is given by:

$$M^*_{col} = \beta\ \omega\ \phi_o\ M_{Ec,above} - 0.3\ h_b\ V^*_{col,above} \qquad \text{(Eq. CD–2)}$$

Where, as illustrated in Figure CD.3(b), ω is the dynamic magnification factor defined in D3.2.3, ϕ_o $M_{E,c,above}$ is the bending moment found from an equivalent static or first mode analysis immediately above the intersection point of the beam and column centrelines.

CD3.2.5 Reduction *in design moments for cases of small axial compression*

Where a column is subjected to small axial compression or net axial tension in the critical load case some plastic deformation is acceptable. For such cases the column design moments in the columns may be reduced, with the extent of the permissible reduction depending on the level of axial load and the dynamic magnification factor. To allow for this effect the bending moments acting at the critical sections in the column shall be multiplied by the factor R_m, to give the reduced design moments, given by:

$$M^*_{col} = R_m\ (\beta\ \omega\ \phi_o\ M_{Ec,above} - 0.3\ h_b\ V^*_{col,above}) \qquad \text{(Eq. CD–3)}$$

Where the value of R_m is given in Table D.1.

CD3.2.6 Design *shears in columns*

In first storey elongation of the beams, associated with the development of plastic hinges, pushes the columns outwards forcing secondary plastic regions to form in the columns, either immediately below or above the first elevated level. Consequently in the first storey the critical shear force is determined assuming that plastic regions develop at the top and bottom of the storey. On this basis the column shear force is given by Equation D-3.

Conventional frame analyses indicate that moments at the top of first storey columns are smaller than the corresponding values at the base. However, these analyses do not include actions induced by elongation of the beams and hence they may be misleading. Consequently, the formation of a secondary plastic region must be anticipated in this location.

The shear force is estimated from a probable and critical moment gradient along the column. However, in recognition of the more serious consequences of a shear failure the equations for V^*_{col}, (Eq. D-3 and D–4), have been increased by approximately 15 % with some allowance for the different reliability of the design equations for shear and flexure.

In columns, which intersect with beams on two or more axes, the simultaneous action of the shear forces applied by the beams on each axis should be considered in the design for shear in the column. In such cases the shear resisted by the concrete should be proportioned between the two axes of the column.

CD3.3 Design moments and shears in columns by Method B

CD3.3.1 General

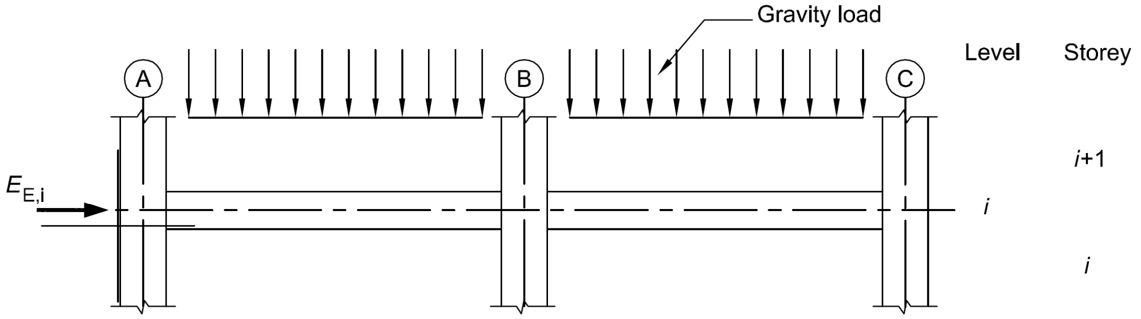
The theory behind Method B is outlined in the following paragraphs.

The individual steps are described in general terms in D3.3.1 and in detail in D3.3.3 to D3.3.7. With Method B, full confinement is provided in the columns. Consequently redistribution of structural actions, which involves limited inelastic deformation in the columns (formation of secondary plastic hinges) may be assumed in determining the critical column actions. As described below the analysis of the required column strengths can proceed on a level by level basis.

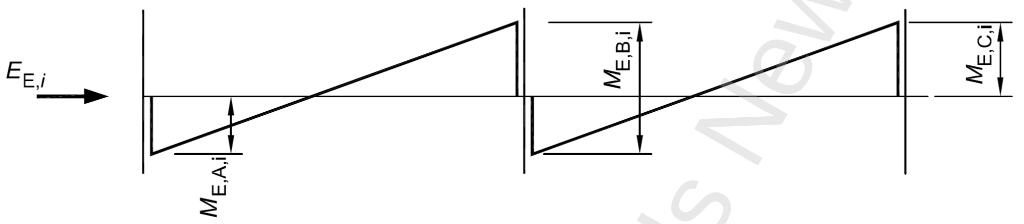
In the analysis for this method each column is assumed to have a point of inflection, which in general will lie within the storey. (However, in extreme cases it is not restricted to the storey). The most convenient location for the assumed point of inflection is the mid-height of the storey. The chosen location for this assumed point is restricted as follows;

> An equivalent static or first mode analysis is examined and the position of the analysis point of inflection predicted is noted. If the analysis point of inflection lies within the column then the assumed position of the point of inflection for method B may be assumed to be anywhere within the middle half of the storey height. The implied inelastic deformation associated with the redistribution of moments in this case is small, and consequently it is not necessary to check the ductility required for this. If the analysis point of inflection lies outside the column then the inelastic deformation associated with redistribution may be significant. Consequently in this case the inelastic rotation should be determined and the required section ductility checked to ensure it is within acceptable limits (2.6.1).

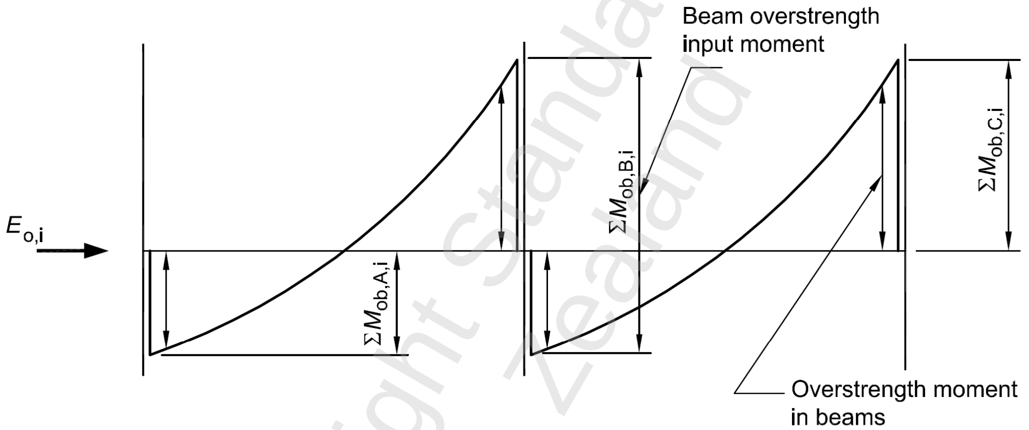
Consider a level of a moment resisting frame, as illustrated in Figure CD.5(a). As shown the forces acting on the level consist of gravity loads and a lateral seismic force, $E_{e,i}$. In part (c) of this figure the same level is shown when beam overstrength moments act in the plastic regions. In addition to the bending moments the level sustains a lateral force of $E_{o,i}$, where the subscript "$_o$" indicates that this force acts with the overstrength beam moments and the subscript "$_i$" indicates it is on level *i*. This lateral force is distributed to the individual beam-column joints in this level ($E_{o,A,i}$, $E_{o,B,i}$ and $E_{o,C,i}$) in the frame as shown in Figure CD.5(d). Points of inflection are assumed to form in the columns. Generally it is assumed that these points are located at the mid-height of each storey. However, in a limited number of cases a different assumption may be required, see D3.3.2.



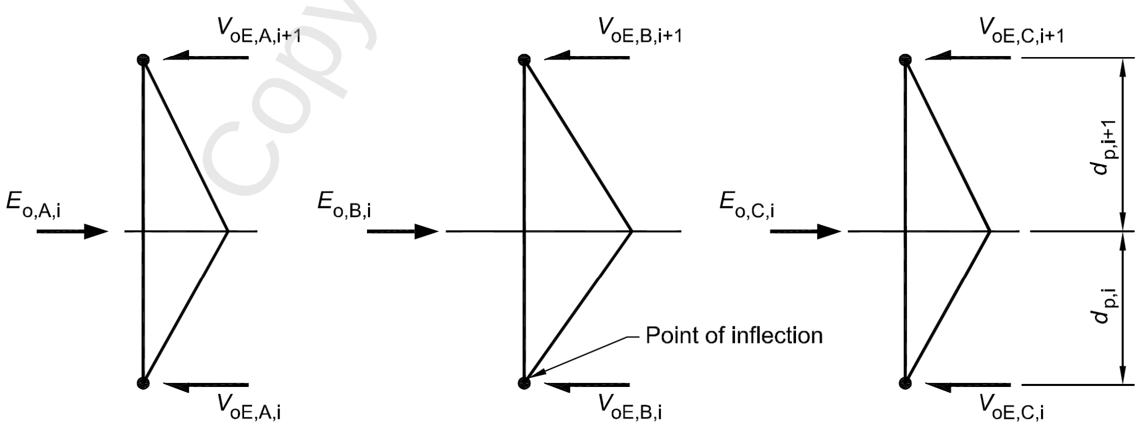
(a) Elevation level *i* in frame



(b) Beam moments level *i* due to earthquake actions

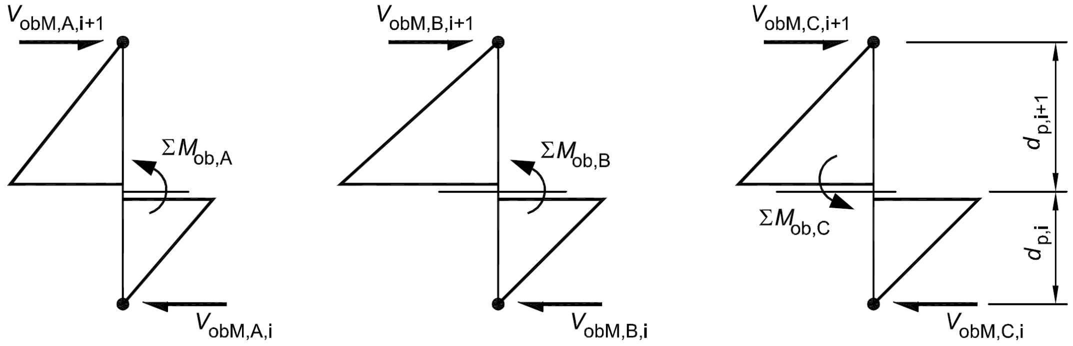


(c) Beam overstrength actions

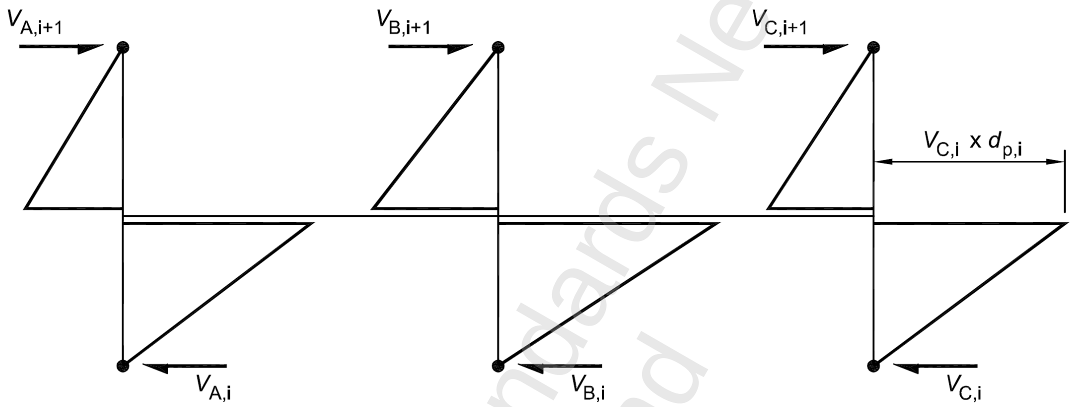


(d) Column shears and moments due to lateral force at level

Figure CD.5 – Capacity design moments and shears in columns – Method B



(e) Column shears due to beam overstrength input moments



$$V_{A,i} = \beta\omega\left[V_{oE,A,i} + V_{obM.A,i}\right]$$

(f) Resultant column moments and shears found by adding values in (d) and (e) and multiplying by dynamic magnification factor and modification factors $\omega\beta$

Figure CD.5 – Capacity design moments and shears in columns – Method B (Continued)

The actions at each column at the intersection of the beam and column centrelines, may be identified as:

(a) The beam input overstrength moment to the joint zone, shown for example as $\Sigma M_{ob,A}$ in column A, in Figure CD.5(e);

A proportion of the seismic lateral force, $E_{o,i}$, assigned to each column in the level, for example $E_{o,A,i}$ is assigned to column A in Figure CD.5(d).

The structural actions arising from the input moment from the beams and the lateral force on each column may be evaluated separately.

(i) The beam overstrength input moment induces equal and opposite shears in the column being considered. These shears are equal to the beam input overstrength moment divided by the distance between the points of inflection, as illustrated in Figure CD.5(e) as $V_{obM,A,i}$ etc.

(ii) The proportion of the seismic lateral force, which acts at the level of the beam centreline induces shears in the upper and lower portions of the column as illustrated in Figure CD.5(d). The values of these shears are found from equilibrium requirements. The sum of all the shear forces above and below the column equals the lateral force acting on the level. The bending moments induced in the column by these shears at the beam centreline, are equal.

In each column the shear forces arising from the beam input overstrength moment and the lateral force acting on the joint zone are added together and multiplied by an appropriate dynamic magnification and modification factors.

The following points for this method should be noted;

(a) The local lateral force acting at a level of a frame is assessed by scaling the corresponding lateral force from either an equivalent static or a first mode analysis of the structure. The method of scaling is set out in D3.3.3.

(b) Increasing the strength of a column in a level at either the top or bottom of a storey above that required to sustain the bending moment corresponding to the shear force in that member does not invalidate the process. This merely increases the column strength above the minimum level required to ensure that a storey column sway mode does not develop.

(c) Multiplying these actions by the product of the modification factor and dynamic magnification factor is required to provide an adequate margin of safety against:

  (i) A possible underestimate of the lateral force acting at the level being considered due to higher mode effects;

  (ii) The premature formation of a combined beam-column sway failure mode.

CD3.3.3 Lateral *seismic forces at a level*

(a) The lateral force corresponding to overstrength actions at the level of a frame being considered, $E_{o,i}$ is found by scaling the corresponding value, $E_{e,i}$, from an equivalent static or first mode analysis for seismic actions. The subscript "$_E$" stands for a value from the analysis for earthquake design actions, while the subscript "$_i$" refers to the level in the frame. To obtain the corresponding lateral force when overstrength actions are sustained in the beam the lateral force, $E_{E,i}$, is multiplied by a ratio, $\phi_{ol,i}$, which is equal to the ratio of the sum of the beam overstrength input moments into the columns in level i to the sum of the corresponding moments found in analysis for seismic action. The process is illustrated in Figure CD.5(b) and (c). The resultant lateral force, $E_{o,i}$ at level "i" in this figure is given by:

$$E_{o,i} = E_{E,i}\frac{\left(\sum M_{ob,A,i} + \sum M_{ob,B,i} + \sum M_{ob,C,i}\right)}{\left(\sum M_{E,A,i} + \sum M_{E,B,i} + \sum M_{E,C,i}\right)} \quad \text{(Eq. CD–4)}$$

where the subscript "$_{ob}$" refers to a value related to beam input overstrength actions, the second subscript defines the column line and the third subscript the level. The factor multiplying $E_{E,i}$ in equation CD–4 is equal to $\phi_{ol,i}$. .

(b) At all levels, except the top, the lateral force ($E_{o,i}$)is distributed to the beam-column joints in the level. There is considerable freedom in how much of this force is allocated to each joint zone, but the requirements of D3.3.8 must be satisfied. A simple guide is to distribute it in proportion to the beam overstrength input moment. However, where the chosen ductile failure mechanism includes primary plastic regions in the columns following this guide may not be possible. The distribution of $E_{o,i}$ gives the individual values of $E_{o,A,i}$, $E_{o,B,i}$ etc. as shown in Figure CD.5(d).

(c) The lateral force at each joint zone induces shear into the columns framing into the joint zone. The sum of all these column shears equals the lateral force $E_{o,i}$. The individual shear force in each column is calculated from statics assuming:

  (i) no moment is transferred between the beams and columns

  (ii) the columns are supported by shear forces acting at the selected points of inflection in the storeys immediately above and below the level being considered.

For example the component of shear in column B in level i due to the lateral force $E_{o,B,i}$ is given by:

$$V_{oE,B,i} = E_{o,B,i}\left(\frac{d_{p,i+1}}{d_{p,i} + d_{p,i+1}}\right) \quad \text{(Eq. CD–5)}$$

CD3.3.4 Column *shear due to beam overstrength moments*

The beam input overstrength moments are calculated for all the columns intersecting the level as detailed in D3.5. The input moment for each column is considered in turn. Thus for example the beam overstrength input moment, $\Sigma M_{ob,A}$, is resisted by equal and opposite shear forces in each column, ($V_{obM,A,i}$ and $V_{obM,A,i+1}$, etc.) such that:

$$V_{obM,A,i} = -V_{obM,A,i+1} = \frac{\sum M_{ob,A,i}}{\left(d_{p,i} + d_{p,i+1}\right)} \quad \text{(Eq. CD–6)}$$

This is illustrated in Figure CD.5(e).

CD3.3.5 Resultant *column shears*

The dynamic magnification factor is introduced to give the frame a high level of protection against the premature formation of a mixed beam-column sway mode. The modification factor β allows for the situation where the beam overstrength moment is appreciably higher than the nominal strength. In such cases the overstrength moment can only be sustained at high levels of inter-storey drift, and this reduces the potential of dynamic magnification.

Where a column forms part of 2 or more frames the moments are applied to the column it is unlikely that the maximum dynamic magnification factors will act simultaneously along both axes. Consequently in determining the critical biaxial moments in a column the $\beta\omega$ factors need be applied to only the beams from one of the frames, unless the enclosed angle between two of the frames is small (<45°). The actions induced by beams in other frames shall correspond to $\beta\omega$ values of 1.0 provided the angle between the frames exceeds 45°.

CD3.3.6 Capacity *design column moments*

The step of finding the critical design moments in the columns from the shear forces is illustrated in Figure CD.5 (f).

CD3.3.7 Design *shear strength for columns*

The 1.15 factor used to calculate the column shear forces maintains the margin between shear and flexural strengths implied by the differing strength reduction factors used in the ultimate limit state.

In the first storey plastic regions may be expected to form at both the top and bottom of each column due to elongation of the beams. For this reason the minimum shear strength is based on the maximum value that is consistent with simultaneous plastic deformation in these two locations.

CD3.3.8 *Limit on distribution of column shear forces*

There is considerable freedom on how the lateral force on each level, which is associated with overstrength actions, is distributed to the joint zones in D3.3.3. However, it is essential that this distribution does not invalidate the selected ductile failure mechanism in D3.1 by allowing primary plastic regions to migrate from their chosen locations.

CD3.4 Capacity design axial forces for Methods A and B

The capacity design axial load applies when the structure is sustaining extensive inelastic deformation. Consequently linear elastic theory does not apply and the axial forces cannot be obtained from an elastic based analysis.

The design axial forces in the columns are calculated from the assumption that all the primary plastic regions sustain their overstrength actions. The simplest way of achieving this objective is to calculate the axial force as three separate components, namely:

(a) The dead of the column and any element attached to it;
(b) The shear force transferred to the column due to gravity loads acting on the beam, but neglecting any shear force arising from end moments. These values are found by assuming that pins are located in the beams at the face of the columns as illustrated in Figure CD.6(b) and (c);
(c) The shear force in each beam due to the end moments that are sustained when overstrength moments act at the critical sections of the potential plastic regions in the beams as illustrated in Figure CD.6(c).

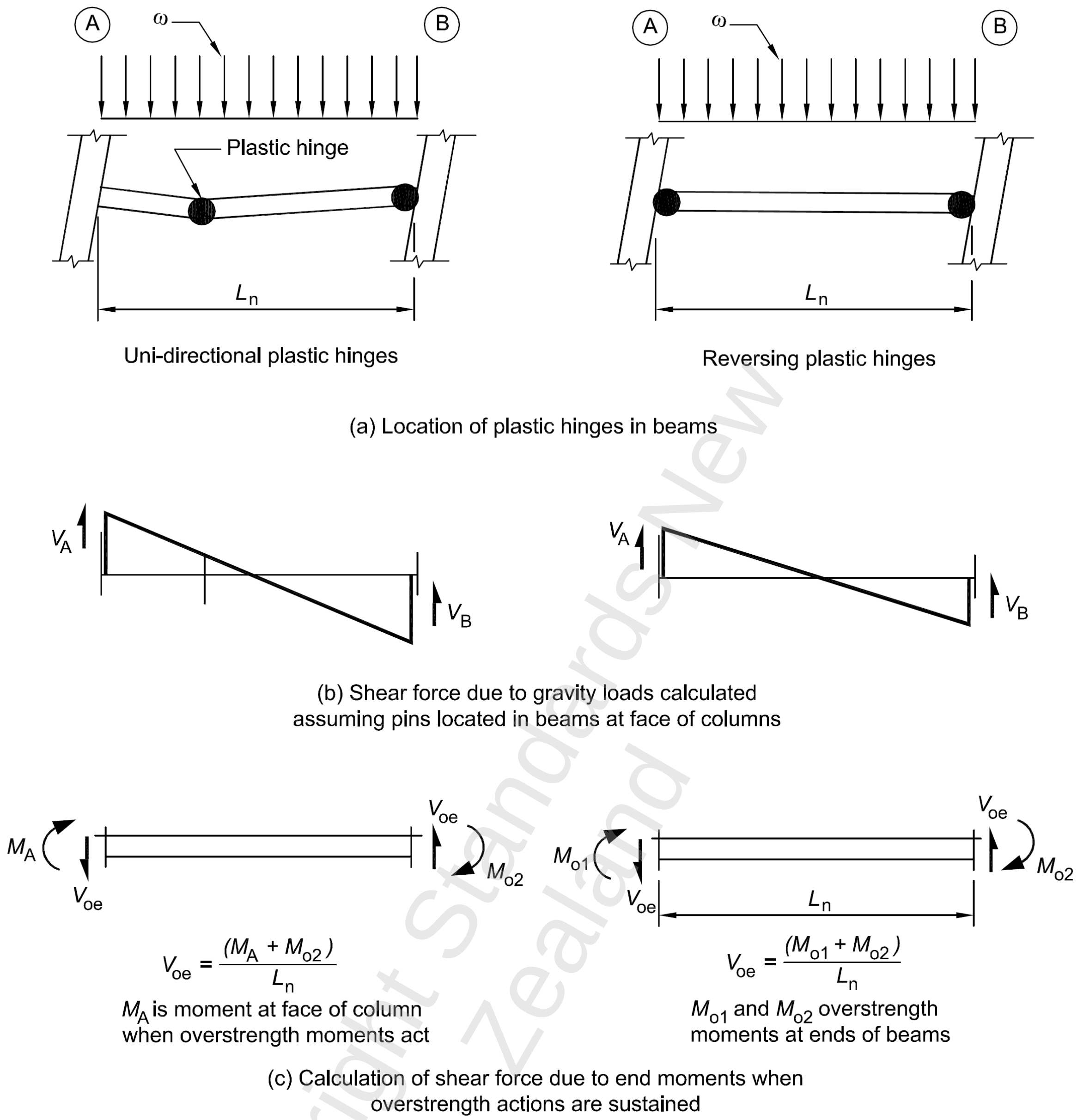


Figure CD.6 – Calculation of axial forces in columns

The resultant axial force at any level is found by summing the resultant forces acting on the columns above the level being considered. However, as not all the plastic hinges in the beams will in general sustain their overstrength actions simultaneously, and some reduction in axial force component, N_{oe}, (equal to ΣV_{oe}) calculated from the end moments in the beams in (c) above, may be made. To allow for this reduction this component of the axial force may be multiplied by the factor R_v, which is given by Equation D–8. It should be noted that the component of axial load, $R_v N_{oe}$, can be tensile or compressive and it is likely to vary in magnitude with the direction of sway sustained by the structure.

In calculating the critical axial load in a column, which forms part of two or more frames, the components of axial load from all the frames should be included. Any additional axial load associated with vertical ground motion is neglected on the basis that this does not exist for a sufficient length of time to significantly affect the performance of the column.

In designing the column to sustain the bending moments and axial forces care is required to ensure that the critical combinations of moment and axial force are chosen. It should be noted that often the bending moments associated with minimum axial load are different from those associated with the maximum level of axial load.

CD4 Ductile and limited ductile walls

CD4.1 General

Capacity design of walls is required to ensure that in the event of a major earthquake a ductile flexural failure mechanism forms in preference to non-ductile failure modes. A ductile failure mechanism is chosen, which generally consists of a plastic hinge at the base of the wall. However, where a wall in a high rise structure is supported at an intermediate height, by lower walls or a frame, such that the bending moment decreases below this level (such as many podium structures, the selected position of the plastic hinge may be located at the height of the intermediate support.

To ensure that inelastic deformation is predominately confined to the chosen location the design moments for the wall need to be modified from those found in an equivalent static or modal response spectrum analysis to allow for higher mode effects. This is considered D4.2 and CD4.2. Higher mode effects also have a significant effect on the distribution of design shear forces. This aspect is considered in D4.3 and CD4.3.

CD4.2 Design moment envelope

Higher mode effects in the walls, which occur when a plastic hinge forms at the base of a wall, cause the bending moments sustained in the mid-height regions to increase. To prevent, or limit, the formation of secondary plastic regions in zones that have not been detailed to sustain plastic deformation, the design moment envelope needs to be modified. Design rules for regular multi-storey walls have been established, and one of these is given below. However, to date design rules for the general case have not been established either for flexure or shear.

Diagonal cracking associated with shear stresses increases the magnitude of the tension force sustained by reinforcement (see 8.6.11.3). To allow for this effect reinforcement should be extended for a distance of the wall length, ℓ_w, plus a development length beyond the moment envelope.

The capacity design moment envelope for a structural wall, which is regular in elevation, is shown in Figure CD.7. The bending moment envelope is based on the nominal flexural strength of the wall at the critical section of the primary plastic hinge, which is identified as point B in Figure CD.7. This is located either at the base of the wall, or at a level above the base where the wall is supported by stiff structural elements such that the bending moment below this point reduces in magnitude. The design envelope is defined by connecting the required flexural strength at points A, B, C and D by straight lines, as illustrated in Figure CD.7. Point A is at the base of the wall, point B is at the critical section of the plastic hinge, point C is at mid-height between point B and point D, which is at the top of the wall. For most non-podium type walls points A and B will be co-incident. The required nominal flexural strength of the wall at the identified points are given by:

(a) At point B the bending moment is equal to the nominal flexural strength of the wall, $M_{n,B}$;

(b) At point D (top of wall) the bending moment is zero;

(c) At point C the bending moment, M^*_c, is the larger of half the moment at point B, which is $(M_{n,B})/2$, or the value given by:

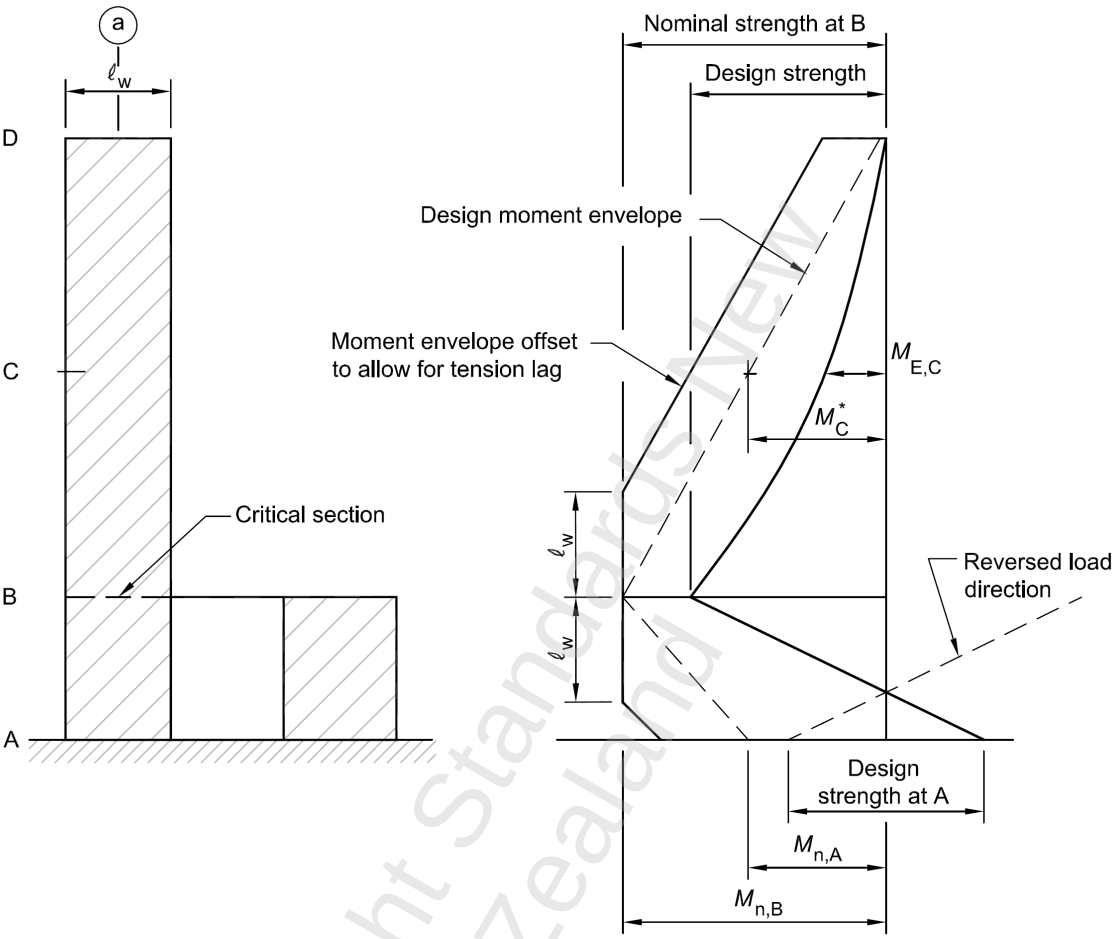
$$M^*_c = \frac{M_{E,C}}{0.85}\left[1+\frac{n_t-1}{4}\right] \leq 2.0 M_{E,C} \quad \text{(Eq. CD–7)}$$

Where n_t is the total number of storeys and $M_{E,C}$ is the bending moment at point C found in an equivalent static or modal response spectrum analysis for design actions at point C.

Equation CD–7 allows for a limited amount of non-uniformity in the seismic masses over the height of the wall.

The bending moment envelope between points A and B, is taken as the line connecting the required nominal flexural strength at A ($M_{n,A}$) and the required minimum nominal flexural strength at B ($M_{n,B}$).

The design envelope is shown on Figure CD.7 as a dashed line. In determining where longitudinal reinforcement may be cut off in a wall, allowance must be made for tension lag associated with diagonal cracking. To allow for this action, except at the top of the wall, the reinforcement should be extended for a distance of one wall length (ℓ_w) plus a development length for the bar beyond the point where the reinforcement is no longer required by standard flexural theory and the design envelope, see Figure CD.7.



(a) Elevation on wall (b) Bending moments

Figure CD.7 – Capacity design bending moment envelope for a structural wall

CD4.3 Design shear force envelope

A design envelope for shear in a regular high rise wall has been established and used extensively (see NZS 3101:1995), However, rational design envelopes for the general case where strength, stiffness or seismic weights vary over the height of the wall have not been developed. The approach detailed below is based on a recommended envelope for uniform walls. It should be used with caution and conservatism for cases where uniform conditions do not apply.

The design shear force at any level above the critical section of the primary plastic region in a structural wall, V^*_o, shall be taken as not less than the corresponding shear force found form an equivalent static analysis multiplied by an overstrength factor, ϕ_o, and a dynamic magnification factor, ω_v, such that:

$$V^*_o = \omega_v \phi_o V_E \qquad \text{(Eq. CD–8)}$$

where ω_v is the dynamic shear magnification factor, which is given by:

$$\omega_v = 0.9 + \frac{n_t}{10} \qquad \text{(Eq. CD–9)}$$

for buildings up to 6 storeys, and

$$\omega_v = 1.3 + \frac{n_t}{30} \leq 1.8 \quad \text{(Eq. CD–10)}$$

for buildings over 6 storeys,

where n_t is the number of storeys

ϕ_o is the overstrength factor related to flexural actions at B given by:

$$\phi_o = \frac{\text{Flexural overstrength at B}}{\text{Bending moment at B from an equivalent static analysis}} \quad \text{(Eq. CD–11)}$$

And V_E is the shear force found from the analysis for seismic actions from an equivalent static analysis.

The shear force envelope below the critical section of the plastic hinge in the high rise wall, shall be taken as the larger of:

(a) The calculated shear force when the overstrength actions are applied to the critical section of the primary plastic hinge; or

(b) 1.6 the shear force found in an equivalent static analysis.

A number of analyses have been made to investigate the distribution of storey shear in structures, which contain walls of different length or stiffness. These have shown that elastic based methods of analysis cannot predict realistic shear force envelopes[D.4]. Hence in designing such structures a conservative approach is strongly recommended. In particular designers should be aware that significant redistribution of shear force can occur between walls. This is most acute in the first and second storeys and in the highest storey for the wall or walls. At present accurate analytical values of shear and moments cannot be accurately predicted in these regions. Hence designers should anticipate that some limited inelastic deformation may occur in these zones, outside the primary plastic regions. As the flexural inelastic deformation will be limited full ductile detailing is not required, but detailing which could severely confine a plastic region should be avoided. However, to prevent a brittle shear failure shear reinforcement should be assessed conservatively and it should be well in excess of the minimum value for walls. It should be noted that the analyses reported in reference [D.4] did not allow for changes in shear stiffness, which occur when diagonal cracks form, or for the limited levels of shear deformation ductility, that is available in members that contain moderate amounts of shear reinforcement.

CD5 Wall-frame structures – Ductile and limited ductile

This form of building has a number of advantages over pure frame or wall buildings. Guidelines for the design of these structures are given in reference D3.

REFERENCES

D.1 NZS 1170.5:2004, "Structural Design Actions - Part 5: Earthquake Actions, New Zealand", Standards New Zealand.

D.2 Bulletin of NZ National Society for Earthquake Engineering, "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames", Sections A, E, F, C and H, Vol. 10, No. 2, June 1977, pp 69-105.

D.3 Paulay, T. and Priestley, M.J.N., "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley & Sons, 1992, 767 pp

D.4 Rutenberg, A and Leibovich, E., "On the lateral force distribution among structural walls in multistorey buildings", Bulletin of NZSEE, Vol. 35, No. 4, Dec. 2002.

NOTES

APPENDIX CE – SHRINKAGE AND CREEP

A3

CE1 General

The requirements for shrinkage and creep have been adapted from AS 3600:2009[E.1] and the draft revision (2014) of AS 5100.5 [E.2]. This commentary has been adapted from draft commentary prepared for AS 3600-2009.

Creep and shrinkage in concrete depends on the composition of the concrete, the effective thickness of the elements making up a member and the environment. Increasing the damp curing period of the concrete reduces the shrinkage. The amount of creep that occurs under a given load is smaller as the age of the concrete at the time the load is applied is increased [E.1, E.2, E.3, E.4].

Reinforcement in a section partially restrains movements due to creep and shrinkage. Methods of allowing for this action are discussed in References E.3 and E.4 and a simple approach is given in Appendix CE.

CE2 Shrinkage

Shrinkage of concrete is the time-dependent strain in an unloaded and unrestrained specimen at constant temperature. It is important to distinguish between *plastic* shrinkage, *chemical* shrinkage, *thermal* shrinkage and *drying* shrinkage. Plastic shrinkage occurs in the wet concrete before setting, whereas chemical, thermal and drying shrinkage all occur in the hardened concrete after setting.

Drying shrinkage is the reduction in volume caused principally by the loss of water during the drying process. It increases with time at a gradually decreasing rate and takes place in the months and years after casting. The magnitude and rate of development of drying shrinkage depend on all the factors that affect the drying of concrete, including the relative humidity, the mix characteristics (in particular, the type and quantity of the binder, the water content and water-to-cement ratio, the ratio of fine-to-coarse aggregate, and the type of aggregate), and the size and shape of the member[E.5, E.6, E.7].

Chemical (or autogenous) shrinkage results from various chemical reactions within the cement paste and includes hydration shrinkage that is related to the degree of hydration of the binder in a sealed specimen with no moisture exchange. Chemical shrinkage occurs rapidly in the days and weeks after casting and is less dependent on the environment and the size of the specimen than drying shrinkage. *Thermal shrinkage* is the contraction that results in the first few hours (or days) after setting as the heat of hydration gradually dissipates. The term *endogenous shrinkage* is used to refer to that part of the shrinkage of the hardened concrete that is not associated with drying (i.e. the sum of autogenous and thermal shrinkage).

Drying and autogenous shrinkage both increase with time at a decreasing rate. Drying shrinkage may continue for many years depending on the size and shape of the specimen, but autogenous shrinkage is essentially complete at about 50 days after setting.

Drying shrinkage is dependent on the rate and magnitude of water loss from the hardened concrete. Shrinkage increases with the proportion of free water used in the mix and on the ability of this water to escape from the concrete due to its permeability. If the aggregate content remains constant, drying shrinkage increases when the water-cement ratio increases, when the relative humidity decreases and when the ratio of the exposed surface area to volume increases. Temperature rises accelerate drying and therefore increase shrinkage. Higher shrinkage also arises in concrete that has an effective curing period of less than seven days (or equivalent maturity). By contrast, endogenous shrinkage increases as the cement content increases and the water-cement ratio decreases. In addition, endogenous shrinkage is not affected significantly by the ambient relative humidity.

The effect of a member's size on drying shrinkage should be emphasised. For a thin member, such as a slab, the drying process may be complete after several years, but for the interior of a larger member, the drying process may continue throughout its lifetime. For example, at a relative humidity of 60 % it typically takes 18 months for 50 % of the total shrinkage to develop in a member with a thickness of 400 mm, whereas with a member with a thickness of 100 mm the corresponding time is of the order of one month. For uncracked mass concrete structures, there is no significant drying (shrinkage) except for about

A3 300 mm from each exposed surface. By contrast, the chemical shrinkage is not affected by the size and shape of the specimen.

Shrinkage is also affected by the volume and type of aggregate. Aggregate provides restraint to shrinkage of the cement paste, so that an increase in the aggregate content reduces shrinkage. Shrinkage is also smaller when stiffer aggregates are used, i.e. aggregates with higher elastic moduli. Thus shrinkage is considerably higher in lightweight concrete than in normal weight concrete (often by more than 50 %).

Shrinkage may be reduced by:

(a) The use of special additives;
(b) Reducing the proportion of free water in the mix;
(c) Increasing the proportion of course aggregate in the concrete;
(d) Increasing the stiffness of the course aggregate;
(e) Increasing the thickness of the member;
(f) Reducing the drying characteristics of the environment surrounding the member;
(g) Reducing the porosity of the concrete;
(h) Increasing the damp curing period of the concrete (or at maturity when damp curing ceases).

CE2.1 Calculation of design shrinkage strain

For estimating the design shrinkage strain of concrete (ε_{cs}), one of three approaches may be used. The first approach is to determine ε_{cs} from measurements on similar local concrete. The second is to determine ε_{cs} by measuring the shrinkage strain that develops in a standard prism after 8 weeks of drying in accordance with AS 1012.13 [E.8] and then to separate out the probable endogenous shrinkage from the total shrinkage measured to provide an estimate of the basic drying shrinkage, $\varepsilon_{csd,b}$. The final long-term value for ε_{cs} can then be determined using the procedure outlined in E1.2. The third approach is to calculate ε_{cs} using the procedure outlined in E1.2 and the default values for $\varepsilon_{csd,b}$ given in Table E.2.

The first and second options will give a more reliable value than the third. The second option involves the standard shrinkage test, where the shrinkage of a prism, of length 280 mm and with cross section 75 mm by 75 mm, is measured over a period of 56 days at a relative humidity of 50 %. The results of such a test can be used to estimate the long-term design shrinkage (the 30-year value) of a particular element in a structure using the model outlined in E1.2. An example of this calculation is provided in CE2.2.

It is worth noting that the product of k_1 and k_4 for a test conducted in accordance with AS 1012.13[E8] is equal to 1.0. Therefore the basic drying shrinkage will equal the test result minus any autogenous shrinkage that occurred over the measurement period.

CE2.2 Design shrinkage strain

The predictive model adopted in the Standard was originally developed by Gilbert (Reference E.9) for estimating the shrinkage strain in normal and high strength concrete. The model divides the total shrinkage strain, ε_{cs}, into two components, the autogenous shrinkage strain, ε_{cse}, and the drying shrinkage strain, ε_{csd}, (i.e. $\varepsilon_{cs} = \varepsilon_{cse} + \varepsilon_{csd}$).

For high-strength concrete, autogenous (or chemical) shrinkage is significantly more than for normal-strength concrete. The autogenous shrinkage is assumed to develop exponentially, rapidly approaching a final value ε^*_{cse} that varies linearly with the concrete strength, ranging from $\varepsilon^*_{cse} = 10 \times 10^{-6}$ when $f'_c = 20$ MPa to $\varepsilon^*_{cse} = 250 \times 10^{-6}$ when $f'_c = 100$ MPa. In the first month after setting, 95 % of the autogenous shrinkage is assumed to have occurred. Unlike drying shrinkage, the autogenous shrinkage is assumed to be independent of both the environmental conditions and the size and shape of the concrete member.

At any time t after the commencement of drying, the drying shrinkage strain is given by

$$\varepsilon_{csd} = k_1 k_4 \varepsilon_{csd,b} \quad \text{(Eq. CE–1)}$$

The term $\varepsilon_{csd,b}$ depends on the quality of the local aggregates. Values for various aggregate types and locations around New Zealand are presented in Table E.2 and have been derived from the drying shrinkage test results reported by Mackechnie in CCANZ Technical Report TR11 [E.10] modified by subtracting the endogenous shrinkage occurring between day 7 and day 63 at the completion of the AS 1012.13 [E.8] test as determined from Equation E–2. The categorisation of the various aggregate types is outlined by Mackechnie [E.11].

The factor k_1 describes the development of drying shrinkage with time and accounts for the dependence of drying shrinkage on the size and shape of the member. It is calibrated in terms of the hypothetical thickness t_h (in mm) and the time t after the commencement of drying (in days). The rate of development of drying shrinkage, as well as the final magnitude, increases as the hypothetical thickness reduces. The factor k_1 is given by: A3

$$k_1 = \frac{\alpha_1 t^{0.8}}{t^{0.8} + 0.15 t_h} \quad \text{(Eq. CE–2)}$$

where. $\alpha_1 = 0.8 + 1.2e^{-0.005 t_h}$.

The factor k_4 depends on the average humidity of the environment. Table CE.1 presents average relative humidities for various locations around New Zealand based on an analysis of data from NIWA's Cliflo climate database.

Table CE.1 – Average relative humidities for various New Zealand locations

Location	**RH** (%)	**Location**	**RH** (%)	**Location**	**RH** (%)	**Location**	**RH** (%)
Kaikohe	85	Taupo	79	Wellington	76-81	Ashburton	77
Whangarei	80	Taumarunui	82	Nelson	77	Timaru	80
Dargaville	80	Gisborne	78	Blenheim	75	Franz Josef	84
Auckland	77-82	New Plymouth	83*	Westport	83	Queenstown	70*
Hamilton	82	Waiouru	82	Kaikoura	75	Alexandra	66*
Tauranga	77	Napier	76	Greymouth	79	Oamaru	80
Whakatane	80	Whanganui	77	Hokitika	85*	Dunedin	78
Rotorua	83*	Palmerston Nth	80	Hanmer	76	Invercargill	81
Te Kuiti	77	Masterton	78	Christchurch	78		

NOTE –

RH values are average RH assessed from NIWA CliFlo data for the period 2008 – 12.

* Conservative (low) estimates based on incomplete NIWA datasets. Data missing: Rotorua, New Plymouth, Hokitika – early hours of the morning; Queenstown – night-time hours; Alexandra – night-time hours and large and irregular gaps in daytime hours.

In general, humidity is usually higher during the hours of darkness, peaking a little before dawn.

For specimens with an assumed basic drying shrinkage strain of $\varepsilon_{csd,b} = 1000 \times 10^{-6}$ and with t_h = 200 mm, the shrinkage strain components predicted by the above model at 28 days after the commencement of drying and after 30 years (t = 10,950 days) are given in Table CE.2.

It is well known that in addition to the environment the water-cement ratio and the size and shape of the specimen, shrinkage is also highly dependent on the amount and type of aggregate and the proportions of the binder. The influence of aggregate type on shrinkage is shown in Figure CE.1 (taken from Reference E.5 for concretes of fixed mix proportions but with different aggregates, and stored in air at 21 °C and 50 % relative humidity).

A3 **Table CE.2 – Design shrinkage strain components (t_h = 200mm and $\varepsilon_{csd,b} = 1000 \times 10^{-6}$)**

Environment	f'_c (MPa)	Shrinkage strain components (× 10^{-6})					
		t = 28 days			*t* = 10,950 days (30 years)		
		ε_{cse}	ε_{csd}	ε_{cs}	ε_{cse}	ε_{csd}	ε_{cs}
50 % RH k_4=0.68	25	23	274	297	25	830	855
	30	38	274	311	40	830	870
	40	66	274	339	70	830	900
	50	94	274	367	100	830	930
	60	122	274	396	130	830	960
	70	150	274	424	160	830	990
	80	178	274	452	190	830	1020
70 % RH k_4=0.50	25	23	201	225	25	610	635
	30	38	201	239	40	610	650
	40	66	201	267	70	610	680
	50	94	201	295	100	610	710
	60	122	201	323	130	610	740
	70	150	201	351	160	610	770
	80	178	201	380	190	610	800
80 % RH k_4 = 0.39	25	23	157	180	25	476	501
	30	38	157	194	40	476	516
	40	66	157	223	70	476	546
	50	94	157	251	100	476	576
	60	122	157	279	130	476	606
	70	150	157	307	160	476	636
	80	178	157	335	190	476	666

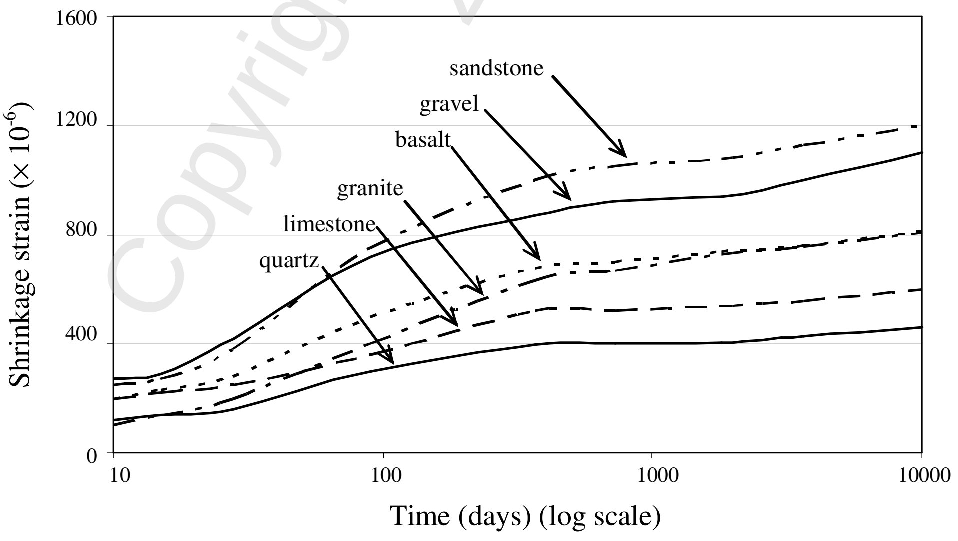


Figure CE.1 – Effect of aggregate type on shrinkage

The method outlined in the Standard requires only a short calculation, but designers should be aware the estimate of ε_{cs} is within a range of ±30 %, and this should be allowed for in the specification of shrinkage requirements for concrete production. Being unduly restrictive in specifying the required limit on shrinkage can significantly increase the cost of concrete production.

Because details of the mix proportions are not included in the prediction model for shrinkage and the influence of aggregate type is only modelled approximately, a better estimate of ε_{cs} will usually be determined from long-term tests on samples kept in the same environmental conditions as the concrete in the structure and made from concrete that is similar to that intended to be used in the structure. A3

Relationship between 56-day shrinkage measured in accordance with AS 1012.13 and the design shrinkage strain, ε_{cs}

A good estimate of the final design shrinkage strain in a particular member can be obtained from the results of a standard 56-day shrinkage test. The standard test is conducted on a 75 mm × 75 mm × 280 mm prism at a relative humidity of 50 %. For the standard test specimen, the hypothetical thickness is t_h = 37.5 mm, k_4 = 0.68 and, from Equation CE–2 at 56 days, k_1 = 1.466.

To illustrate the procedure consider the following example:

Example calculation: In a standard prism of concrete with f'_c = 40 MPa, if the shrinkage strain measured after 56 days is 650×10^{-6}, determine the final design shrinkage strain ε_{cs} after 30 years in a 200 mm thick slab of the same concrete in a temperate inland environment of 60 % relative humidity and drying from both the top and bottom surfaces.

All the autogenous shrinkage will have occurred at 56 days but half of it will have occurred in the initial 7-day period of wet curing. For 40 MPa concrete:

$$\varepsilon_{cse} = (0.06\ f'_c - 1.0) \times 50 \times 10^{-6} = 70 \times 10^{-6}.$$

The drying shrinkage strain at 56 days is therefore:

$$\varepsilon_{csd,b} = \varepsilon_{cs} - 0.5\varepsilon_{cse} = 615 \times 10^{-6}$$

For the 200 mm thick concrete slab, t_h = 200 mm, k_4 = 0.61 and from Equation CE–2 at 30 years (t = 10,950 days), k_1 = 1.22. With $\varepsilon_{csd,b} = 615 \times 10^{-6}$, the final drying shrinkage is obtained from Equation E–5:

$$\varepsilon_{csd} = k_1\ k_4\ \varepsilon_{csd,b} = 1.22 \times 0.61 \times 615 \times 10^{-6} = 458 \times 10^{-6}$$

With the autogenous shrinkage equal to 70×10^{-6}, the final design shrinkage strain is

$$\varepsilon_{cs} = \varepsilon_{cse} + \varepsilon_{csd} = 528 \times 10^{-6}$$

CE3 Creep

When concrete is subjected to a sustained stress, creep strain develops gradually with time. Creep increases with time at a decreasing rate. In the period immediately after first loading, creep develops rapidly, but the rate of increase slows appreciably with time. Creep is generally thought to approach a limiting value as the time after first loading approaches infinity. About 50 % of the final creep develops in the first 2 to 3 months and about 90 % after 2 to 3 years. After several years under load, the rate of change of creep with time is very small. Creep has its origins in the hardened cement paste and is caused by several different mechanisms[E.12].

Many factors influence the magnitude and rate of development of creep, including the properties of the concrete mix and its constituent materials. In general, as the concrete quality increases, the capacity of concrete to creep decreases. For a particular stress level, creep in higher-strength concrete is less than that in lower-strength concrete. An increase in either the aggregate content or the maximum aggregate size reduces creep, as does the use of a stiffer aggregate type. Creep also decreases as the water-to-cement ratio and amount of free water in the mix is reduced.

Creep depends on the environment and increases as the relative humidity decreases. Creep is also greater in thin members with large surface area-to-volume ratios, such as slabs. However, the dependence of creep on both the relative humidity and the size and shape of the specimen decreases as the concrete strength increases. Near the surface of a member, creep takes place in a drying environment and is therefore greater than in regions remote from a drying surface. In addition to the relative humidity, creep is dependent on the ambient temperature. A temperature rise increases the deformability of the cement paste and accelerates drying, and thus increases creep.

A3 In addition to the environment and the characteristics of the concrete mix, creep depends on the loading history, in particular the magnitude of the stress and the age of the concrete when the stress is first applied. When the sustained concrete stress is less than about 0.5 f'_c (and this is usually the case in concrete structures at service loads), creep is proportional to stress and is known as linear creep. The age of the concrete when the stress is first applied has a marked influence on the magnitude of creep. Concrete loaded at an early age creeps more than concrete loaded at a later age [E.7].

CE3.1 General

The capacity of concrete to creep is usually measured in terms of the creep coefficient, given the symbol φ_{cc} in the Standard. In a concrete specimen subjected to a constant sustained compressive stress σ_0, first applied at age τ_0, the creep coefficient at time t is the ratio of creep strain (ε_{cc}) to instantaneous strain (ε_e) at that time and is given by $\varphi_{cc} = \varepsilon_{cc}/\varepsilon_e$. Therefore, the creep strain at time t is:

$$\varepsilon_{cc} = \varphi_{cc}\,\varepsilon_e = \varphi_{cc}\,\sigma_0/E_c \quad \text{(Eq. CE–3)}$$

For stress levels less than about 0.5 f'_c, the creep coefficient is a pure time function, independent of the applied stress, and the creep coefficient increases with time at an ever-decreasing rate. As time approaches infinity, the creep coefficient is assumed to approach a final value, $\varphi^*_{cc} = \varphi_{cc}(\infty)$, which usually falls within the range 1.5 to 4.0 (Reference E.7). In design, the Standard specifies that the value of the elastic modulus E_c in Equation CE–3 is the 28-day value.

Another measure of the capacity of concrete to creep is known as *specific creep*. Specific creep is the creep strain per unit stress and, from Equation CE–3, is equal to the creep coefficient divided by the elastic modulus.

CE3.2 Basic creep coefficient

The most accurate way of determining the final creep coefficient is by testing or by using results obtained from measurements on similar local concretes. Testing is often not a practical option for the structural designer. In the absence of long-term test results, the final creep coefficient may be determined by extrapolation from relatively short-term test results in accordance with AS 1012.16 [E.13], where creep is measured over a relatively short period (say 28 days) in specimens subjected to constant stress. Various mathematical expressions for the shape of the creep coefficient versus time curve are available from which long-term values may be predicted from the short-term measurements. The longer the period of measurement, the more accurate are the long-term predictions. Some of the more useful expressions for φ_{cc} are presented in Reference E.12.

If testing is not an option, the Standard specifies a procedure to provide a quick and approximate estimate of the design creep coefficient.

The Standard defines the basic creep coefficient ($\varphi_{cc.b}$) as the mean value of the final creep strain to elastic strain for a standard specimen (i.e. a 100 mm diameter cylinder with hypothetical thickness, t_h = 50 mm at a relative humidity of 70 %) under a constant compressive stress of 0.4f'_c first applied at age 28 days. Prescribed values of $\varphi_{cc.b}$ for each standard strength grade of concrete were proposed in Reference E.9 and are given in Table E.3, where values range from $\varphi_{cc.b}$ = 5.2 for 20 MPa concrete to $\varphi_{cc.b}$ = 1.5 for 100 MPa concrete. Alternatively, the basic creep coefficient may be obtained from measurements on similar local concrete or determined by testing standard 100 mm diameter cylinders in accordance with AS 1012.16 [E.13].

CE3.3 Design creep coefficient

The standard permits the creep coefficient at any time φ_{cc} to be calculated from the basic creep coefficient using any accepted mathematical model of the development of creep strain with time provided that the model is calibrated so that it predicts the final creep coefficient for a 100 mm thick cylinder loaded at 28 days at a relative humidity of 70 % to be equal to $\varphi_{cc.b}$. The prescribed model for calculating the design creep coefficient at any time was first proposed by Gilbert in Reference E.9 and involves multiplying the basic creep coefficient by four different factors: k_2, k_3, k_4 and k_5.

The factor k_2 (graphed in Figure 5.2) describes the development of creep with time (t in days) and is similar to, but not identical to, the shrinkage factor k_1, and is given by:

A3

$$k_2 = \frac{\alpha_2 t^{0.8}}{t^{0.8} + 0.15 t_h} \quad \text{(Eq. CE–4)}$$

where $\alpha_2 = 1.0 + 1.12 e^{-0.008 t_h}$.

The factor k_3 depends on the age at first loading, τ. Gilbert and Ranzi have proposed the following expression for k_3, which they claim is an improvement over the previous trilinear relation presented in AS 3600 [E.1] and more convenient for inclusion in mathematical modelling of the creep coefficient [E.7]:

$$k_3 = \frac{2.7}{1 + \log(\tau)} \quad \text{(Eq. CE–5)}$$

The factor k_4 is identical to that specified for drying shrinkage and accounts for the environment, while the factor k_5 accounts for the reduced influence of both the relative humidity and the specimen size on the creep of concrete as the concrete strength increases (or more precisely, as the water-binder ratio decreases).

The factor k_6 has been developed from the limited information presented in Mackechnie and Fenwick (Reference E.14) of testing undertaken on a limited range of concrete mixes with a constant water-cement ratio of 0.5 made with different aggregates from major centres around New Zealand. Aggregate types have been classified as described in Mackechnie (Reference E.11). For only one aggregate type, central greywacke, was more than one data point available, for which the derived k_6 factor ranged from 0.8 to 1.4. For all other aggregate types the suggested k_6 factor is based on a single data point. Thus considerable variability from the suggested values is likely.

A family of creep coefficient versus duration of loading curves obtained using the prescribed model is shown in Figure CE.2 for a concrete specimen located in a temperate environment, with a hypothetical thickness t_h = 150 mm, concrete strength f'_c = 40 MPa and loaded at different ages, τ.

The above discussion is concerned with compressive creep. In many practical situations, creep of concrete in tension is also of interest. Tensile creep plays an important part in delaying the onset of cracking caused by restrained shrinkage. The mechanisms of tensile creep are different from those of compressive creep, but at the same stress levels the magnitudes are similar. In design, it is usual to assume that the creep coefficients in tension and in compression are identical. Although not strictly correct, this assumption simplifies calculations and does not usually introduce serious inaccuracies.

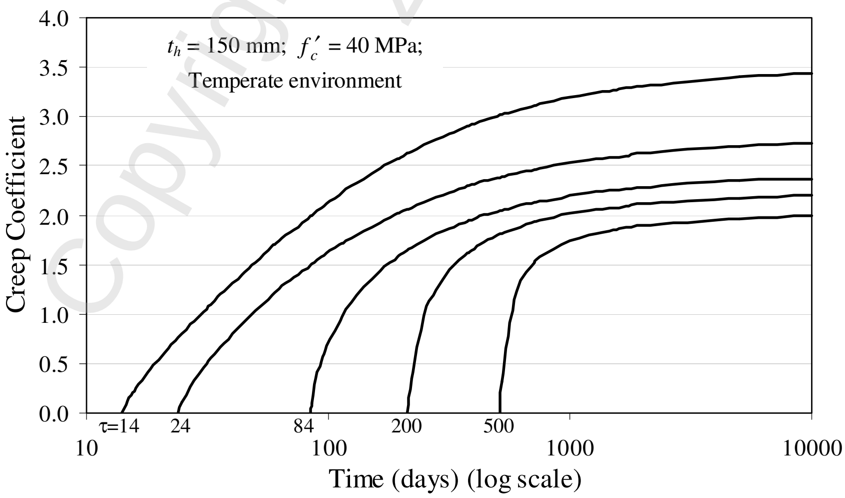


Figure CE.2 – Typical creep coefficient versus time curves

It should be emphasised that creep of concrete is highly variable with significant differences in the measured creep strains in seemingly identical specimens, tested under identical conditions (both in terms of load and environment). The model for the design creep coefficient in E2.3 does not account for such factors as cement type, cement replacement materials and more, but it does provide a ballpark estimate of the creep coefficient for both normal and high-strength concrete with a range of approximately ±30 %. The standard cautions that this range may be exceeded where the temperature is greater than 25 °C for a

A3 prolonged period or when the sustained concrete stress exceeds about 50 % of the characteristic strength of concrete apply.

CE4 Analysis of prestressed concrete structures for creep and shrinkage

Many methods of analysis have been proposed for assessing the effects of creep and shrinkage in concrete structures. The problem is very complex and there is not a single theoretical solution that can be used. However, most typical situations that arise in design can be assessed with sufficient accuracy for design purposes by a method that is known as either "the age-adjusted effective modulus method" [E.15] or the "modified effective modulus method" [E.16]. This method is briefly outlined below. More comprehensive analytical methods of analysis of a wide variety of problems may be found in the References E.3 and E.17 and a number of other texts.

Where creep and shrinkage may have an important influence on the behaviour of concrete members the designer should assess appropriate creep and shrinkage values for use in any analysis and examine the influence of varying these properties.

CE4.1 *Modified effective modulus method of analysis*

When concrete is subjected to a stress, σ_c, for a short period of time the strain in the concrete, ε_c, is given by:

$$\varepsilon_c = \sigma_c / E_c \quad \text{(Eq. CE–6)}$$

where E_c is the elastic modulus. If this stress is maintained the strain increases due to creep. If the creep factor is ϕ, than the creep strain is equal to $\phi\varepsilon_c$. The total strain in the concrete, ε_t, is equal to the sum of the elastic plus creep strains and hence:

$$\varepsilon_t = \varepsilon_c (1 + \phi) \quad \text{(Eq. CE–7)}$$

which can be rewritten as:

$$\varepsilon_t = \sigma / E_{eff}$$

where E_{eff}, the effective modulus of elasticity, is given by:

$$E_{eff} = \frac{1}{1+\phi} E_c \quad \text{(Eq. CE–8)}$$

The effective modular ratio, n_{eff}, is calculated for any particular case using the E_{eff} elastic modulus. The basis of the modified effective modulus method is to analyse the structure as an elastic body but replacing the elastic modulus of the concrete with E_{eff}. For load cases where the actions are induced over a period of time the creep factor, ϕ, is replaced by $k\phi$, where k is a coefficient which allows for the reduction in creep potential over the period in which the action is being gradually introduced. Shrinkage of concrete is one such case, with the shrinkage developing at a similar rate that creep displacements occur in concrete subjected to constant stress. For shrinkage an appropriate value of k is typically 0.6. However, different texts give a range of values depending on how the development of creep is modelled. Experimental work at the University of Auckland and elsewhere has shown that predictions based on using a value of k of 0.6 give realistic assessments of shrinkage-induced actions in many common situations [E.3, E16]. However, it should be noted that in thick structural members shrinkage develops at an appreciably slower rate than creep, consequently for these cases a smaller value of k may be appropriate.

Creep and shrinkage can give rise to a number of structural effects, such as those listed below:

(a) Creep of the concrete can cause redistribution of structural actions for cases where the load, or some of the load, is applied and the structure is subsequently modified. Two examples of this include the addition of a flange to a prestressed member and the connection of two simply supported beams after some of the load has been applied;

(b) Deflection and stresses induced by shrinkage of concrete;

(c) Creep, including differential creep, and shrinkage, including differential shrinkage, of concrete can induce significant stresses and deflection of members; A3

(d) Reduction of actions induced by imposed displacements, such as differential settlement of foundations, with time;

(e) The restraint that reinforcement provides against creep and shrinkage movements can lead to reinforcement being subjected to high compression stresses, such that when concrete first cracks the reinforcement can be under appreciable compression.

The effective modulus method can be used to assess the actions corresponding to all the situations described above. However, it should be noted that this is an approximate method. More information on this approach and other methods of analysis may be found in References E.3, E.4, E.12, E.15, E.16, E.17 and E.18.

CE4.2 *Example of modified effective modulus method*

Consider a beam with a span of 15 m with a rectangular cross section as shown in Figure CE.3. The beam is supported laterally along its length to ensure stability against buckling. The section is 200 mm wide and the depth is 500 mm. It contains two 12 mm bars near the top of the section and two 20 mm bars near the bottom of the section, and it is prestressed by six pretensioned strains, as shown in Figure CE.3, each of which has an area of 100 mm^2, and the height of the centroid of the strands is 140 mm from the base. The initial stress in the strands just before transfer is assumed to be 1250 MPa. This value has been reduced to allow for anticipated loss of prestress due to creep in the strands (not to be confused with creep in the concrete). The concrete strength is taken as 40 MPa, the free shrinkage of the concrete as 700 × 10^{-6} and the creep factor for the concrete as 3.0. The dead load of the beam is taken as 2.5 kN/m.

The beam is analysed to find the stresses when the prestress is first applied, the stresses after creep has ceased and the stresses induced by shrinkage of the concrete. In addition the initial and long-term deflection of the beam is calculated.

CE4.2.1 *Section properties*

First the relevant section properties have to be found for the initial condition, the long-term creep calculation and the shrinkage calculation. For each of these, the effective elastic modulus of the concrete, E_{eff}, changes, and consequently the effective modular ratio varies, which changes the transformed section properties. The change in the position of the neutral axis in particular should be noted, as this changes the effective prestress moment acting on the section. The calculated values are given in the table below. The elastic modulus of both the passive and prestressed reinforcement has been taken as 200,000 MPa.

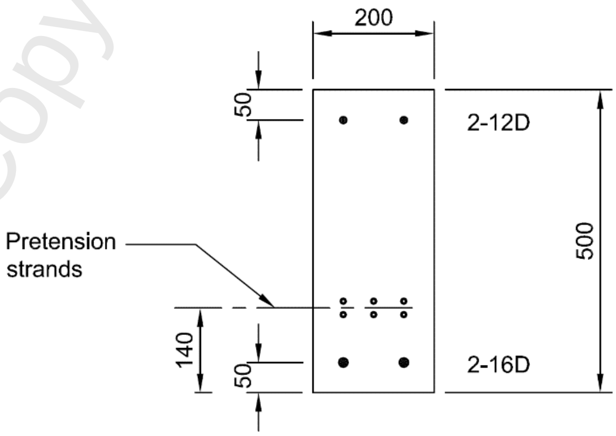


Figure CE.3 – Beam section

A3

Table CE.3 – Transformed section properties

Property	Initial condition	Long-term creep	Shrinkage of concrete
E_{eff} (MPa)	27,900	6974	9963
n_{eff}	7.17	28.68	20.07
A_t (mm^2)	108,970	140,240	127,730
Ht. to neutral axis (mm)	241.7	221.1	228.1
I_t (mm^4)	2.331×10^9	3.113×10^9	2.819×10^9
$Z_{top\ fibre}$ (mm^3)	0.9026×10^7	1.116×10^7	1.037×10^7
$Z_{bottom\ fibre}$ (mm^3)	-0.9645×10^7	-1.408×10^7	-1.235×10^7
$Z_{top\ bars}$ (mm^3)	-1.119×10^7	1.360×10^7	1.270×10^7
$Z_{bottom\ bars}$ (mm^3)	-1.216×10^7	-1.819×10^7	-1.582×10^7
$Z_{prestress}$ (mm^3)	-2.292×10^7	-3.838×10^7	-3.198×10^7

CE4.2.2 *Initial stresses*

Just before transfer buttresses hold the strands with a force of 750 kN (600 × 1250 MPa) at an eccentricity of 101.7 mm (241.7 – 140.0), which gives a *Pe* value (so-called prestress moment) of 76.28 kN m. To cancel the buttress force standard structural theory is used. That is an equal and opposite force is applied to the transformed section, of 750 kN at an eccentricity of 0.1017 m. The resultant stresses are found by adding the stresses sustained when the buttress was in place (zero in the concrete and –1250 MPa in the reinforcement) to the stresses sustained when the buttress forces are cancelled. The results of these calculations are given in Table CE.4. It should be noted that with this method of calculation the so-called elastic loss, due to elastic shortening of the pretension strands, is incorporated in the method.

Table CE.4 – Stresses in section (MPa)

Item	Initial condition	Long-term creep	Shrinkage stresses	Resultant long-term
Top fibre	−1.57	−0.1	0.1	0.0
Bottom fibre	14.8	9.7	−3.0	6.7
Top bars	0.5	25	136	161
Bottom bars	94.3	249	86	335
Prestress	−1,177	−1,051	96	−955

CE4.2.3 *Long-term creep*

The calculations are the same as for the initial condition except that the transformed section properties with long-term creep are used. Note the reinforcement has a restraining effect on the reinforcement. Two effects should be noted. Firstly the neutral axis moves towards the centroid of the reinforcement. This reduces the effective eccentricity of the prestress force, and secondly the reinforcement has a much greater influence on section properties than was the case in the calculations for the initial state.

CE4.2.4 *Shrinkage calculations*

Assume a buttress holds the beam so that no strain can develop in the concrete due to shrinkage. To achieve this the buttresses will need to apply a tensile stress equal to the free shrinkage strain times the effective elastic modulus. Hence the held stress is $700 \times 10^{-6} \times 9\,963 = -6.974$ MPa (tension). This stress corresponds to a force of −687.3 kN acting at a height of 253.8 mm. This gives an eccentricity of 25.7 mm above the neutral axis. As in the previous cases the buttress forces are eliminated by applying an equal but opposite force to the transformed section. The resultant stresses are found by adding the stresses sustained when the buttress forces acted (−6.974 MPa in concrete and zero in reinforcement) to the stresses induced in the transformed section when these forces were cancelled.

Note the loss in prestress in the concrete due to creep and shrinkage arises from the action of the reinforcement absorbing a high proportion of the prestress force. Stress calculations which ignore the influence of reinforcement on creep and shrinkage can give misleading values in situations where there is an appreciable amount of prestressed and passive reinforcement, as is the case in this example. A3

CE4.2.5 *Deflection calculations*

Standard theory can be used to calculate the deflections corresponding to different conditions. In each case the appropriate material properties should be used:

(a) *Dead load deflections*

(i) The initial dead load deflection is ....+ 25.43 (downwards)

(ii) Long-term dead load deflection allowing for creep is ....+ 75.91 mm (downwards)

(iii) Long-term deflection due to shrinkage is ....+ 4.72 mm (downwards);

(b) *Prestress deflections*

(i) The initial prestress deflection is ....− 33.0 mm (upwards)

(ii) Long-term prestress deflection allowing for creep is ....− 78.8 mm (upwards);

(c) *Resultant deflections*

(i) Initial deflection is ....− 7.6 mm (upwards)

(ii) Long-term deflection is ....+ 1.8 mm (downwards).

It should be noted that ignoring the influence of reinforcement on section properties gives very appreciable errors in deflection calculations in cases where there is an appreciable quantity of longitudinal reinforcement present. This is the inherent assumption when deflection calculations are based on gross section properties and the long-term deflection is taken as the creep factor times the short-term deflection.

REFERENCES

E.1 AS 3600:2009 *Concrete Structures*, Standards Australia.

E.2 DR AS 5100.5 *Bridge Design – Part 5: Concrete* (Revision of AS 5100.5 – 2004), Standards Australia, Draft for Public Comment, 13 January 2014.

E.3 Bryant, A.H., Wood, J.A. and Fenwick, R.C., “Creep and Shrinkage in Concrete Bridges”, Road Research Unit Bulletin No. 70, National Roads Board, New Zealand, 1984.

E.4 ACI Committee 209R-92, “Predicting Creep, Shrinkage and Thermal Effects in Concrete Structures, Manual of Concrete practice, Vol.1, 1993 and later editions.

E.5 Neville, A.M., “Properties of Concrete”, Pitman (3rd Edition), London, 1981.

E.6 Brooks, J.J., “30-Year Creep and Shrinkage of Concrete”, Magazine of Concrete Research, Vol. 57, No. 9, November 2005, pp. 545-556.

E.7 Gilbert, R.I. and Ranzi, G., “Time-Dependent Behaviour of Concrete Structures”, Spon Press, London, 2011.

E.8 AS 1012.13 – 1992 *Methods of testing concrete – Method 13: Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory*, Standards Australia.

E.9 Gilbert, R.I., “Creep and Shrinkage Models for High Strength Concrete: Proposals for Inclusion in AS3600”, Australian Journal of Structural Engineering, IEAust, Vol. 4, No. 2, December 2002, pp. 95-106.

E.10 Mackechnie, J.R., “Properties of New Zealand Concrete Aggregates”, Cement and Concrete Association of New Zealand, Technical Report 11 (TR 11).

E.11 Mackechnie, J.R., “Hardened Properties of Concrete Containing New Zealand Aggregates”, SESOC Journal, Vol. 16, No. 2, 2003.

E.12 Gilbert, R.I., “Time Effects in Concrete Structures”, Elsevier Science, New York, 1988.

E.13 AS 1012.16 – 1996 *Methods of testing concrete – Method 16: Determination of creep of concrete cylinders in compression*, Standards Australia.

E.14 Mackechnie, J.R. and Fenwick, R.C., “Assessing the Dimensional Stability of New Zealand Concrete”, SESOC Journal, Vol. 22, No. 2, 2009.

E.15 Branson, D E., “Deformation of Concrete Structures”, McGraw-Hill, NY, 1977, p. 546.

E.16 Sritharan, S. and Fenwick, R.C., “Creep and Shrinkage Effects in Prestressed Beams”, Magazine of Concrete Research, Vol. 47, No. 170, Mar. 1995, pp. 45-55.

A3 E.17 Rusch, H., Jungwirth, D. and Hilsdolf, H.K., "Creep and Shrinkage: Their Effect on Behaviour of Concrete Structures", Springer-Verlorg, New York, 1983, p. 284.

E.18 CEB-FIP, "Model Code 1990", Thomas Telford, London, 1993, p. 473.

Approved by the Standards Council on 17 March 2006 to be a New Zealand Standard pursuant to the provisions of section 10 of the Standards Act 1988.

Amendment No. 1 was approved on 5 July 2006 by the Standards Council.

Amendment No. 2 was approved on 6 August 2008 by the Standards Council.

Amendment No. 3 was approved on 5 July 2017 by the New Zealand Standards Approval Board to be an amendment to NZS 3101:2006 pursuant to section 12 of the Standards and Accreditation Act 2015.

First published: 17 March 2006

Reprinted incorporating Amendment No. 1: 18 July 2006

Reprinted incorporating Amendment No. 1 and 2: 21 August 2008

Reprinted incorporating Amendment No. 1, 2 and 3: 30 August 2017

The following references relate to this standard:

Project No. P 3101

Draft for comment No. DZ 3101

Typeset by: Standards New Zealand

STANDARDS®
NEW ZEALAND
PAEREWA AOTEAROA