NZS 3101.1:1982

Amendment No. 1, 2, and 3 appended

New Zealand Standard

Code of practice for the design of concrete structures

Part 1

NZS 3101.1:1982

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NEW ZEALAND STANDARD

Code of practice for
THE DESIGN OF CONCRETE STRUCTURES

Superseding NZS 3101P and NZSR 32

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COMMITTEE REPRESENTATION

This Standard was prepared under the supervision of the Concrete Industry Sectional Committee (31/–) for the Standards Council, established under the Standards Act 1965. The committee consisted of representatives of the following:

Association of Consulting Engineers New Zealand
Building Research Association of New Zealand
Department of Scientific and Industrial Research
Harbours Association of New Zealand
Institution of Professional Engineers New Zealand
Ministry of Works and Development
Municipal Association of New Zealand
New Zealand Concrete Masonry Association
New Zealand Concrete Research Association
New Zealand Concrete Society
New Zealand Counties Association
New Zealand Master Builders Federation
New Zealand Portland Cement Association
New Zealand Ready Mixed Concrete Association
New Zealand Universities

The Concrete Design Committee (31/12) was responsible for the preparation of the Standard and consisted of the following persons:

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Mr P.D. Leslie
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Prof. T. Paulay
Mr L.M. Robinson
Mr M.A. Wesseldine

RELATED DOCUMENTS

In this document reference is made to the following:

NEW ZEALAND STANDARDS

NZS 1900 : - - - -	*Model building bylaw –*
Chapter 5 : 1963	*Fire-resisting construction and means of egress*
Chapter 9 : - - - -	*Design and construction –*
Division 9.3.:1981	*Concrete construction*
NZS 3109 : 1980	*Concrete construction*
NZS 3152 : 1974 ® 1980	*Manufacture and use of structural and insulating lightweight concrete*
NZS 3402P : 1973	*Hot rolled steel bars for concrete reinforcement*
NZS 3404 : 1977	*Code for design of steel structures*
NZS 4203 : 1976	*Code of practice for general structural design and design loadings for buildings*
NZS 4702 : 1982	*Metal-arc welding of Grade 275 reinforcing bar*

AMERICAN STANDARD

American Concrete Institute

ACI 318–77	*Building code requirements for reinforced concrete*

The users of this document should ensure that their copies of the above-mentioned New Zealand Standards or of overseas standards endorsed as suitable for use in New Zealand are the latest revisions or include the latest amendments. Such amendments are listed in the annual SANZ *Catalogue* which is supplemented by lists contained in the monthly magazine *Standards* issued free of charge to committee and subscribing members of SANZ.

ACKNOWLEDGEMENT

Grateful acknowledgement is made to the American Concrete Institute for permission to use extracts from ACI 318-77: "Building Code Requirements for Reinforced Concrete".

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FOREWORD

The objectives in drafting this Code NZS 3101 Part 1 and its commentary NZS 3101 Part 2 have been to provide an up-to-date design code which covers the design of buildings, bridges and other civil engineering structures. In writing all the sections, particular attention has been given to producing provisions which would be appropriate for use with the modern New Zealand design loading codes -- particularly with NZS 4203:1976, *Code of Practice for general structural design and design loadings for buildings.* The Code is a revision of NZS 3101P:1970 and it has been extended to cover the design requirements for prestressed concrete. Concurrently with the publication of this document, NZS 3101P:1970 and NZSR 32:1968 *Prestressed concrete,* are revoked.

Generally the design requirements of each section of the Code are presented under five clauses in the following order:

Clause 1	Notation
Clause 2	Scope
Clause 3	General principles and requirements for design
Clause 4	Principles and requirements additional to Clause 3 for members *not* designed for seismic loading
Clause 5	Principles and requirements additional to clause 3 for members designed for seismic loading.

This arrangement of clauses represents a significant change in format from the previous code with the aim of producing a more workable document.

The intended order of usage is that after proceeding through Notation, Scope and General principles and requirements which apply to all structures, the designer then goes either to: Principles and requirements additional to Clause 3 for members *not* designed for seismic loading, or to: Principles and requirements additional to Clause 3 for members designed for seismic loading, that is, only *one* of the last two clauses is used, not both. (See diagram below.)

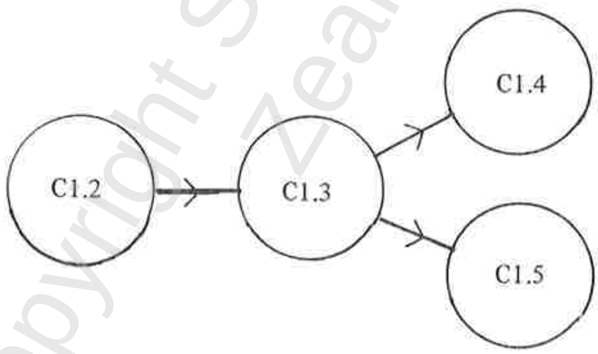


DIAGRAM INDICATING ORDER OF USAGE OF CLAUSES

Section 3, General design requirements, has a particular importance in the Code for two reasons:

(a) It covers the use of all other sections which should not be used in isolation, but should be read together with Section 3

(b) It establishes the relationship of this Code to the Loadings Code NZS 4203 and to the Ministry of Works and Development Highway Bridge Design Brief.

It should be noted that some provisions in this Code are based on proposed amendments to NZS 4203 which at the time of publication are being finalized.

Section 14 gives the design and detailing provisions for members in structures of limited ductility subjected to earthquake induced loading. This Section recognizes that less stringent ductility requirements are appropriate because of the larger lateral design loads applicable to such structures.

The Code permits considerable simplification in design procedures to be achieved if a structure is treated as responding elastically to earthquakes, under the provisions of 3.5.1.1 (c). This exempts the structure from the additional seismic requirements of all relevant sections of the Code. There will be many small structures, and some structural forms having substantial total lengths of wall in each direction, where the larger design seismic loads required for elastically responding structures will not result in significant cost increase. Alternatively, significant simplification can be obtained by the use of the procedures for design of structures of limited ductility set out in Section 14.

With the exception of the provisions for seismic loading, ACI 318-77 *Building code requirements for reinforced concrete,* has been used with minor modification. Following the practice of ACI 318-77 all sections commence with a list of notation used in that section. In addition, a list of the entire set of symbols used in the Code is presented in Appendix A. It should be noted that some symbols can have different meanings in different sections.

Appendix B presents an alternative design method which is based on working stress design whereas the main body of the code is based on the strength method of design with serviceability checks. In particular the strength method of design is mandatory for seismic design.

A comprehensive commentary is published with the code and it is strongly recommended that the two documents should be read together. This commentary is presented in some length with the aim of providing guidelines without unnecessary restriction. The appendix to Commentary Section 3 (C3.A) "A method for the evaluation of column action in multistorey frames" is a special example of this intention. This appendix is included to give designers guidance in the assessment of the maximum actions on columns resulting from capacity design considerations. Because of its developmental stage and as it is possible to use other methods, it is not a mandatory provision. At the end of several commentary sections a list of references is provided to assist designers in areas where standard design procedures have not yet been formulated.

NEW ZEALAND STANDARD

Code of practice for THE DESIGN OF CONCRETE STRUCTURES

1 GENERAL

1.1 Scope. This New Zealand Standard Code of Practice specifies minimum requirements for the design of reinforced and prestressed concrete structures. It serves as a means of compliance with the relevant requirements of NZS 1900, Chapter 9.3.

It is applicable only to structures and parts of structures complying with the materials and workmanship requirements of NZS 3109.

For special structures such as shells, arches, tanks, reservoirs, bins and silos, blast-resistant structures and chimneys, the provisions of this Standard Code of Practice shall govern where applicable.

1.2 Interpretation

1.2.1 In this Standard the word "shall" indicates a requirement that is to be adopted in order to comply with the standard, while the word "should" indicates a recommended practice.

1.2.2 Cross-references to other clauses or clause subdivisions within this Standard quote the number only, for example: " . . . as required by 4.4.1.3 (d) for shored construction."

1.2.3 The full titles of reference documents cited in this Standard are given in the "List of related documents" immediately preceding the Foreword.

1.2.4 Where any other standard named in this Standard has been declared or endorsed in terms of the Standards Act 1965, then:

(a) Reference to the named standard shall be taken to include any current amendments declared or endorsed in terms of the Standards Act 1965; or

(b) Reference to the named standard shall be read as reference to any standard currently declared or endorsed in terms of the Standards Act 1965 as superseding the named standard, including any current amendments to the superseding standard, declared or endorsed in terms of the Standards Act 1965.

NOTE – The date at which an amendment or superseding standard is regarded as "current" is a matter of law depending upon the particular method by which that standard becomes legally enforceable in the case concerned. In general, if this is by contract the relevant date is the date on which the contract is created, but if it is by Act, regulation, or bylaw then the relevant date is that on which the Act, regulation, or bylaw is promulgated.

2 DEFINITIONS

2.1 General. The following terms are defined for general use in this Code. Specialized 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. See Section 5. Also, the means by which the prestress force is permanently transferred to the concrete.

BEAM. An element subjected primarily to loads producing flexure.

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

CAPACITY DESIGN. In the capacity design of earthquake resistant structures, elements of the primary lateral load resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural elements 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 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.

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

CONCURRENCY. The occurrence of simultaneous seismic actions along both principal axes of the structure.

CONSTRUCTION JOINT. An intentional joint in concrete work detailed to ensure adequate strength and serviceability.

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

DEFORMED REINFORCEMENT. Reinforcing bars conforming to NZS 3402P.

DEVELOPMENT LENGTH. The embedded length of reinforcement required to develop the design strength of the reinforcement at a critical section (see 5.3).

DIAPHRAGM. A horizontal member composed of a web (such as floor or roof slab) or a horizontal truss which distributes horizontal forces to the vertical resisting elements.

DUCTILE FRAME. A structural frame possessing ductility (refer NZS 4203).

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; stresses remaining in prestressing tendons after all losses have occurred excluding effects of dead load and superimposed load.

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

EMBEDMENT LENGTH, EQUIVALENT. The embedded length of reinforcement which can develop the same stress in the reinforcing as that which can be developed by a hook or mechanical anchorage.

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.

ENGINEER. The Local Authority's principal Engineer who shall be registered under the Engineers Registration Act 1924 and who is the holder of a current annual practicing certificate; his deputy or assistant appointed by the Local Authority to control the erection of buildings, or the registered engineer appointed by the Highway or Railway Authority to control the erection of bridges.

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

LOAD:

LOAD, DEAD. Includes the weight of all permanent components of a structure, for example, for buildings – includes walls, partitions, columns, floors, roofs, finishes and fixed plant and fittings that are an integral part of the structure.

LOAD, DESIGN. Combinations of factored loads used in design as set out in NZS 4203 or other appropriate loadings code. In seismic design the design load may be either the factored loads or the load resulting from the capacity design procedure depending on the case being considered.

LOAD, EARTHQUAKE. Loads assumed to simulate earthquake effects as defined by NZS 4203 or other appropriate loadings code.

LOAD, FACTORED. Load, multiplied by appropriate load factors, used to proportion members by the strength design method of this Code. See 3.3.1 and 4.2.

LOAD, LIVE. The load assumed or known to result from the use of a structure as specified in NZS 4203 or other appropriate loadings code. For buildings this includes the loads on floors, loads on roofs other than wind or snow loads, on balustrades and loads from movable goods, machinery, and plant that are not an integral part of the structure.

LOAD, SERVICE. The unfactored dead and live loads. This means service loads are not necessarily the same as Alternative Method Design loads which do include load factors.

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

P-DELTA EFFECT. Implies or refers to the increase in overturning moment at any level of the structure, caused by the gravity load which is laterally displaced in the deformed structure due to seismic or wind load or other effects.

PIER. A vertical element (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 Code.

PLASTIC HINGE REGION. Regions in a member as defined in this Code where significant rotations due to inelastic strains can develop under flexural actions.

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

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

PRESTRESSED CONCRETE. Concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from loads are counteracted to a desired degree.

PRE-TENSIONING. A method of prestressing in which the tendons are tensioned before the concrete is placed.

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

SEGMENTAL MEMBER. A structural member made up of individual elements designed together to act as a monolithic unit under service loads.

SPAN LENGTH. See 3.3.3.5 or 11.1

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

STIRRUP OR TIES. Reinforcement used to resist shear and torsion in a structural member; typically bars, wires, or welded wire fabric (smooth or deformed) bent into L, U, or rectangular shapes 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 refer to closed stirrups which play a confining role in addition to acting as shear steel.

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, DEPENDABLE OR RELIABLE STRENGTH. The ideal strength multiplied by the appropriate strength reduction factor.

STRENGTH, OVER. See Overstrength.

STRENGTH, IDEAL. The ideal or nominal strength of a section of a member is calculated using the section dimensions as detailed and minimum specified material strengths.

STRENGTH, SPECIFIED COMPRESSIVE OF CONRETE. 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.

STRUCTURAL. A term used to denote an element or elements which provide resistance to forces acting on the building or bridge.

SUPPLEMENTARY CROSS TIES. Additional ties placed around stirrup ties or longitudinal bars. See Sections 6 and 10.

TENDON. Steel elements such as wire, cable, bar, rod, or strand used to impart prestress to concrete when the element is tensioned.

TIES. See Stirrups.

TRANSFER. Act of transferring stress in prestressing tendons from jacks or pre-tensioning 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.

WALL. Means a structural wall, which because of its position and shape is designed to contribute to the rigidity and strength 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.

3 GENERAL DESIGN REQUIREMENTS

3.1 Notation

β the ratio of the reduction in moment of resistance to the numerically largest moment given anywhere by the elastic analysis for that particular member covering all appropriate combinations of design load

c neutral axis depth measured from extreme compression fibre, mm

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

E earthquake loads as defined by NZS 4203

E_c modulus of elasticity of concrete, MPa

E_p earthquake loads for parts and portions, specified in NZS 4203, applied as inertia loading to the secondary elements

E_s modulus of elasticity of steel, MPa

f'_c specified compressive strength of concrete, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

ℓ_n clear span for positive moment or shear and the average adjacent clear spans for negative moment, mm

M structural material factor as defined in NZS 4203

S structural type factor as defined in NZS 4203

ν modification factor by which deformations $\triangle$ are multiplied, as specified in NZS 4203

v_c ideal shear stress provided by concrete, MPa

w_u factored load per unit length of beam or per unit area of slab

w density of concrete, kg per m^3

$\triangle$ displacement or deformation (angular or lineal) of the primary elements due to the loading E

$\triangle_p$ displacement or deformation (angular or lineal) of the secondary elements due to the loading E_p

3.2 Scope

3.2.1 In the design of reinforced and prestressed concrete structures for buildings and bridges, the methods of design, loading arrangements, assumptions for analysis, material and stiffness properties, load combinations and geometric limitations for structural systems and members shall be as specified in this Section.

3.3 General principles and requirements for analysis and design

3.3.1 *Methods of design.* Members shall be proportioned for adequate strength in accordance with the provisions of this Code using the factored loading specified in NZS 4203, or other appropriate loadings code and strength reduction factors, as specified in Section 4.

3.3.2 *Arrangement of live load for buildings*

3.3.2.1 In frame analysis for gravity loading the live load may be considered to be applied only to the floor or roof under consideration, and the far ends of the columns of continuous frames may be assumed as fixed.

3.3.2.2 In arranging gravity loads, consideration may be limited to combinations of:

(a) Design dead load on all spans with full design live load on two adjacent spans; and

(b) Design dead load on all spans with full design live load on alternate spans.

3.3.3 *Assumptions and methods of analysis*

3.3.3.1 All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified according to 3.3.3.4. The redistribution of moments permitted in 3.3.3.4 shall not be applied to the approximate moments of 3.3.3.3.

3.3.3.2 Except for prestressed concrete, approximate methods of frame analysis may be used for buildings of usual types of construction, spans, and storey heights.

3.3.3.3 In lieu of a more accurate method of frame analysis for gravity loading, provided:

(a) There are two or more spans;

(b) Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20%;

(c) Loads are uniformly distributed; and

(d) Unit live load does not exceed three times unit dead load;

the following approximate moments and shears may be used in design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction):

(1) Positive moment:

End spans

Discontinuous end unrestrained . . . . $w_u\ell_n{}^2/11$

Discontinuous end integral with support . $w_u\ell_n{}^2/14$

Interior spans . . . . $w_u\ell_n{}^2/16$

(2) Negative moment at exterior face of first interior support:

Two spans . . . . $w_u\ell_n{}^2/9$

More than two spans . . . . $w_u\ell_n{}^2/10$

(3) Negative moment at other faces of interior supports . . . . $w_u\ell_n{}^2/11$

(4) Negative moment at face of all supports for:

Slabs with spans not exceeding 3 m; and

Beams where ratio of sum of column stiffness to beam stiffness exceeds eight at each end of the span . . . . $w_u \ell_n^2/12$

(5) Negative moment at interior face of exterior support for members built integrally with support:

Where support is a spandrel beam . . . . . $w_u \ell_n^2/24$

Where support is a column . . . . $w_u \ell_n^2/16$

(6) Shear in end members at face of first interior support . . . . $1.15\ w_u \ell_n/2$

(7) Shear at face of all other supports . . . . $w_u \ell_n/2$

3.3.3.4 Redistribution of the design moments obtained by elastic analysis may be carried out for non-prestressed flexural members in accordance with all the following provisions:

(a) Equilibrium between the internal forces and the external loads must be maintained under each appropriate combination of factored loads

(b) The dependable strength after redistribution provided at any section of a member shall not be less than 70% of the moment for that section obtained from an elastic moments envelope covering all appropriate combinations of loads

(c) The elastic moment at any section in a member due to a particular combination of factored loads shall not be reduced by more than 30% of the numerically largest moment given anywhere by the elastic moments envelope for that particular member, covering all combinations of factored loads

(d) The neutral axis depth c, of a section resisting a reduced moment of resistance due to moment redistribution must not be greater than:

$$c = (0.6 - B)d \qquad \text{(Eq. 3-1)}$$

where B is the ratio of the reduction in moment of resistance to the numerically largest moment given anywhere by the elastic analysis for that particular member covering all appropriate combinations of a factored load.

3.3.3.5 For the purpose of computing moments, shears, deflections or stiffnesses the following span lengths shall be used:

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

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

(c) For beams built integrally with supports, moments at faces of support may be used for design

(d) Solid or ribbed slabs built integrally with supports, with clear spans 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.

3.3.3.6 In computing 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 in 3.3.6.2.

3.3.3.7 When 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 requirements of Section 8

(b) All concrete floor finishes may be part of required cover or total thickness for non-structural considerations.

3.3.4 *Material properties*

3.3.4.1 The modulus of elasticity E_c for concrete may be taken as $0.043w^{1.5}\sqrt{f'_c}$ (in MPa) for values of w between 1400 and 2500 kg/m^3. For normal weight concrete, E_c may be considered as $4700\sqrt{f'_c}$.

3.3.4.2 The modulus of elasticity E_s of non-prestressed steel reinforcement may be taken as 200 GPa.

3.3.4.3 The modulus of elasticity E_s of prestressing tendons shall be determined by tests or supplied by the manufacturer.

3.3.5 *Stiffness*

3.3.5.1 Computation of the relative flexural, shear and torsional stiffnesses of structural members shall be based on recognized engineering principles. Assumptions shall be consistent throughout analysis.

3.3.5.2 Effect of stiff panel zones at the intersection of deep members and haunches shall be considered both in determining bending moments and in design of members.

3.3.6 *Structural members*

3.3.6.1 In design of columns, consideration shall be given to:

(a) Resistance of axial forces from factored loads on all floors or roof and the maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Consideration shall also be given to loading conditions giving the maximum ratio of moment to axial load

(b) The effect of unbalanced floor or roof loads on both exterior and interior columns in frames or continuous construction, and eccentric loading due to other causes

(c) Computing moments in columns due to gravity loading, when the far ends of columns built integrally with the structure may be considered fixed. Moments at the faces of beams may be used for the design of columns

(d) Resistance to moments at any floor or roof level, which shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffness and conditions of restraint.

3.3.6.2 In T-beam construction, the slab and web shall be built integrally or otherwise effectively bonded together and the following requirements shall also be satisfied:

(a) The width of slab effective as a T-beam flange resisting stresses due to flexure, shall not exceed one-quarter the span length of the beam, and the effective overhanging slab width on each side of the web shall not exceed:

(1) Eight times the slab thickness, nor

(2) Half the clear distance to the next web

(b) For beams with a flange on one side only, the effective overhanging slab width considered in flexural resistance, shall not exceed:

(1) One-twelfth the span length of the beam, nor

(2) Six times the slab thickness, nor

(3) Half the clear distance to the next web

(c) Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one-half the width of web and an effective flange width not more than four times the width of web. In such beams transverse reinforcement placed perpendicularly to the beam shall be provided so as to:

(1) Carry the factored load on the overhanging slab width assumed to act as a cantilever

(2) Act as shear reinforcement when necessary to ensure flange action

(3) Be placed not further apart than five times the slab thickness, nor 450 mm.

3.4 Principles and requirements additional to 3.3 for members not designed for seismic loading

3.4.1 *Method of design.* As an alternative to 3.3.1, for non-prestressed members designed in accordance with allowable stresses under the governing design load, the "Alternative Method" provided in Appendix B may be used. For this the design load combinations specified in NZS 4203, or other appropriate loadings code must be used.

3.4.2 *Joist construction*

3.4.2.1 Joist construction consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

3.4.2.2 Ribs shall not be less than 100 mm in width; and shall have a depth of not more than three and one-half times the minimum width of rib used.

3.4.2.3 Clear spacing between ribs shall not exceed 750 mm.

3.4.2.4 Joist construction not meeting the limitations of 3.4.2.1 to 3.4.2.3 inclusive shall be designed as slabs and beams.

3.4.2.5 When permanent burned clay or concrete tile fillers or material having a unit compressive strength at least equal to that of the specified strength of concrete in the joists are used:

(a) Vertical shells of fillers in contact with the ribs may be included in strength computations for shear and negative moment. Other portions of fillers shall not be included in strength computations

(b) Slab thickness over permanent fillers shall not be less than one-twelfth the clear distance between ribs, nor less than 40 mm

(c) Width of ribs shall not be less than 80 mm and depth shall not be more than six times the minimum width of the rib

(d) In one-way joists, reinforcement normal to the ribs shall be provided in the slab as required by 5.3.32.

3.4.2.6 When removable forms or fillers not complying with 3.4.2.5 are used:

(a) Slab thickness shall not be less than one-twelfth the clear distance between ribs, nor less than 50 mm

(b) Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations if any, but not less than required by 5.3.32.

3.4.2.7 Where conduits or pipes are embedded within the slab, slab thickness shall be at least 25 mm greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not impair significantly the strength of the construction.

3.4.2.8 Shear stress carried by concrete v_c for the ribs may be taken as 10% greater than provided in Section 7. Shear strength may be increased by use of shear reinforcement or by widening the ends of the ribs.

3.5 Principles and requirements additional to 3.3 for the analysis and design of structures subjected to seismic loading

3.5.1 *Methods of design*

3.5.1.1 To provide minimum resistance for the appropriate combination of gravity and seismic loads specified by NZS 4203 or other appropriate loading code, design methods shall be used which are applicable to the structural systems as follows:

(a) Ductile structures resisting seismic loading and undergoing inelastic displacements are required to dissipate energy by ductile flexural yielding in specified localities of the structure. Ductile structures shall be subject to capacity design as defined in Section 2. Adequate ductility and hysteretic dissipation of seismic energy may be considered to have been provided for, if all primary earthquake resisting elements of such structures are designed and detailed in accordance with this Code

(b) Structures of limited ductility are assumed to have low inelastic deformation demand and are designed to resist seismic loads derived with the use of larger structural type factors, as specified in NZS 4203 or other appropriate loading code. Member strength is determined either with capacity or strength design procedures according to Section 14

(c) Elastically responding structures are not expected to develop inelastic deformations while resisting the largest seismic loads specified by NZS 4203, or other appropriate loading code. Accordingly they may be designed to conform to 3.3 and are exempt from the seismic requirements for detailing for ductility.

3.5.1.2 For structures subjected to seismic loading, the alternative method of design, given in Appendix B, shall not be used.

3.5.1.3 Wherever the requirements of a capacity design procedure apply, the maximum member actions to be expected during large inelastic deformations of a structure shall be based on the overstrength of the potential plastic hinges.

3.5.1.4 The interaction of all structural and non-structural elements which, due to seismic displacements, may affect the response of the structure or the performance of non-structural elements, shall be considered in the design of that structure.

3.5.1.5 Consequences of failure of elements that are not a part of the intended primary system for resisting seismic forces shall also be considered.

3.5.1.6 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.

3.5.1.7 Structural systems and design methods, other than those covered in this Code, may be used only if it can be shown by analysis or experiment, based on accepted engineering principles, that adequate strength, stiffness and ductility for the anticipated seismic movements have been provided for.

3.5.2 *Seismic loading*

3.5.2.1 In the derivation of the lateral seismic loading, to be considered with the appropriately factored gravity load, the structural type factor S, the structural material factor M, specified by NZS 4203 or other approved codes, shall be used. The same structural type factor S shall be substituted in all relevant equations of the additional seismic requirements of this Code.

3.5.2.2 Where modified capacity design procedures are used, the appropriate factors for member overstrength, dynamic moment and shear magnification shall be used to determine the design actions on members.

3.5.2.3 In considering the concurrency of seismic effects in two-way horizontal force resisting systems the following requirements shall be satisfied:

(a) Columns and walls, including their joints and foundations, which are part of a two-way horizontal force resisting system, shall be designed, in accordance with the requirements of NZS 4203, for concurrent effects resulting from the simultaneous yielding of all beams or diagonal braces framing into such columns or walls from all directions at the level under consideration and as appropriate at other levels

(b) When the design actions on columns, walls or foundations have been derived from capacity design procedures with appropriate magnifications for dynamic, concurrency and other extreme seismic effects, the intent of 3.5.2.3 (a) may be deemed to have been satisfied if components of such two-way framing systems are designed separately for the maximum actions so derived for each of the principal directions of the seismic loading

(c) Bridge members shall be designed for any additional forces resulting from seismic actions along both major axes of the structure concurrently, such as those due to friction or shear stiffness of devices intended to prevent horizontal movement in a direction perpendicular to that being considered.

3.5.3 *Assumptions and methods of analysis*

3.5.3.1 In determining the minimum strengths for members, designed for the maximum effects of factored static loads determined by elastic analysis, or for effects derived from dynamic analysis, as permitted by NZS 4203 or other appropriate loading code, the strength reduction factors specified in Section 4 shall be used.

3.5.3.2 Structures classified in 3.5.1.1 (a), such as ductile frames composed of beams and columns with or without shear walls, and also cantilever or coupled shear walls and bridge piers, shall be assumed to be forced into lateral deformations sufficient to create reversible plastic hinges by actions of a severe earthquake.

3.5.3.3 Whenever capacity design procedures are used to determine the strengths of members of structures classified in 3.5.1.1 (a) and (b), strength reduction factors need not be used.

3.5.3.4 In ductile structures or structures with limited ductility, that are continuous, a redistribution of moments, derived from an elastic analysis for factored gravity and the seismic load, may be made, provided:

(a) The amount of moment that is redistributed in any span of continuous beams shall not exceed 30% of the absolute maximum moment derived for that span from elastic analyses for any combination of earthquake and appropriately factored gravity loading

(b) The redistribution of beam terminal moments shall not reduce the combined end moments in any column, taken at the axis of the beam, to less than 70% of the value derived from elastic analysis for the design earthquake load only. This limitation is satisfied if the redistribution of shear forces between columns, due to the design earthquake load only, is limited to 30% reduction of the shear force derived from elastic analysis for the column affected

(c) The positive span moments for all design load combinations shall be modified in beams when terminal moments are changed, to satisfy the requirements of statics

(d) Moment redistribution shall not be used where terminal beam moments for any load combinations are based on nominal values

(e) The requirements of 3.3.3.4 are to be satisfied when the structure is subjected to gravity and wind load only

(f) Redistribution of moments due to lateral seismic load only, between cantilever or coupled shear walls, with or without ductile frames, shall not change the maximum value of the moment derived from elastic analysis for any wall by more than 30%.

3.5.4 *Material properties*

3.5.4.1 The structural material factors M, to be used together with the appropriate structural type factors shall be those specified in NZS 4203 or other appropriate loadings code for reinforced concrete and prestressed concrete.

3.5.4.2 Specified compressive strength of the concrete, f'_c, shall not be less than 20 MPa and shall not exceed 55 MPa unless the requirements of 13.5.2 are satisfied.

3.5.4.3 Specified yield strength of reinforcement, f_y, used in potential plastic hinge regions, shall not exceed 415 MPa.

3.5.4.4 Grade of reinforcement used shall be only that specified except that substitution of higher grades of reinforcement may be made with the approval of the designer.

3.5.4.5 Only deformed bars shall be used for longitudinal non-prestressed reinforcement.

3.5.4.6 Grade 275 plain round bars shall be used for transverse reinforcement, except that Grade 380 plain bars of up to one-half the diameter of the longitudinal bars may be used as transverse reinforcement, provided that such plain bars are permanently identified.

3.5.5 *Stiffness*

3.5.5.1 For the purpose of estimating periods of vibration and structural distortions, to comply with requirements of NZS 4203 or other appropriate loading code, allowances shall be made for the effects of cracking on the stiffness of various structural members.

3.5.5.2 In the estimation of stiffness or deformations of shear walls and other deep members, allowance shall be made for shear distortions, and distortions of anchorages and foundations, where appropriate.

3.5.6 *Ductile moment resisting space frames*

3.5.6.1 In fully ductile space frames where the gravity and lateral load is resisted entirely by frame action, without the contribution of shear walls, primary members and their connections shall comprise cast-in-place monolithic reinforced or prestressed concrete, except that precast members with connections formed on site may be used, provided that the energy dissipation properties of the system, in accordance with 3.5.1.1 (a), are verified by analysis or tests to the approval of the Engineer.

3.5.6.2 Taking into account the flexural overstrength of beams, the flexural strength of columns in frames with more than two storeys shall be sufficient to preclude the possibility of simultaneous plastic hinge formation in the top and bottom of all columns in any storey with the exception of the top storey of a bent.

3.5.6.3 With the exception of the top storey, the likelihood of yielding in columns, before the yielding of beams, shall be minimized in frames with more than two storeys, unless the requirements of 3.5.6.10 are satisfied.

3.5.6.4 In determining the flexural strength of columns the most adverse combination of earthquake and gravity induced axial load, consistent with the origin of column moment, shall be considered.

3.5.6.5 The design column bending moment may be reduced in a bent in which a ductile column is subject to small axial compression or to tension, provided that the corresponding reduction in shear resistance is insignificant in terms of the seismic shear resistance of the entire bent.

3.5.6.6 In evaluating the earthquake induced axial design loads on columns, the shear forces in the beams, derived from capacity design procedures, shall be considered with factored gravity loads in accordance with NZS 4203 or other appropriate loading code.

3.5.6.7 In estimating the earthquake induced axial design loads on columns, allowance may be made for the reduction of accumulated beam shear forces associated with flexural overstrength with increasing number of storeys.

3.5.6.8 The design shear forces across columns shall be based on an adverse moment gradient consistent with the development of plastic hinges at flexural overstrength in the adjoining beams or columns where plastic hinges are expected.

3.5.6.9 The ideal shear strength of columns shall not be less than 1.7 times the shear force calculated from the application of the design seismic loads of NZS 4203 or other appropriate loading code.

3.5.6.10 Interior columns of gravity load dominated ductile frames, three storeys or higher, may be designed to develop plastic hinges in any storey simultaneously at the top and the bottom ends, while beam hinges develop at or near the exterior columns only. The total dependable flexural strength of such a mechanism with respect to lateral loading shall be at least twice that which would be required by the application of the design seismic loads of NZS 4203 or other appropriate loading code. Exterior columns shall have adequate flexural reserve strength to absorb without yielding, the overstrength moments generated in adjacent beam hinges.

3.5.6.11 In single storey or two-storey structures and in the top storey of a multistorey frame, column hinge mechanisms are permitted. Where such mechanisms are used and where the adjacent beams are proportioned according to capacity design procedures so that no beam yielding can occur under the most adverse loading condition or inelastic displacements, the requirements of 3.5 with respect of such beams only, need not be satisfied.

3.5.6.12 Components of the structure, which are not intended to act as primary lateral load resisting members, may be precast, cast-in-place, composite or of any other approved system, provided that the connections to the ductile frame permit the expected inelastic deformations to occur without irreparable damage to the primary member of the frame to which they may be attached, and without any reduction below the required gravity load carrying capacity of such secondary component. Where relevant, the requirements of 3.5.14 shall be satisfied.

3.5.6.13 Provisions shall be made for *P*-delta effects in accordance with the requirements of NZS 4203 or other appropriate loading code.

3.5.7 *Ductile shear wall structures*

3.5.7.1 In ductile structures where the lateral earthquake load is resisted by a system of cantilever or coupled walls, the appropriate structural type factor *S* specified by NZS 4203 shall be used, and where applicable, allowance for the dynamic magnification of shear forces shall be made.

3.5.7.2 All walls to which lateral earthquake load is assigned shall be designed to be capable of dissipating seismic energy by flexural yielding.

3.5.7.3 Appropriately modified capacity design procedures shall be used to ensure that the ideal shear strength of walls is in excess of the shear force when flexural overstrength is reached.

3.5.7.4 When two or more cantilever walls are interconnected in the same plane at intervals by substantial ductile beams, part of the seismic energy to be dissipated shall be assigned to the coupling system. Capacity design procedures shall be used to ensure that the energy dissipation in the coupling system can be maintained at its flexural overstrength.

3.5.8 *Ductile hybrid structures*

3.5.8.1 Whenever 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. In this, attention shall be given to the likely energy dissipation capacity of each element, and the ensuing local damage in relation to the ductility demand on the element when the desired ductility for the building as a whole is attained.

3.5.9 *Ductile bridge structures*

3.5.9.1 Bridge structures shall be designed for the loading specified in the appropriate loadings code. Where the structure can be classed as "ductile" or some of its members are intended to perform as such, the detailing of appropriate members shall be in accordance with this Clause and the relevant clauses of Sections 6 and 7. Where the design loadings chosen apply to bridge structures with "limited ductility", the provisions of Section 14 apply. Where mechanical energy dissipating devices are incorporated, the provisions of 3.5.13 apply.

3.5.9.2 Primary lateral load resisting members shall be subject to capacity design procedures. The dependable flexural strength of primary energy dissipating members shall be not less than the bending moments assigned to their locations from an elastic analysis with the design loadings applied. The structure shall then be analysed as a plastic mechanism assuming all intended or potential plastic hinges to have developed their flexural overstrength, and the strength of resisting members shall be made such as to minimize the likelihood of yielding in such members under these conditions.

3.5.9.3 In determining the required flexural strength of piers, the most adverse combination of earthquake and gravity induced axial load, consistent with the plastic hinge mechanism, shall be considered.

3.5.9.4 The ideal flexural and shear strength of members resisting the moments caused by frictional forces in sliding bearings shall have a suitable margin over the moment and shear induced at the maximum likely coefficient of friction.

3.5.9.5 The ideal flexural and shear strength of members resisting the moments caused by shear forces in elastomeric bearings, where the members are not intended to form plastic hinges as part of the primary seismic energy dissipating mechanism, shall have a suitable margin over the actions corresponding to the horizontal limit displacement appropriate to the design.

3.5.9.6 In order to protect the bridge against brittle failure, reserve shear capacity shall be provided. Accordingly, the ideal shear strength in members which are part of the primary seismic dissipating mechanism, such as piers, and in the resisting members, such as shear keys, shall be not less than the forces corresponding to a plastic mechanism in which all intended plastic hinges are assumed to have developed their overstrength.

3.5.9.7 Transverse reinforcement for confinement in potential plastic hinge regions shall be not less than that required by 6.5.3.3 or 6.5.4.3 whichever is appropriate. It shall be established that the structure ductility capability is not less than the structure ductility demand appropriate to the design.

3.5.9.8 Attention to detail shall be given to minimize secondary damage during strong earthquake motions. Due allowance shall be made to accommodate the anticipated relative movements between structural components.

3.5.9.9 Positive horizontal linkage shall be provided between adjacent sections of superstructure at supports and hinges, and between superstructures and their supporting abutments. Holding-down devices shall be provided at all supports and hinges where horizontal deflection of the superstructure can cause an appreciable reduction in the gravity load reaction between superstructure and bearings.

3.5.10 *Structures with limited ductility*

3.5.10.1 In structures with limited ductility, the system as a whole or the primary lateral load resisting components are not considered to be capable of sustaining the inelastic displacements that are expected in fully ductile structures, without significant loss of strength or reduction in energy dissipating capacity. Therefore in the design of such structures, in accordance with 3.5.1.1 (b):

(a) Larger structural type factor *S* shall be used to derive the total design earthquake load to the requirements of NZS 4203. Where specified the appropriate code loading for structures of limited ductility shall also apply to bridge structures

(b) Appropriate detailing of potential plastic regions, in accordance with Section 14, shall ensure that the reduced ductility demands can be met

(c) Capacity design procedures may be used, in combination with modifications for the additional seismic design and detailing requirements of this code for fully ductile structures, as permitted by 14.3.3.1

(d) Strength design procedures in accordance with the general principles and requirements of the relevant sections of this Code may be used only in conjunction with the limitations imposed by Section 14.

3.5.11 *Elastically responding structures.* Structures which are expected to respond elastically to large earthquake motions, in accordance with 3.5.1.1 (c), are exempt from the additional seismic requirements of all relevant sections of this Code, provided that the earthquake design load used is that specified for these types of structures by NZS 4203 or other appropriate loading code.

3.5.12 *Foundations*

3.5.12.1 The concrete foundation system shall maintain its ability to support the design gravity loads while maintaining the chosen earthquake energy dissipating mechanisms in the structure.

3.5.12.2 *Elastic foundations supporting ductile superstructures.* For structures in which the entire dissipation of expected seismic energy has been assigned to the ductile superstructure, the foundations may be designed to remain elastic. The following conditions apply:

(a) The actions in the foundation structure shall be derived from the earthquake induced axial loads and corresponding moments and shear forces in columns and walls at the top of the foundation structure, at the development of flexural overstrength in the chosen energy dissipating mechanism in the superstructure consistent with the appropriately factored gravity loads, in accordance with capacity design procedures

(b) When the superstructure is of limited ductility, the design actions in the foundation structure shall be 1.8 times those resulting from the lateral design load applied to the superstructure, combined with the appropriately factored gravity loads

(c) Every component of the foundation structure shall have a minimum ideal strength to transmit these actions to the supporting soil

(d) Components of the foundation structure so designed need not meet the additional seismic design or detailing requirements of relevant sections of this Code.

3.5.12.3 *Elastic foundations supporting elastic superstructures.* Components of foundation structures designed to transmit actions elastically from elastically responding superstructure, defined in 3.5.1.1 (c), need not meet the additional seismic requirements of this Code.

3.5.12.4 *Ductile foundation structures.* The foundations of buildings, in which the dissipation of seismic energy is assigned entirely or partly to the foundation system, while the specified earthquake and factored gravity loads are maintained by the entire structural system, shall comply with:

(a) The additional principles and requirements for structures designed for seismic loading, wherever the actions that could be transmitted by the superstructure at the top of the foundations are less than those which would result from the application of lateral earthquake loading to the superstructure corresponding with $SM = 1.6$

(b) The requirements of Section 14, wherever the actions that could be transmitted by the superstructure at the top of the foundations are equal or larger than those which would result from the application of lateral earthquake loading to the superstructure corresponding with $SM = 1.6$.

3.5.12.5 *Rocking foundations.* When special studies are carried out to the satisfaction of the Engineer, structural walls may be assumed to limit the seismic loads induced in the structure by rocking with their foundations, provided that:

(a) The vertical design loads on the foundations are determined from factored gravity loads together with overstrength contributions of adjacent slabs, beams and other elements which may be yielding during the rocking of the wall system, and having regard to all accelerations induced in the superstructure during rocking

(b) The lateral design load acting simultaneously with the vertical forces, in accordance with 3.5.12.4 (a), are determined from special studies.

3.5.12.6 *Lateral forces on retaining walls and piles.* Particular attention shall be given to forces that might develop against retaining walls and piles during earthquakes.

3.5.12.7 *Uplift forces.* Uplift forces that may act on foundation pads during earthquakes, shall be considered to ensure that, when necessary, adequate flexural tension reinforcement is provided in the top of isolated footing pads or in other localities of continuous or combined footings or rafts, where under gravity load compression stresses would prevail. Such reinforcement shall not be less than 0.001 times the gross sectional area of such a pad.

3.5.13 *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:

(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 Code 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.

3.5.14 *Secondary structural elements*

3.5.14.1 Secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads 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 loading but which, by virtue of their detailed separations, are not subjected to loading 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 loadings, as for Group 1, and to loadings induced by the deformation of the primary elements.

3.5.14.2 Group 1 elements shall be detailed for separation to accommodate deformations $\nu\triangle$ and $\triangle_p$. 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) Loading E_p used in the design shall be that specified in NZS 4203

(b) Analysis may be by any rational method

(c) Detailing shall be such as to allow ductile behaviour and in accordance with the assumptions made in the analysis. Fixings for precast units shall be designed and detailed in accordance with 3.5.15.

3.5.14.3 Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis. For elements of Group 2:

(a) Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations $\nu\triangle$, specified in NZS 4203, and the assumptions of elastic behaviour

(b) Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $\nu\triangle$

(c) Inertia loading E_p shall be that specified by NZS 4203

(d) Loadings induced by the deformation of the primary elements shall be those arising from the level of deformation $\nu\triangle$, specified in NZS 4203 having due regard to the pattern and likely simultaneity of deformation

(e) Analysis may 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 amplified deformation, $\nu\triangle$, of the primary elements, as specified in NZS 4203

(f) Where elastic theory is applied in accordance with (e) for deformation corresponding to 0.5 $\nu\Delta$ or larger, the design and detailing requirements of Section 14 may be applied, but otherwise the additional seismic requirements of other sections shall apply.

3.5.15 *Fixings for precast non-structural elements*

3.5.15.1 When seismic deflection of the structure results in relative movement between a precast element and the points on the structure to which it is fixed, the fixings shall be designed to give clearance for the relative movements at these fixing points, corresponding to the seismic deflection computed by NZS 4203.

3.5.15.2 In buildings where the relative movements at the fixing points, computed in accordance with 3.5.15.1, are provided for by the capacity of the steel fittings for ductile deformation, and the relative movements do not require deflections in the fixings in excess of twice their yield deflection, the clearances required by 3.5.15.1 need not be provided.

3.5.15.3 For exterior elements and elements adjacent to any means of egress, the fixings, together with their anchorages shall be designed to deform in a ductile manner under movements exceeding the clearances required by 3.5.15.1.

4 STRENGTH AND SERVICEABILITY

4.1 Notation

A average effective area of concrete in tension around each reinforcing bar, calculated from the effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, mm²

A_g gross area of section, mm²

A_s area of non-prestressed tension reinforcement, mm²

A'_s area of compression reinforcement, mm²

d' distance from extreme compression fibre to centroid of compression reinforcement, mm

d_s distance from extreme tension fibre to centroid of tension reinforcement, mm

E_c modulus of elasticity of concrete, MPa. See 3.3.4.1

f'_c specified compressive strength of concrete, MPa

$\sqrt{f'_c}$ square root of specified compressive strength of concrete, MPa

f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa

f_r modulus of rupture of concrete, MPa

f_s steel stress at service load, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

h overall thickness of member, mm

h_1 distance from the centroid of the tension steel to the neutral axis, mm

h_2 distance from the extreme tension fibre to the neutral axis, mm

I_{cr} moment of inertia of cracked section, mm⁴

I_e effective moment of inertia for computation of deflection, mm⁴

I_g moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement, mm⁴

K_{cp} factor used in computing deflections allowing for long-time effects

ℓ span length of beam, girder or one-way slab, as defined in 3.3.3.5 (a); clear projection of cantilever, mm

ℓ_n length of clear span in long direction of two-way construction, measured face-to-face of columns in slabs without beams and face-to-face of beams or other supports in other cases, mm

ℓ_s shortest span length of bridge deck slab, mm

M_a maximum moment in member at stage for which deflection is being computed, N mm

M_{cr} cracking moment, N mm

P_e maximum design axial load in compression with given eccentricity due to gravity and seismic loading acting on the member during an earthquake

P_i ideal axial load strength at given eccentricity

P_u factored axial load at given eccentricity (not including any prestressing force) $\leqslant \phi P_i$, N

t_b distance from extreme tension fibre to the centre of the adjacent bar, mm

U required strength in accordance with appropriate design loadings code

w density of concrete, kg per m³

$w_{max.}$ maximum crack width at the surface of the member, mm

y_t distance from centroidal axis of gross section, neglecting the reinforcement, to extreme fibre in tension

α ratio of flexural stiffness of beam section to the flexural stiffness of a width of slab bounded laterally by the centreline of the adjacent panel (if any) on each side of the beam. See Section 11.

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_s ratio of length of continuous edges to total perimeter of a slab panel

γ ratio of distance between centroids of tensile and compressive reinforcement to overall depth of the member

ϕ strength reduction factor

4.2 General

4.2.1 Structures and structural members shall be designed to have dependable strengths at least equal to the required strengths calculated for the factored loads and applied forces in such combinations as are stipulated in NZS 4203 or other appropriate loadings code.

4.2.2 Members also shall meet all other requirements of this Code to ensure adequate performance at service loads in such combinations as are stipulated in the appropriate loadings code.

4.3 Strength

4.3.1 *General requirements*

4.3.1.1 The design dependable strength of a member or cross-section in terms of load moment, shear, or stress shall be taken as the ideal strength calculated in accordance with the requirements and assumptions of this Code, multiplied by a strength reduction factor, ϕ. The design dependable strength of a member or cross-section shall be equal to or greater than the required strength U resulting from the design loads of NZS 4203 or other appropriate loadings code.

4.3.1.2 The strength reduction factor ϕ shall be as follows:

(a) Flexure, with or without axial tension 0.90

(b) Axial tension 0.90

(c) Axial compression, with or without flexure: Members with spirals, hoops or special transverse reinforcement complying with 6.4.7.1 (a), 6.4.7.2 (a) or 6.5.4.3 . . . . 0.90

Other members 0.70

except that ϕ may be increased linearly to 0.9 as P_u decreases from $0.10 f'_c A_g$ to zero.

(d) Flexure in walls subjected to seismic loading and designed in accordance with 10.5 . . . . 0.90

(e) Shear and torsion 0.85

(f) Bearing on concrete 0.70

(g) Flexure in plain concrete 0.65

4.3.1.3 Development lengths specified in Section 5 already allow for understrength.

4.3.1.4 Designs shall not be based on a yield strength for reinforcing steel, f_y, in excess of 550 MPa.

4.3.2 *Additional requirements for members designed for seismic loading.* When the design moments, axial loads and shear forces for a section are derived from overstrengths of adjacent members or sections, in accordance with capacity design, a ϕ factor of unity may be used for that section.

4.4 Serviceability

4.4.1 *Deflection*

4.4.1.1 *General.* Members subject to flexure shall be designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the serviceability of the structure.

4.4.1.2 *Minimum thickness.* The minimum thickness specified in this Clause shall apply unless the computation of deflection according to 4.4.1.3 and 4.4.1.4 indicates that lesser thicknesses may be used without adverse effects:

(a) *One-way construction (non-prestressed) for buildings:* The minimum thicknesses stipulated in table 4.1 may be used in lieu of calculation of deflections for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections

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

(1) The minimum thickness of slabs or other two-way construction designed in accordance with the provisions of Section 11, and having a ratio of long to short span not exceeding 2, shall be governed by equations 4-1, 4-2 and 4-3, and the other provisions of this clause

Table 4.1
MINIMUM THICKNESSES OF NON-PRESTRESSED BEAMS OR ONE-WAY SLABS

f_y *MPa*	*Member*	*Minimum thickness,* h — *Members not supporting or attached to partitions or other construction likely to be damaged by large deflections*			
		Simply supported	*One end continuous*	*Both ends contin.*	*Cantilever*
275	Solid one-way slabs	$\ell/25$	$\ell/30$	$\ell/35$	$\ell/13$
	Beams or ribbed one-way slabs	$\ell/20$	$\ell/23$	$\ell/26$	$\ell/10$
380	Solid one-way slabs	$\ell/21$	$\ell/25$	$\ell/29$	$\ell/11$
	Beams or ribbed one-way slabs	$\ell/17$	$\ell/19$	$\ell/22$	$\ell/8$

NOTE – The values given shall be used directly for members with normal density concrete (w = 2400 kg/m^3) and Grades 275 or 380 reinforcement. For other conditions, the values shall be modified as follows:

For structural lightweight concrete having a density in the range 1450–1850 kg/m^3, the values shall be multiplied by (1.65–.0003 w) but not less than 1.09, where w is the density in kg per m^3.

For other than 275 or 380 MPa, the values for f_y = 380 MPa shall be multiplied by $(0.4 + f_y/630)$.

$$h = \frac{\ell_n (5.52 + 0.005 f_y)}{250 + 35\beta\,[\alpha_m - 0.5\,(1 - \beta_s)\,(1 + \frac{1}{\beta})]} \quad \text{(Eq. 4-1)}$$

but not less than

$$h = \frac{\ell_n (5.52 + 0.005 f_y)}{250 + 35\beta(1 + \beta_s)} \quad \text{(Eq. 4-2)}$$

and need not be more than

$$h = \frac{\ell_n (5.52 + 0.005 f_y)}{250} \quad \text{(Eq. 4-3)}$$

However, the thickness shall not be less than the following values:

For slabs without beams or drop panels . . . . 125 mm

For slabs without beams but with drop panels conforming to (2) below . . . . 100 mm

For slabs having beams on all four edges with a value of α_m at least equal to 2.0 . . . . . . 90 mm

(2) For slabs without beams, but with drop panels extending in each direction from the centre line of support a distance not less 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 4-1, 4-2 or 4-3 may be reduced by 10%

(3) At discontinuous edges, an edge beam shall be provided with a stiffness ratio α not less than 0.80; or the minimum thickness required by equations 4-1, 4-2 or 4-3, or by (2) above, shall be increased by at least 10% in the panel with a discontinuous edge

(c) *Composite construction for buildings:*
If the thickness of non-prestressed composite members meets the requirements of table 4.1, deflection need not be computed except as required by 4.4.1.3 (d) for shored construction. The portion of the member in compression shall determine whether the values in table 4.1 for normal density or lightweight concrete apply

(d) *Bridge structure members:*
The minimum thickness stipulated in table 4.2 shall apply to flexural members of bridge structures unless computation of deflection and design for the effects of traffic-induced vibration in accordance with 4.4.3 indicates that lesser thickness may be used without adverse effect.

Table 4.2
MINIMUM THICKNESSES OF CONTINUOUS PRISMATIC* FLEXURAL MEMBERS OF BRIDGE STRUCTURES

Superstructure type	*Minimum thickness (mm)*
Bridge deck slabs	$100 + \frac{\ell_s}{30}$
T-Girders	$150 + \frac{\ell}{18}$
Box-Girders	$150 + \frac{\ell}{20}$

* For non-prismatic members, that is, members with variable depth or width, the values given may be adjusted to account for change in relative stiffness of positive and negative moment sections.

4.4.1.3 *Computation of deflection*

(a) *One-way construction (non-prestressed):*

(1) *Computation of immediate deflection.* Where deflections are to be computed, the deflections that occur immediately on application of service load, in accordance with appropriate loading code requirements, shall be computed by the usual methods or formulae for elastic deflections considering effects of cracking and reinforcement on member stiffness.

Unless stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be computed with the modulus of elasticity E_c for concrete as specified in 3.3.4.1 (normal density or lightweight concrete) and with the effective moment of inertia as follows, but not greater than I_g

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

where

$$M_{cr} = \frac{f_r I_g}{y_t} \quad \text{(Eq. 4-5)}$$

and for normal density concrete

$$f_r = 0.6\sqrt{f'_c} \quad \text{(Eq. 4-6)}$$

When lightweight aggregate concrete is used, one of the following modifications shall apply:

either, when f_{ct} is specified and the concrete mix is designed in accordance with NZS 3152, f_r shall be modified by substituting $1.8 f_{ct}$ for $\sqrt{f'_c}$, but the value of $1.8 f_{ct}$ shall not exceed $\sqrt{f'_c}$;

or, when f_{ct} is not specified, f_r shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

For continuous spans, the effective moment of inertia may be taken as the average of the values obtained from eq. 4-4 for the critical positive and negative moment sections

(2) *Computation of long-time deflection.* Unless values are obtained by a more comprehensive analysis, the additional long-time deflection for flexural members (normal density and lightweight concrete) shall be obtained by multiplying the immediate deflection caused by the sustained load considered, computed in accordance with (1) above, by a factor no less than

$$K_{cp} = [2 - 1.2\,(A'_s/A_s)] > 0.6 \quad \text{......(Eq. 4-7)}$$

(b) *Two-way construction (non-prestressed):*

(1) *Computation of immediate deflection.* Deflections shall be computed, taking into account the size and shape of the panel, the conditions of support and the nature of restraints at the panel edges. For deflection computations, the modulus of elasticity, E_c, for concrete shall be as specified in 3.3.4.1. The effective moment of inertia shall be that given by eq. 4-4; other values may be used if the computed deflection is in reasonable agreement with the results of comprehensive tests

(2) *Computation of long-time deflection.* Additional long-time deflection shall be computed in accordance with 4.4.1.3 (a) (2)

(c) *Prestressed concrete construction:*

(1) *Computation of immediate deflection.* For flexural members designed in accordance with the provisions of Section 13, immediate deflection shall be computed by the usual methods or formulae for elastic deflections, and the moment of inertia of the gross concrete section may be used for uncracked sections

(2) *Computation of long-time deflection.* The additional long-time deflection of prestressed concrete members shall be computed taking into account the stresses in the concrete and the steel under the sustained load and including the effects of creep and shrinkage of the concrete and relaxation of the steel

(d) *Composite construction:*

(1) *Shored construction.* 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 comptuation 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

(2) If the thickness of a non-prestressed composite member meets the requirements of table 4.1, deflection occurring after the member becomes composite need not be computed, but the long-time deflection of the precast member shall be investigated for the magnitude and duration of load prior to the beginning of effective composite action.

4.4.1.4 *Allowable deflection.* The deflections computed in accordance with 4.4.1.3 shall not exceed the requirements of the appropriate general design code.

4.4.2 *Cracking*

4.4.2.1 *General.* Cracking of concrete under service load shall be limited so that the appearance or durability of the structure is not adversely affected, having regard to the requirements of the particular structure.

The calculation of crack widths according to 4.4.2.2 and 4.4.2.3 shall be required only where any of the following conditions apply:

(a) The environment is aggressive

(b) The specified yield strength of the reinforcing steel exceeds 275 MPa

(c) The diameter of flexural reinforcement exceeds 32 mm

(d) The design of prestressed concrete members is not based on limits on flexural tensile stresses but rather on the provisions of 13.3.2.2.

4.4.2.2 *Computation of crack widths.* The calculated maximum crack widths on the surface of members reinforced by deformed bars shall be taken as not less than

$$w_{max.} = 1.1 \sqrt[3]{t_b A}\, \frac{h_2}{h_1} f_s \times 10^{-5} \text{ mm} \quad \text{(Eq. 4-8)}$$

The crack widths at the surface of prestressed concrete members shall be calculated by suitable methods.

4.4.2.3 *Allowable crack widths.* The crack widths computed in accordance with 4.4.2.2 shall not exceed the limits specified in table 4.3.

For members incorporating a combination of significant quantities of reinforcing and prestressing steel, the allowable crack widths shall be chosen from table 4.3 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.

4.4.3 *Vibration.* Where there is a likelihood of a structure being subjected to vibration from causes such as wind forces, machinery or traffic movements, measures shall be taken to prevent discomfort or alarm to persons, damage to the structure or interference with its proper function.

4.5 Other considerations

4.5.1 *Fatigue*

4.5.1.1 The effects of fatigue shall be considered where the imposed load on a structure is frequently repetitive in nature.

4.5.1.2 At sections where frequent stress reversals occur, caused by live load plus impact at service load, the range between the maximum and minimum stress in straight reinforcement shall not exceed 150 MPa unless a special study is made. For prestressed sections refer table 13.2.

4.5.1.3 In slabs subject to frequently repetitive loads, the minimum diameter of any bends in the reinforcing steel shall be increased above the values specified in 5.3.3 to 20 bar diameters.

4.5.2 *Fire resistance.* The provisions of NZS 1900 : Chapter 5, as they apply to fire resisting concrete construction, shall be satisfied.

Table 4.3 ALLOWABLE SURFACE WIDTH OF CRACKS UNDER SERVICE LOAD

Material	*Load category*				*Type of Environment*
	I Immediately after transfer before time dependent losses	*II* Permanent loads plus variable loads of long duration; or permanent loads plus frequently repetitive loads, for example, highway bridge normal loads	*III* Specified service loads for buildings where Load Category II does not apply	*IV* Permanent loads plus infrequent combinations of transient loads, for example, highway bridge overloads	
Reinforced concrete	–	0.4 mm	0.4 mm	0.5 mm	Internal
Prestressed concrete	0.3 mm	0.2 mm	0.3 mm	0.4 mm	
Reinforced concrete	–	0.3 mm	0.3 mm	0.4 mm	External
Prestressed concrete	0.2 mm	0.1 mm	0.2 mm	0.3 mm	
Reinforced concrete	–	0.2 mm	0.2 mm	0.3 mm	Aggressive
Prestressed concrete	Zero	Zero	0.1 mm	0.2 mm	

5 REINFORCEMENT – DETAILS, ANCHORAGE AND DEVELOPMENT

5.1 Notation

A_b area of an individual bar, mm²

A_g gross area of section, mm²

A_{sp} area of flexural reinforcement provided, mm²

A_{sr} area of required flexural reinforcement, mm²

A_t area of bar formed into spiral reinforcement, mm²

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, mm² if longitudinal bars are enclosed within spiral reinforcement, A_t, mm²

A_v area of shear reinforcement within a distance s, mm²

A_w area of an individual wire to be developed or spliced, mm²

b_w web width, or diameter of circular section, mm

c the smaller of c_c or c_s, mm

c_c distance measured from extreme tension fibre to centre of bar, mm

c_s the smaller of the distance from the face of the concrete to the centre of bar measured along the line through the layer of bars, or half the centre-to-centre distance of bars in the layer, mm

For splices, c_s shall be the smaller of the distance from the concrete side face to the centre of the outside bar, or one-half the clear spacing of bars spliced at the same location plus a half bar diameter, 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_h tensile stress developed by standard hook, MPa

f_{ps} calculated stress in prestressing steel at design load, MPa

f_s steel stress, MPa

f_{se} effective stress in prestressing steel after losses, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

f_{yt} specified yield strength of transverse reinforcement, MPa

h overall thickness of member, mm

h_b beam depth, mm

h_c column depth parallel to the longitudinal beam bars being considered, mm

k_b multiplier applied to the permitted beam bar sizes through column joints in non-yielding beams

k_{tr} an index of the transverse reinforcement provided along the anchored bar, $A_{tr} f_{yt}/10s$, expressed as mm

ℓ_a additional embedment length at support or at point of inflection, mm

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

ℓ_{hb} basic development length for a hooked bar, mm

ΣM_b sum of the moments at ideal strength in non-yielding beams at opposite faces of the joint, summed in the same vector sense, and related to the centre of the intersecting column, Nmm

ΣM_c sum of the moments at ideal strength in hinging columns at opposite faces of the joint, summed in the same vector sense, and related to the centre of the intersect-beam, Nmm

M_i ideal flexural strength of section, Nmm

n number of bars in a layer

$P_{min.}$ minimum axial load on a column at its junction with a beam in which plastic hinges form, N

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, 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 d_b, mm

s_w spacing of wires to be developed or spliced, mm

V_u factored shear force at section, N

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.

5.2 Scope. Provisions of Section 5 shall apply to detailing of reinforcement, including spacing and cover, and design of anchorage, development and splices.

5.3 General principles and requirements

5.3.1 *Steel reinforcement*

5.3.1.1 All reinforcement other than ties, stirrups, spirals, 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.

5.3.1.2 Reinforcing bars shall conform to NZS 3402P, unless special design provisions are made.

5.3.2 *Hooks.* 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 not less 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; or

(c) A 135° stirrup hook, which is defined as a 135° turn around a longitudinal bar plus an extension of at least 8 stirrup bar diameters at the free end of the bar embedded in the core concrete of the member.

5.3.3 *Minimum bend diameter.* The diameter of bend, measured to the inside of the bar, shall not be less than the appropriate value given in table 5.1 or the value given by eq. 5-1 except that eq. 5-1 need not apply in the case where two transverse bars are placed in contact with the inside of the bend or where $\ell_b > \ell_d/2$. The transverse bars shall have a diameter at least as great as that of the bent bar.

Table 5.1 MINIMUM DIAMETERS OF BEND

Steel grade	*Bar dia. (mm)* d_b	*Minimum dia. of bend* d_i
275	6–28	$5d_b$
	32–40	$6d_b$
380	6–20	$8d_b$
	24–40	$10d_b$

The diameter of bend measured to the inside of the bar shall not be less than:

$$d_i \geqslant d_b\,(0.5 + \frac{d_b}{s_b})(1 - \frac{\ell_b}{\ell_d})\frac{f_y}{f'_c} \quad \ldots\ldots \quad \text{Eq. 5-1}$$

5.3.4 *Stirrup and tie bends*

5.3.4.1 Inside diameter of bends of stirrups and ties shall be equal to the diameter of the enclosed bar except that it shall be not less than the values given in table 5.2.

Table 5.2 MINIMUM DIAMETERS OF BENDS FOR STIRRUPS AND TIES

Steel grade	*Bar dia. (mm)* d_b	*Minimum diameter of bend* d_i — *Plain bars*	*Minimum diameter of bend* d_i — *Deformed bars*
275	6-24	$2d_b$	$4d_b$
380	6-20	$4d_b$	$8d_b$

where d_b is the stirrup or tie bar diameter.

5.3.4.2 Inside diameter of bends in welded wire fabric, plain or deformed, for stirrups and ties shall not be less than four wire diameters for deformed wire larger than 7 mm and two wire diameters for all other wires. Bends with inside diameter of less than eight wire diameters shall not be less than four wire diameters from the nearest welded intersection.

5.3.5 *Spacing of reinforcement*

5.3.5.1 The clear distance between parallel reinforcing bars in a layer shall be not less than the nominal diameter of the bars, nor 25 mm.

5.3.5.2 The nominal maximum size of the aggregate shall not be larger than three-fourths of the minimum clear spacing between individual reinforcing bars or bundles or pre-tensioning tendons or post-tensioning ducts.

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 not less than 25 mm nor the nominal diameter of the bars.

5.3.5.3 Groups of parallel reinforcing bars bundled in contact, assumed to act as a unit, not more than four in any one bundle, may be used only when the bundle is within the perimeter of stirrups or ties. Bars larger than 35 mm shall not be bundled in beams or girders of buildings. 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 limitation 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.

5.3.5.4 In walls and slabs other than concrete joist constructions, the principal reinforcement shall be spaced not farther apart than three times the wall or slab thickness, nor more than 450 mm, subject to the further restrictions for bridge structures in 5.3.5.5. Minimum bar spacing in basement walls shall comply with the provisions of 5.3.36.3.

5.3.5.5 In bridge decks or abutment walls the maximum spacing between adjacent bars, in both directions, in the outermost layer shall not exceed 300 mm. This requirement may be relaxed to 450 mm when either:

(a) The adjacent concrete surfaces are not exposed to direct sunlight and the bars parallel to the span of a member are always in flexural compression; or

(b) The adjacent concrete surfaces are not exposed to the weather and the reinforcement is not stressed in tension by repetitive live loads.

5.3.5.6 In spirally reinforced and tied compression members, the clear distance between longitudinal bars shall be not less than 1.5 d_b, nor 40 mm.

5.3.5.7 The limit on clear distance between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

5.3.5.8 The clear distances between pre-tensioning reinforcement at each end of the member shall be not less than 4 d_b of individual wires nor 3 d_b of strands. Closer vertical spacing and bundling of strands is permitted in the middle portion of the spans, but the requirements of 5.3.5.2 shall be satisfied.

5.3.5.9 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.

5.3.6 *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.

5.3.7 *Development length of deformed bars and deformed wire in tension*

5.3.7.1 The development length, ℓ_d, of deformed bars in tension shall be computed as the product of the basic development length, ℓ_{db}, from 5.3.7.2 and the applicable modification factor or factors in 5.3.7.3, but ℓ_d shall not be less than 300 mm.

5.3.7.2 The basic development length, ℓ_{db}, for Grade 275 reinforcement shall be computed as follows:

(a) The basic development length for any of the bars in a layer where the cover to the bars is not less than 40 mm and the centre-to-centre spacing of such bars is not less than 100 mm

$$\ell_{db} = 24 d_b \text{ when } d_b \leqslant 20 \text{ mm and } f'_c \geqslant 20 \text{ MPa} \qquad \text{Eq. 5-2}$$

$$\ell_{db} = \frac{7.6 A_b}{\sqrt{f'_c}} \text{ when } d_b > 20 \text{ mm} \qquad \text{Eq. 5-3}$$

and the contribution of transverse reinforcement as in 5.3.7.3 (d) shall not be considered

(b) When the limitations of 5.3.7.2 (a) are not satisfied in beams and columns, and several bars not larger than 40 mm diameter are used with a centre-to-centre spacing of not less than 3 d_b

$$\ell_{db} = \frac{200 d_b}{\sqrt{f'_c}} \qquad \text{Eq. 5-4}$$

or with a centre-to-centre spacing of bars not less than 80 mm

$$\ell_{db} = \frac{400}{\sqrt{f'_c}} (d_b - 11) \qquad \text{Eq. 5-5}$$

(c) When the limitations of 5.3.7.2 (a) are not satisfied, then the basic development length need not be greater than

$$\ell_{db} = \frac{380 A_b}{c \sqrt{f'_c}} \qquad \text{Eq. 5-6}$$

where c shall not be taken larger than 3 d_b

5.3.7.3 The basic development length shall be multiplied by the applicable factor or factors for:

(a) Reinforcement having yield strength other than 275 MPa . . . . $f_y/275$

(b) Top horizontal reinforcement where more than 300 mm of fresh concrete is cast in the member below the bar 1.3

(c) Reinforcement in a flexural member (not subjected to seismic loads nor required for temperature or shrinkage in restrained members) in excess of that required . . . . A_{sr}/A_{sp}

(d) Transverse reinforcement where at least three bars, transverse to the bar being developed, and outside it, are provided within ℓ_d, the factors 1.0 or $\frac{c}{c + k_{tr}}$ may be used:

where k_{tr} shall be taken not more than d_b,

and ℓ_{db} is calculated by eq. 5-6, and c and $c + k_{tr}$ shall each be taken as not more than 3 d_b.

Transverse reinforcement used for shear, flexure or temperature may be included in A_{tr}.

5.3.8 *Development length of plain bars and wire in tension.* The development length for plain bars and wire shall be twice the calculated value of ℓ_d for a deformed bar or wire but not less than 24 bar diameters.

5.3.9 *Development length of deformed bars in compression*

5.3.9.1 Development length ℓ_d for deformed bars in compression shall be computed as the product of the basic development length of 5.3.9.2 and applicable modification factors of 5.3.9.3, but ℓ_d shall not be less than 200 mm.

5.3.9.2 Basic development length in compression shall be . . . . $0.24 d_b f_y / \sqrt{f'_c}$

but not less than . . . . $0.044 d_b f_y$

5.3.9.3 Basic development length in compression may be multiplied by applicable factors for:

(a) Reinforcement in excess of that required by analysis . . . . A_{sr}/A_{sp}

(b) Reinforcement enclosed within spiral reinforcement or rectangular ties provided that at least three sets of ties or turns of spirals are present over ℓ_d and

$\frac{A_{tr}}{s} \geqslant \frac{A_b}{1000}$ 0.75

5.3.10 *Development length of plain bars in compression.* The development length, ℓ_d, for plain bars in compression shall be twice the calculated value for a deformed bar of the same diameter.

5.3.11 *Development of bundled bars.* Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20% for a three-bar bundle, and 33% for a four-bar bundle.

5.3.12 *Development of welded deformed wire fabric in tension*

5.3.12.1 Development length, ℓ_d, of welded deformed wire fabric measured from point of critical section to end of wire shall be computed as the product of the basic development length of 5.3.12.2 or 5.3.12.3 and applicable modification factor or factors of 5.3.7.3, but ℓ_d shall not be less than 200 mm except in computing lap splices by 5.3.23.

5.3.12.2 Basic development length of welded deformed wire fabric, with at least one cross wire within the development length not less than 50 mm from point of critical section, shall be

$$\ell_{db} = 0.36 d_b (f_y - 138) / \sqrt{f'_c} \qquad \text{Eq. 5-7}$$

but shall also satisfy

$$\ell_{db} \geqslant 2.40 \frac{A_w f_y}{s_w \sqrt{f'_c}} \qquad \text{Eq. 5-8}$$

5.3.12.3 Basic development length of welded deformed wire fabric, with no cross wires within the development length as required by 5.3.12.2, shall be determined as for deformed wire in tension.

5.3.13 *Development of welded smooth wire fabric in tension.* The yield strength of smooth wires of welded wire fabric shall be considered developed by embedding at least two cross wires, with the closer one at least 50 mm from point of critical section. However, development length ℓ_d measured from point of critical section to outermost cross wire shall not be less than

$$\frac{3.25 A_w f_y}{s_w \sqrt{f'_c}}$$

multiplied by A_{sr}/A_{sp} for reinforcement in excess of that required by analysis, but ℓ_d shall not be less than 150 mm except in computing lap splices by 5.3.22.

5.3.14 *Development of prestressing strand*

5.3.14.1 Three or seven-wire pre-tensioning strand shall be bonded beyond the critical section for a development length

$$\ell_d \geqslant (f_{ps} - \frac{2}{3} f_{se}) d_b / 7 \qquad \text{Eq. 5-9}$$

5.3.14.2 Investigation may be limited to the cross-sections nearest each end of the member that are required to develop full design strength under specified factored loads.

5.3.14.3 Where bonding of a strand does not extend to the end of a member, bonded development length specified in 5.3.14.1 shall be doubled.

5.3.15 *Standard hooks in tension*

5.3.15.1 The development length ℓ_{dh} of a deformed bar in tension terminating in a standard hook shall be computed as the product of the basic development length ℓ_{hb} from 5.3.15.2 and the applicable factor or factors in 5.3.15.3 but ℓ_{dh} shall not be taken less than 8 d_b or 150 mm, whichever is greater.

5.3.15.2 The basic development length for Grade 275 hooked deformed bars shall be computed by:

$$\ell_{hb} = \frac{66 d_b}{\sqrt{f'_c}} \qquad \text{Eq. 5-10}$$

5.3.15.3 The basic development length shall be multiplied by the applicable factor or factors for:

(a) Reinforcement having yield strength other than 275 MPa . . . . $f_y/275$

(b) Confinement; for 32 mm bars or smaller with side cover normal to the plane of the hooked bar not less than 60 mm and cover on the tail extension of 90° hooks not less than 40 mm 0.7

For confinement by closed stirrups or hoops at a maximum spacing of $6 d_b$ or less, where

$\frac{A_{tr}}{s} \geqslant \frac{A_b}{1000}$ 0.8

(c) Reinforcement in flexural members (not subjected to seismic loads nor required for temeprature or shrinkage in restrained members) in excess of that required . . . . A_{sr}/A_{sp}

5.3.15.4 Hooks shall not be considered effective in developing reinforcement in compression.

5.3.16 *Mechanical anchorage*

5.3.16.1 Any mechanical device capable of developing the design force in the reinforcement may be used as anchorage.

5.3.16.2 Certified test results showing adequacy of such mechanical devices shall be available to the Engineer.

5.3.17 *Splices in reinforcement – General*

5.3.17.1 Splices of reinforcement shall be made only as required or permitted on the design drawings or in specifications, or as authorized by the Engineer. Except as provided herein, all welding shall conform to NZS 4702.

5.3.17.2 Grade 380 bars to NZS 3402P shall not be welded except with the approval of the Engineer who shall be satisfied prior to approval that the welding technique and local control of conditions shall have been demonstrated by tests to produce welds that have the required mechanical and metallurgical properties.

5.3.17.3 Lap splices shall not be used for bars larger than 35 mm.

5.3.17.4 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 5.3.18 or 5.3.20 shall be increased by 20% for a three-bar bundle and 33% for a four-bar bundle.

5.3.17.5 Bars spliced by non-contact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required length of lap nor 150 mm.

5.3.17.6 Welded splices or mechanical connections satisfying the following conditions, may be used:

(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 butt welded to develop in tension 1.6 f_y or the breaking strength of the bar, whichever is smaller

(c) A mechanical connection is defined as a connection which relies on mechanical interlock with the bar deformations to develop the connection capacity. A high strength mechanical connection shall develop in tension or compression, as required, not less than 1.6 f_y, or the breaking strength of the bar, whichever is smaller. When tested in tension or compression as appropriate, the change of length at a stress of 0.7 f_y in the bar, and measured over the full length of the connection system shall be not more than twice that of an equal length of unspliced bar

(d) Welded splices or mechanical connections not meeting the requirements of 5.3.17.6 (b) or 5.3.17.6 (c) may be used in regions of low computed stress in conformance with 5.3.19.2.

5.3.18 *Lap splices of bars and wire in tension*

5.3.18.1 The minimum length for lap splices of bars in tension shall be taken equal to the development length ℓ_d in 5.3.7 for deformed bars and equal to the development length in 5.3.8 for plain bars.

5.3.18.2 Bars spliced greater than $\ell_d/2$ from another splice shall not be considered in the computation of c_s.

5.3.19 *Welded splices or mechanical connections in tension*

5.3.19.1 Welded splices or mechanical connections shall meet the requirements of 5.3.17.6 (b) or (c) or 5.3.19.2.

5.3.19.2 The requirements of 5.3.17.6 (b) and (c) may be waived when splices satisfy all the following requirements:

(a) Are staggered at least 600 mm

(b) Can develop at least twice the calculated force at the section

(c) Can develop not less than 0.7 f_y based on the total area of effective bars across the section

(d) Satisfy the change of length requirements at 0.7 f_y of 5.3.17.6 (c) except that 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.

5.3.19.3 In computing the strength developed at each section, spliced bars may be rated at the specified splice strength.

5.3.19.4 Unspliced bars cut off near the section shall be rated only at a fraction of f_y, defined by the ratio of the development length provided to ℓ_d required to develop f_y.

5.3.20 *Lap splices in compression*

5.3.20.1 The minimum length of a lap splice in compression shall be the development length in compression ℓ_d, in accordance with 5.3.9 and 5.3.10, but not less than 0.073 $f_y d_b$ for f_y of 415 MPa or less, nor $(0.13 f_y - 24) d_b$ for f_y greater than 415 MPa, nor 300 mm. When the specified concrete strength is less than 20 MPa the lap length shall be increased by one-third.

5.3.20.2 In tied compression members where at least three sets of ties are present over the length of the lap and

$$\frac{A_{tr}}{s} \geqslant \frac{A_b}{1000}$$

or where transverse reinforcement as required by 6.5.4.3(c) has been provided, 0.80 of the lap length specified in 5.3.20.1 may be used but the lap length shall be not less than 300 mm.

5.3.20.3 In spirally reinforced compression members, if at least three turns of spiral are present over the length of the lap and

$$\frac{A_{tr}}{s} \geqslant \frac{A_b}{600}$$

0.80 of the lap length specified in 5.3.20.1 may be used, but the lap length shall not be less than 300 mm.

5.3.21 *Welded splices or mechanical connections in compression.* Welded splices or mechanical connections used in compression shall meet the requirements of 5.3.19.

5.3.22 *Splices of welded smooth wire fabric in tension*

5.3.22.1 Lapped splices in regions where the area of steel, A_{sp}, provided at the splice is less than twice that required by analysis shall be so made that the overlap measured between outermost cross wires of each fabric sheet is not less 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 5.3.13.

5.3.22.2 Lapped splices in regions where the area of steel, A_{sp}, provided at the splice is at least twice that required by analysis, shall be so made that the overlap measured between outermost cross wires of each fabric sheet is not less than 1.5 ℓ_d nor 50 mm where ℓ_d is the development length for f_y as given in 5.3.13.

5.3.23 *Splices of welded deformed wire fabric in tension.* Lapped splices shall be so made that the overlap measured between outermost cross wires of each fabric sheet is not less than 50 mm. The overall lapped splice length measured between the ends of each fabric sheet shall be not less than 1.7 ℓ_d nor 200 mm, where ℓ_d is the development length as given in 5.3.12.1.

5.3.24 *Development of flexural reinforcement – General*

5.3.24.1 Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

5.3.24.2 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. Provisions of 5.3.25.3 must be satisfied.

5.3.24.3 Except at supports of simple spans and at the free end of cantilevers, tension reinforcement shall extend beyond the point at which, according to the appropriate bending moment envelope, it is:

(a) Required at full strength 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.

5.3.24.4 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 does not exceed two-thirds that permitted, including shear strength of shear reinforcement provided; or

(b) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of member. Excess stirrup area A_v shall not be less than $0.4 b_w s/f_y$. Spacing s shall not exceed $d/8\beta_b$; or

(c) For 35 mm bar and smaller, continuing reinforcement provides double the area required for flexure at the cut-off point and shear does not exceed three-fourths that permitted.

5.3.24.5 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 compression face.

5.3.25 *Development of positive moment reinforcement*

5.3.25.1 At least one-third the positive moment reinforcement in simple members and one-fourth the 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.

5.3.25.2 When a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by 5.3.25.1 shall be anchored to develop the specified yield strength f_y in tension at the face of support.

5.3.25.3 The positive tension reinforcement at simple supports and both the positive and negative tension reinforcement at points of inflection shall be limited to a diameter such that ℓ_d computed for f_y by 5.3.7 satisfies the following

$$\ell_a \geqslant \ell_d - \frac{M_i}{V_u} \quad \text{. . . . . . Eq. 5-11}$$

where ℓ_a at a support shall be the sum of the embedment length beyond the centre of support and the equivalent embedment length of any hook or mechanical anchorage provided.

ℓ_a at a point of inflection shall be limited to the effective depth of member or 12 d_b, whichever is greater.

Value of M_i/V_u may be increased 30% when the ends of reinforcement are confined by a compressive reaction.

5.3.26 *Development of negative moment reinforcement*

5.3.26.1 Negative moment reinforcement in a continuous, restrained or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks or mechanical anchorage.

5.3.26.2 Negative moment reinforcement shall have an embedment length into the span as required by 5.3.6 and 5.3.24.3.

5.3.26.3 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 of not less than 1.3 times the effective depth of the member.

5.3.27 *Special details for columns and piers*

5.3.27.1 Where longitudinal bars are offset, the slope of the inclined portion of the bar with the axis of the column shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the axis of the column. Adequate horizontal support at the offset bends shall be treated as a matter of design, and shall be provided by ties, spirals or parts of the floor construction. Ties or spirals so designed shall be placed not more than 150 mm from the point of bend. The horizontal thrust to be resisted shall be assumed as one and one-half times the horizontal component of the nominal force in the inclined portion of the bar, assumed to be stressed to f_y.

5.3.27.2 Where column faces are offset 75 mm or more, splices of vertical bars adjacent to the offset face shall be made by separate dowels lapped as required herein.

5.3.27.3 Where the design load stress in the longitudinal bars in a column calculated for any loading condition exceeds 0.5 f_y in tension, lap splices designed for full yield stress in tension, or high strength welded splices or high strength mechanical connections in accordance with 5.3.17.6 (b) and (c) shall be used.

5.3.27.4 Steel cores in composite columns shall be accurately finished to bear at end bearing splices, and positive provision shall be made for alignment of one core above another. Bearing shall be considered effective to transfer 50% of the total compressive stress in the steel core. At the column base, provision shall be made to transfer the load to the footing, in accordance with 12.3.7.

The base of the metal section shall be designed to transfer the load from the entire composite column to the footing, or it may be designed to transfer the load from the metal section only, provided it is so placed as to leave ample section of concrete for the transfer of load from the reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression of the concrete.

The steel core shall comply with NZS 3404.

5.3.28 *Connections.* At connections of principal framing elements such as beams and columns, enclosure shall be provided for splices of continuing reinforcement and for end anchorage of reinforcement terminating in such connections. Such enclosure may consist of external concrete or internal closed ties, spirals or stirrups. Joints shall be subject to rational analysis in accordance with Section 9.

5.3.29 *Spiral or circular hoop reinforcement for columns and piers*

5.3.29.1 Spiral or circular hoop reinforcement shall be of such size and so assembled to permit handling and placing without distortion from designed dimensions.

5.3.29.2 For cast-in-place construction, size of spiral or circular hoop bar shall not be less than 6 mm diameter.

5.3.29.3 Anchorage of a spiral bar at the termination of the length of spiral shall be provided by an extra one-half turn of spiral bar plus either a 135° stirrup hook or welding the spiral bar on to the previous turn to develop in tension 1.6 f_y or the breaking strength of the bar, whichever is smaller.

5.3.29.4 Spiral or circular hoop bar shall not be lap spliced.

5.3.29.5 Ends of circular hoop bar, or spiral bar within the length of the spiral, shall either be welded to develop the breaking strength of the bar, or anchorage may be provided by at least a 135° stirrup hook.

5.3.29.6 Spacing and arrangement of spiral or circular hoop reinforcement are covered in 5.4.1 and 5.5.4.

5.3.30 *Rectangular hoop and tie reinforcement for columns and piers*

5.3.30.1 Rectangular hoop or tie reinforcement shall be at least 6 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 36 mm in diameter or larger and for bundled longitudinal bars.

5.3.30.2 Rectangular hoop or tie reinforcement shall enclose all longitudinal bars.

5.3.30.3 Spacing, arrangement and anchorage of rectangular hoop and tie reinforcement are covered by 5.4.2 and 5.5.5.

5.3.31 *Stirrup and tie reinforcement in beams*

5.3.31.1 Stirrup or tie reinforcement shall satisfy the size limitations in 5.3.30.1.

5.3.31.2 Stirrup or tie reinforcement shall enclosed the longitudinal compression reinforcement in beams.

5.3.31.3 Spacing, arrangement and anchorage of rectangular stirrup and tie reinforcement in flexural members are covered by 5.4.3, 5.5.6 and 7.3.5.

5.3.32 *Shrinkage and temperature reinforcement*

5.3.32.1 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 specified yield strength in conformance with 5.3.6 or 5.3.18. Such reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0014 and in no case shall such reinforcement be placed farther apart than five times the slab thickness nor more than 450 mm in buildings, nor 300 mm in bridges.

Slabs where Grade 275 deformed bars are used . . . 0.0020
Slabs where Grade 380 bars or welded wire fabric deformed or plain, are used 0.0018
Slabs where reinforcement with yield strength exceeding 380 MPa measured at a yield strain of 0.35% is used . . . . $\frac{0.0018 \times 380}{f_y}$

5.3.32.2 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 1000 mm^2 per metre width in each direction, with bars no not farther apart than 300 mm.

5.3.33 *Concrete protection for reinforcement*

5.3.33.1 The following minimum concrete cover shall be provided for reinforcing bars, prestressing tendons, or ducts. For bar bundles, the minimum cover shall equal the equivalent diameter of the bundle but need not be more than 50 mm or the tabulated minimum, whichever is greater:

(a) *Cast-in-place concrete*

	Minimum cover, mm
Cast against and permanently exposed to earth	75
Exposed to earth or weather:	
Slabs, walls, ribs:	
24 mm bars and larger	45
20 mm bars or wire, and smaller	35
Beams and columns:	
Principal longitudinal reinforcement	50
Ties, stirrups and spirals	40
Shells and folded plate members:	
24 mm bars and larger	45
20 mm bars and smaller	35
Not exposed to weather or in contact with the ground:	
Slabs, walls, ribs:	
40 mm and larger	40
24 mm through 32 mm	30
20 mm bars or wire, and smaller	20
Beams and columns:	
Principal longitudinal reinforcement	40
Ties, stirrups and spirals	25
Shells and folded plate members:	
20 mm bars and larger	20
16 mm bars or wire, and smaller	15

(b) *Precast concrete (manufactured under plant control conditions)*

	Minimum cover, mm
Exposed to earth or weather:	
Slabs, walls, ribs:	
24 mm through 40 mm bars	40
20 mm bars or wire, and smaller	30
Beams and columns:	
Principal longitudinal reinforcement	40
Ties, stirrups and spirals	30
Shells and folded plate members	30
Not exposed to weather or in contact with the ground:	
Slabs, walls, ribs:	
40 mm and larger bars	35
24 mm through 32 mm	25
20 mm bars or wire, and smaller	15
Beams and columns:	
Principal reinforcement:	
16 mm and larger bars	35
12 mm and smaller bars	30
Ties, stirrups, and spirals:	
16 mm and larger bars	20
12 mm and smaller bars	15
Shells and folded plate members:	
Reinforcement 16 mm and smaller	15
Other reinforcement	20

5.3.33.2 Covers specified in 5.3.33.1 apply to prestressed members, subject to the following provisions:

(a) For walls, slabs and ribs exposed to earth and weather, the cover over prestressing tendons and ducts may be reduced by 5 mm

(b) In pre-tensioned prestressed concrete members the cover adjacent to the anchorage length shall not be less than 3 diameters of the respective tendons except in the case of prestressed flat slabs where it may be reduced to 2.5 diameters

(c) Covers specified in 5.3.33.1 apply to prestressed members with stresses less than or equal to the limits of 13.4.1.1 (a). When tensile stresses exceed this value for members exposed to weather, earth or aggressive environment, cover shall be increased 50%.

5.3.33.3 In conditions of aggressive environment, particular attention shall be given to providing a resistant, dense concrete and, where appropriate, the covers specified in 5.3.33.1 shall be increased or special surface protection shall be provided.

5.3.34 *Protection of exposed reinforcing bars and fittings.* Exposed reinforcing bars, inserts and plates intended for bonding with future extensions shall be protected from corrosion.

5.3.35 *Covers required for fire protection.* When NZS 1900 : Chapter 5 requires a fire-protection covering greater than the concrete protection specified in 5.3.33, such greater thickness shall be used.

5.3.36 *Wall reinforcement*

5.3.36.1 All concrete walls shall have reinforcement placed in two directions at an angle of approximately 90° which shall be continuous at all angles and through all intersections. However, bars shall not be bent round re-entrant angles unless special provisions are made for positive resistance of bursting forces at bends of bars.

5.3.36.2 The area of horizontal reinforcement of reinforced concrete walls shall be not less than 0.0025 and that of the vertical reinforcement not less than 0.0015 times the area of the wall. These values may be reduced to 0.0020 and 0.0012 respectively, if the reinforcement is not larger than 16 mm in diameter and consists of either welded wire fabric or deformed bars with a specified yield strength of 380 MPa or greater.

5.3.36.3 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. One layer consisting of not less than one-half and not more than two-thirds the total required shall be placed not less than 50 mm nor more than one-third the thickness of the wall from the exterior surface. The other layer, comprising the balance of the required reinforcement shall be placed not less than 20 mm and not more than one-third the thickness of the wall from the interior surface. Bars, if used, shall not be less than 10 mm bars, nor shall they be spaced more than 2½ times the thickness of the wall or 450 mm on centres, whichever is least. Welded wire fabric reinforcement for walls shall be in flat sheet form.

5.3.36.4 In addition to the minimum as prescribed in 5.3.36.2 there shall be not less than 2 sq mm of Grade 275 bar, or its equivalent, 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.

5.4 Principles and requirements additional to 5.3 for members not designed for seismic loading

5.4.1 *Spiral or circular hoop reinforcement for columns and piers*

5.4.1.1 Volumetric ratio and limits of spacing of spirals or circular hoops in members shall conform to 6.4.7.1.

5.4.1.2 Spiral or circular hoop reinforcement shall extend from top of footing or slab in any storey to level of lowest horizontal reinforcement in members supported above.

5.4.1.3 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.

5.4.1.4 In columns with capitals, spirals or circular hoops shall extend to a level at which the diameter or width of capital is two times that of the column.

5.4.1.5 Transverse reinforcement in composite columns shall be in accordance with 6.4.12.7.

5.4.2 *Rectangular hoop and tie reinforcement for columns and piers*

5.4.2.1 Spacing and arrangement of tie reinforcement in members shall conform to 6.4.7.2.

5.4.2.2 Tie reinforcement shall be anchored by at least a 135° stirrup hook.

5.4.2.3 Tie reinforcement shall be located vertically not more than one-half a tie spacing above the top of the footing or slab in any storey, and shall be spaced as provided herein to not more than one-half a tie spacing below the lowest horizontal reinforcement in members supported above.

5.4.2.4 Where beams or brackets frame into all sides of a column, ties may be terminated not more than 75 mm below the lowest reinforcement in such beams or brackets.

5.4.2.5 Welded wire fabric of equivalent area may be used.

5.4.2.6 Transverse reinforcement in composite columns shall be in accordance with 6.4.12.8.

5.4.3 *Stirrup and tie reinforcement in beams*

5.4.3.1 Spacing and arrangement of stirrup or tie reinforcement shall conform to 6.4.4.

5.4.3.2 Stirrup and tie reinforcement shall be anchored by at least a 90° bend around a longitudinal bar plus an extension beyond the bend of at least 8 stirrup or tie bar diameters. A 135° stirrup hook shall be used rather than a 90° bend where there is a possibility of cover concrete being lost at the strength of the member due to flexure or torsion.

5.4.3.3 Transverse reinforcement in regions of members subject to significant flexural or torsional stresses shall consist of closed stirrups or closed ties extending around the longitudinal reinforcement. Closed stirrups or closed ties may be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced or with welded splices conforming to NZS 4702 which can develop in tension the breaking strength of the bar, or 1.6 f_y, whichever is smaller.

5.4.3.4 Welded wire fabric of equivalent area may be used.

5.5 Principles and requirements additional to 5.3 for members designed for seismic loading

5.5.1 *Splices in reinforcement*

5.5.1.1 Welded splices meeting the requirements of 5.3.17.6 (a) may be used in any location. For all other splices the following restrictions apply:

(a) No portion of any splice shall be located within the beam/column joint region, or within twice the effective member depth from the critical section of a potential plastic hinge in a beam

(b) In a column the centre of the splice must be within the middle quarter of the storey height of the column unless it can be shown that plastic hinges cannot develop in the column ajdacent to the beam faces.

5.5.1.2 Tensile reinforcement in beams or columns shall not be spliced by lapping in a region of tension or reversing stress unless the region is confined by stirrup-ties so that

$$\frac{A_{tr}}{s} > \frac{8d_b}{f_{yt}}.$$

5.5.1.3 Welded splices or other mechanical connections complying with 5.3.17.6 (b) and 5.3.17.6 (c) respectively may be used. Mechanical splices shall be tested through eight full cycles of loading to a maximum stress of 0.95 f_y in the bar, and at maximum load in both tension and compression shall show a change of length, measured over the full length of the connection system, not more than 10% in excess of the extension in an equal length of unspliced bar. Splices not satisfying this stiffness requirement may 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.

5.5.2 *Development of flexural reinforcement*

5.5.2.1 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.

5.5.2.2 When longitudinal beam bars are anchored in column cores or beam stubs, the anchorage shall be deemed to commence at one-half of the relevant depth of the column or 10 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 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.

5.5.2.3 For the development length the reduction provision of 5.3.7.3 (c) and 5.3.15.3 (c) of A_{sr}/A_{sp} shall not apply.

5.5.2.4 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. Beam bars that are intended to be terminated at an interior column shall be passed right through the core of the column and terminated with a standard hook immediately outside the far side of the ties round the perimeter of the core. Top beam bars may only be bent down and bottom bars must be bent up.

5.5.2.5 The diameter of longitudinal beam bars passing through interior joints shall comply with requirement (a) below. It shall also comply with one of the following requirements (b), (c), (d) and (e):

(a) The diameter of bars in that part of the slab specified in 6.5.3.2 (e) shall not exceed $\frac{1}{5}$th of the slab thickness unless it can be shown that they can never be subjected to compression

(b) Where beams frame into opposite sides of a column the maximum diameter of the longitudinal beam bars shall not exceed $\frac{1}{25}$th of the column depth h_c for Grade 275 steel nor $\frac{1}{35}$th of the column depth h_c for Grade 380 steel

(c) Where the axial compressive load, $P_{min.}$ is greater than 0.4 $f'_c A_g$, the ratio of the maximum diameter of the longitudinal beam bar to the column depth parallel to the bar shall satisfy the following equations:

For Grade 275 steel –

$$\frac{1}{20} \geqslant \frac{d_b}{h_c} \leqslant \frac{1}{25}\left[1 + \frac{5}{4}\left(\frac{P_{min.}}{A_g f'_c} - 0.4\right)\right] \quad \text{...... Eq. 5-12}$$

For Grade 380 steel –

$$\frac{1}{25} \geqslant \frac{d_b}{h_c} \leqslant \frac{1}{35}\left[1 + 2\left(\frac{P_{min.}}{A_g f'_c} - 0.4\right)\right] \quad \text{...... Eq. 5-13}$$

(d) When it can be shown that the critical section of a plastic hinge resulting from inelastic seismic displacements is at a distance from the column face of at least the depth of the beam, or 500 mm, whichever is less, bar diameters up to $\frac{1}{20}$th of the relevant column depth of Grade 275 steel, and $\frac{1}{25}$th of the relevant column depth for Grade 380 steel respectively, may be used

(e) In beams of structures in which storey sway mechanisms are intended, in accordance with 3.5.6.11, the maximum size of beam bars passing through beam-column joints as given by 5.5.2.5 (b) and (c) may be multiplied by k_b where

$$k_b = 2 - \Sigma M_c / \Sigma M_b \geqslant 1 \quad \text{...... Eq. 5-14}$$

5.5.2.6 Where beam bars at exterior columns are terminated in beam stubs, reinforcement within the stub shall be provided where necessary to ensure that the bar strength can be developed also in compression and that bursting of the concrete at bends of the beam bars or anchorage devices, is prevented.

5.5.2.7 Where anchorage is to be provided to the requirements of 5.5.2.2, the basic development ℓ_{dh} of a deformed bar terminating in a standard hook and determined from 5.3.15.2 and 5.3.15.3, may be reduced by 20% provided that two transverse bars are placed in contact with the inside of the bend, having a diameter at least as great as that of the bent bar.

5.5.3 *Development of column reinforcement*

5.5.3.1 When columns terminate in beam to column joints or joints between columns and foundation members, 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 10 d_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 hinge adjacent to the beam face cannot occur, the development length may be considered to commence from the beam face of entry.

5.5.3.2 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. The direction of the horizontal leg of the bend must always be towards the far face of the column, except in footings.

5.5.3.3 When columns are designed to develop plastic hinges in the end regions, the maximum diameter of column bars passing through the beams shall be not greater than $\frac{1}{20}$th of the beam depth for Grade 275 steel, and $\frac{1}{25}$th of the beam depth for Grade 380 steel.

5.5.3.4 When columns are not intended to develop plastic hinges the maximum diameter of longitudinal column bars at any level shall be $\frac{1}{15}$th of the depth, h_b, of the deepest beam framing into the column at that level, for Grade 275 steel and $\frac{1}{20}$th of the depth h_b of any beam framing into the column at that level for Grade 380 steel. 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.

5.5.3.5 Longitudinal column bars extending through the joint must be extended straight through joints of the type covered by 5.5.3.3. Where longitudinal column bars in compression members at joints of the type covered by 5.5.3.4 are offset, the slope of the inclined bars with the axis of the compression member shall not exceed 1 in 6, and horizontal ties at the bend in addition to those otherwise required by 9.5.4 shall be provided to carry 1.5 times the horizontal thrust developed by the column bars at yield stress.

5.5.4 *Spiral or circular hoop reinforcement for columns and piers*

5.5.4.1 Volumetric ratio and limits on spacing of spirals or circular hoops in members shall conform to 6.5.4.3 (a), (c) and (d) and in beam-column connections to 9.5.4.3 and 9.5.6.

5.5.5 *Rectangular hoop and tie reinforcement for columns and piers*

5.5.5.1 Area and limits on spacing and arrangement of rectangular hoops with or without supplementary cross-ties in members shall conform to 6.5.4.3 (b), (c) and (d) and in beam-column joints to 9.5.4.3 and 9.5.6.

5.5.5.2 Diameter of supplementary cross-ties shall not be less than the diameter of the peripheral hoop.

5.5.5.3 In the portion of a column or pier defined by 6.5.4.3 (d) (2), hoop reinforcement shall be anchored by at least a 135° stirrup hook. Alternatively the ends of the hoop bar shall be welded to develop the breaking strength of the bar or 1.6 f_y, whichever is smaller. Each end of a supplementary cross-tie shall engage a longitudinal bar with at least a 135° stirrup hook.

5.5.5.4 In the portions of a column of pier defined by 6.5.4.3 (b), (c) and (d) (1), anchorage of hoop reinforcement and cross-ties shall be as in 5.5.5.3. Where the ends of the hoop bar are welded, the weld shall develop the breaking strength of the bar.

5.5.6 *Stirrup and tie reinforcement in beams*

5.5.6.1 Limits on spacing and arrangement of stirrups in members conform to 6.4.4, and limits on spacing and arrangement of stirrup-ties in members shall conform to 6.5.3.3.

5.5.6.2 Stirrups shall be anchored by at least a 135° stirrup hook. Alternatively, the ends of the stirrup shall be spliced by welding to develop the breaking strength of the bar or 1.6 f_y, whichever is smaller.

5.5.6.3 Stirrup-ties shall be anchored as in 5.5.6.2. Where the ends of the stirrup-tie are welded, the weld shall develop the breaking strength of the bar.

6 FLEXURE WITH OR WITHOUT AXIAL LOAD

6.1 Notation

a depth of equivalent rectangular stress block as defined in 6.3.1.7, mm
ΣA_b sum of areas of longitudinal bars, mm²
A_c area of concrete core of section measured to outside of peripheral spiral or hoop, mm²
A_g gross area of section, mm²
A_s area of non-prestressed tension reinforcement, mm²
A'_s area of non-prestressed compression reinforcement, mm²
A_{sh} total effective area of hoop bars and supplementary cross ties in direction under consideration within spacing s_h, mm²
A_{st} total area of longitudinal reinforcement, (bars or steel shapes), mm²
A_t area of structural steel shape, pipe, or tubing in a composite section, mm²
A_{te} area of one leg of stirrup-tie, mm²
A_1 loaded area, mm²
A_2 maximum area of the portion of the supporting surface that is geometrically similar to, and concentric with, the loaded area, mm²
b width of compression face of member, mm
b_c width of column section, mm
b_w width of web, mm
c distance from extreme compression fibre to neutral axis, mm
C_m a factor relating actual moment diagram to an equivalent uniform moment diagram
d distance from extreme compression fibre to centroid of tension reinforcement, mm
D dead load as defined by NZS 4203
E earthquake load as defined by NZS 4203
E_c modulus of elasticity of concrete, MPa. See 3.3.4.1
E_s modulus of elasticity of reinforcement, MPa. See 3.3.4.2
EI flexural stiffness of a member. See equations 6–8 and 6–9 for columns and piers
f'_c specified compressive strength of concrete, MPa
f_y specified yield strength of non-prestressed reinforcement, MPa
f_{yh} specified yield strength of spiral, hoop or supplementary cross tie reinforcement, MPa
f_{yt} specified yield strength of stirrup-tie reinforcement, MPa
h overall depth of member, mm
h'' dimension of concrete core of section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop, mm
I_g moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm⁴
I_{se} moment of inertia of reinforcement about centroidal axis of member cross-section, mm⁴
I_t moment of inertia of structural steel shape, pipe or tubing about centroidal axis of composite member cross-section, mm⁴
k effective length factor for a column or pier
ℓ_n clear length of member measured from face of supports, mm
ℓ_u unsupported length of a column or pier, mm
L_R reduced live load as defined by NZS 4203
M_c moment to be used for design of a column or pier
M_u design moment for a member
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
M_2 value of larger design end moment on a column or pier calculated by conventional elastic frame analysis, always positive
P_c critical load. See eq. 6–7
P_e design axial load in compression at given eccentricity due to gravity and seismic loading acting on the member during an earthquake
P_o ideal axial load compressive strength when the load is applied with zero eccentricity
P_u design axial load at given eccentricity
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
U design load
β_d ratio of maximum design dead load moment to maximum design total load moment, always positive
β_1 factor defined in 6.3.1.7
δ moment magnification factor. See 6.4.11.5 and 6.4.11.6.
ρ ratio of non-prestressed tension reinforcement $= A_s/bd$
ρ_b reinforcement ratio producing balanced strain conditions in a beam or slab. See 6.4.1.2
ρ' ratio of non-prestressed compression reinforcement $= A'_s/bd$
$\rho_{max.}$, $\rho_{min.}$ maximum and minimum values of the ratio of non-prestressed tension reinforcement computed using width of web
ρ_s ratio of volume of spiral or circular hoop reinforcement to total volume of concrete core (out-to-out of spirals or hoops)
ϕ strength reduction factor. See 4.3.1 and 4.3.2.

6.2 Scope

6.2.1 Provisions of section 6 shall apply to design of members for flexure with or without axial loads. Members subject primarily to flexure shall be designed as beams or slabs. Members subject primarily to axial load and flexure shall be designed as columns or piers.

6.3 General principles and requirements

6.3.1 *General design assumptions*

6.3.1.1 Strength design of members for flexure with or without axial loads shall be based on assumptions given in 6.3.1.2 to 6.3.1.7, and on satisfaction of applicable conditions of equilibrium and compatibility of strains.

6.3.1.2 Strain in reinforcement and concrete shall be assumed directly proportional to the distance from the neutral axis, except, for deep beams with overall depth to clear span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a non-linear distribution of strain shall be considered (see 6.3.3).

6.3.1.3 Maximum usable strain at extreme concrete compression fibre shall be assumed equal to 0.003.

6.3.1.4 Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

6.3.1.5 Tensile strength of concrete shall be neglected in flexural calculations of reinforced concrete, except when meeting requirements of table 13.1.

6.3.1.6 Relationship between concrete compressive stress distribution and concrete strain may be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

6.3.1.7 Requirements of 6.3.1.6 may be considered satisfied by an equivalent rectangular concrete stress distribution defined by the following:

(a) Concrete stress of $0.85 f'_c$ 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) Distance c from fibre of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

(c) 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 reduced continuously at the rate of 0.04 for each 5 MPa of strength in excess of 30 MPa, but β_1 shall be not taken as less than 0.65.

6.3.2 *Distribution of flexural reinforcement in beams and slabs*

6.3.2.1 This clause prescribes rules for distribution of flexural reinforcement to control flexural cracking in beams and slabs.

6.3.2.2 In two-way slabs the distribution of flexural reinforcement shall be as required by section 11.

6.3.2.3 In beams and one-way slabs the flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross-section. When required by 4.4.2.1 the crack widths at sections of maximum positive and negative moment at the service load shall be calculated and shall not exceed the limits specified in 4.4.2.2 and 4.4.2.3.

6.3.2.4 Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement may be distributed over an effective flange width as defined in 3.3.6.2, or a width equal to 1/10 the span, whichever is smaller.

6.3.2.5 If the overall depth of a beam exceeds 1 m, longitudinal reinforcement having a total area equal to at least 10% of the area of the flexural tension reinforcement shall be placed near the side faces of the web and distributed in the zone of flexural tension with a vertical spacing not more than the web width, nor 300 mm. Such reinforcement may be included in strength computations only if a strain compatibility analysis is made to determine stresses in the individual bars or wires.

6.3.3 *Deep beams*

6.3.3.1 Beams with overall depth to clear span ratios greater than 2/5 for continuous spans, or 4/5 for simple spans, shall be designed as deep beams taking into account non-linear distribution of strain and lateral buckling.

6.3.3.2 Design of deep beams for shear effects shall be in accordance with 7.3.12.

6.3.3.3 Minimum flexural tension reinforcement shall conform to the requirements for beams.

6.3.3.4 Minimum horizontal and vertical reinforcement in the side faces of deep beams shall be the greater of the requirements of 7.3.12.8 and 7.3.12.9 or 7.3.14.9.

6.3.4 *Transmission of column loads through floor systems*

6.3.4.1 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 6.3.4.2 or 6.3.4.3 or 6.3.4.4.

6.3.4.2 Concrete of strength specified for the column shall be placed in the floor about the column for an area four times the column area. Column concrete shall be well integrated into floor concrete.

6.3.4.3 Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

6.3.4.4 For columns laterally supported on four sides by beams of approximately equal depth or by slabs, strength of the column may be based on an assumed concrete strength in the column joint equal to 75% of column concrete strength plus 35% of floor concrete strength.

6.3.5 *Bearing strength*

6.3.5.1 Ideal bearing strength of concrete shall not exceed $0.85\, f'_c A_1$, except as permitted in 6.3.5.2 to 6.3.5.4.

6.3.5.2 When the supporting surface is wider on all sides than the loaded area, ideal bearing strength of the loaded area may be multiplied by $\sqrt{A_2/A_1}$, but not more than 2.

6.3.5.3 When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

6.3.5.4 Clause 6.3.5 does not apply to post-tensioning anchorages.

6.4 Principles and requirements additional to 6.3 for members not designed for seismic loading

6.4.1 *Strength calculations*

6.4.1.1 Strength design of cross-sections subject to flexure with or without axial loads shall be based on stress and strain compatibility using assumptions in 6.3.1.

6.4.1.2 Balanced strain conditions exist at a cross-section of a beam or slab when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.

6.4.1.3 Compression reinforcement in conjunction with additional tension reinforcement may be used to increase the strength of beams.

6.4.1.4 Columns and piers shall be designed for the most unfavourable combination of design moment M_u and design axial load P_u. The maximum design moment M_u shall be magnified for slenderness effects in accordance with 6.4.10.

6.4.1.5 For columns and piers the maximum design axial load in compression at a given eccentricity shall not exceed 0.85 ϕP_o for members with transverse reinforcement conforming to either 6.4.7.1 (a) or 6.4.7.2 (a) and composite members conforming to 6.4.12.7, nor 0.80 ϕP_o for members with transverse reinforcement conforming to either 6.4.7.1 (b) or 6.4.7.2 (b) and composite members conforming to 6.4.12.8, where

$$P_o = 0.85 f'_c \, (A_g - A_{st}) + f_y A_{st} \quad \ldots\ldots \quad \text{(Eq.6-1)}$$

6.4.2 *Maximum longitudinal reinforcement in beams and slabs*

6.4.2.1 For beams and slabs the ratio of longitudinal reinforcement ρ provided shall not exceed 0.75 of the ratio ρ_b that would produce balanced strain conditions for the section under flexure without axial load. For members with compression reinforcement, the portion of ρ_b equalized by compression reinforcement need not be reduced by the 0.75 factor.

6.4.3 *Minimum longitudinal reinforcement in beams and slabs*

6.4.3.1 At any positive moment section of a beam, except as provided in 6.4.3.2 and 6.4.3.3, where tension reinforcement is required by analysis, the ratio ρ shall not be less than that given by

$$\rho_{min.} = \frac{1.4}{f_y} \quad \ldots\ldots \quad \text{(Eq. 6-2)}$$

In T-beams and joists where the web is in tension, the ratio ρ shall be computed for this purpose using width of web.

6.4.3.2 Alternatively, the area of reinforcement provided at every section of a beam, for positive and negative moment, shall be at least one-third greater than that required by analysis.

6.4.3.3 For structural slabs of uniform thickness, minimum area and maximum spacing of reinforcement in the direction of the span shall be as required for shrinkage and temperature according to 5.3.32.

6.4.4 *Limits for transverse reinforcement in beams and slabs*

6.4.4.1 Stirrups or ties conforming to 6.4.4.2 and 6.4.4.3 shall be present throughout the distance of a beam or slab where longitudinal compression reinforcement is required.

6.4.4.2 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, 16 longitudinal bar diameters, or 48 stirrup or tie bar diameters.

6.4.4.3 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 with an included angle of not more than 135° and no such bar shall be further than 150 mm clear on each side along the stirrup or tie from such a laterally supported bar.

6.4.5 *Distance between lateral supports of beams*

6.4.5.1 Spacing of lateral supports for a beam shall not exceed 50 times the least width b of compression flange or face.

6.4.5.2 Effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

6.4.6 *Limits for longitudinal reinforcement in columns and piers*

6.4.6.1 Area of longitudinal reinforcement for non-composite columns or piers shall not be less than 0.008 nor more than 0.08 times gross area A_g of section.

6.4.6.2 Minimum number of longitudinal reinforcing bars in columns and piers shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement.

6.4.7 *Limits for transverse reinforcement in columns and piers*

6.4.7.1 Spiral or circular hoop reinforcement for columns and piers shall conform to 5.3.29 and 5.4.1 and shall be placed as follows:

(a) When a strength reduction factor ϕ of 0.9 is used:

(1) Volumetric ratio ρ_s shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}} \quad \ldots\ldots \quad \text{(Eq. 6-3)}$$

where f_{yh} shall not exceed 400 MPa.

(2) Centre-to-centre spacing of spirals or circular hoops along the member shall not exceed the smaller of one-fifth of the diameter of the cross-section of the member or 16 longitudinal bar diameters. Clear spacing shall not be less than 25 mm.

(b) When a strength reduction factor ϕ of 0.7, or between 0.7 and 0.9 for $P_u \leqslant 0.1 f'_c A_g$, is used:

(1) Volumetric ratio may be less than the value given by eq. 6-3

(2) Centre-to-centre spacing of spirals or circular hoops along the member shall not exceed the smaller of the diameter of the cross-section of the member, 16 bar diameters or 48 transverse bar diameters.

6.4.7.2 Hoop or tie reinforcement not of circular shape for columns and piers shall conform to 5.3.30 and 5.4.2 and shall be placed as follows:

(a) When a strength reduction factor ϕ of 0.9 is used:

(1) The total area in each of the principal directions of the cross-section within spacing s_h shall not be less than

$$A_{sh} = 0.3\, s_h h'' \left(\frac{A_g}{A_c} - 1\right) \frac{f'_c}{f_{yh}} \quad \ldots\ldots \text{(Eq. 6-4)}$$

where f_{yh} shall not exceed 400 MPa

(2) Centre-to-centre spacing of hoop sets along the member shall not exceed the smaller of one-fifth of the least lateral dimension of the cross-section or 16 longitudinal bar diameters

(3) Centre-to-centre spacing across the cross-section between cross linked bars shall not exceed 200 mm

(4) Each longitudinal bar or bundle of bars shall be laterally supported by a corner of a hoop having an included angle of more than 135° or by a cross tie

(b) When a strength reduction factor ϕ of 0.7, or between 0.7 and 0.9 for $P_u \leqslant 0.1 f'_c A_g$, is used:

(1) The total area in each of the principal directions of the cross-section within spacing s_h may be less than the value given by eq. 6-4.

(2) Centre-to-centre spacing of hoop or tie sets along the member shall not exceed the smaller of the least lateral dimension of the cross-section of the member, 16 longitudinal bar diameters, or 48 transverse bar diameters

(3) Hoops or ties shall be arranged so that every corner and alternate longitudinal bar shall be laterally supported by a corner of a hoop or tie with an included angle of not more than 135° and no such bar shall be further than 150 mm clear on each side along the hoop or tie from such a laterally supported bar.

6.4.8 *Columns and piers supporting slab systems*

6.4.8.1 Columns and piers supporting a slab system included within the scope of 11.3.5 and 11.9 shall be designed as provided in Section 6 and in accordance with the additional requirements of Section 11.

6.4.9 *Design dimensions for columns and piers*

6.4.9.1 Outer limits of the effective cross-section of an isolated column or pier with two or more interlocking spirals shall be taken at a distance outside the extreme limits of the spirals equal to the minimum concrete cover required by 5.3.33.

6.4.9.2 Outer limits of the effective cross-section of a column or pier, with spiral or circular hoop reinforcement built monolithically with a concrete wall, shall be taken either as a circle at least 40 mm outside the spiral or hoop reinforcement, or as a square or rectangle with sides at least 40 mm outside the spiral or hoop reinforcement.

6.4.9.3 In lieu of using full gross area for design, a column or pier 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. Gross area considered, required percentage of reinforcement, and design strength shall be based on that circular section.

6.4.9.4 For a column or pier with a larger cross-section than required by considerations of loading, a reduced effective area A_g not less than one-half the total area may be used to determine minimum reinforcement and design strength.

6.4.10 *Slenderness effects in columns and piers braced against sidesway*

6.4.10.1 Design of columns and piers shall be based on forces and moments determined from analysis of the structure. Such analysis shall take into account influence of axial loads and variable moments of inertia on member stiffness and fixed-end moments, effect of deflections on moments and forces, and the effects of duration of loads.

6.4.10.2 In lieu of the detailed procedure prescribed in 6.4.10.1, slenderness effects in columns and piers braced against sidesway may be evaluated in accordance with the approximate procedure presented in 6.4.11.

6.4.11 *Approximate evaluation of slenderness effects for columns and piers braced against sidesway*

6.4.11.1 Unsupported length of a column or pier shall be determined as follows:

(a) 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

(b) Where column capitals or haunches are present, unsupported length shall be measured to the lower extremity of capital or haunch in the plane considered.

6.4.11.2 Effective length factor k of a column or pier braced against sidesway shall be taken as 1.0, unless analysis shows that a lower value may be used.

6.4.11.3 Radius of gyration r may be taken equal to 0.30 times the overall dimension in the direction stability is being considered for a rectangular column or pier, and 0.25 times the diameter for circular column or pier. For other shapes r may be computed for the gross concrete section.

6.4.11.4 For columns and piers braced against sidesway

(a) If $k\ell_u/r < 34 - 12M_1/M_2$, the effects of slenderness may be neglected

(b) If $34 - 12M_1/M_2 \leqslant k\ell_u/r \leqslant 100$, the effects of slenderness may be evaluated using 6.4.11.5

(c) If $k\ell_u/r > 100$, the effects of slenderness shall be evaluated by an analysis as defined in 6.4.10.1.

6.4.11.5 Design actions including the effects of slenderness shall be determined as follows:

(a) Columns and piers shall be designed using the design axial load P_u from a conventional frame analysis and a magnified design moment M_c defined by

$$M_c = \delta M_2 \quad \text{(Eq. 6-5)}$$

where

$$\delta = \frac{C_m}{1-(P_u/\phi P_c)} \geqslant 1.0 \quad \text{(Eq. 6-6)}$$

and

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \quad \text{(Eq. 6-7)}$$

(b) In lieu of a more accurate calculation, EI in eq. 6-7 may be taken either as

$$EI = \frac{(E_c I_g/5) + E_s I_{se}}{1+\beta_d} \quad \text{(Eq. 6-8)}$$

or conservatively as

$$EI = \frac{E_c I_g/2.5}{1+\beta_d} \quad \text{(Eq. 6-9)}$$

(c) In eq. 6-6 for members braced against sidesway and without transverse loads between supports, C_m may be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \quad \text{(Eq. 6-10)}$$

but not less than 0.4. If transverse loads exist between supports C_m shall be taken as 1.0.

(d) If computations show that there is no moment at both ends of a column or pier or that computed end eccentricities are less than $(15 + 0.03h)$ mm, M_2 in eq. 6-5 shall be based on a minimum eccentricity of $(15 + 0.03\,h)$ mm about each principal axis separately. Ratio M_1/M_2 in eq. 6-10 shall be determined by either of the following:

(1) When computed end eccentricities are less than $(15 + 0.03h)$ mm, computed end moments may be used to evaluate M_1/M_2 in eq. 6-10.

(2) If computations show that there is essentially no moment at both ends of a column or pier, the ratio M_1/M_2 shall be taken equal to one.

6.4.11.6 For columns and piers subject to bending about both principal axes, moment about each axis shall be magnified by δ, computed from corresponding conditions of restraint about that axis.

6.4.12 *Composite columns and piers*

6.4.12.1 Composite columns and piers shall include all such members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars.

6.4.12.2 Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

6.4.12.3 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.

6.4.12.4 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.

6.4.12.5 For evaluation of slenderness effects, radius of gyration of a composite section shall not be greater than the value given by

$$r = \sqrt{\frac{(E_c I_g/5) + E_s I_t}{(E_c A_g/5) + E_s A_t}} \quad \text{(Eq. 6-11)}$$

For computing P_c in eq. 6-7, EI of the composite section shall not be greater than

$$EI = \frac{(E_c I_g/5) + E_s I_t}{1+\beta_d} \quad \text{(Eq. 6-12)}$$

6.4.12.6 Composite members with concrete core encased by structural steel shall conform to the following:

(a) Thickness of the steel encasement shall not be less than

$b\sqrt{\dfrac{f_y}{3E_s}}$, for each face of width b

nor

$h\sqrt{\dfrac{f_y}{8E_s}}$, for circular cross-sections of diameter h

(b) Longitudinal bars located within the encased concrete core may be considered in computing A_t and I_t.

6.4.12.7 A composite member with concrete reinforced with spirals or circular hoops around a structural steel core shall conform to the following:

(a) Specified compressive strength of concrete f'_c shall not be less than 17.5 MPa

(b) Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 350 MPa

(c) Spiral or circular hoop reinforcement shall conform to 6.4.7.1 (a)

(d) Longitudinal bars located within the spiral or circular hoop reinforcement shall not be less than 0.008 nor more than 0.08 times net area of concrete section

(e) Longitudinal bars located within the spiral or circular hoop reinforcement may be considered in computing A_t and I_t.

6.4.12.8 A composite member with laterally tied concrete around a structural steel core shall conform to the following:

(a) Specified compressive strength of concrete f'_c shall not be less than 17.5 MPa

(b) Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 350 MPa

(c) Lateral ties shall extend completely around the structural steel core

(d) Lateral ties shall be at least 16 mm diameter bars, or smaller bars with a diameter not less than 1/50 times the greater side dimension of the composite member, but not smaller than 10 mm diameter. Welded wire fabric of equivalent area may be used

(e) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or one half times the least side dimension of the composite member

(f) Longitudinal bars located within the ties shall not be less than 0.008 nor more than 0.08 times the net area of concrete section

(g) A longitudinal bar shall be located at every corner of a rectangular cross-section, with other longitudinal bars spaced not further apart than one-half the least side dimension of the composite member

(h) Longitudinal bars located within the ties may be considered in computing A_t for strength but not in computing I_t for the evaluation of slenderness effects.

6.5 Principles and requirements additional to 6.3 for members designed for seismic loading

6.5.1 *Strength calculations*

6.5.1.1 Strength design of cross-sections subjected to flexure with or without axial loads shall be based on stress and strain compatibility using the assumptions in 6.3.1.

6.5.1.2 Full member cross-section shall be considered to contribute to the strength of the cross-section.

6.5.1.3 Columns and piers shall be designed for the most unfavourable combination of design moment M_u and design axial load P_u or P_e.

6.5.1.4 For frames where sidesway mechanisms with plastic hinges forming only in columns are not permitted by NZS 4203, the design moments and axial loads on columns shall include the effect of possible beam overstrength, concurrent seismic loading, and magnification of column moments due to dynamic effects, in order to provide a high degree of protection against plastic hinging of the columns.

6.5.1.5 For columns and piers the maximum design axial load in compression at a given eccentricity P_e shall not exceed $0.7\,\phi f'_c A_g$, unless it can be shown that P_e is less than $0.7\,\phi P_o$

where

$$P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \quad \text{(Eq. 6-13)}$$

6.5.2 *Dimensions*

6.5.2.1 Depth, width and clear length between the faces of supports of members with rectangular cross-section, to which moments are applied at both ends by adjacent beams, columns, or both, shall be such that

$$\frac{\ell_n}{b_w} \leqslant 25 \quad \text{(Eq. 6-14)}$$

and

$$\frac{\ell_n h}{b_w^2} \leqslant 100 \quad \text{(Eq. 6-15)}$$

6.5.2.2 Depth, width and clear length from the face of support of cantilever members with rectangular cross-section, excluding bridge piers, shall be such that

$$\frac{\ell_n}{b_w} \leqslant 15 \quad \text{(Eq. 6-16)}$$

and

$$\frac{\ell_n h}{b_w^2} \leqslant 60 \quad \text{(Eq. 6-17)}$$

6.5.2.3 Width of web and depth 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 6-14 and 6.16 are not exceeded by more than 50%.

6.5.2.4 The width of the compression face of a member with rectangular, T, L or I section shall not be less than 200 mm.

6.5.2.5 When narrow beams frame into wide columns only a limited width of a column shall be assumed to resist load transmitted by such narrow beams in accordance with 9.5.7.

6.5.3 *Reinforcement in beams*

6.5.3.1 Potential plastic hinge regions in beams shall be considered to be 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 midspan, at each end of the beam where a plastic hinge may develop

(b) Where the critical section is located at a distance not less than 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 0.5 h or 250 mm from the critical section, and extends at least 1.5 h past the critical section toward midspan

(c) Where, within the span, yielding of flexural tension steel may occur in one face of the beam only as a result of inelastic displacements of the frame: over the lengths equal to the beam depth on both sides of the critical section.

6.5.3.2 Longitudinal reinforcement in beams shall be as follows:

(a) At any section of a beam within a potential plastic hinge region, as defined in 6.5.3.1, the tension reinforcement ratio shall not exceed the smaller of

$$\rho_{max.} = \frac{1 + 0.17\,(\frac{f'_c}{7} - 3)}{100}\,(1 + \frac{\rho'}{\rho})\ . . . \text{(Eq. 6-18)}$$

or

$$\rho_{max.} = \frac{7}{f_y}\ . \ . \ . \ . \ . \ . \ \text{(Eq. 6-19)}$$

where the reinforcement ratio shall be computed using the width of the web

(b) At any section of a beam within a potential plastic hinge region, as defined in 6.5.3.1, the compression reinforcement area A'_s shall not be less than one half of the tension reinforcement area A_s at the same section

(c) At any section of a beam throughout its length the reinforcement shall not be less than

$$\rho_{min.} = \frac{1.4}{f_y}\ . \ . \ . \ . \ . \ . \ . \ \text{(Eq. 6-20)}$$

where the reinforcement ratio shall be computed using the width of the web. At least two 16 mm diameter bars shall be provided both top and bottom throughout the length of the beam

(d) 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

(e) In T and L beams built integrally with slabs, the slab longitudinal reinforcement to be considered effective as beam reinforcement, in addition to those bars placed within the web width of the beam, shall be as follows:

(1) At interior columns where a transverse beam of similar dimensions frames into the column, all reinforcement within that part of the slab which extends a distance of four times the slab thickness from each side of the column

(2) At interior columns where no transverse beam exists, all reinforcement within that part of the slab which extends a distance of two and a half times the thickness of the slab from each side of the column

(3) At exterior columns where a transverse beam of similar dimensions frames into the column, and where the beam reinforcement is to be anchored, all reinforcement within that part of the slab which extends a distance of twice the slab thickness from each side of the column

(4) At exterior columns where no transverse beam exists, all reinforcement within the width of the column

(5) In all cases, at least 75% of the longitudinal reinforcement in each face, providing the required flexural strength, must pass through or be anchored in the column core. When longitudinal reinforcement is governed by the load combination $U = 1.4\,D + 1.7\,L_R$ then only 75% of the reinforcement required for the load combination $U = D + 1.3\,L_R + E$ is required to pass through or be anchored in the column core.

6.5.3.3 Transverse reinforcement in the form of stirrup-ties should be placed in potential plastic hinge regions, as defined in 6.5.3.1 in beams 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) Diameter of the stirrup-ties shall not be less than 6 mm, and area of one leg of a stirrup-tie in the direction of potential buckling of the longitudinal bar, shall not be less than

$$A_{te} = \frac{\Sigma A_b f_y}{16 f_{yt}} \cdot \frac{s}{100} \quad \text{. . . . . . . (Eq. 6-21)}$$

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 6.5.3.3 (a). 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 layer of longitudinal bars is centred further than 100 mm from the inner face of stirrup-ties, the outermost bars shall be tied laterally as required in 6.5.3.3 (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 6.5.3.1 (a) and (b) the centre-to-centre spacing of stirrup-ties shall not exceed the smaller of $d/4$, or six times the diameter of any longitudinal bar to be restrained in the outer layers, or 150 mm. Where 6.5.3.1 (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 6.5.3.1 (c) the centre to centre spacing of stirrup-ties shall not exceed either $d/3$ or 12 times the diameter of any longitudinal compression bar to be restrained, and the clear spacing between sets of stirrup-ties shall not exceed 200 mm

(f) Stirrup-ties may be assumed to contribute fully to the shear strength of the beam.

6.5.4 *Reinforcement in columns and piers*

6.5.4.1 Potential plastic hinge regions in columns and piers shall be considered to be the end regions adjacent to moment resisting connections over a length from the face of the connection as follows:

(a) When $P_e \leqslant 0.3\,\phi f'_c A_g$, not less than the larger of the longer cross-section dimension in the case of a rectangular cross-section or the diameter in the case of a circular cross-section, or where the moment exceeds 0.8 of the maximum moment at that end of the member

(b) When $P_e > 0.3\,\phi f'_c A_g$, not less than the larger of 1.5 times the longer member cross-section dimension in the case of a rectangular cross-section or 1.5 times the diameter in the case of a circular cross-section, or where the moment exceeds 0.7 of the maximum moment at that end of the member.

6.5.4.2 Longitudinal reinforcement in columns and piers shall be as follows:

(a) Area of longitudinal reinforcement shall be not less than $0.008\,A_g$

(b) Area of longitudinal reinforcement shall be not greater than $0.06A_g$ for Grade 275 steel nor greater than 0.045 for Grade 380 steel, except that in the region of lap splices the total area shall not exceed $0.08\,A_g$ for Grade 275 steel nor $0.06\,A_g$ for Grade 380 steel

(c) For longitudinal bars in potential plastic hinge regions, as defined in 6.5.4.1, the bars shall not be spaced further apart between centres than 200 mm. In any row of bars the smallest bar diameter used shall not be less than two-thirds of the largest bar diameter used. The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement.

6.5.4.3 Transverse reinforcement in columns and piers shall conform to 5.5.4 and 5.5.5 and shall be placed as follows:

(a) In potential plastic hinge regions, as defined in 6.5.4.1, when spirals or circular hoops are used:

(1) Volumetric ratio ρ_s shall not be less than the greater of:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\,\frac{P_e}{\phi f'_c A_g}\right) \quad \text{(Eq. 6-22)}$$

or

$$\rho_s = 0.12\,\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\,\frac{P_e}{\phi f'_c A_g}\right) \quad \text{(Eq. 6-23)}$$

except that, where permitted by 6.5.4.3 (c), ρ_s may be reduced by one-half

(2) In equations 6-22 and 6-23, P_e shall not exceed the value permitted in 6.5.1.5 and f_{yh} shall not exceed 400 MPa

(3) Centre-to-centre spacing of spirals or circular hoops along the member shall not exceed the smaller of one-fifth of the diameter of the cross-section of the member, or six times the diameter of the longitudinal bar to be restrained, or 200 mm

(b) In potential plastic hinge regions, as defined in 6.5.4.1, when rectangular hoops with or without supplementary cross-ties are used:

(1) Total area of hoop bars and supplementary cross-ties in each of the principal directions of the cross-section within spacing s_h shall not be less than the greater of

$$A_{sh} = 0.3\,s_h h''\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\,\frac{P_e}{\phi f'_c A_g}\right) \quad \text{(Eq. 6-24)}$$

or

$$A_{sh} = 0.12\, s_h h'' \frac{f'_c}{f_{yh}} \left(0.5 + 1.25 \frac{P_e}{\phi f'_c A_g}\right)$$

(Eq. 6-25)

except that where permitted by 6.5.4.3 (c) A_{sh} may be reduced by one-half

(2) In equations 6-24 and 6-25, P_e shall not exceed the value permitted in 6.5.1.5 and f_{yh} shall not exceed 400 MPa

(3) Centre-to-centre spacing of hoop sets along the member shall not exceed the smaller of one-fifth of the least lateral dimension of the cross-section, or six times the diameter of the longitudinal bar to be restrained, or 200 mm

(4) Centre-to-centre spacing across the cross-section between cross linked bars shall not exceed 200 mm

(5) 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 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 200 mm between centres

(ii) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inner face of hoops

(6) Yield force in the hoop bar or supplementary cross-tie at the specified yield strength shall be at least equal to one-sixteenth of the yield force in the bar or bars it is to restrain, including the contribution from the tributary area of any bar or bars exempted under 6.5.4.3 (b) (5)

(c) 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 potential plastic hinge regions in 6.5.4.1 may be one-half of that required by equations 6-22 and 6-23 or equations 6-24 and 6-25, but all other provisions of 6.5.4 shall be conformed to. This reduction in the quantity of transverse steel in potential plastic hinge regions shall not be permitted at the base of columns nor in any storey in which a column sidesway mechanism could occur with plastic hinges forming in the columns.

(d) Outside potential plastic hinge regions of a column or pier the transverse reinforcement shall be as follows:

(1) Over the length of column or pier adjacent to the potential plastic hinge region and equal in length to the potential plastic hinge region:

(i) Centre-to-centre spacing of transverse reinforcement along the member shall not exceed the smaller of two-fifths of the diameter in the case of a circular cross-section or two-fifths of the least lateral dimension in the case of a rectangular cross-section, or 12 times the diameter of the longitudinal bar to be restrained, or 400 mm

(ii) The quantity of transverse reinforcement shall not be less than one-half of that required in the potential plastic hinge region

(2) Over the remainder of the column or pier the centre-to-centre spacing of transverse reinforcement shall not exceed 0.5 d

(e) Spirals, hoops and ties placed according to 6.5.4.3 extending around longitudinal bars in the compression and tension faces of the member cross-section may be assumed to fully contribute to the shear strength of the member.

7 SHEAR AND TORSION

7.1 Notation

a shear span, distance between concentrated load and face of support or as defined in 7.3.12.3, mm

A_{co} area enclosed by perimeter of section, mm^2 (see 7.3.7)

A_{ct} area of reinforcement to resist a tensile force in corbels, mm^2

A_f area of flexural tension reinforcement required in corbels, mm^2

A_g gross area of section, mm^2

A_h area of shear reinforcement in brackets and corbels parallel to flexural tension reinforcement, mm^2

A_ℓ total area of longitudinal reinforcement to resist torsion, mm^2

A_o area enclosed by line connecting the centres of the longitudinal bars in the corners of the closed stirrips, mm^2 (see 7.3.7)

A_{ps} area of prestressed reinforcement in tension zone, mm^2

A_s area of non-prestressed tension reinforcement, mm^2

A_t area of one leg of a closed stirrup resisting torsion within a distance s, mm^2

A_v area of shear reinforcement within a distance s, or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s for deep beams, mm^2

A_{vf} area of shear-friction reinforcement, mm^2

A_{vh} area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, mm^2

b width of compression face of member, mm

b_o perimeter of critical section for slabs and footings, mm

b_w web width, or diameter of circular section, 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 longitudinal tension reinforcement, but need not be less than 0.8 h for prestressed members or 0.8 ℓ_w for walls, mm. (For circular sections, d need not be less than the distance from extreme compression fibre to centroid of tension reinforcement in opposite half of member.)

d_c distance from extreme compression fibre to centroid of prestressed reinforcement, mm

d_s distance from centroid of total vertical reinforcement to extreme compression fibre of wall section, mm

f'_c specified compressive strength of concrete, MPa

$\sqrt{f'_c}$ square root of specified compressive strength of concrete, MPa

f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa

f_{pc} compressive stress in concrete (after allowance for all prestress losses) at centroid of cross-section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, MPa. (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone), MPa

f_{pe} compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fibre of section where tensile stress is caused by externally applied loads, MPa

f_{pu} ultimate tensile strength of prestressing tendons, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

f_{yh} specified yield strength of horizontal non-prestressed shear reinforcement in a wall, MPa

f_{yn} specified yield strength of vertical non-prestressed wall reinforcement, MPa

h overall thickness of member, mm

h_v total depth of shearhead cross-section, mm

h_w total height of wall from base to top, mm

I_x moment of inertia of section resisting externally applied factored loads, mm^4

ℓ_s distance between point of zero shear and the support of a deep beam and as specified in 7.3.12.4, mm

ℓ_v length of shearhead arm from centroid of concentrated load or reaction, mm

ℓ_w horizontal length of wall, mm

M_o moment which causes zero stress at extreme fibre at which tensile stress is induced by applied load as defined by eq. 7.10 , Nmm

M_p required plastic moment strength of shearhead cross-section, Nmm

M_u factored moment at section, Nmm

M_{uv} factored moment occurring simultaneously at a section with V_u, Nmm

M_v moment resistance contributed by shearhead reinforcement, Nmm

N_u factored tensile force on bracket or corbel acting simultaneously with V_u, to be taken positive for tension, Nmm

p_c perimeter of section, mm

p_o perimeter of area, A_o, mm

P_e design axial load in compression with given eccentricity due to gravity and seismic loading, acting on the member simultaneously with shear stress v_i during an earthquake, N

P_u factored axial load normal to cross-section occurring simultaneously with V_u to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage, N

r the algebraic ratio at the plastic hinge section of the maximum shear force developed with positive moment hinging to the maximum shear force developed with negative moment hinging, always taken negative

s spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, mm

s_1 spacing of vertical reinforcement in wall, mm

s_2 spacing of shear reinforcement in direction perpendicular to longitudinal reinforcement or spacing of horizontal reinforcement in wall, mm

S structural type factor as defined by NZS 4203

t_c $0.75\, A_{co}/p_c$, mm

t_o $0.75\, A_o/p_o$, mm

T_i ideal torsional strength of section, N mm

T_u factored torsional moment at section, N mm

v_b basic shear stress, MPa

v_c ideal shear stress provided by concrete, MPa

v_{ci} ideal shear stress provided by concrete when diagonal cracking results from combined shear and moment, MPa

v_{cw} ideal shear stress provided by concrete when diagonal cracking results from excessive principal tensile stress in web, MPa

v_i total shear stress, MPa

v_{in} shear stress sustained in members with variable depth defined by eq. 7.7, MPa

v_{ti} torsional shear stress, MPa

V_{di} design shear force to be resisted by diagonal shear reinforcement, N

V_i ideal shear strength of section, N

V_p transverse component of effective longitudinal prestress force at section, N

V_u factored shear force at section, N

y_t distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension, mm

α angle between inclined stirrups and longitudinal axis of member

α_v ratio of stiffness of shearhead arm to surrounding composite slab section. (See 7.3.15.4)

β for reinforced concrete the angle the tension reinforcement makes with the plane of the extreme compression fibre. For prestressed members the angle the prestressing tendons make with the plane of the beam centroid

β_c ratio of short side to long side of concentrated load or reaction area

μ coefficient of friction. (See 7.3.11.5)

ρ_h ratio of horizontal shear reinforcement area to gross concrete area of vertical section

ρ_n ratio of total vertical reinforcement area in wall section to gross area of horizontal section of the web only

ρ_w $(A_s + A_{ps})/b_w d$

ϕ strength reduction factor. (See 4.3.1.2)

ϕ_o ratio of overstrength moment of resistance to moment resulting from code specified loading, where both moments refer to the base section of wall.

7.2 Scope. Provisions of this Section 7 apply to design of members for shear and torsion with flexure and with or without axial load. Special provisions are made for deep beams, brackets and corbels, walls, slabs and footings and also for members subjected to earthquake loading.

7.3 General principles and requirements

7.3.1 *Shear strength*

7.3.1.1 Design of cross-sections of members subject to shear shall be based on

$$V_u < \phi V_i \qquad \text{(Eq. 7-1)}$$

where V_u is the shear force at the section derived from factored load on the structure and V_i is the ideal shear strength of the section computed with the total shear stress v_i from

$$V_i = v_i b_w d \qquad \text{(Eq. 7-2)}$$

7.3.1.2 The total shear stress v_i shall be assumed to consist of the contribution of the concrete v_c in accordance with 7.3.2 or 7.3.3 and the contribution of the shear reinforcement $(v_i - v_c)$ in accordance with 7.3.6.

7.3.1.3 In determining shear stress v_c whenever applicable, effects of axial tension due to creep and shrinkage and those of differential temperature shall be considered.

7.3.1.4 When the reaction, in direction of applied shear, introduces compression into the end regions of simply supported or continuous or cantilever members, other than brackets and corbels, calculation of maximum factored shear force V_u shall be as follows:

(a) For non-prestressed members, sections located less than a distance d from face of support may be designed for the same shear V_u as that computed at a distance d

(b) For prestressed members, sections located less than a distance $h/2$ from face of support may be designed for the same shear V_u as that computed at a distance $h/2$.

7.3.1.5 When v_i exceeds v_c, shear reinforcement shall be provided according to 7.3.6.

7.3.1.6 For deep beams, brackets and corbels, walls, and slabs and footings, the special provisions of 7.3.12 to 7.3.15 shall apply.

7.3.1.7 Provisions for shear stress v_c apply to normal weight concrete. When lightweight aggregate concrete is used one of the following modifications shall apply:

(a) When f_{ct} is specified and the concrete mix is designed in accordance with NZS 3152, provisions for v_c shall be modified by substituting $1.8\,f_{ct}$ for $\sqrt{f'_c}$ but the value of $1.8\,f_{ct}$ shall not exceed $\sqrt{f'_c}$

(b) When 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 may be used when partial sand replacement is used.

7.3.1.8 The total shear stress v_i shall not be taken larger than $0.2\,f'_c$ or 6 MPa.

7.3.2 *Shear strength provided by concrete for non-prestressed members*

7.3.2.1 Shear stress v_c for all members except slabs and deep flexural members shall be computed from the basic shear stress

$$v_b = (0.07 + 10\,\rho_w)\sqrt{f'_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-3)}$$

the value of which shall not be more than $0.2\sqrt{f'_c}$, nor need it be taken less than $0.08\sqrt{f'_c}$, as follows:

(a) For members subjected to shear and flexure only

$$v_c = v_b + v_{in} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-4)}$$

(b) For members subjected also to axial compression load v_b may be replaced by

$$\left(1 + \frac{3\,P_u}{A_g f'_c}\right) v_b \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-5)}$$

(c) For members subjected to axial tension load v_b shall be replaced by

$$\left(1 + \frac{12\,P_u}{A_g f'_c}\right) v_b \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-6)}$$

where P_u is negative for tension

(d) In members with varying depth not subjected to axial load the shear stress sustained by the internal forces induced by flexure shall be taken as

$$v_{in} = \pm\frac{M_{uv}}{b_w d^2}\tan\beta \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-7)}$$

unless more detailed analysis is carried out. The positive sign applies when the bending moment M_{uv} increases numerically in the same direction in which the depth d increases and the negative applies where the moment decreases in this direction.

7.3.2.2 In one way slabs or other elements having a width of four times the effective depth or greater and a total depth of 300 mm or less, the shear stress v_c is given by eq. 7-4 but need not be taken less than $0.17\sqrt{f'_c}$. For one-way slabs or elements having width between three and four times the effective depth and effective depth less than 300 mm, the value of v_b shall be interpolated between the value given by eq. 7-3 and the value for slab with width four times its effective depth.

7.3.3 *Shear strength provided by concrete for prestressed members*

7.3.3.1 Irrespective of the shear provided by concrete, the total shear stress in prestressed members shall not exceed $0.2\,f'_c$ or 6 MPa unless special studies are performed. If the web contains bars, cables or grouted ducts of diameter greater than $b_w/8$, the value of b_w taken for the purpose of assessing total shear stress shall be reduced by one half of the sum of the diameters of all such bars, cables or grouted ducts that are present at the most unfavourable level of the web.

7.3.3.2 For members with effective prestress force not less than 40% of the design tensile strength of flexural reinforcement, unless a more detailed calculation is made in accordance with 7.3.3.3:

$$v_c = 0.05\sqrt{f'_c} + 5\,\frac{V_u d_c}{M_{uv}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-8)}$$

but v_c need not be taken less than $0.08\sqrt{f'_c}$ nor shall v_c be taken greater than the value of v_{cw} given by eq. 7.11 nor the value given in 7.3.3.4. The quantity $V_u d_c/M_{uv}$ shall not be taken greater than 1.0, where M_{uv} is factored moment occuring simultaneously with V_u at section considered.

7.3.3.3 Shear stress v_c may be taken as the lesser of v_{ci} or v_{cw} given as follows:

(a) Shear stress provided by concrete when diagonal cracking results from combined shear and moment, v_{ci}, is the basic shear stress v_b plus the shear stress sustained under that Moment M_o which causes zero stress at the extreme fibre at which tensile stresses are induced by applied loads. For non-composite uniformly loaded beams:

$$v_{ci} = v_b + \frac{V_u M_o}{b_w d_c M_u} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-9)}$$

where

$$M_o = \frac{I_x}{y_t}\,f_{pe} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. 7-10)}$$

(b) Shear stress provided by the concrete when diagonal cracking results from excessive principal tensile stress in web v_{cw} shall be computed by

$$v_{cw} = 0.3\left(\sqrt{f'_c} + f_{pc}\right) + \frac{V_p}{b_w d_c} \quad . \ . \ . \ . \ . \quad \text{(Eq. 7-11)}$$

Alternatively, v_{cw} may be computed as the shear stress corresponding to dead load plus live load that results in a principal tensile stress of $0.33\sqrt{f'_c}$ at centroidal axis of member, or at intersection of flange and web when centroidal axis is in the flange. In composite members principal tensile stress shall be computed using the cross-section that resists live load.

7.3.3.4 In a pre-tensioned member in which the section at a distance $h/2$ from face of support is closer to end of member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing v_{cw}. This value of v_{cw} shall also be taken as the maximum limit for eq. 7-8. Prestress force may be assumed to vary linearly from zero at end of tendon to a maximum at a distance from end of tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

7.3.3.5 With the exception of the allowance in eq. 7-11 the transverse component of the longitudinal prestressing force V_p, shall not be considered to contribute to the shear resistance of prestressed concrete beams.

7.3.4 *Shear reinforcement – Minimum requirements*

7.3.4.1 A minimum area of shear reinforcement shall be provided in all reinforced, prestressed and non-prestressed concrete where shear stress v_i required to resist V_u exceeds half the shear strength provided by concrete v_c, except:

(a) Slabs and footings

(b) Concrete joist construction defined by 3.4.2

(c) Beams with total depth not greater than 250 mm, two and a half times thickness of flange, or one-half the width of web, whichever is greater.

7.3.4.2 Minimum shear reinforcement requirements of 7.3.4.1 may be waived if shown by test that required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

7.3.4.3 Where shear reinforcement is required by 7.3.4.1 or by analysis, minimum area of shear reinforcement for prestressed (except as provided in 7.3.4.4) and non-prestressed members shall be computed by

$$A_v = 0.35 \frac{b_w s}{f_y} \qquad \text{(Eq. 7-12)}$$

where b_w and s are in millimetres.

7.3.4.4 For prestressed members with effective prestress force not less than 40% of the design tensile strength of flexural reinforcement, minimum area of shear reinforcement may be computed by eq. 7-12 or 7-13

$$A_v = \frac{A_{ps}}{80} \cdot \frac{f_{pu}}{f_y} \cdot \frac{s}{d} \sqrt{\frac{d}{b_w}} \qquad \text{(Eq. 7-13)}$$

7.3.5 *Shear reinforcement details*

7.3.5.1 Shear reinforcement may consist of:

(a) Stirrups perpendicular to axis of member

(b) Welded wire fabric with wires located perpendicular to axis of member

(c) Stirrups making an angle of 45° or more with the longitudinal tension bars

(d) Vertical or inclined prestressing.

7.3.5.2 For non-prestressed members, shear reinforcement may also consist of:

(a) Longitudinal reinforcement with bent portion making an angle of 30° or more with the longitudinal tension reinforcement

(b) Combinations of stirrups and bent longitudinal reinforcement

(c) Spirals.

7.3.5.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fibre and shall be anchored at both ends according to 5.4.3.2 to develop the yield strength of reinforcement.

7.3.5.4 Spacing limits for shear reinforcement shall be as follows:

(a) Spacing of shear reinforcement, placed perpendicular to axis of member, shall not exceed the lesser of either (1) and (2) as appropriate or (3):

  (1) 0.5 d in non-prestressed members

  (2) 0.75 h in prestressed members and non-prestressed members provided that P_u/A_g exceeds $0.12 f'_c$

  (3) 600 mm

(b) Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45° line, extending towards the reaction from mid-depth of member 0.5 d to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement

(c) When $(v_i - v_c)$ exceeds $0.07 f'_c$ maximum spacings given in 7.3.5.4 (a) and (b) shall be reduced by one-half.

7.3.6 *Design of shear reinforcement*

7.3.6.1 Design yield strength of shear reinforcement shall not exceed 415 MPa.

7.3.6.2 When the total shear stress v_i exceeds the ideal shear stress provided by concrete, v_c, shear reinforcement shall be provided for the difference $(v_i - v_c)$.

7.3.6.3 When shear reinforcement perpendicular to axis of member is used, the required area of shear reinforcement within a distance s shall be not less than

$$A_v = \frac{(v_i - v_c)\ b_w s}{f_y} \qquad \text{(Eq. 7-14)}$$

7.3.6.4 When inclined stirrups are used as shear reinforcement, the required area shall not be less than

$$A_v = \frac{(v_i - v_c)\ b_w s}{f_y\ (\sin\alpha + \cos\alpha)} \qquad \text{(Eq. 7-15)}$$

7.3.6.5 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, the required area shall not be less than

$$A_v = \frac{(v_i - v_c)\ b_w d}{f_y \sin\alpha} \qquad \text{(Eq. 7-16)}$$

in which the term $(v_i - v_c)$ shall not exceed $0.05\ f'_c$.

7.3.6.6 When shear reinforcement 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 not less than that computed by eq. 7-15.

7.3.6.7 Only the centre three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

7.3.6.8 Where more than one type of shear reinforcement is used to reinforce the same portion of the web:

(a) Stirrups shall be provided to carry at least one-third of the total shear to be resisted by web reinforcement

(b) The required area shall be computed as the sum for the various types separately with v_c being included only once.

7.3.6.9 When a beam or beams frame monolithically into a supporting girder, stirrups having a total dependable strength $\phi \Sigma A_w f_y$ equal to or greater than the total reaction transferred from the beams to the girder shall be placed in the girder within 0.5 times the depth of the beams on either side of the beams. This requirement is waived if:

(a) The beam shear stress at the interface is less than v_b or $0.17\sqrt{f'_c}$

(b) The lower face of the girder is more than the depth of the beam below the bottom of the beam

(c) The girder is supported on its lower face at the beam-girder joint.

7.3.7 *Members loaded in torsion*

7.3.7.1 The provisions of this Clause do not apply to slabs, footings or concrete joist floor construction defined by 3.4.2.

7.3.7.2 If the torsion in the member is required to maintain equilibrium in the structure, and if the magnitude of the ideal torsional strength required exceeds $0.1 A_{co} t_c \sqrt{f'_c}$, torsional reinforcement designed in accordance with 7.3.10 shall be provided.

7.3.7.3 If the torsion on the member arises because the member must twist to maintain compatibility, the effect of torsion on the member may be neglected, provided that the moments and shears in the structure are computed assuming no torsional stiffness of the member, and that the following provisions are satisfied:

(a) Minimum torsional steel shall be provided in the member in accordance with the provisions of 7.3.8 and detailed in accordance with 7.3.9, except for those members where it is shown that the stress calculated from eq. 7-18 does not exceed $0.08\sqrt{f'_c}$, in which case torsional reinforcement may be omitted

(b) In those sections of adjoining members, where moments will occur due to the torsional restraint provided by the member, minimum flexural reinforcement as specified by 6.4.3 and 11.4 and anchored so as to provide full development shall be provided.

7.3.7.4 Sections located less than a distance d from the face of the support may be designed for the same torsion as that computed at a distance d.

7.3.7.5 Design of cross-sections subject to torsion shall be based on the relationship

$$T_u < \phi T_i \qquad \text{(Eq. 7-17)}$$

where T_u is the torsion at the section derived from factored load on the structure and T_i is the ideal torsional strength of the section.

7.3.7.6 The torsional shear stress v_{ti} shall be computed from

$$v_{ti} = \frac{T_i}{2 A_o t_o} \qquad \text{(Eq. 7-18)}$$

where the value of t_o shall not exceed the actual wall thickness of hollow sections and v_{ti} shall not exceed $0.2\ f'_c$ or 6 MPa.

7.3.7.7 When torsional shear stress and torsion 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.

7.3.7.8 When torsional and flexural shear stresses occur together at a section the condition

$$v_i + v_{ti} < 0.2\ f'_c < 6\ \text{MPa} \qquad \text{(Eq. 7-19)}$$

shall be satisfied.

7.3.8 *Torsion reinforcement – Minimum requirements*

7.3.8.1 If torsion reinforcement is required, an amount of closed stirrup and longitudinal reinforcement shall be provided such that

$$\sqrt{\frac{A_t}{s} \cdot \frac{A_\ell}{p_o}} \geqslant \frac{1.5}{f_y} \cdot \frac{A_{co}\, t_c}{A_o} \qquad \text{(Eq. 7-20)}$$

7.3.8.2 For hollow sections t_c shall not exceed the actual wall thickness.

7.3.8.3 In calculating the term A_t/s in eq. 7-20 any closed stirrups provided for shear resistance or to satisfy minimum requirements may be included.

7.3.8.4 In calculating the term A_ℓ/p_o in eq. 7-20, longitudinal steel used to resist flexure may be included provided that such steel is anchored to provide full development. The term A_ℓ/p_o shall not be taken greater than $7\,A_t/s$.

7.3.9 *Torsion reinforcement details*

7.3.9.1 Torsion reinforcement shall consist of closed stirrups perpendicular to the axis of the member combined with longitudinal bars.

7.3.9.2 Spacing of closed stirrups shall not exceed $p_o/8$, or 300 mm whichever is the smaller.

7.3.9.3 Spacing of longitudinal bars, distributed around the perimeter of the stirrups shall not exceed 300 mm.

7.3.9.4 At least one longitudinal bar having a diameter of not less than $s/16$ and not less than 10 mm, shall be placed inside each corner of the closed stirrups. These corner bars shall be anchored to provide full development.

7.3.9.5 Torsion reinforcement shall be provided at least a distance $p_o/2$ beyond the point of zero torsion.

7.3.9.6 Closed stirrups shall be anchored with 135° standard hooks.

7.3.9.7 When flanged sections are used, in accordance with 7.3.7.7, 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}.

7.3.10 *Design of torsion reinforcement*

7.3.10.1 When required by 7.3.7.2 torsional reinforcement consisting of closed stirrups perpendicular to the axis of the member and longitudinal bars distributed symmetrically around the section shall be provided in addition to the reinforcement required to resist the shear, flexure, and axial forces acting in combination with the torsion.

7.3.10.2 The provisions of this Clause apply to prestressed concrete members, provided that the value of v_c used in determining the area of shear reinforcement shall not exceed $0.17\sqrt{f'_c}$, unless special studies are made.

7.3.10.3 The required area of closed stirrups shall be computed by

$$A_t = \frac{v_{ti}\, t_o\, s}{f_y} \qquad \text{(Eq. 7-21)}$$

7.3.10.4 The required area of longitudinal bars distributed symmetrically around the section shall be computed by

$$A_\ell = \frac{v_{ti}\, t_o\, p_o}{f_y} \qquad \text{(Eq. 7-22)}$$

7.3.10.5 In the flexural compression zone of a member the area of longitudinal torsion steel required may be reduced by an amount equal to $M_u/0.9\,df_y$, where M_u is the design moment at the section acting in combination with T_u.

7.3.11 *Shear friction*

7.3.11.1 Provisions of 7.3.11 may be applied where it is appropriate to consider shear transfer across a given plane such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

7.3.11.2 A crack shall be assumed to occur along the shear plane, with relative displacement along the assumed crack resisted by friction maintained by shear-friction reinforcement across the assumed crack. Shear-friction reinforcement shall be placed approximately perpendicular to the assumed crack.

7.3.11.3 The required area of shear friction reinforcement A_{vf} shall be computed from

$$A_{vf} = \frac{V_u - P_u}{\phi \mu f_y} \qquad \text{(Eq. 7-23)}$$

where μ is coefficient of friction in accordance with 7.3.11.5.

7.3.11.4 Shear stress v_i shall not exceed $0.2\, f'_c$ nor 6 MPa.

7.3.11.5 Coefficient of friction μ in eq. 7-23 shall be:

(a) Concrete placed monolithically or placed against previously hardened concrete where the interface for shear transfer is clean, free of laitance, and intentionally roughened to a full amplitude of not less than 5 mm 1.4

(b) Concrete placed against hardened concrete, where the interface is free of laitance and roughened to a full amplitude of less than 5 mm but more than 2 mm 1.0

(c) Concrete placed against as-rolled structural steel in accordance with 7.3.11.9, or hardened concrete free of laitance 0.7

7.3.11.6 Design yield strength of shear-friction reinforcement shall not exceed 415 MPa.

7.3.11.7 Direct tension across the assumed crack shall be provided for by additional reinforcement.

7.3.11.8 Shear-friction reinforcement shall be well distributed across the assumed crack and shall be adequately anchored on both sides by embedment, hooks, or welding to special devices. All vertical reinforcement within the effective section, resisting flexure and axial load and crossing the potential sliding plane, may be included in determining A_{vf}.

7.3.11.9 When shear is transferred between as-rolled steel and concrete, steel shall be clean and free of paint.

7.3.12 *Special provisions for deep beams*

7.3.12.1 The following definitions and limitations apply to the interpretation of the shear provisions for deep beams:

(a) The principal load, defined in 7.3.12.2 is located so that a/d is less than 2, where a is the shear span defined by 7.3.12.3

(b) When no principal load is supported, or when portions of beams support distributed loads and the distance ℓ_s, between the point of zero shear and the support, as defined in 7.3.12.4, is less than three times the effective depth d

(c) Load is applied to the top face of the beam which is supported on its bottom face

(d) For beams loaded by members framing into the sides, the load may be assumed to be applied at the top of the supported member provided that reinforcement satisfying 7.3.6.9 is provided

(e) Beams supported on members framing into the sides may be assumed to be supported at the level of the bottom of the supporting member.

7.3.12.2 For a given shear span, a, the principal load is a concentrated load which causes 50% or more of the shear at the support or the deep beam.

7.3.12.3 Shear span, a, shall be taken as the distance from the centre of the principal load to the centre of the support but not more than 1.15 times the clear distance from the face of the load to the face of the support.

7.3.12.4 Distance ℓ_s shall be taken as the distance from the point of zero shear to the centre of the support but not more than 1.15 times the clear distance from the point of zero shear to the face of the support.

7.3.12.5 Shear strength of deep beams shall be based on the principles that:

(a) Total shear strength consists of the sum of the contributions of the ideal shear stress provided by the concrete, v_c, and of the vertical and the horizontal shear reinforcement

(b) Stress limits of 7.3.11.4 are not exceeded

7.3.12.6 Shear strength of deep beams supporting principal loads in accordance with 7.3.12.1 (a) shall be determined as follows:

(a) Shear force resulting from factored load on the beam, V_u, shall be computed at $a/2$ and the shear reinforcement required at this section shall be used throughout the entire span

(b) Ideal shear stress v_c shall be computed from

$$v_c = v_b \left(2\frac{d}{a}\right) \quad \ldots\ldots \quad \text{(Eq. 7-24)}$$

and from eq. 7-3. The value of v_c need not be taken less than v_b

(c) Area of vertical and horizontal shear reinforcement shall be computed from

$$\frac{A_v}{s}\left(\frac{a}{d}-\frac{1}{2}\right)+\frac{A_{vh}}{s_2}\left(\frac{3}{2}-\frac{a}{d}\right)=\frac{(v_i-v_c)\,b_w}{f_y} \quad \text{(Eq. 7-25)}$$

with the value of a/d not taken larger than 3/2.

7.3.12.7 Shear strength of deep beams not supporting principal load and of beams loaded with distributed load in the shear span, in accordance with 7.3.12.1 (b), shall be determined as follows:

(a) Shear force resulting from factored load on the beam, V_u, shall be computed at distance $\ell_s/3$ from face of support and the shear reinforcement required at this section shall be used throughout the entire span

(b) Shear stress v_c shall be computed from

$$v_c = v_b \left(3\frac{d}{\ell_s}\right) \quad \ldots\ldots \quad \text{(Eq. 7-26)}$$

and from eq. 7-3. The value of v_c need not be taken less than v_b

(c) Areas of vertical and horizontal shear reinforcement shall be computed from

$$\frac{A_v}{s}\left(\frac{2}{3}\frac{\ell_s}{d}-\frac{1}{2}\right)+\frac{A_{vh}}{s_2}\left(\frac{3}{2}-\frac{2}{3}\frac{\ell_s}{d}\right)=\frac{(v_i-v_c)\,b_w}{f_y} \quad \text{(Eq. 7-27)}$$

with the value of ℓ_s/d not taken larger than 9/4.

7.3.12.8 The area of shear reinforcement, A_v, perpendicular to the main reinforcement of deep beams shall not be less than that required by 7.3.4.3 and the horizontal spacing, s, shall not exceed $d/4$ or 450 mm.

7.3.12.9 In deep beams having spans such that a/d is less than 2 or ℓ_s/d is less than 3, the area of horizontal shear reinforcement, A_{vh}, shall not be less than

$$A_{vh} = 0.35\frac{b_w s_2}{f_y} \quad \ldots\ldots \quad \text{(Eq. 7-28)}$$

and the spacing s_2 shall not exceed $d/3$ or 400 mm.

7.3.13 *Special provisions for brackets and corbels*

7.3.13.1 Provisions of 7.3.13 apply to brackets and corbels having a shear-span-to-depth ratio, a/d of unity or less, which are subjected to a design horizontal tensile force N_u less than or equal to the design shear force V_u. The distance d shall be measured at a section adjacent to the face of the support.

7.3.13.2 The reinforcement A_{vf} required to resist the factored shear V_u by friction shall be calculated using the design provisions of 7.3.11.

7.3.13.3 Design of main tension reinforcement shall be as follows:

(a) The section adjacent to the face of the support shall be designed to resist simultaneously a factored shear force V_u, a factored horizontal tensile force N_u and a factored moment M_u about the main tension reinforcement, where M_u shall be computed by

$$M_u = V_u a + N_u (h - d) \quad \text{(Eq. 7-29)}$$

(b) The reinforcement A_f required to resist the factored moment shall be calculated using the provisions of 6.3.1

(c) The area of reinforcement A_{ct} required to resist the factored tensile force N_u shall be computed by

$$A_{ct} = \frac{N_u}{\phi f_y} \quad \text{(Eq. 7-30)}$$

where $\phi = 0.85$. The design tensile force N_u shall not be taken as less than 0.2 V_u unless special provisions are made to avoid tension forces due to restrained shrinkage and creep, so that the member is subject to shear and moment only. The tensile force N_u shall be regarded as a live load even when it results from creep, shrinkage, or temperature change

(d) The area of main tension reinforcement A_s shall be made equal to $(A_{ct} + A_f)$ or $(2A_{vf}/3 + A_{ct})$, whichever is greater

(e) The ratio
$\rho = A_s/(bd)$ shall be not less than 0.04 (f'_c/f_y)

(f) The main tension reinforcement shall be anchored as close to the outer face of the corbel as cover requirements permit, by welding a bar of equal diameter across the ends of the main reinforcing bars, or by some other means of positive anchorage. The bearing area of the load shall not project beyond the straight portion of the bars forming the main tension reinforcement.

7.3.13.4 Closed stirrups or ties parallel to the main tension reinforcement, having a total cross-sectional area A_h not less than 0.50 $(A_s - A_{ct})$, or $aA_v/3s$ shall be uniformly distributed within two-thirds of the effective depth adjacent to the main tension reinforcement, where a is the shear span.

7.3.13.5 Depth of the corbel or bracket at the outside edge of the bearing area shall be not less than one-half the effective depth of the corbel or bracket at the section adjacent to the face of the support.

7.3.14 *Special provisions for walls*

7.3.14.1 Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 7.3.15. Design for horizontal shear forces in plane of wall shall be in accordance with 7.3.14.2 to 7.3.14.9.

7.3.14.2 Design of horizontal section for shear in plane of wall shall be based on 7.3.1.1 and 7.3.1.2, where shear stress v_c shall be in accordance with 7.3.14.5 or 7.3.14.6 and shear reinforcement shall be in accordance with 7.3.14.9.

7.3.14.3 Shear stress v_i at any horizontal section for shear in plane of wall and based on the minimum net wall thickness shall not be taken greater than 0.2 f'_c or 6 MPa.

7.3.14.4 For design for horizontal shear forces in plane of wall, d shall be taken as equal to 0.8 ℓ_w. A larger value of d, equal to the distance from extreme compression fibre to centre of force of all reinforcement in tension, may be used when determined by a strain compatibility analysis.

7.3.14.5 Unless a more detailed calculation is made in accordance with 7.3.14.6, shear stress v_c shall not be taken greater than $0.2\sqrt{f'_c}$ for walls subject to P_u in compression, or v_c shall not be taken greater than

$$v_c = 0.2 \left(\sqrt{f'_c} + \frac{P_u}{A_g}\right) \quad \text{(Eq. 7-31)}$$

for walls subject to P_u in tension, in which case P_u is negative in eq. 7.31.

7.3.14.6 Shear stress v_c may be computed by equations 7.32 and 7.33 where v_c shall be the lesser of eq. 7-32 or eq. 7-33.

$$v_c = 0.27\sqrt{f'_c} + \frac{P_u}{4A_g} \quad \text{(Eq. 7-32)}$$

or

$$v_c = 0.05\sqrt{f'_c} + \frac{\ell_w\left(0.1\sqrt{f'_c} + 0.2\dfrac{P_u}{A_g}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}} \quad \text{(Eq. 7-33)}$$

where P_u is negative for tension. When $(M_u/V_u - \ell_w/2)$ is negative, eq. 7.33 shall not apply.

7.3.14.7 Sections located closer to wall base than a distance $\ell_w/2$ or one-half the wall height, whichever is less, may be designed for the same v_c as that computed at a distance $\ell_w/2$ or one-half the height.

7.3.14.8 Irrespective of whether total shear stress v_i is more or less than $v_c/2$, reinforcement shall be provided in accordance with 7.3.14.9.

7.3.14.9 Design of shear reinforcement for walls shall satisfy the following requirements:

(a) Where total shear stress v_i exceeds shear stress v_c, horizontal shear reinforcement shall be computed from

$$A_v = \frac{(v_i - v_c)\, b_w s_2}{f_{yh}} \qquad \text{(Eq. 7-34)}$$

where A_v is the area of horizontal shear reinforcement within a distance s_2

(b) Irrespective of the requirements of 7.3.14.9 (a) area of horizontal shear reinforcement in a wall shall not be less than

$$A_v = \frac{0.7\, b_w s_2}{f_{yh}} \qquad \text{(Eq. 7-35)}$$

(c) Spacing of horizontal shear reinforcement s_2 shall not exceed $\ell_w/5$, $3h$, nor 450 mm

(d) Ratio ρ_n of vertical reinforcement area to gross concrete area of horizontal section shall not be less than $0.7/f_{yn}$

(e) Spacing of vertical shear reinforcement s_1 shall not exceed $\ell_w/3$, $3h$, nor 450 mm.

7.3.15 *Special provisions for slabs and footings*

7.3.15.1 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 slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with 7.3.1 to 7.3.6

(b) Two-way action for slab or footing, with a critical section perpendicular to plane of slab and located so that its perimeter b_o is a minimum, but need not approach closer than $d/2$ to perimeter of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with 7.3.15.2 to 7.3.15.5

7.3.15.2 Design of slab or footing for two-way action shall be based on

$$v_i = \frac{V_i}{b_o d} \qquad \text{(Eq. 7-36)}$$

where shear stress v_i shall not be taken greater than shear stress v_c given by eq. 7-37, unless shear reinforcement is provided in accordance with 7.3.15.3 or 7.3.15.4.

$$v_c = 0.17\,(1 + 2\,\beta_c)\sqrt{f_c'} \qquad \text{(Eq. 7-37)}$$

but not greater than $0.33\sqrt{f_c'}$. β_c is the ratio of short side to long side of concentrated load or reaction area and b_o is perimeter of critical section defined in 7.3.15.1 (b).

7.3.15.3 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

(a) Required total shear strength shall be provided in accordance with 7.3.1.2. Shear stress v_c shall be in accordance with 7.3.15.3 (d) and shear reinforcement shall be provided as required by 7.3.15.3 (e)

(b) Shear stress v_i shall not be taken greater than $0.5\sqrt{f_c'}$ where b_o is perimeter of critical section defined in 7.3.15.1 (b)

(c) Shear strength shall be investigated at the critical section defined in 7.3.15.1 (b) and at successive sections more distant from the support

(d) Shear stress v_c at any section shall not be taken greater than $0.17\sqrt{f_c'}$, taking b_o as the perimeter of critical section defined in 7.3.15.1 (b)

(e) Where ideal shear stress v_i exceeds shear stress v_c as given in 7.3.15.3 (d), required area A_v of shear reinforcement shall be calculated in accordance with 7.3.6 and anchored in accordance with 5.4.3.2.

7.3.15.4 Shear reinforcement consisting of steel I or channel shapes (shearheads) may be used in slabs. Provisions of this Clause shall apply where shear is transferred at interior column supports. Where shear is transferred at edge or corner column supports, special designs are required:

(a) Each shearhead shall consists of steel shapes fabricated by welding into four identical arms at right angles. Shearhead arms shall be continuous through the column section

(b) Shearhead shall not be deeper than 70 times the web thickness of the steel shape

(c) Ends of each shearhead arm may be cut at angles not less 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 0.3 d of compression surface of slab

(e) Ratio α_v between the stiffness for each shearhead arm and that for surrounding composite cracked slab section of width $(c_2 + d)$ shall not be less than 0.15

(f) Plastic moment strength M_p required for each arm of the shearhead shall be computed by

$$\phi M_p = \frac{V_u}{8}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \qquad \text{(Eq. 7-38)}$$

where ϕ is strength reduction factor for flexure and ℓ_v is minimum length of each shearhead arm required to comply with requirements or 7.3.15.4 (g) and (h)

(g) Critical slab section for shear shall be perpendicular to plane of slab and shall cross each shearhead arm three-quarters the distance $[\ell_v - (c_1/2)]$ from column face to end of shearhead arm. Critical section shall be located so that its perimeter b_o is a minimum, but need not approach closer than $d/2$ to perimeter of column section

(h) Shear stress v_i shall not be taken greater than $0.33\sqrt{f'_c}$ on the critical section defined in 7.3.15.4 (g). When shearhead reinforcement is provided, shear stress v_i shall not be taken greater than $0.6\sqrt{f'_c}$ on the critical section defined in 7.3.15.1(b)

(j) A shearhead may be assumed to contribute a moment resistance M_v to each slab column strip computed by

$$M_v = \frac{\phi \alpha_v V_u}{8}\left(\ell_v - \frac{c_1}{2}\right) \quad \text{(Eq. 7-39)}$$

where ϕ is the strength reduction factor for flexure and ℓ_v is the length of each shearhead arm actually provided. However, M_v shall not be taken larger than the smaller of:

(1) 30% of total factored moment required for each slab column strip

(2) Change in column strip moment over length ℓ_v

(3) Value of M_p computed by eq. 7-38.

7.3.15.5 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 11, the critical slab section for shear defined in 7.3.15.1 (b) and 7.3.15.4 (g) 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 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).

7.3.16 *Transfer of moments to columns*

7.3.16.1 When gravity load, wind, earthquake or other lateral forces cause transfer of moment at connections of framing elements to columns, shear resulting from moment transfer shall be considered in design of shear reinforcement in the joint.

7.3.16.2 The following special provisions for slabs apply:

(a) When gravity load, wind, earthquake or other lateral forces cause transfer of moment between slab and column, a fraction of the unbalanced moment shall be transferred by eccentricity of shear in accordance with 7.3.16.2 (c) and (d)

(b) Fraction of unbalanced moment not transferred by eccentricity of shear shall be transferred by flexure in accordance with 11.3.5

(c) A fraction of the unbalanced moment given by

$$\gamma_v = 1 - \frac{1}{1 + 2/3\sqrt{\left(\frac{c_1 + d}{c_2 + d}\right)}} \quad \text{(Eq. 7-40)}$$

shall be considered transferred by eccentricity of shear about centroid of a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than $d/2$ to perimeter of column

(d) Shear stresses resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly about centroid of critical section defined in 7.3.16.2 (c). Maximum total shear stress due to factored shear forces and moments shall not exceed $0.17(1+2\beta_c)\sqrt{f'_c}$ nor $0.33\sqrt{f'_c}$

(e) When unbalanced gravity load, wind, earthquake or other lateral forces require a bending moment to be transferred between slab and column which exceeds the value allowed by 7.3.16.2 (c) the connection may be strengthened by shear reinforcement designed using a beam analogy for the strength of the slab.

7.4 Principles and requirements additional to 7.3 for members not designed for seismic loading

7.4.1 *Transfer of moments to columns.* Lateral reinforcement not less than that required by eq. 7-12 shall be provided within connections of framing elements to columns, except for connections not part of a seismic load-resisting system that are restrained on four sides by beams or slabs of approximately equal depth.

7.5 Principles and requirements additional to 7.3 for members designed for seismic loading

7.5.1 *Shear strength*

7.5.1.1 The design shear in members subjected primarily to flexure shall be determined from considerations of static transverse forces on the member, with 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 the appropriate load factor.

7.5.1.2 The design shear force in members subjected to combined flexure and axial load shall be determined from considerations of static forces on the member, with a rational adverse combination of the maximum likely end moments.

7.5.2 *Shear strength provided by concrete*

7.5.2.1 In beams subjected to flexure, shear stress v_c shall be assumed to be zero for any seismic load combination in all regions where stirrup ties are required in accordance with 6.5.3.3.

7.5.2.2 In beams and columns subjected to flexure, axial load and shear, stress v_c in the end regions, defined in 7.5.3.2, shall be taken zero unless the design axial compression, P_e produces an average stress of not less than 0.1 f'_c over the gross concrete area, in which case

$$v_c = 4 v_b \sqrt{\frac{P_e}{A_g f'_c} - 0.1} \quad \text{(Eq. 7-41)}$$

7.5.3 *Shear reinforcement details*

7.5.3.1 In members subjected to flexure and axial load the transverse reinforcement shall satisfy the requirements of 7.3.1.2 and 7.3.6 for shear resistance, and in end regions also the requirements of 6.5.3.3 and 6.5.4.

7.5.3.2 The end region of a member, where in accordance with 7.5.2, the limitations of shear stress v_c apply, shall not be less than:

(a) The depth of member in the plane in which a frame resists the earthquake forces under consideration

(b) One-sixth of the clear span or height of the member

(c) 450 mm.

7.5.4 *Design of shear reinforcement*

7.5.4.1 With the limitations of 7.5.2 on shear stress v_c, all requirements of 7.3.6 apply.

7.5.4.2 At critical sections of potential plastic hinge regions of flexural members, where due to reversed seismic load the top and bottom flexural reinforcement may be subjected to tensile yielding, the following requirements shall also be satisfied:

(a) Unless the conditions of 7.5.4.5 are satisfied total shear stress shall not exceed $0.9\sqrt{f'_c}$

(b) When the total shear stress exceeds $0.3\,(2 + r)\sqrt{f'_c}$, diagonal shear reinforcement shall be provided across the web in the plastic hinge in one or both directions to resist the shear force

$$V_{di} = 0.7\left(\frac{v_i}{\sqrt{f'_c}} + 0.4\right)(-r)\, V_i \quad \text{(Eq. 7-42)}$$

where, by taking into account the sense of the shear forces resulting from the two directions of earthquake loading, V_{di} need only be considered when

$$-1 < r < -0.2$$

(c) A plastic hinge shall be assumed to extend to a distance of not less than d from the face of the support or from a similar cross-section where maximum yielding due to reversed loading can be expected

(d) Only the diagonal reinforcement acting in tension may be included in accordance with 7.3.6.4, when the shear reinforcement to prevent diagonal tension failure in the plastic hinge region is determined.

7.5.4.3 Rational analysis must show that the shear resistance, V_{di}, in accordance with 7.5.4.2, at each cross-section of the potential plastic hinge zone is provided by the transverse component of the inclined steel forces only. When diagonal bars cross a section in two directions, the transverse components of the diagonal tension and compression steel forces may be considered together.

7.5.4.4 The requirements of 7.5.4.2 do not apply to members in which the minimum axial compression stress on the gross concrete area, associated with the maximum shear on the member, is more than $0.10 f'_c$;

7.5.4.5 Total shear stress v_i may exceed $0.9\sqrt{f'_c}$ if the entire load on the member is resisted by diagonal reinforcement.

7.5.5 *Special provisions for earthquake resisting walls and diaphragms*

7.5.5.1 In the evaluation of shear strength of earthquake resisting walls the general requirements of 10.5.5 shall also be satisfied.

7.5.5.2 In end regions of shear walls, defined in 10.5.5.3, shear stress v_c shall not be taken larger than

$$v_c = 0.6\sqrt{\frac{P_e}{A_g}} \quad \text{(Eq. 7-43)}$$

and the total shear stress v_i shall not exceed

$$v_i = (0.3\,\phi_o\, S + 0.16)\sqrt{f'_c} \quad \text{(Eq. 7-44)}$$

7.5.5.3 Ratio of ρ_h of horizontal shear reinforcement to gross concrete area of vertical wall section shall not be less than

$$\rho_h = \frac{4}{3}\left[\frac{d_s V_u}{M_u}\frac{f_{yn}}{f_{yh}}\rho_n - \frac{v_c}{f_{yh}}\right] \quad \text{(Eq. 7-45)}$$

unless it is shown that yielding of the horizontal shear reinforcement cannot occur before the development of the ideal flexural strength of the wall section. The reinforcement ratio ρ_h in eq. 7-45 need not be taken larger than the value that would result in the ideal flexural strength of the wall corresponding with $S = 4$. The reinforcement ratio ρ_n refers to the critical wall region only where yielding of the vertical reinforcement due to flexure may be expected.

7.5.5.4 In the evaluation of shear strength of precast elements with cast *in situ* reinforced concrete topping, used also for diaphragm action, the requirements of 10.5.6.6 shall also be considered.

7.5.5.5 In coupling beams of ductile coupled shear wall structures the arrangement of reinforcement shall be in accordance with the shear stress limitations of 10.5.7.2.

7.5.6 *Openings in the web*

7.5.6.1 Adjacent openings for services in the web of flexural members shall be arranged so that potential failure planes across such openings cannot occur.

7.5.6.2 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 not less 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 0.004 d^2 when the effective depth is more than 500 mm.

7.5.6.3 Webs with openings larger than that permitted by 7.5.6.2 shall be subject to rational design to ensure that the forces and moments are adequately transferred in the vicinity of the openings.

7.5.6.4 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, nor where the total shear stress exceeds 0.4 $\sqrt{f_c'}$, or in potential plastic hinge zones. In no case shall the height of the opening exceed 0.4 d nor shall its edge be closer than 0.33 d to the compression face of the member.

7.5.6.5 For openings defined by 7.5.6.4, longitudinal and transverse reinforcement shall be placed in the compression side of the web at one side of the opening to resist one and one-half times the shear and moment generated by the shear across the opening. Shear transfer in the tension side of the web on the other side of the opening shall be neglected.

7.5.6.6 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 across the opening.

8 COMPOSITE CONCRETE FLEXURAL MEMBERS

8.1 Notation

b_v the width of the cross-section being investigated for horizontal shear, mm

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

v_{dh} design horizontal shear stress at any cross-section, MPa

v_h permissible horizontal shear stress, MPa

V_i ideal shear strength of section, N

V_u factored shear force at section, N

ϕ strength reduction factor. See 4.3.1.2.

8.2 Scope

8.2.1 Provisions of this Section apply for design of composite concrete flexural members defined as precast or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to loads as a unit.

8.2.2 All provisions of this Code shall apply to composite concrete flexural members, except as specifically modified in this Section.

8.3 General principles and requirements

8.3.1 *General considerations*

8.3.1.1 An entire composite member or portions thereof may be used in resisting shear and moment.

8.3.1.2 Individual elements shall be investigated for all critical stages of loading.

8.3.1.3 If the specified strength, unit weight, or other properties of the various elements are different, properties of the individual elements or the most critical values shall be used in design.

8.3.1.4 In strength computations of composite members, no distinction shall be made between shored and unshored members.

8.3.1.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

8.3.1.6 Reinforcement shall be provided as required to control cracking and to prevent separation of individual elements of composite members.

8.3.1.7 Composite members shall meet requirements for control of deflections in accordance with 4.4.1.

8.3.2 *Shoring.* When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and cracking at time of shoring removal.

8.3.3 *Vertical shear*

8.3.3.1 When an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Section 7 as for a monolithically cast member of the same cross-sectional shape.

8.3.3.2 Shear reinforcement shall be fully anchored into interconnected elements in accordance with 6.4.4 and 6.5.3.3.

8.3.3.3 Extended and anchored shear reinforcement may be included as ties for horizontal shear.

8.3.4 *Horizontal shear*

8.3.4.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

8.4 Principles and requirements additional to 8.3 for members not designed for seismic loading

8.4.1 *Horizontal shear*

8.4.1.1 Full transfer of horizontal shear forces may be assumed when all of the following are satisfied:

(a) Contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 5 mm;

(b) Minimum ties are provided in accordance with 8.4.2;

(c) Web members are designed to resist total vertical shear; and

(d) All shear reinforcement is fully anchored into all interconnected elements.

8.4.1.2 If all requirements of 8.4.1.1 are not satisfied, horizontal shear shall be investigated in accordance with 8.4.1.4 or 8.4.1.6.

8.4.1.3 The horizontal shear stress v_{dh} may be calculated at any cross-section as

$$v_{dh} = \frac{V_i}{b_v d} \qquad \text{(Eq. 8-1)}$$

in which d is for the entire composite section.

8.4.1.4 The design shear force may be transferred at contact surfaces using the permissible horizontal shear stresses v_h as follows:

(a) When ties are not provided, but the contact surfaces are clean and intentionally roughened, permissible v_h = 0.55 MPa

(b) Where the minimum tie requirements of 8.4.2 are provided and the contact surfaces are clean but not intentionally roughened, permissible v_h = 0.55 MPa

(c) When the minimum tie requirements of 8.4.2 are provided and the contact surfaces are clean and intentionally roughened, permissible v_h = 2.4 MPa

(d) When v_{dh} exceeds 2.4 MPa, design for horizontal shear shall be made in accordance with 7.3.11.

8.4.1.5 When tension exists perpendicular to any surface, shear transfer by contact may be assumed only when the minimum tie requirements of 8.4.2 are satisfied.

8.4.1.6 Horizontal shear may be investigated by computing the actual compressive or tensile force in any segment, and provisions made to transfer that force as horizontal shear to the supporting element. The design or factored horizontal shear stress in this case shall not exceed the permissible stresses given by 8.4.1.4 (a) to (d) inclusive.

8.4.1.7 In bridge superstructures the minimum tie requirements of 8.4.2 or more shall always be provided, and the contact surfaces shall always be intentionally roughened.

8.4.2 *Ties for horizontal shear*

8.4.2.1 When vertical bars or extended stirrups are used to transfer horizontal shear, the tie area shall not be less than that required by 7.3.6 and the spacing shall neither exceed four times the least dimension of the supported element nor 600 mm.

8.4.2.2 Ties for horizontal shear may consist of single bars, multiple leg stirrups, or the vertical legs of welded wire fabric. All ties shall be fully anchored into the components in accordance with 5.4.3 and 5.5.6.

8.4.3 *Intentional roughness.* Intentional roughness may be assumed only when the contact surface is roughened, clean, and free of laitance. When using 8.4.1.4 (c) and 8.4.1.4 (d) the roughness shall have a full amplitude of approximately 5 mm.

8.5 Principles and requirements additional to 8.3 for members designed for seismic loading

8.5.1 *General.* Where composite members are required to resist flexural loading the provisions of this Code shall apply. In particular shear reinforcing shall be provided in accordance with 7.5.1 to 7.5.4 inclusive.

8.5.2 *Diaphragm action.* Where diaphragm action is required through cast *in situ* topping the requirements of 10.5.6.6 shall be satisfied.

9 BEAM-COLUMN JOINTS

9.1 Notation

A_g gross area of section, mm²

A_{jh} total area of effective horizontal joint shear reinforcement, mm²

A_{jv} total area of effective vertical joint shear reinforcement, mm²

A_s area of non-prestressed tension beam reinforcement, mm²

A_s' area of non-prestressed compression beam reinforcement, mm²

A_{sc} area of non-prestressed tension reinforcement in one face of the column section, mm²

A_{sc}' area of non-prestressed compression reinforcement in one face of the column section, mm²

b_c overall width of column, mm

b_j effective width of joint, mm

b_w web width of beam, mm

C_j $\dfrac{V_{jh}}{V_{jx} + V_{jz}}$

e eccentricity between the centre lines of the webs of a beam and a column at a joint, mm

f_c' specified compressive strength of concrete, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

h_b 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 passing through a joint, N

P_e design axial load in compression with given eccentricity due to gravity and seismic loading acting on the member during an earthquake, N

P_u design axial compression column load including vertical prestressing where applicable occurring simultaneously with V_{jh}, N

v_{jh} nominal horizontal shear stress in joint core, MPa

V_{col} horizontal shear force across a column, N

V_{ch} ideal horizontal joint shear strength provided by concrete shear resisting mechanism only, N

V_{cv} ideal vertical joint shear strength provided by concrete shear resisting mechanism only, N

V_{jh} total horizontal shear force across a joint, N

V_{jv} total vertical shear force across a joint, N

V_{jx} total horizontal joint shear force in x direction, N

V_{jz} total horizontal joint shear force in z direction, N

V_{sh} ideal horizontal joint shear strength provided by horizontal joint shear reinforcement, N

V_{sv} ideal vertical joint shear strength provided by vertical joint shear reinforcement, N

ϕ strength reduction factor:
1.0 where joint forces are derived from overstrength member actions, or 0.85 in other cases

9.2 Scope

9.2.1 Provisions of this Section apply to design of beam-column joints subject to shear induced by gravity or earthquake loads or both. Design for shear in slab-column connections is to be in accordance with 7.3.15 and 7.3.16.

9.3 General principles and requirements

9.3.1 Beam–column joints shall satisfy the following criteria:

(a) A joint shall perform under service loads at least as well as the members that it joins

(b) A joint shall have a dependable strength sufficient to resist the most adverse load combinations sustained by the adjoining members, as specified by the appropriate loadings code, several times where necessary.

9.4 Principles and requirements additional to 9.3 for joints not designed for seismic loadings

9.4.1 *General.* Provisions in this Clause 9.4 apply to beam–column joints where gravity load actions govern. If the joint is also subject to seismic load reversals it shall be checked for compliance with the provisions of 9.5.

9.4.2 *Design forces.* The design forces acting on a beam–column joint shall be evaluated from the maximum stresses generated by all members meeting at the joint, subjected to the most adverse combination of loads as required by the appropriate loadings code, with the joint in equilibrium. 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.

9.4.3 *Strength reduction factor.* In determining the shear strength of the joint the value of the strength reduction factor ϕ shall be 0.85.

9.4.4 *Maximum permissible horizontal stress.* The nominal horizontal shear stress in the joint shall not exceed that specified in 9.5.3.2.

9.4.5 *Design principles.* The joint shear shall be assumed to be resisted by a concrete mechanism plus a truss mechanism, comprising horizontal and vertical stirrups or bars and diagonal concrete struts, in accordance with 9.4.6 and 9.4.7, except that corner joints of portal frame structures and in other appropriate applications joints may be detailed by rational analysis so that shear forces are transferred by an acceptable mechanism and so that anchorage of the flexural reinforcement within the joint is assured.

9.4.6 *Horizontal joint shear reinforcement.* The horizontal design shear force to be resisted by the horizontal joint shear reinforcement shall be

$$V_{sh} = \frac{V_{jh}}{\phi} - V_{ch} \qquad \text{(Eq. 9-1)}$$

where

$$V_{ch} = 0.5\, V_{jh} \left(1 + \frac{C_j P_u}{0.4\, A_g f_c'}\right) \qquad \text{(Eq. 9-2)}$$

except that in joints where the overall depth of the column, h_c, is at least two times the overall depth of the beam, h_b, V_{ch} need not be taken less than

$$V_{ch} = 0.2\, b_j\, h_c \sqrt{f_c'} \qquad \text{(Eq. 9-3)}$$

The area of horizontal shear reinforcement shall be determined in accordance with 9.5.4.3.

9.4.7 *Vertical joint shear reinforcement.* The vertical design shear force to be resisted by the vertical joint shear reinforcement shall be

$$V_{sv} = \frac{V_{jv}}{\phi} - V_{cv} \qquad \text{(Eq. 9-4)}$$

where

$$V_{cv} = \frac{A_{sc}'}{A_{sc}}\, V_{jv} \left(0.6 + \frac{C_j P_u}{A_g f_c'}\right) \qquad \text{(Eq. 9-5)}$$

except that V_{cv} need not be taken less than

$$V_{cv} = 0.2\, b_j\, h_b \sqrt{f_c'} \qquad \text{(Eq. 9-6)}$$

The area of vertical shear reinforcement shall be determined in accordance with 9.5.5.3 and 9.5.5.4.

9.4.8 *Confinement.* The horizontal transverse confinement reinforcement in beam-column joints shall not be less than that required by 6.4.7, 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 6.4.7, but in no case shall the stirrup-tie spacing in the joint core exceed ten times the diameter of the column bar or 200 mm, whichever is less.

9.5 Principles and requirements additional to 9.3 for joints designed for seismic loading

9.5.1 *General.* Special provisions are made in this Section for beam-column joints that are subjected to forces arising as a result of inelastic lateral displacements of ductile frames. 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.

9.5.2 *Design forces*

9.5.2.1 The design forces acting on a beam-column joint core shall be evaluated from the maximum stresses 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. 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.

9.5.2.2 The magnitude of the horizontal shear force, V_{jh}, and the vertical shear force, V_{jv}, in the joint shall be evaluated from a rational analysis taking into account the effect of all forces acting on the joint.

9.5.3 *Design assumptions*

9.5.3.1 The design of the shear reinforcement in the joint shall be based on the effective control of a potential failure plane that extends from one edge of the joint to the diagonally opposite edge. In determining the shear strength of the joint the value of the strength reduction factor ϕ shall be 1.0 where design forces are derived from overstrength member forces.

9.5.3.2 The nominal horizontal shear stress in the joint in either principal direction, v_{jh}, shall not exceed $1.5\sqrt{f_c'}$

where

$$v_{jh} = \frac{V_{jh}}{b_j h_c} \qquad \text{(Eq. 9-7)}$$

The effective joint width, b_j, shall be taken as

(a) when $b_c > b_w$
either $b_j = b_c$
or $b_j = b_w + 0.5\, h_c$, whichever is the smaller

(b) when $b_c < b_w$
either $b_j = b_w$
or $b_j = b_c + 0.5\, h_c$, whichever is the smaller.

9.5.3.3 The shear strength of a joint shall be assessed as follows:

(a) When plastic hinges could develop immediately adjacent to a joint the entire shear shall be assumed to be resisted by a truss mechanism, consisting of horizontal and vertical stirrups or bars and diagonal concrete struts, with the exception of joints where gravity load or prestressing enable transmission of shear by diagonal concrete compression forces, in which case some shear may be allocated to a concrete mechanism alone in accordance with 9.5.4.2.

(b) For the plastic hinge conditions of 9.5.3.3 (a) diagonal bars, bent across the joint in one or both directions, or other special devices, may be used if it is shown by rational analysis or tests, or both, to the satisfaction of the Engineer, that the shear forces that may be induced during large inelastic deformations of adjacent beams are adequately transferred by an acceptable mechanism and that anchorage of the flexural reinforcement across the joint is assured.

(c) When beams are detailed so that, in accordance with 6.5.3.1 (b), plastic hinges cannot develop immediately adjacent to the joint, a larger proportion of joint shear resistance may be allocated to the concrete mechanism only, provided that a rational analysis is used or the requirements of 9.5.4.2 (c) and 9.5.5 are satisfied.

9.5.3.4 The required horizontal and vertical joint shear reinforcement shall be placed within the effective width of the joint, defined in 9.5.3.2, relevant to each direction of loading.

9.5.4 *Horizontal joint shear*

9.5.4.1 The horizontal design shear force to be resisted by the horizontal joint shear reinforcement shall be

$$V_{sh} = V_{jh} - V_{ch} \quad \ldots\ldots\ldots \quad \text{(Eq. 9-8)}$$

9.5.4.2 The value of V_{ch} shall be assumed to be zero except in the following cases:

(a) When the minimum average compression stress on the gross concrete area of the column above the joint, including prestress where applicable, exceeds $0.1\, f'_c/C_j$

$$V_{ch} = \frac{2}{3}\sqrt{\frac{C_j P_e}{A_g} - \frac{f'_c}{10}}\,[b_j h_c] \quad \ldots\ldots \quad \text{(Eq. 9-9)}$$

(b) When beams are prestressed through the joint

$$V_{ch} = 0.7 P_{cs} \quad \ldots\ldots\ldots \quad \text{(Eq. 9-10)}$$

where P_{cs} is the force after all losses in the prestressing steel that is located within the central third of the beam depth. The values of V_{ch} obtained from eq. 9-9 and eq. 9-10 may be added when applicable.

(c) When the design precludes the formation of any beam plastic hinges at a joint, or when all beams at the joint are detailed so that the critical section of the plastic hinge is located at a distance from the column face in accordance with 6.5.3.1 (b), or for external joints where the flexural steel is anchored outside the column core in a beam stub in accordance with 5.5.2

$$V_{ch} = 0.5\frac{A'_s}{A_s} V_{jh}\left(1 + \frac{C_j P_e}{0.4 A_g f'_c}\right) \quad \ldots\ldots \quad \text{(Eq. 9-11)}$$

where the ratio A'_s/A_s shall not be taken larger than 1.0.

When the axial column load results in tensile stresses over the gross concrete area exceeding $0.2\, f'_c$, the entire joint shear shall be resisted by reinforcement. For axial tension smaller than these limits the value of V_{ch} may be linearly interpolated between zero and the values given by eq. 9-11 with P_e taken as zero.

(d) For external joints without beam stubs, where beam bars are anchored in the joint core with at least a 90° standard hook and in accordance with the requirements of 5.5.2, V_{ch} given by eq. 9-11 may be used when multiplied by the ratio

$$\frac{3\, h_c\, (A_{jv}\ \text{provided})}{4\, h_b\, (A_{jv}\ \text{required})}$$

the value of which shall not be taken larger than unity. The vertical joint reinforcement required, A_{jv}, shall be computed by eq. 9-16.

(e) When the overall depth of the column, h_c, is at least two times the overall depth of the beam, h_b, V_{ch} need not be taken less than

$$V_{ch} = 0.2\, b_j h_c \sqrt{f'_c} \quad \ldots\ldots\ldots \quad \text{(Eq. 9-12)}$$

9.5.4.3 The horizontal shear reinforcement shall be capable of carrying the design joint shear force given by eq. 9-8 across the corner to corner potential failure plane. The effective total area of the horizontal reinforcement that crosses the critical diagonal plane, determined according to the orientation of the individual tie legs with respect to this failure plane, that are situated within the effective joint width, b_j, shall not be less than

$$A_{jh} = \frac{V_{sh}}{f_y} \quad \ldots\ldots\ldots \quad \text{(Eq. 9-13)}$$

Horizontal sets of stirrup ties shall be placed between the outermost layers of the top and bottom beam reinforcement and shall be distributed as uniformly as practicable. Any tie leg between bends around column bars that does not cross the potential failure plane, or is shorter than one-third of the dimension of the column in the appropriate direction of action, shall be neglected.

9.5.5 *Vertical joint shear*

9.5.5.1 The vertical design shear force to be resisted by the vertical joint shear reinforcement shall be

$$V_{sv} = V_{jv} - V_{cv} \quad \ldots\ldots\ldots \quad \text{(Eq. 9-14)}$$

9.5.5.2 The value V_{cv} shall be determined from

$$V_{cv} = \frac{A'_{sc}}{A_{sc}} V_{jv}\left(0.6 + \frac{C_j P_e}{A_g f'_c}\right) \quad \ldots\ldots\ldots \quad \text{(Eq. 9-15)}$$

except

(a) Where axial load results in tensile stresses over the column section, the value V_{cv} shall be linearly interpolated between the value given by eq. 9-15 with P_e taken as zero, and zero when the axial tension over the gross concrete area is $0.2\, f'_c$;

and

(b) Where plastic hinges are expected to form in the column above or below a joint as part of the primary seismic energy dissipating mechanism, but not where elastic action is assured in the column or column stub on the opposite side of the joint, V_{cv} shall be assumed to be zero for any axial load on the column.

9.5.5.3 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.

9.5.5.4 The total area of vertical joint shear reinforcement within the effective joint width, b_j, shall not be less than

$$A_{jv} = \frac{V_{sv}}{f_y} \quad \text{(Eq. 9-16)}$$

9.5.5.5 The spacing of vertical joint reinforcement in each plane of any beam framing into a joint shall not exceed 200 mm, and in no case shall there be less than one intermediate bar in each side of the column in that plane.

9.5.6 *Confinement*

9.5.6.1 The horizontal transverse confinement reinforcement in beam-column joints shall not be less than that required by 6.5.4.3 with the exception of joints connecting beams at all four column faces that are designed according to 9.5.4.2 (c) in which case the transverse joint reinforcement may be reduced to one-half of that required in 6.5.4.3 (b), but in no case shall the stirrup tie spacing in the joint core exceed ten times the diameter of the column bar or 200 mm, whichever is less.

9.5.7 *Joints with wide columns and narrow beams*

9.5.7.1 When the width of the column is larger than the effective joint width specified in 9.5.3.2 or 9.5.8, 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 6.5.4.2 (c). Transverse reinforcement outside of the effective joint area shall be in accordance with the confinement provisions of 9.5.6.

9.5.8 *Eccentric beam-column joints*

9.5.8.1 All design provisions of this section apply, except that in the case of the eccentricity of a beam relative to the column into which it frames, as measured by the distance e between the centre lines of the webs of the beam and the column, the effective joint width, b_j, shall not be taken larger than $0.5\,(b_w + b_c + 0.5h_c) - e$.

10 WALLS AND DIAPHRAGMS

10.1 Notation

ΣA_b sum of area of individual bars, mm^2

A_c^* area of concrete core extending over the outer half of the neutral axis depth which is subjected to compression, measured to outside of peripheral hoop legs, mm^2

A_g gross area of section, mm^2

A_g^* gross area of concrete section extending over outer half of the neutral axis depth which is subjected to compression, mm^2

A_s area of longitudinal wall reinforcement spaced horizontally at s_v, mm^2

A_{sh} total effective area of hoop bars and supplementary cross ties in direction under consideration, within spacing s_h, mm^2

A_{te} area of one leg of stirrup-tie, mm^2

b width or thickness of wall section, mm

c computed distance of neutral axis from the compression edge of the wall section, mm

c_c distance of the critical neutral axis from the compression edge of the wall section, mm

f_c' specified compressive strength of concrete, MPa

f_{yh} specified yield strength of hoop or supplementary cross tie steel, MPa

h overall depth of beam, mm

h'' dimension of concrete core of section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop, mm

ℓ_d development length, mm

h_w total height of wall from base to top, mm

ℓ_n the clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span, mm

ℓ_w horizontal length of wall, mm

P_{iw} ideal axial load-carrying capacity of a bearing wall designed by 10.4.2, N

s_h centre-to-centre spacing of hoop sets, mm

s_v horizontal spacing of longitudinal reinforcement along the length of a wall, mm

S structural type factor defined by NZS 4203

v_i total shear stress, MPa

ρ_ℓ the ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section $= a_s/bs_v$

ϕ_o ratio of overstrength moment of resistance to moment resulting from code specified loading, where both moments refer to the base section of wall

10.2 Scope. Provisions of this Section shall apply to design of walls subjected to axial load, with or without flexure, and to the design of diaphragms transferring in-plane forces during earthquake loading.

10.3 General principles and requirements

10.3.1 *General design principles*

10.3.1.1 Walls shall be designed for any vertical or lateral in-plane and face loading to which they may be subjected.

10.3.1.2 Proper provisions shall be made for eccentric loads.

10.3.1.3 Unless designed in accordance with this Section, walls subjected to combined flexure and axial loads shall be designed under the provisions of 6.3.1.

10.3.2 *Dimenional limitations*

10.3.2.1 Overall thickness of bearing walls shall not be less than 1/25 the unsupported height or width, whichever is shorter.

10.3.2.2 Bearing walls shall not be less than 150 mm thick for uppermost 4 m of wall height and for each successive 7.5 m downward (or fraction thereof), minimum thickness shall be increased by 25 mm. Bearing walls for two-storey dwellings may be 150 mm thick for total wall height, provided that the compression stress over the gross area of the wall due to factored axial load does not exceed $0.2 f_c'$.

10.3.2.3 Exterior basement walls and foundation walls shall not be less than 150 mm thick.

10.3.2.4 Overall thickness of non-loadbearing wall panels and enclosure walls shall not be less than 100 mm, nor less than 1/30 the distance between supporting or enclosing members.

10.3.2.5 Where bearing walls consist of studs or ribs tied together by other reinforced concrete members at each floor or roof level, such studs or ribs may be considered as columns.

10.3.2.6 Length of wall to be considered as effective for each concentrated load or reaction shall not exceed centre-to-centre distance between loads, nor width of bearing plus four times wall thickness.

10.3.2.7 Limits of thickness and quantity of reinforcement required by 10.3.2 and 5.3.36 respectively may be waived where, instead of the empirical rules of 10.4, structural analysis or test shows adequate strength and stability.

10.3.3 *Anchorage of walls.* Walls shall be anchored to floors, roofs, or columns, pilasters, buttresses, and intersecting walls.

10.3.4 *Foundation walls*

10.3.4.1 Walls designed as foundation beams shall have top and bottom reinforcement as required for moment in accordance with the provisions of 6.3. Design for shear shall be in accordance with provisions of Section 7.

10.3.4.2 Portions of foundation walls exposed above ground also shall meet requirements of 5.3.33 and 10.3.2.

10.3.5 *Ties around vertical reinforcement.* Vertical wall reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

10.4 Principles and requirements additional to 10.3 for walls not designed for seismic loading

10.4.1 *Empirical design.* Gravity load bearing walls may be designed by the empirical provisions of 10.4.2 if resultant of the factored axial load is located within the middle third of the overall thickness of wall and all limits of 10.3.2 are satisfied except as provided by 10.3.2.7.

10.4.2 *Axial load strength.* Ideal axial load strength P_{iw} of a wall, within limitations of 10.4.1, shall be computed by

$$P_{iw} = 0.55 f'_c A_g \left[1 - \left(\frac{\ell_n}{40b}\right)^2\right] \quad \ldots\ldots \text{(Eq. 10-1)}$$

10.5 Principles and requirements additional to 10.3 for walls and diaphragms designed for seismic loading

10.5.1 *General seismic design requirements*

10.5.1.1 Cantilever walls, coupled shear walls and diaphragms 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.

10.5.1.2 In the design of earthquake resisting ductile walls, in accordance with 10.5, the requirements of 3.5.7 shall also be satisfied.

10.5.1.3 Walls with limited ductility shall be designed in accordance with Section 14.

10.5.2 *Dimensional limitations*

10.5.2.1 Unless permitted by 10.5.2.2 and 10.5.2.3 the thickness of any part of structural walls, three storeys or higher, located in the outer half of the plastic hinge region subjected to compression strains by the combination of axial load and flexure due to design load, shall not be less than $\ell_n/10$.

10.5.2.2 When the neutral axis depth computed for the design loading is located within a distance of the lesser of 4 b or 0.3 ℓ_w from the compression edge of a wall section, the requirement of 10.5.2.1 need not be complied with.

10.5.2.3 Any part of a wall that lies within a distance of 3 b from the inside of a continuous line of lateral support, provided by a flange or cross wall, need not satisfy the requirements of 10.5.2.1. The width of the flange providing effective lateral support shall not be less than $\ell_n/5$.

10.5.3 *Longitudinal reinforcement*

10.5.3.1 The ratio ρ_ℓ of longitudinal reinforcement over any part of a wall shall not be less than $0.7/f_y$ nor more than $16/f_y$.

10.5.3.2 In walls thicker than 200 mm or when the design shear stress exceeds $0.3\sqrt{f'_c}$, at least two layers of reinforcement shall be used, one near each side of the wall.

10.5.3.3 The diameter of the bars used in any part of a wall shall not exceed one tenth of the thickness of the wall.

10.5.4 *Transverse reinforcement*

10.5.4.1 The requirements for minimum reinforcement ratio, placing of reinforcement, diameter of transverse bars used and their spacing shall be in accordance with 5.3.36 and as for longitudinal bars in accordance with 10.5.3.2 and 10.5.3.3.

10.5.4.2 Transverse reinforcement shall be provided to resist shear forces resulting from earthquake loading in accordance with 7.3.14.9 and shall be adequately anchored at the wall edges or in boundary elements as required by 5.5.6 for stirrups in beams.

10.5.4.3 In regions of potential compression yielding of the longitudinal reinforcement within a wall with two layers of reinforcement, where the longitudinal reinforcement ratio ρ_ℓ, computed from

$$\rho_\ell = \frac{A_s}{b s_v} \quad \ldots\ldots \text{(Eq. 10-2)}$$

exceeds $2/f_y$, transverse tie reinforcement satisfying the following requirements shall be provided:

(a) Ties suitably shaped shall be so arranged 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. When 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 area of one leg of a tie, A_{te}, in the direction of potential buckling of the longitudinal bar, shall be computed from eq. 6-21 where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tirbutary area of any bars exempted from being tied in accordance with 10.5.4.3 (a). 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) The spacing of ties along the longitudinal bars shall not exceed six times the diameter of the longitudinal bar to be restrained.

10.5.4.4 In areas where the compressive yield strength of the longitudinal bars, required for the ideal strength of the wall section in accordance with the appropriate bending moment envelope, cannot be developed, lateral reinforcement shall be provided as required for beams by 6.4.7.2(b) unless:

(a) For the same load conditions the steel compression stresses will not exceed 0.5 f_y

(b) The wall is exempted from the requirements for transverse reinforcement in accordance with 10.5.4.3.

10.5.4.5 When the neutral axis depth in the potential yield regions of a wall, computed for the appropriate design loading, exceeds

$$c_c = 0.10\,\phi_o\,S\ell_w \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{Eq. 10-3})$$

or the value obtained from more detailed calculation

$$c_c = \frac{8.6\,\phi_o\,S\,\ell_w}{(4-0.7S)\left(17+\dfrac{h_w}{\ell_w}\right)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{Eq. 10-4})$$

the following requirements shall be satisfied in the outer half of that part of the wall section which is subjected to compression strains due to the design loading:

(a) Rectangular or polygonal closed hoops, surrounding longitudinal bars, shall be used as in confined columns so that

$$A_{sh} = 0.3\,s_h h''\left(\frac{A_g^*}{A_c^*}-1\right)\frac{f'_c}{f_{yh}}\left(0.5+0.9\frac{c}{\ell_w}\right) \quad (\text{Eq.10-5})$$

or

$$A_{sh} = 0.12\,s_h h''\frac{f'_c}{f_{yh}}\left(0.5+0.9\frac{c}{\ell_w}\right) \quad . \, . \, . \, . \, . \, . \, (\text{Eq. 10-6})$$

whichever is greater, where the ratio c/ℓ_w need not be taken more than 0.8

(b) Longitudinal bars shall be restrained against possible buckling in accordance with 10.5.4.3 (a)

(c) The centre-to-centre spacing of hoops along longitudinal bars shall not exceed six times the diameter of the longitudinal bar, nor one-half of the thickness of the confined region of the wall, nor 150 mm

(d) The potential yield region of the wall, over which the requirements for hoops in accordance with 10.5.4.5 (a) to (c) is to be satisfied, shall be assumed to extend above the critical section by ℓ_w or 1/6 of height of wall measured to the top of the wall, whichever is larger

(e) Walls with a single layer of reinforcement shall not be used.

10.5.5 *Shear strength*

10.5.5.1 The evaluation of shear strength of, and the determination of shear reinforcement for, walls shall be in accordance with Section 7. For ductile walls, conforming with requirements of Section 10, the shear stress shall not be greater than permitted by 7.3.14.

10.5.5.2 In the end region of ductile walls the shear stress limitations of 7.5.5.2 shall not be exceeded.

10.5.5.3 The height of the end region in walls, for which the special shear stress limitations apply, shall be taken as the length of the wall ℓ_w or 1/6 of the height of the wall, whichever is larger, measured from the section at which the first flexural yielding is expected. The height of the end region need not be taken larger than 2 ℓ_w.

10.5.5.4 Where applicable, ties may be assumed to contribute to the shear strength of a wall element.

10.5.6 *Diaphragms*

10.5.6.1 Diaphragms, intended to transfer earthquake induced horizontal floor forces to primary lateral load resisting elements or which are required to transfer horizontal seismic shear forces from one vertical primary lateral load resisting element to another, shall be designed for the maximum forces that can be resisted by the vertical primary load resisting system, or for forces corresponding with the seismic design coefficients specified by NZS 4203 for parts or portions of buildings, whichever is smaller.

10.5.6.2 Diaphragms shall be reinforced in both directions with not less than the minimum reinforcement required for two-way slabs in accordance with 5.3.32.

10.5.6.3 When it is shown that a diaphragm can introduce forces required to develop the overstrength of the primary lateral load resisting system, without yielding in the diaphragm; or at dependable strength the forces specified in NZS 4203, the special requirements of seismic detailing of the diaphragm for ductility need not be complied with.

10.5.6.4 When the design forces to be transmitted by diaphragms do not lead to the development of the full strength of the primary lateral load resisting system, diaphragms shall be designed in accordance with the requirements of 14.9.

10.5.6.5 Where joints across diaphragms are provided, only the effective area over which interface shear transfer, in accordance with 7.3.11 can occur, shall be considered.

10.5.6.6 Where precast elements are used for floor construction, cast-in-place reinforced concrete topping, at least 50 mm thick, may be used to transfer seismic shear forces through diaphragm action, provided that:

(a) Minimum reinforcement in two directions in accordance with 5.3.32 is placed in the topping slab

(b) When composite action of the precast elements and topping for gravity load is relied on, proper bonding of the cast *in situ* topping to the surface of the precast elements, as required by 8.4.1 for composite members, is assured

(c) When composite action for gravity loading is not relied on, the surface of the precast element is clean and free from laitance, and is intentionally roughened as required by 8.4.3.

(d) When the requirements of 10.5.6.6 (b) or (c) are satisfied and the shear stress due to diaphragm shear transfer by the topping alone exceeds $0.3\sqrt{f'_c}$, or when the surface of the precast element is not specifically prepared for proper bonding:

(1) Ties with an effective area of 40 mm² per m² of floor area, or equivalent connectors, shall connect the topping to the precast element

(2) Spacing of connectors shall not exceed 1500 mm, and the tributary area of topping reliant on each connector shall not exceed 2.25 m²

(3) Connectors shall engage horizontal reinforcement, or shall be otherwise effectively anchored into both the topping and the precast element.

10.5.7 *Walls and diaphragms with openings*

10.5.7.1 Openings in structural walls and diaphragms shall be so arranged that unintentional failure planes across adjacent openings do not reduce the shear or flexural strength of the structure. The behaviour and strength of such walls or diaphragms shall be evaluated by rational analysis.

10.5.7.2 Walls of ductile coupled shear walls shall be connected by ductile coupling beams or diaphragms. In such coupling beams the entire earthquake induced shear and flexure shall be resisted by diagonal reinforcement in both directions unless the earthquake induced shear stress is less than

$$v_i = 0.1\frac{\ell_n}{h}\sqrt{f'_c} \qquad \text{(Eq. 10-7)}$$

The diagonal reinforcement shall be enclosed by rectangular ties or spirals that satisfy the requirements of 10.5.4.3, except that the spacing of the ties or the pitch of the rectangular spiral in ductile coupling beams shall not exceed 100 mm.

10.5.8 *Special splice and anchorage requirements*

10.5.8.1 The splicing of the principal vertical flexural tension reinforcement in potential areas of yielding in walls shall be avoided if possible. Not more than one-third of such reinforcement shall be spliced where yielding can occur.

10.5.8.2 Stagger between splices shall be not less than twice the splice length and at least one leg of a lateral tie, not further than 10 times the diameter of a longitudinal bar, satisfying the requirements of 5.5.1.2, shall surround a splice of bars larger than 16 mm.

10.5.8.3 Approved mechanical and welded splices may be used, but not more than one-half of the reinforcement shall be spliced at one section, and the stagger shall not be less than 500 mm.

10.5.8.4 When three or more diagonal or horizontal bars of a coupling beam are anchored in adjacent structural walls, the development length shall be $1.5\ \ell_d$.

11 TWO-WAY SLAB SYSTEMS

11.1 NOTATION

a larger side of rectangular contact area

b smaller side of rectangular contact area

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

C cross-sectional constant to define torsional properties. See eq. 11-12.

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

E_{cb} modulus of elasticity of beam concrete

E_{cc} modulus of elasticity of column concrete

E_{cs} modulus of elasticity of slab concrete

h overall thickness of member, mm

I_b moment of inertia about centroidal axis of gross section of a beam as defined in 11.8.2.4.

I_c moment of inertia of gross cross-section of column

I_s moment of inertia about centroidal axis of gross section of slab

$=$ $h^3/12$ times width of slab specified in definitions of α and β_t

K_b flexural stiffness of beam; moment per unit rotation

K_c flexural stiffness of column; moment per unit rotation

K_{ec} flexural stiffness of equivalent column; moment per unit rotation. See eq. 11-10

K_s flexural stiffness of slab; moment per unit rotation

K_t torsional stiffness of torsional member; moment per unit rotation

ℓ_n length of clear span, in the direction moments are being determined, measured face-to-face of supports

ℓ_x length of clear span in short direction of rectangular slab

ℓ_y length of clear span in long direction of rectangular slab

ℓ_1 length of span in the direction that moments are being determined, measured centre-to-centre of supports

ℓ_2 length of span transverse to ℓ_1 measured centre-to-centre of supports. See also 11.8.5.3 and 11.8.5.4.

M_o total factored static moment

M_{sx} moment at mid-span or the supports of strips of unit width and span ℓ_x

M_{sy} moment at mid-span or the supports of strips of unit width and span ℓ_y

t thickness of surfacing and filling material

u larger side of rectangular loaded area allowing for load spread

v smaller side of rectangular load area allowing for load spread

w_u total factored uniformly distributed load per unit area $= w_d + w_l$

w_d factored uniformly distributed dead load per unit area

w_ℓ factored uniformly distributed live load per unit area

x shorter overall dimension of a rectangular part of cross-section

y longer overall dimension of a rectangular part of cross-section

α ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centre lines of adjacent panels (if any) on each side of the beam

$$= \frac{E_{cb} I_b}{E_{cs} I_s}$$

α_c ratio of flexural stiffness of the columns above and below the slab to the combined flexural stiffness of the slabs and beams at a joint taken in the direction of the span for which moments are being determined

$$= \frac{\Sigma K_c}{\Sigma (K_s + K_b)}$$

α_{ec} ratio of flexural stiffness of equivalent column to the combined flexural stiffness of the slab and beams at a joint taken in the direction of the span for which moments are being determined

$$= \frac{K_{ec}}{\Sigma (K_s + K_b)}$$

$\alpha_{min.}$ minimum α_c to satisfy 11.8.13.1 (a)

α_1 α in direction of ℓ_1

α_2 α in direction of ℓ_2

β_a ratio of dead load per unit area to live load per unit area (in each case without load factors)

β_{sx}, β_{sy} moment coefficients shown in table 11.1

β_t ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of the beam, centre-to-centre of supports

$$= E_{cb} C / 2 E_{cs} I_s$$

γ_f fraction of unbalanced moment transferred by flexure at slab-column connections. See 11.3.5.

δ_s factor defined by eq. 11-9. See 11.8.13

ρ ratio of tension steel per unit width

11.2 Scope

11.2.1 Provisions of Section 11 shall apply to the design of slab systems reinforced for flexure in more than one direction with or without beams between supports.

11.2.2 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.

11.2.3 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 Section 11.

11.2.4 Slabs with panelled ceilings are included within the scope of Section 11, provided the panel of reduced thickness lies entirely within middle strips, and is not less than two-thirds of the thickness of the remainder of the slab, exclusive of the drop panel, nor less than 100 mm thick.

11.2.5 Minimum thickness of slabs designed in accordance with Section 11 shall be as required by 4.4.1.2 (b).

11.2.6 For the design of prestressed concrete slabs refer to 13.4.2.

11.2.7 The provisions of Section 11 shall apply to the design of slab systems subject predominantly to loading acting at right angles to the plane of the slab.

11.3 Design procedures

11.3.1 *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 by either NZS 4203, or other appropriate loading code, and that all serviceability conditions, including specified limits on deflections, are met.

11.3.2 *Design methods.* The design moments and shears resulting from distributed or concentrated loads shall be determined using either:

(a) Elastic theory for thin plates as in 11.5, or

(b) Limit design theory as in 11.6, or

(c) A detailed procedure as in either 11.7 or 11.8 or 11.9.

11.3.3 *Design for flexure.* The slabs and beams (if any) between supports shall be proportioned for the moments prevailing at every section. Design for flexure shall be in accordance with Section 6 or Appendix B. The range of stresses permitted in the reinforcement due to service live load shall also satisfy the limitation specified under 4.5.1.2 if appropriate.

Design equations in 11.7, 11.8 and 11.9 are given in terms of factored loads and moments, but if the sections are to be designed in accordance with Appendix B the appropriate design loads and moments should be used.

11.3.4 *Effective area of concentrated loads.* The moments induced in slabs by concentrated loads shall 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 be determined according to the following equations:

$$u = \sqrt{(a+2t)^2 + h^2} \qquad \text{(Eq. 11-1)}$$

$$v = \sqrt{(b+2t)^2 + h^2} \qquad \text{(Eq. 11-2)}$$

where the load areas derived from equations 11-1 and 11-2 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.

11.3.5 *Moment transfer between slab and column.* When moment is to be transferred between column and slab without beams, the connection shall be designed for adequate strength using an analysis which considers the strength in bending, torsion and shear at the critical slab sections. In lieu of a special analysis, the following procedure may be used:

(a) A fraction

$$\gamma_f = \frac{1}{1 + 2/3\sqrt{\dfrac{c_1 + d}{c_2 + d}}} \qquad \text{(Eq. 11-3)}$$

of the moment shall be considered to be transferred between the slab and column by flexure over an effective slab width between lines that are one and one-half slab or drop panel thickness (1.5 h) outside opposite faces of the column or capital.

(b) Concentration of reinforcement over the column by closer spacing or additional reinforcement may be used to resist the moment on this effective slab width.

(c) Fraction of the moment not transferred by flexure $(1 - \gamma_f)$ shall be transferred by eccentricity of shear in accordance with 7.3.16.

11.3.6 *Shear transfer between slab and supporting elements*

11.3.6.1 Design for transfer of load from slab to supporting beams, columns or wall through shear and torsion shall be in accordance with Section 7.

11.3.7 *Openings in slabs*

11.3.7.1 Openings of any size may be provided in slab systems if shown by analysis that the design strength is at least equal to the required strength and that all serviceability conditions, including the specified limits on deflections, are met. In lieu of special analysis, openings may be provided in uniformly loaded slab systems without beams only in accordance with the following:

(a) Column strip is a strip with a width each side of the column centre line of 0.25 ℓ_1 or 0.25 ℓ_2, whichever is smaller. Middle strip is a strip bounded by two column strips

(b) Openings of any size may be located in the area common to intersecting middle strips, provided total amount of reinforcement required for the panel without the opening is maintained

(c) In the area common to intersecting column strips, not more than one-eighth of the width of column strip in either span shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening

(d) In the area common to one column strip and one middle strip, not more than one-quarter of the reinforcement in either strip shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening

(e) Shear requirements of 7.3.15.5 shall be satisfied.

11.4 Slab reinforcement

11.4.1 Area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections but shall not be less than required by 5.3.32.

11.4.2 Spacing of principal reinforcement shall not exceed two times the slab thickness, except for portions of slab area that may be of cellular or ribbed construction. In the slab over cellular spaces, reinforcement shall be provided as required by 5.3.5.4.

11.4.3 Positive moment reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns, or walls.

11.4.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, to be developed at face of support according to provisions of Section 5.

11.4.5 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 may be within the slab.

11.4.6 In slabs supported on beams or walls reinforcement should be provided for torsional moments in exterior corners. For the design procedure of 11.7 the reinforcement for torsion shall be as in 11.7.1 (h), (j) and (k). For the design procedures of 11.5, 11.6, 11.8 and 11.9, either a special analysis may be used to determine reinforcement for torsion, or alternatively when α is greater than 1.0 special top and bottom slab reinforcement for torsion shall be provided at exterior corners in accordance with the following:

(a) The special reinforcement in both top and bottom of slab shall be sufficient to resist a moment equal to the maximum positive moment (per unit of width) in the slab

(b) Direction of moment shall be assumed parallel to the diagonal from the corner in the top of the slab and perpendicular to the diagonal in the bottom of the slab

(c) The special reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth of the longer span

(d) In either the top or bottom of the slab, the special reinforcement may be placed in a single band in the direction of the moment or in two bands parallel to the sides of the slab.

11.5 Design moments and shear forces from elastic thin plate theory

11.5.1 The design bending moments, torsional moments and shear forces may be determined assuming that the slabs act as thin elastic plates. The assumptions adopted for computing the flexural and torsional rigidities of sections shall be consistent throughout the analysis.

11.6 Design moments and shear forces from limit design theory

11.6.1 The design moments and shear forces may be determined by a limit design theory such as Johansen's yield line theory or Hillerborg's strip method, provided that the ratio 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 steel ratio ρ used shall not exceed 0.4 of the ratio producing balanced conditions as defined by 6.4.1.2.

11.7 Moment coefficients and loads on supporting beams for uniformly loaded two-way rectangular slabs supported on four sides

11.7.1 *Moment coefficients.* In uniformly loaded rectangular slabs spanning two directions where the corners are prevented from lifting, and where provision of torsion in the slab is made, the following design procedure may be used:

(a) Maximum moments per unit width shall be given by the following equations

$$M_{sx} = \beta_{sx} w_u \ell_x^2 \quad \text{(Eq. 11-4)}$$

$$M_{sy} = \beta_{sy} w_u \ell_x^2 \quad \text{(Eq. 11-5)}$$

where β_{sx} and β_{sy} are coefficients given in table 11.1

(b) Slabs shall be considered as divided in each direction into middle strips and edge strips as shown in fig. 11.1, the middle strip being three-quarters of the total width and each edge strip being one-eighth of the total width

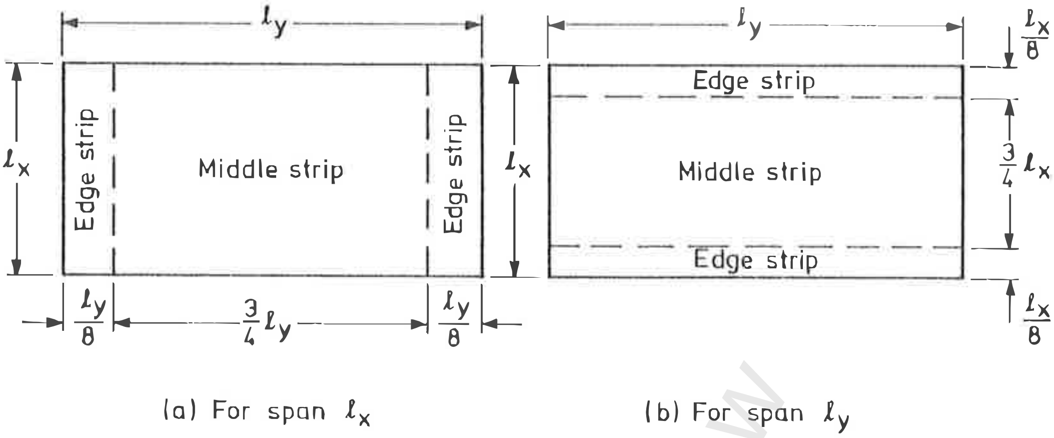


Fig. 11.1 DIVISION OF UNIFORMLY LOADED SLAB INTO MIDDLE AND EDGE STRIPS

(c) The maximum moments calculated above shall apply only to the middle strips and no redistribution of moments shall be used

(d) Tension reinforcement provided at mid-span shall extend in the x and y directions in the lower part of the slab to within at least 0.15 ℓ_x or 0.15 ℓ_y, respectively, of a continuous edge, and in accordance with 11.4.3 at a discontinuous edge

(e) Over the continuous edges the tension reinforcement shall extend in the upper part of the slab a distance of at least 0.15 ℓ_x or 0.15 ℓ_y, as appropriate, from the support, and at least 50% shall extend a distance of at least 0.3 ℓ_x or 0.3 ℓ_y, as appropriate

(f) At a discontinuous edge negative moments may arise, the magnitude depending on the degree of fixity at the edge of the slab. Tension reinforcement equal to at least 50% of that provided at mid-span shall extend in the upper part of the slab a distance of at least 0.1 ℓ_x or 0.1 ℓ_y, as appropriate, from the support into the span

(g) Reinforcement in an edge strip, parallel to that edge, need not exceed the minimum given in 5.3.32, except that the requirements for torsion described in (h), (j) and (k) of this clause need to be complied with at the corners

(h) Torsion reinforcement shall be provided at any corner where the slab is simply supported on 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 required for the maximum mid-span moment in the slab.

(j) Torsion reinforcement equal to half that described in (h) shall be provided at a corner contained by edges over only one of which the slab is continuous

(k) Torsion reinforcement need not be provided at any corner contained by edges over both of which the slab is continuous

(m) Where ℓ_y/ℓ_x is greater than 2, slabs shall be designed as spanning one way only.

11.7.2 *Loads on supporting beams.* The loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads may be assumed to be in accordance with fig. 11.2.

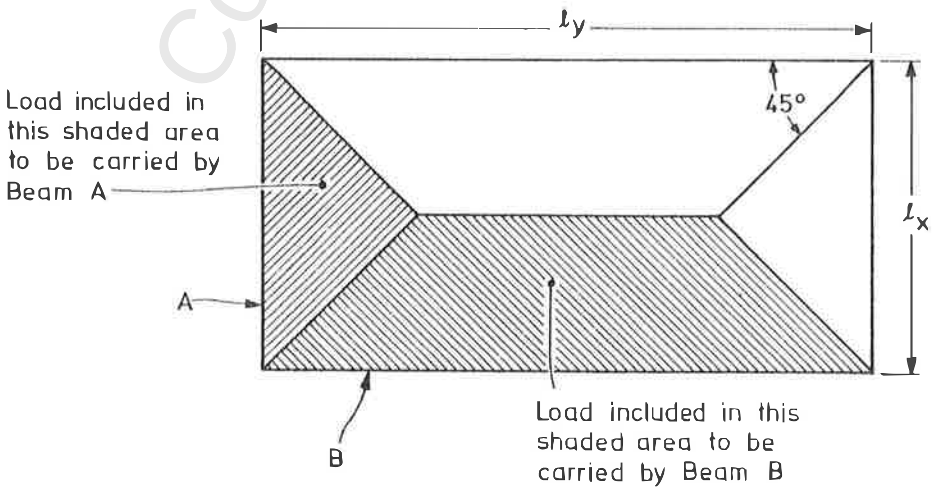


Fig. 11.2 LOAD CARRIED BY SUPPORTING BEAMS OF UNIFORMLY LOADED PANELS

Table 11.1 BENDING MOMENT COEFFICIENTS FOR UNIFORMLY LOADED RECTANGULAR PANELS SUPPORTED ON FOUR SIDES WITH PROVISION FOR TORSION AT CORNERS

Case	*Type of panel and moments considered*	*Short span coefficients β_{sx}*								*Long span coefficients β_{sy} for all values of ℓ_y/ℓ_x*
		Values for ℓ_y/ℓ_x								
		1.0	*1.1*	*1.2*	*1.3*	*1.4*	*1.5*	*1.75*	*2.0*	
1	*Interior panels*									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2	*One short edge discontinuous*									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3	*One long edge discontinuous*									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4	*Two adjacent edges discontinuous*									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5	*Two short edges discontinuous*									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	—
	Positive moment at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6	*Two long edges discontinuous*									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.045
	Positive moment at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7	*Three edges discontinuous (one long edge continuous)*									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	—
	Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8	*Three edges discontinuous (one short edge continuous)*									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.057
	Positive moment at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9	*Four edges discontinuous*									
	Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

11.8 Direct design method for uniformly loaded slab systems with rectangular panels with or without supporting beams or walls

11.8.1 *General.* Design of slab systems by the direct design method shall be based on assumptions given in 11.8.2 to 11.8.13, and all cross-sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

11.8.2 *Definitions*

11.8.2.1 A column strip is a design strip with a width on each side of a column centre line equal to 0.25 ℓ_1 or 0.25 ℓ_2, whichever is less. A column strip includes beams, if any.

11.8.2.2 A middle strip is a design strip bounded by two column strips.

11.8.2.3 A panel is bounded by column, beam, or wall centre lines on all sides.

11.8.2.4 For monolithic or fully composite construction, a beam includes that portion of the slab on each side of the beam measured from the sides of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

11.8.3 *Limitations*

11.8.3.1 Slab systems satisfying all of the requirements of 11.8.3.2 to 11.8.3.9 may be designed by the direct design method.

11.8.3.2 There shall be a minimum of three continuous spans in each direction.

11.8.3.3 Panels shall be rectangular with a ratio of longer to shorter span within a panel not greater than 2.

11.8.3.4 Successive span lengths in each direction shall not differ by more than one-third of the longer span.

11.8.3.5 Columns may be offset a maximum of 10% of the span (in direction of offset) from either axis between centre lines of successive columns.

11.8.3.6 All loads shall be due to gravity only and uniformly distributed over an entire panel. Live load shall not exceed three times dead load.

11.8.3.7 For a panel with beams between supports on all sides, the relative stiffness of beams in two perpendicular directions, the ratio

$$\frac{\alpha_1 \ell_2^{\,2}}{\alpha_2 \ell_1^{\,2}} \qquad \text{(Eq. 11-6)}$$

shall not be less than 0.2 nor greater than 5.0.

11.8.3.8 Moment redistribution as permitted by 3.3.3.4 shall not be applied for slab systems designed by the direct design method. (See 11.8.10.)

11.8.3.9 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 centre line of support a distance not less than one-sixth 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 required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed greater than one-quarter of the distance from edge of drop panel to edge of column or column capital.

11.8.3.10 Variations from the limitations of 11.8.3 may be considered acceptable if demonstrated by analysis that requirements of 11.3.1 are satisfied.

11.8.4 *Slab reinforcement*

11.8.4.1 In addition to the requirements of 11.4, reinforcement in slabs without beams for gravity loading shall comply with the following requirements:

(a) The minimum bend point locations and extensions for reinforcement shall be as prescribed in fig. 11.3

(b) Where adjacent spans are unequal, extension of negative moment reinforcement beyond the face of support as prescribed in fig. 11.3 shall be based on requirements of longer span

(c) Bent bars may be used only when the depth-span ratio permits use of bends 45° or less.

(d) For slabs not braced against sidesway slab reinforcement longer than shown in fig. 11.3 shall be provided when required by analysis.

11.8.5 *Total factored static moment for a span*

11.8.5.1 The total factored static moment for a span shall be determined in a strip bounded laterally by the centre line of the panel on each side of the centre line of supports.

11.8.5.2 The absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{w_u \ell_2 \ell_n^{\,2}}{8} \qquad \text{(Eq. 11-7)}$$

11.8.5.3 Where the transverse span of panels on either side of the centre line of supports varies, ℓ_2 in eq. 11-7 shall be taken as the average of adjacent transverse spans.

11.8.5.4 When the span adjacent and parallel to an edge is being considered, the distance from edge to panel centre line shall be substituted for ℓ_2 in eq. 11-7.

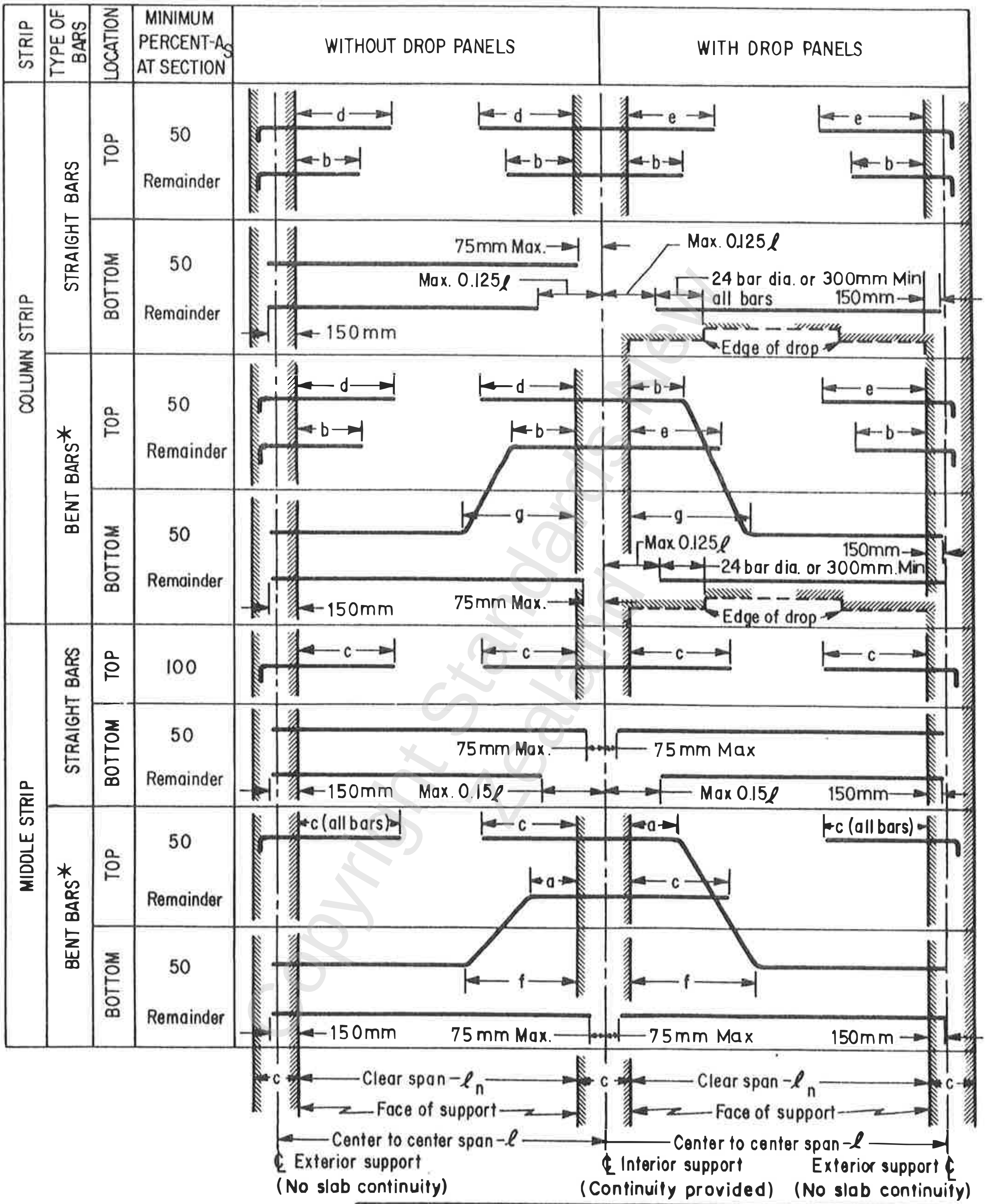
11.8.5.5 Clear span ℓ_n shall extend from face to face of columns, capitals, brackets, or walls. The value of ℓ_n used in eq. 11-7 shall not be less than 0.65 ℓ_1. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

11.8.6 *Negative and positive factored moments*

11.8.6.1 Negative factored moments shall be located at face of rectangular supports. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

11.8.6.2 In an interior span, the total static moment M_o shall be distributed as follows:

Negative factored moment	0.65
Positive factored moment	0.35



* Bent bars at exterior supports may be used if a general analysis is made

	BAR LENGTH FROM FACE OF SUPPORT						
	MINIMUM LENGTH					MAXIMUM LENGTH	
MARK	a	b	c	d	e	f	g
LENGTH	$0.14\ell_n$	$0.20\ell_n$	$0.22\ell_n$	$0.30\ell_n$	$0.33\ell_n$	$0.20\ell_n$	$0.24\ell_n$

Fig. 11.3 MINIMUM BEND POINT LOCATIONS AND EXTENSIONS FOR REINFORCEMENT IN SLABS WITHOUT BEAMS
(see 5.3.25.1 for reinforcement extension into supports)

11.8.6.3 In an end span, the total static moment M_o shall be distributed as follows:

Interior negative factored moment . . . . . . $0.75 - \dfrac{0.10}{1 + \dfrac{1}{\alpha_{ec}}}$

Positive factored moment . . . . $0.63 - \dfrac{0.28}{1 + \dfrac{1}{\alpha_{ec}}}$

Exterior negative factored moment . . . . $\dfrac{0.65}{1 + \dfrac{1}{\alpha_{ec}}}$

where α_{ec} is computed in accordance with 11.9.4 for the exterior column.

11.8.6.4 Negative moment sections shall be designed to resist the larger of the two interior negative factored moments determined for spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with stiffness of adjoining elements.

11.8.6.5 Edge beams or edges of slabs shall be proportioned to resist in torsion their share of exterior negative factored moments.

11.8.7 *Factored moments in column strips*

11.8.7.1 Column strips shall be proportioned to resist the following portions in percent of interior negative factored moments:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	75	75	75
$(\alpha_1\ell_2/\ell_1) \geqslant 1.0$	90	75	45

Linear interpolations shall be made between values shown.

11.8.7.2 Column strips shall be proportioned to resist the following portions in percent of exterior negative factored moments:

ℓ_2/ℓ_1		0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geqslant 2.5$	75	75	75
$(\alpha_1\ell_2/\ell_1) \geqslant 1.0$	$\beta_t = 0$	100	100	100
	$\beta_t \geqslant 2.5$	90	75	45

Linear interpolations shall be made between values shown.

11.8.7.3 Where supports consist of columns or walls extending for a distance equal to or greater than three-quarters of the span length ℓ_2 used to compute M_o, negative moments shall be considered to be uniformly distributed across ℓ_2.

11.8.7.4 Column strips shall be proportioned to resist the following portions in percent of positive factored moments:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	60	60	60
$(\alpha_1\ell_1/\ell_1) \geqslant 1.0$	90	75	45

Linear interpolations shall be made between values shown.

11.8.7.5 For slabs with beams between supports, the slab portion of column strips shall be proportioned to resist that portion of column strip moments not resisted by beams.

11.8.8 *Factored moments in beams*

11.8.8.1 Beams between supports shall be proportioned to resist 85% of column strip moments if $(\alpha_1\ell_2/\ell_1)$ is equal to or greater than 1.0.

11.8.8.2 For values of $(\alpha_1\ell_1/\ell_1)$ between 1.0 and zero, the proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

11.8.8.3 In addition to moments calculated according to 11.8.8.1 and 11.8.8.2, beams shall be proportioned to resist moments caused by loads applied directly on beams.

11.8.9 *Factored moments in middle strips*

11.8.9.1 That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

11.8.9.2 Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

11.8.9.3 A middle strip adjacent to and parallel with an edge supported by a wall shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

11.8.10 *Modification of factored moments*

11.8.10.1 Negative and positive factored moments may be modified by 10% provided the total static moment for a panel in the direction considered is not less than that required by eq. 11-7.

11.8.11 *Factored shear in slab system with beams*

11.8.11.1 Beams with $(\alpha_1\ell_2/\ell_1)$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas bounded by 45° lines drawn from the corners of the panels and the centre lines of the adjacent panels parallel to the long sides.

11.8.11.2 Beams with $(\alpha_1\ell_2/\ell_1)$ less than 1.0 may be proportioned to resist shear obtained by linear interpolation, assuming beams carry no load at $\alpha = 0$.

11.8.11.3 In addition to shears calculated according to 11.8.11.1 and 11.8.11.2 beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

11.8.11.4 Slab shear strength may be computed on the assumption that load is distributed to supporting beams in accordance with 11.8.11.1 or 11.8.11.2. Resistance to total shear occurring on a panel shall be provided.

11.8.11.5 Shear strength shall satisfy requirements of Section 7.

11.8.12 *Factored moments in columns and walls*

11.8.12.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

11.8.12.2 At an interior support, supporting elements above and below the slab shall resist the moment specified by eq. 11-8 in direct proportion to their stiffness unless a general analysis is made:

$$M = \frac{0.08\left[(w_d + 0.5 w_\ell)\,\ell_2 \ell_n^{\,2} - w'_d \ell'_2\,(\ell'_n)^2\right]}{1 + \dfrac{1}{\alpha_{ec}}} \quad \text{......(Eq. 11-8)}$$

where w'_d, ℓ'_2 and ℓ'_n refer to shorter span.

11.8.13 *Provisions for effects of pattern loadings*

11.8.13.1 Where ratio β_a of dead load to live load is less than 2, one of the following conditions shall be satisfied:

(a) Sum of flexural stiffnesses of the columns above and below the slab shall be such that α_c is not less than $\alpha_{min.}$ specified in table 11.2

Table 11.2 VALUES OF $\alpha_{min.}$

β_a	*Aspect ratio* ℓ_2/ℓ_1	*Relative beam stiffness, α*				
		0	*0.5*	*1.0*	*2.0*	*4.0*
2.0	0.5 – 2.0	0	0	0	0	0
1.0	0.5	0.6	0	0	0	0
	0.8	0.7	0	0	0	0
	1.0	0.7	0.1	0	0	0
	1.25	0.8	0.4	0	0	0
	2.0	1.2	0.5	0.2	0	0
0.5	0.5	1.3	0.3	0	0	0
	0.8	1.5	0.5	0.2	0	0
	1.0	1.6	0.6	0.2	0	0
	1.25	1.9	1.0	0.5	0	0
	2.0	4.9	1.6	0.8	0.3	0
0.33	0.5	1.8	0.5	0.1	0	0
	0.8	2.0	0.9	0.3	0	0
	1.0	2.3	0.9	0.4	0	0
	1.25	2.8	1.5	0.8	0.2	0
	2.0	13.0	2.6	1.2	0.5	0.3

(b) If α_c for the columns above and below the slab is less than $\alpha_{min.}$, specified in table 11.2, positive factored moments in panels supported by such columns shall be multiplied by the coefficient δ_s determined from eq. 11-9:

$$\delta_s = 1 + \left(\frac{2 - \beta_a}{4 + \beta_a}\right)\left(1 - \frac{\alpha_c}{\alpha_{min.}}\right) \quad \text{. . . . . . (Eq. 11-9)}$$

where β_a is ratio of dead load to live load, per unit area (in each case without load factors).

11.9 Equivalent frame method for uniformly loaded slab systems with rectangular panels with or without beams

11.9.1 *General*

11.9.1.1 Design of slab systems by the equivalent frame method shall be based on assumptions given in 11.9.1 to 11.9.6, and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

11.9.1.2 The method shall only be used for slab systems loaded predominantly by loading acting at right angles to the plane of the slab.

11.9.1.3 Where steel column capitals are used, account may be taken of their contributions to stiffness and resistance to moment and to shear.

11.9.1.4 Change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected.

11.9.1.5 Drop panels, where used, shall conform to 11.8.3.9.

11.9.1.6 Slab reinforcement shall be in accordance with 11.9.8.

11.9.2 *Equivalent frame*

11.9.2.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building.

11.9.2.2 Each frame shall consist of a row of equivalent columns or supports and slab-beam strips, bounded laterally by the centre line of the panel on each side of the centre line of columns or supports.

11.9.2.3 Frames adjacent and parallel to an edge shall be bounded by that edge and the centre line of the adjacent panel.

11.9.2.4 Each equivalent frame may be analysed in its entirety, or for vertical loading, each floor and the roof (slab-beams) may be analysed separately with far ends of columns considered fixed.

11.9.2.5 Where slab-beams are analysed separately, it may be assumed in determining moment at a given support that the slab-beam is fixed at any support two panels distant therefrom, provided the slab continues beyond that point.

11.9.3 *Slab-beams*

11.9.3.1 Moment of inertia of slab-beams at any cross-section outside of joints or column capitals may be based on the gross area of concrete.

11.9.3.2 Variation in moment of inertia along axis of slab-beams shall be taken into account.

11.9.3.3 Moment of inertia of slab-beams from centre of column to face of column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at face of column, bracket, or capital divided by the quantity $(1 - c_2/\ell_2)^2$ where c_2 and ℓ_2 are measured transverse to the direction of the span for which moments are being determined.

11.9.4 *Equivalent columns*

11.9.4.1 An equivalent column shall be assumed to consist of the actual columns above and below the the slab-beam plus an attached torsional member (see 11.9.5) transverse to the direction of the span for which moments are being determined and extending to bounding lateral panel centre lines on each side of the column.

11.9.4.2 Flexibility (inverse of stiffness) of an equivalent column shall be taken as the sum of the flexibilities of the actual columns above and below the slab-beam and the flexibility of the attached torsional member as expressed by eq. 11-10:

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_c} + \frac{1}{K_t} \quad \text{. . . . . . . (Eq. 11-10)}$$

11.9.4.3 In computing stiffness of columns K_c, moment of inertia of columns at any cross-section outside of joints or column capitals may be based on the gross area of concrete.

11.9.4.4 Variation in moment of inertia along the axis of columns shall be taken into account.

11.9.4.5 Moment of inertia of columns shall be assumed infinite from top to bottom of the slab-beam at a joint.

11.9.5 *Attached torsional members*

11.9.5.1 Attached torsional members shall be assumed to have a constant cross-section throughout their length consisting of the larger of:

(a) A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined

(b) For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab

(c) Transverse beam as defined in 11.8.2.4.

11.9.5.2 Stiffness K_t of an attached torsional member shall be calculated by the following expression:

$$K_t = \Sigma \frac{9E_{cs}C}{\ell_2\left(1 - \dfrac{c_2}{\ell_2}\right)^3} \quad \text{. . . . . . . (Eq. 11-11)}$$

where c_2 and ℓ_2 relate to the transverse spans on each side of the column.

11.9.5.3 The constant C in eq. 11-11 may be evaluated for the cross-section by dividing it into separate rectangular parts and carrying out the following summation:

$$C = \Sigma\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3} \quad \text{. . . . . . . (Eq. 11-12)}$$

11.9.5.4 Where beams frame into columns in the direction of the span for which moments are being determined, the value of K_t as computed by eq. 11-10 shall be multiplied by the ratio of moment of inertia of the slab with such beam to moment of inertia of the slab without such beam.

11.9.6 *Arrangement of live load*

11.9.6.1 When the loading pattern is known, the equivalent frame shall be analysed for that load pattern.

11.9.6.2 When live load is variable but does not exceed three-quarters of the dead load, or the nature of live load is such that all panels will be loaded simultaneously, maximum factored moments may be assumed to occur at all sections with full factored live load on entire slab system.

11.9.6.3 For loading conditions other than those defined in 11.9.6.2, maximum positive factored moment near midspan of a panel may be assumed to occur with three-quarters of the full factored live load on the panel and on alternate panels; and maximum negative factored moment in the slab at a support may be assumed to occur with three-quarters of the full factored live load on adjacent panels only.

11.9.6.4 Factored moments shall not be taken less than those occurring with full factored live load on all panels.

11.9.7 *Factored moments*

11.9.7.1 At interior supports, the critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not greater than 0.175 ℓ_1 from centre of a column.

11.9.7.2 At exterior supports provided with brackets or capitals, the critical section for negative factored moment in the span perpendicular to an edge shall be taken at a distance from the face of supporting element not greater than one-half of the projection of the bracket or capital beyond the face of the supporting element.

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

11.9.7.4 Slab systems within limitations of 11.9.2, when analysed by the equivalent frame method, may have resulting computed moments reduced in such proportion that the absolute sum of the positive and average negative moments used in design need not exceed the value obtained from eq. 11-7.

11.9.7.5 Moments at critical sections across the slab-beam strip of each frame may be distributed to column strips, beams, and middle strips as provided in 11.8.7, 11.8.8 and 11.8.9.

11.9.7.6 Moments determined for the equivalent columns in the frame analysis shall be used in design of the actual columns above and below the slab-beams.

11.9.8 *Slab reinforcement*

11.9.8.1 In addition to the requirements of 11.4 bend point locations and extensions for reinforcement shall be determined from the bending moments derived from the analysis.

11.9.8.2 In meeting the requirements of 11.9.8.1, it may be assumed that the free bending moment diagram in each of the column and middle strips is of the same shape as the free total bending moment diagram derived in the analysis.

11.9.8.3 In lieu of the more detailed calculations of 11.9.8.2, the provisions of 11.8.4 and fig. 11.3, or of 11.7.1, as appropriate, may be applied.

12 FOUNDATIONS

12.1 Notation

A_g gross area of section, mm^2

A_s area of non-prestressed reinforcement, mm^2

b width of compression face of member, mm

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

d_p diameter of pile at footing base, mm

$\sqrt{f_c'}$ square root of specified compressive strength of concrete, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

β_c ratio of short side to long side of foundation

ρ_t ratio of non-prestressed tension reinforcement A_s/bd

ϕ strength reduction factor. See 4.3.1.

12.2 Scope

12.2.1 Provisions of Section 12 shall apply for design of isolated foundations and to combined footings and mats. In addition, basic principles for piles are also given.

12.2.2 Additional requirements for design of combined foundations and mats are given in 12.3.9.

12.3 General principles and requirements

12.3.1 *Loads and reactions*

12.3.1.1 Foundations shall be proportioned to resist the design loads and induced reactions, in accordance with the appropriate design requirements of this Code and as provided in this Section.

12.3.1.2 External forces and moments resulting from factored loads applied to foundations shall be transferred to supporting soil without exceeding allowable soil pressures.

12.3.1.3 For foundations on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at pile centre.

12.3.1.4 The foundation system of all structures (which comprises all parts of the foundation below the superstructure and includes the supporting soil) shall be carried to depths such that adequate bearing is secured. Due allowance shall be made for the effects of seasonal weather changes, lateral movement of the ground and movements of ground in unstable areas.

12.3.1.5 When combinations of deep and shallow foundations are necessary, differential settlement and torsional effects shall be considered.

12.3.1.6 Base area of foundation or number and arrangement of piles shall be determined from the external forces and moments resulting from factored loads (transmitted by foundation to soil or piles) and permissible soil pressure or permissible pile capacity selected through principles of soil mechanics.

12.3.2 *Foundations supporting circular or regular polygon shaped columns or pedestals*

12.3.2.1 Circular or regular polygon shaped concrete columns or pedestals may be treated as square members with the same area for location of critical sections for moment, shear, and development of reinforcement in foundations.

12.3.3 *Moment in foundations*

12.3.3.1 External moment on any section of a foundation shall be determined by passing a vertical plane through the foundation and computing the moment of the forces acting over the entire area of foundation on one side of that vertical plane.

12.3.3.2 Maximum design moment for an isolated foundation shall be computed as prescribed in 12.3.3.1 at critical sections located as follows:

(a) At face of column, pedestal, or wall, for foundations supporting a concrete column, pedestal, or wall

(b) Halfway between middle and edge of wall, for foundations supporting a masonry wall

(c) Halfway between face of column or pedestal and edge of steel base, for foundations supporting a pedestal with steel base plates.

12.3.3.3 In one-way foundations, and two-way square foundations, reinforcement shall be distributed uniformly across entire width of foundation.

12.3.3.4 In two-way rectangular foundations, reinforcement shall be distributed as follows:

(a) Reinforcement in long direction shall be distributed uniformly across the entire width of foundation

(b) For reinforcement in short direction, a portion of the total reinforcement given by eq. 12-1 shall be distributed uniformly over a band width (centred on centreline of column or pedestal) equal to the length of short side of foundation. Remainder of reinforcement required in short direction shall be distributed uniformly outside centre band width of foundation.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2\beta_c}{(\beta_c + 1)} \quad \text{. . . . . . . (Eq. 12-1)}$$

12.3.4 *Shear in footings*

12.3.4.1 Computation of shear in foundations shall be in accordance with Section 7.

12.3.4.2 Location of critical section for shear in accordance with Section 7 shall be measured from face of column, pedestal, or wall, for foundations supporting a column, pedestal, or wall. For foundations supporting a column or pedestal with steel base plates, the critical section shall be measured from the location defined in 12.3.3.2 (c).

12.3.4.3 Computation of shear on any section through a foundation supported on piles shall be in accordance with the following:

(a) Entire reaction from any pile whose centre is located $d_p/2$ or more outside the section shall be considered as producing shear on that section

(b) Reaction from any pile whose centre is located $d_p/2$ 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 full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

12.3.5 *Development of reinforcment in foundations*

12.3.5.1 Computation of development of reinforcement in foundations shall be in accordance with Section 5.

12.3.5.2 Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by proper embedment length, end anchorage, hooks (tension only), or combination thereof.

12.3.5.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in 12.3.3.2 for maximum design moment, and at all other vertical planes where changes of section or reinforcement occur. See also 5.5.

12.3.6 *Minimum foundation depth.* Depth of foundation above bottom reinforcement shall not be less than 150 mm for foundations on soil, nor less than 300 mm for foundations on piles.

12.3.7 *Transfer of force at base of column or reinforced concrete pedestal*

12.3.7.1 All forces and moments applied at base of column or pedestal shall be transferred to top of supporting pedestal or foundation by bearing stress on concrete and by reinforcement. If the required loading conditions include uplift, the total force shall be resisted by reinforcement.

12.3.7.2 Lateral forces shall be transferred to foundations by shear keys, or other means.

12.3.7.3 Bearing stress on concrete at contact surface between supporting and supported member shall not exceed concrete bearing strength for either surface as given in 6.3.5.

12.3.7.4 Reinforcement shall be provided across interface between supporting and supported member either by extending longitudinal bars into supporting member, or by dowels.

12.3.7.5 Reinforcement across interface shall be sufficient to satisfy both of the following:

(a) Reinforcement shall be provided to transfer all force that exceeds concrete bearing strength in supporting or supported member

(b) Area of reinforcement shall not be less than 0.005 times gross area of supported member, with a minimum of four bars.

12.3.7.6 For transfer of force by reinforcement, development of reinforcement in supporting and supported member shall be in accordance with Section 5.

12.3.8 *Sloped or stepped foundations*

12.3.8.1 In sloped or stepped foundations, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section.

12.3.8.2 Sloped or stepped foundations designed as a unit shall be constructed to assure action as a unit.

12.3.9 *Combined foundations and mats*

12.3.9.1 Foundations supporting more than one column, pedestal, or wall (combined foundations or mats) shall be proportioned to resist the design loads and induced reactions, in accordance with appropriate design requirements of this Code.

12.3.9.2 The design method of Section 11 shall not be used for design of combined foundations and mats.

12.3.9.3 Distribution of soil pressure under combined foundations and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

12.3.10 *Plain concrete pedestals and foundations*

12.3.10.1 Maximum compressive stress in plain concrete pedestals and foundations on soil shall not exceed concrete bearing stress as given in 6.3.5. Where concrete bearing stress is exceeded, reinforcement shall be provided and the member designed as a reinforced concrete member.

12.3.10.2 Design ideal stresses in plain concrete pedestals and foundations on soil shall not exceed the following:

Flexure	– extreme fibre stress in tension	$0.30\sqrt{f'_c}$
Shear	– beam action	$0.17\sqrt{f'_c}$
	– two-way action	$0.33\sqrt{f'_c}$

12.3.10.3 Plain concrete shall not be used for foundations on piles.

12.3.10.4 Depth of plain concrete foundations shall not be less than 200 mm.

12.4 Principles and requirements additional to 12.3 for members not designed for seismic loading

12.4.1 *General*

12.4.1.1 The design of the foundation system shall be based upon conventional methods of design.

12.4.1.2 Due consideration shall be given to long-term settlements to ensure that the service ability of the super-structure is not impaired by cracking or movement.

12.4.1.3 For piled foundation systems the shell or casing of a pile may be considered as providing a proportion of the strength of the pile. However, for steel casing due allowance shall be made for loss of wall thickness by corrosion.

12.5 Principles and requirements additional to 12.3 for members designed for seismic loading

12.5.1 *Designing for ductility*

12.5.1.1 The foundation design shall be in accordance with the provisions of this Code using the factored loading specified in NZS 4203, or other appropriate loadings code.

12.5.1.2 The foundation system shall maintain its ability to support the design gravity loads while maintaining the chosen earthquake energy dissipating mechanisms in the structure.

12.5.1.3 All members shall comply with the additional principles and requirements for members designed for seismic loadings as set down in the relevant Sections of this Code. However, flexural members other than piles which have an ideal strength not less than the greatest total seismic load that can be transmitted to them from the super-structure, need not comply with these requirements.

12.5.2 *Potential plastic hinge regions*

12.5.2.1 The upper end of every pile shall be reinforced as a potential plastic hinge region, except where it can be established that there is no possibility of any significant moments in the pile resulting from either movement of the structure relative to the ground or from ground deformation.

12.5.2.2 The potential plastic hinge region of a pile shall be considered to be the end region adjacent to the moment resisting connection and extending from the underside of the pile cap over a length of not less than either the longest cross-section of the pile or 450 mm.

12.5.3 *Reinforcement in piles*

12.5.3.1 Minimum longitudinal reinforcement ratio, ρ_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, ρ_t shall not be less than $2.2/f_y$

(b) For piles having a cross-sectional area, A_g, equal to or greater than 2×10^6 mm^2, ρ_t shall not be less than $1.1/f_y$

(c) For piles having a cross-sectional area, A_g, between 0.5×10^6 mm^2, and 2×10^6 mm^2, ρ_t shall not be less than given by eq. 12-1

$$\rho_{t,\ \mathrm{min.}} = \frac{2200}{f_y \sqrt{2A_g}} \quad \ldots\ldots\ (\text{Eq. 12-1})$$

12.5.3.2 Maximum longitudinal reinforcement ratio, ρ_t, in potential plastic hinge regions, shall be as specified for columns in 6.5.4.2 (b), and shall be arranged as specified for columns in 6.5.4.2 (c).

12.5.3.3 Transverse reinforcement in piles shall comply with the following:

(a) In potential plastic hinge regions of piles, the transverse reinforcement shall be as required for columns and piers, as defined in 6.5.4.3

(b) Adjacent to the potential plastic hinge regions and for a distance equal to the greatest of three pile diameters or the length of the potential plastic hinge region, transverse reinforcement shall be as required for columns and piers, as defined in Section 6.5.4.3 (d) (1)

(c) Over the remainder of the pile the centre-to-centre spacing of transverse reinforcement shall not exceed d, unless the pile-soil interaction characteristics determine a lesser spacing

(d) Transverse reinforcement placed in accordance with (a), (b) and (c) above extending around longitudinal bars in the compression and tension faces of the member cross-section may be assumed to contribute fully to the shear strength of the member.

13 PRESTRESSED CONCRETE

13.1 Notation

a depth of equivalent rectangular stress block, mm

A area of that part of the cross-section between the flexural tension face and the centre of gravity of the gross section

A_c area of concrete at the cross-section considered

A_{ps} area of prestressed reinforcement in tension zone, mm²

A_s area of non-prestressed tension reinforcement, mm²

A'_s area of compression reinforcement, mm²

b width of compression face of member, mm

B the ratio of the reduction in resistance moment to the numerically largest moment. Refer 13.3.10.1 (d)

c distance from extreme compression fibre to neutral axis, mm

d distance from extreme compression fibre to centroid of prestressing steel, or to combined centroid when non-prestressing tension reinforcement is included, mm

D dead loads as defined by NZS 4203

e base of Napierian logarithm

E earthquake loads as defined by NZS 4203

f'_c specified compressive strength of concrete, MPa

f'_{ci} compressive strength of concrete at time of initial prestress, MPa

f_{ps} calculated stress in prestressing steel at design load, MPa

f_{pu} ultimate strength of prestressing steel, MPa

f_{py} specified yield strength of prestressing steel, MPa or the 0.2% proof stress

f_{se} effective stress in prestressing steel, after losses, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

h overall thickness of member, mm

K wobble friction coefficient per m of prestressing steel

L_R reduced live loads as defined by NZS 4203

ℓ length of prestressing steel element from jacking end to any point x, length of span in two way flat plates in the direction parallel to that of the reinforcement being determined, mm

N_c tensile force in the concrete under service load

P_e design axial load in compression with given eccentricity due to gravity and seismic loading acting on the member simultaneously with shear stress v_i during an earthquake

P_i ideal axial load strength at zero eccentricity

P_o axial load strength of member when the external load is applied without eccentricity, that is, when uniform strain exists across section

P_s prestress at jacking end

P_x prestress at any point

U required strength in accordance with appropriate design loadings code

α total angular change of prestressing steel profile in radians from jacking end to any point x

μ curvature friction coefficient

ρ A_s/bd ratio of non-prestressed tension reinforcement

ρ' A'_s/bd

ρ_p A_{ps}/bd ratio of prestressed reinforcement

ϕ strength reduction factor. (See 4.3.1.2)

13.2 Scope

13.2.1 Provisions in this Section apply to structural members prestressed with high strength steel meeting the requirements for prestressing steels of the Concrete Construction Code NZS 3109.

13.2.2 The following provisions of the Code shall not apply to prestressed concrete unless specifically noted: Clauses 3.3.3.4, 3.3.6.2 (a), 3.3.6.2 (b), 3.3.6.2 (c), 3.4.2, 6.4.1.2, 6.4.2, 6.4.3, 6.4.6 and Section 11. All other provisions of this Code shall apply.

13.3 General principles and requirements

13.3.1 *General considerations*

13.3.1.1 Members shall meet the strength and serviceability requirements specified in this Code. Design shall be based on strength and on behaviour at service conditions at all stages that may be critical during the life of the structure from the time the prestress is first applied.

13.3.1.2 Stress concentrations due to the prestressing shall be considered in the design.

13.3.1.3 The effects on parts of the structure or adjoining structure of elastic and plastic deformation and deflections shall be provided for. The effects of temperature, creep, and shrinkage shall be considered.

13.3.1.4 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

13.3.1.5 In calculations of section properties prior to bonding of tendons the effect of loss of area due to open ducts shall be considered. In pre-tensioned members and in post-tensioned members after grouting, section properties may be based on gross sections, net sections, or effective sections using transformed areas of bonded tendons and reinforcing steel.

13.3.1.6 Where tendons are subjected to deviations, the forces caused by these deviations shall be allowed for.

13.3.1.7 Reinforcement for shrinkage and temperature stresses normal to the direction of prestress shall be provided, where appropriate, in accordance with 5.3.32.

13.3.2 *Basic assumptions*

13.3.2.1 In designing for strength, the assumptions provided in 6.3.1 shall apply.

13.3.2.2 In investigating sections as service loads, after transfer of prestress and at cracking load, elastic theory shall be used with the following assumptions:

(a) Strains vary linearly with depth through the entire load range

(b) At any section where the permissible concrete tensile stresses are exceeded the section shall be assumed to be cracked and to have no tension capacity in any part of that concrete section.

13.3.2.3 For prestressed members ideal axial load strength P_i shall not be taken greater than 0.85 or 0.80 of the ideal axial load strength at zero eccentricity P_o for members with spiral and tie reinforcement respectively.

13.3.3 *Unbonded tendons*:

(a) The use of unbonded tendons is permitted provided they are in accordance with NZS 3109 and the environment is not aggressive except that for bridge structures special approval will be required for their use

(b) Bonded reinforcement shall be provided in accordance with 13.3.8

(c) Design strength shall be computed in accordance with 13.3.6.

13.3.4 *Permissible stresses in steel at jacking*:

(a) Due to jacking force at any stage of the stressing operation . . . . $0.80 f_{pu}$ or $0.94 f_{py}$
whichever is the smaller, but not greater than the maximum value recommended by the manufacturer of the steel or of the anchorages

(b) Pretensioning tendons immediately after transfer, or post-tensioning tendons immediately after anchoring . . . . $0.70 f_{pu}$

(c) Under service load the requirements of 13.4.1.1 (b) shall be satisfied.

13.3.5 *Loss of prestress*

13.3.5.1 To determine the effective prestress, allowance for the following sources of loss of prestress shall be considered:

(a) Anchorage seating
(b) Elastic shortening of concrete
(c) Creep of concrete
(d) Shrinkage of concrete
(e) Relaxation of steel stress
(f) Frictional loss due to intended or unintended curvature in the tendons..

13.3.5.2 Friction losses in post-tensioned steel shall be based on experimentally determined wobble and curvature coefficients, and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongations, shall be defined. These friction losses shall be calculated as follows:

$$P_x = P_s e^{-(K\ell + \mu\alpha)} \qquad \text{(Eq. 13-1)}$$

When $(K\ell + \mu\alpha)$ is not greater than 0.3, eq. 13-2 may be used:

$$P_x = \frac{P_s}{(1 + K\ell + \mu\alpha)} \qquad \text{(Eq. 13-2)}$$

13.3.5.3 When prestress in a member may be reduced through its connection with adjoining elements, such reduction shall be allowed for in the design.

13.3.6 *Flexural strength*

13.3.6.1 Flexural strength of members shall be computed by the strength design methods given in this Code. For prestressing steel, f_{ps} shall be substituted for f_y. In lieu of a more precise determination of f_{ps}, and provided that f_{se} is not less than 0.5 f_{pu}, the following approximate values shall be used:

(a) Bonded prestressing steel

$$f_{ps} = f_{pu} \left(1 - 0.5 \rho_p \frac{f_{pu}}{f'_c}\right) \qquad \text{(Eq. 13-3)}$$

(b) Unbonded prestressing steel

$$f_{ps} = f_{se} + 100 \qquad \text{(Eq. 13-4)}$$

13.3.6.2 Non-prestressed reinforcement, when used in combination with prestressed steel, may be considered to contribute to the tensile force in a member at design load moment, a force equal to its area times its yield stress.

13.3.7 *Size and number of prestressing tendons*

13.3.7.1 The maximum number and size of prestressing tendons shall be governed by strain compatibility calculations performed in accordance with the assumptions listed in 6.3.1.

13.3.7.2 The minimum number and size of prestressing tendons and non-prestressed reinforcement shall be adequate to develop a design moment in flexure greater than the moment at the cracking moment of the concrete when allowance is made for likely variations in prestress and strength of materials. In the absence of special studies, it may be assumed that the maximum concrete tensile stress prior to cracking is 1.0 $\sqrt{f'_c}$ provided allowance is made for a variation of 10% in the calculated level of prestress at the section under consideration.

13.3.8 *Minimum bonded reinforcement where unbonded tendons are used*

13.3.8.1 Except for two-way flat plates defined as solid slabs of uniform thickness, the minimum amount of bonded reinforcement, A_s, in members containing unbonded prestressing tendons shall be:

$$A_s = 0.004A \quad \text{(Eq. 13-5)}$$

The bonded reinforcement shall be uniformly distributed over the pre-compressed tension zone as close as practicable to the extreme tension fibre. This bonded reinforcement shall be required regardless of the service load stress condition.

13.3.8.2 In two-way flat plates containing unbonded prestressing tendons, the minimum amount and distribution of bonded reinforcement, A_s shall be as follows:

(a) Bonded reinforcement shall not be required in positive moment areas where the concrete tensile stress at service load, after all prestress losses, does not exceed $0.17\sqrt{f_c'}$

(b) In positive moment areas, where the concrete tensile stress at service load is greater than $0.17\sqrt{f_c'}$ the minimum area of bonded reinforcement, A_s shall be

$$A_s = \frac{N_c}{0.5 f_y} \quad \text{(Eq. 13-6)}$$

and f_y shall not exceed 410 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

$$A_s = 0.00075\,hl \quad \text{(Eq. 13-7)}$$

where l is the length of the span in the direction parallel to that of the reinforcement being determined.

The bonded reinforcement shall be distributed within a slab width between lines that are 1.5 h outside opposite column faces and shall be spaced not greater than 300 mm and not less than four bars or wires shall be provided in each direction.

13.3.8.3 Bonded reinforcement required by 13.3.8.1 and 13.3.8.2 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. The reinforcement shall be centred in the positive moment area

(c) When bonded reinforcement is required for flexural strength in accordance with 13.3.6 or for tensile stress conditions in 13.3.8.2 (b) the length of this reinforcement shall also conform to the provisions of Section 5.

13.3.9 *End regions*

13.3.9.1 Reinforcements shall be provided when required in the anchorage zone to resist bursting, horizontal splitting, and spalling forces induced by the tendon anchorages. Regions of abrupt change in section shall be adequately reinforced.

13.3.9.2 End blocks shall be provided when required for end bearing or for distribution of concentrated prestressing forces.

13.3.9.3 Post-tensioning anchorages and the supporting concrete shall be designed to support the maximum jacking load at the concrete strength at time of prestressing, and the end anchorage region shall be designed to develop at least 95% of the guaranteed ultimate tensile strength of the tendons, or the calculated tensile force whichever is the greater with a value of $\phi = 0.90$.

13.3.10 *Redistribution of design moments*

13.3.10.1 Redistribution of the design moments obtained by elastic analysis may be carried out in accordance with all the following provisions:

(a) Equilibrium between the internal forces and the external loads shall be maintained under each appropriate combination of design loads

(b) The design strength after redistribution provided at any section of a member shall be not less than 80% of the moment for that section obtained from an elastic moments envelope covering all appropriate combinations of design loads

(c) The elastic moment at any section in a member due to a particular combination of design loads shall not be reduced by more than 20% of the numerically largest moment given anywhere by the elastic moments envelope for that particular member, covering all appropriate combinations of design load

(d) Where, as a result of redistribution, the design strength of a section is reduced, and where special confining steel has not been provided in the concrete compression zone,

$$c \leqslant (0.5 - B)\,d \quad \text{(Eq. 13-8)}$$

where B is the ratio of the reduction in resistance moment to the numerically largest moment given anywhere by the design elastic analysis for that particular member covering all appropriate combinations of design load

(e) No moment redistribution is permitted for service loads.

13.3.10.2 The moments to be used in design shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to design dead and live loads including redistribution as permitted in 13.3.10.1.

13.3.11 *Shear strength.* The requirements of 7.3.3 shall be satisfied.

13.4 Principles and requirements additional to 13.3 for members not designed for seismic loading

13.4.1 *Serviceability*

13.4.1.1 *General.* In general the requirement for adequate performance under service load will be achieved either by designing on the basis of the homogeneous or uncracked section or by performing an analysis based on the cracked section in accordance with 13.3.2:

(a) When the section is analysed on the basis of the uncracked (that is, homogeneous) section the flexural concrete stresses shall be not more than appropriate values in table 13.1

(b) When the section is analysed on the basis of the cracked section in accordance with the principles of 13.3.2 bonded steel (whether non-prestressed or prestressing steel) must be present. For this case the compressive stresses in the concrete and the range of stress (that is the total maximum variation from compression to tension for non-prestressed steel and the increase in tension for prestressing steel) in the bonded steel shall be not more than the appropriate values in table 13.2. In addition the maximum tension in the prestressing steel at transfer and in the final service condition after all design losses shall not exceed $0.70f_{pu}$.

(c) The exception for which the stress limits of table 13.2 do not apply and special studies are mandatory are:
   (1) Two way slab system
   (2) Structures that are located in an aggressive environment.

13.4.1.2 *Crack widths.* The crack width limits of 4.4.2 shall be satisfied. The spacing of tendons or a combination of tendons and non-prestressed reinforcing shall not exceed the maximum spacing allowed for reinforcing bars in 5.3.5.

13.4.1.3 *Deflections.* The deflection limits of the appropriate general design code (refer 4.4.1.4) shall be satisfied. For calculations of deflection it is permissible to assume that the section is uncracked provided the concrete tensile flexural stress is less than $1.0\sqrt{f'_c}$. Refer 4.4.1.3 (c).

13.4.1.4 *Frequently repetitive loads.* For members subject to frequently repetitive loads the possibility of inclined diagonal tension cracks forming under lesser stresses than under static loading shall be taken into account in the design.

13.4.1.5 *Prestress variation.* In all cases where curvature occurs in post-tensioning tendons allowance shall be made for variation of the tendon force at any point of ±5% from the calculated values.

13.4.2 *Slab systems.* Prestressed slab systems reinforced for flexure in more than one direction shall be analysed and designed by a method which will account for column stiffnesses, rigidity of slab-column connection, and for the effect of prestressing in accordance with 13.3.1. Approximate moment coefficients developed for reinforced concrete slab design are not applicable.

13.4.3 *Compression members – Combined axial load and bending*

13.4.3.1 Members with average prestress of 1.5 MPa or higher shall be subject to the other provisions of this Clause. Members with average prestress less than 1.5 MPa shall have minimum reinforcement in accordance with Section 5. Average prestress is defined as the total effective prestress force divided by the gross area of the concrete.

13.4.3.2 *Combined axial load and bending.* Prestressed concrete members under combined axial load and bending, with or without non-prestressed reinforcement, shall be proportioned by the strength design method given in this Code for members without prestressing. The effects of prestress, shrinkage, and creep shall also be included and where appropriate serviceability shall be verified. The minimum amounts of reinforcement specified in Section 5 may be waived where average prestress is over 1.5 MPa and a structural analysis shows adequate strength and stability.

13.4.3.3 *Transverse reinforcement.* Except for walls, all prestressing steel shall be enclosed by transverse reinforcement conforming to 6.4.7.1 or 6.4.7.2. Transverse reinforcement shall be located at not more than one-half a spacing above the floor or footing and at not more than one-half a spacing below the lowest horizontal reinforcement in the slab or drop panel above. Where beams or brackets provide enclosure on all sides of the column, the transverse reinforcement may be terminated not more than 75 mm below the lowest reinforcement in such beams or brackets.

13.5 Principles and requirements additional to 13.3 for prestressed members designed for seismic loading

13.5.1 *General.* This Section covers the design of prestressed and partially prestressed concrete members of ductile moment resisting frames and joints between such members.

13.5.2 *Concrete:*

(a) f'_c shall not exceed 55 MPa unless special transverse reinforcement in accordance with 6.5.4.3 is provided in potential plastic hinge regions

(b) Post-tensioned tendons in moment resisting frame members shall be grouted, except as allowed by 13.5.5.3.

Table 13.1 PERMISSIBLE CONCRETE STRESSES IF SECTION DESIGNED ON THE BASIS THAT IT IS UNCRACKED

Stresses in MPa

Stress case	*Load category*			
	I Immediately after transfer before time dependent losses	*II* Permanent loads plus variable loads of long duration, or permanent loads plus frequently repetitive loads	*III* Specified service loads for buildings where load category II does not apply	*IV* Permanent loads plus infrequent combinations of transient loads
Compression	$0.6\,f'_{ci}$	$0.4\,f'_c$	$0.45\,f'_c$	$0.55\,f'_c$
Tension across construction joints	zero	zero	zero	zero
Tension in monolithic concrete	$0.5\sqrt{f'_{ci}}$	zero	$0.5\sqrt{f'_c}$	$0.5\sqrt{f'_c}$

Table 13.2 PERMISSIBLE CONCRETE COMPRESSIVE STRESSES AND STEEL STRESS RANGE FOR CRACKED SECTION DESIGN

Stresses in MPa

Stress case	*Load category*			
	I Immediately after transfer before time dependent losses	*II* Permanent loads plus variable loads of long duration, or permanent loads plus frequently repetitive loads	*III* Specified service loads for buildings where load category II does not apply	*IV* Permanent loads plus infrequent combinations of transient loads
Compression	$0.6\,f'_{ci}$	$0.4\,f'_c$	$0.45\,f'_c$	$0.55\,f'_c$
Steel stress range	200	100	200	200

13.5.3 *Design of beams*

13.5.3.1 Dimensions of prestressed beams shall be in accordance with the provisions of 6.5.2.

13.5.3.2 Provided the limits to flexural steel are in accordance with 13.5.3.3, elastically derived bending moments may be redistributed in accordance with the provisions of 3.5.3.4.

13.5.3.3 The content of prestressed plus non-prestressed flexural steel shall be such that at the flexural capacity of sections in potential plastic hinge zones

$$a/h \leqslant 0.2 \qquad \text{(Eq. 13-9)}$$

unless special transverse reinforcement is provided in accordance with 6.5.4.3, in which case

$$a/h \leqslant 0.3 \qquad \text{(Eq. 13-10)}$$

13.5.3.4 The effective flexural steel near column faces in T and L beams built integrally with slabs shall be subject to the provisions of 6.5.3.2 (e) (1) to (4) inclusive. In all cases at least 75% of the tensile force capacity in each face, providing the required flexural capacity, shall be provided by steel passing through, or anchored in, the column core. When longitudinal steel is governed by the load combination $U = 1.4D + 1.7L_R$, then only 75% of the tensile force capacity required for the load combination $U = D + 1.3L_R + E$ is required to be provided by steel passing through or anchored in the column core.

13.5.3.5 Stirrup ties shall be provided in potential plastic hinge zones in accordance with the provisions of 6.5.3.3.

In all cases in potential plastic hinge zones the stirrup ties shall not be less than 10 mm diameter and the distance between vertical legs of stirrup ties across the section shall not exceed 200 mm between centres in each set of stirrup ties.

13.5.4 *Design of columns*

13.5.4.1 Design of prestessed columns in moment resisting frames shall be subject to the provisions of 6.5.1 and 6.5.2 and to the additional requirements of 13.5.4.2, 13.5.4.3, and 13.5.4.4.

13.5.4.2 The flexural strength of a column section shall be greater than the maximum likely column cracking moment, as calculated in accordance with 13.3.6 including the effect of axial load and prestress; except that this requirement shall not apply if cracking of the column does not occur under the action of the maximum anticipated flexural demand.

13.5.4.3 Special transverse reinforcement in accordance with 6.5.4.3 shall be provided

13.5.4.4 Longitudinal prestressed or non-prestressed steel in potential plastic hinge regions shall not be spaced further apart between centres than one-third of the length of the section dimension in that direction, but need not be less than 200 mm apart. Where longitudinal reinforcing bars are also utilized as vertical shear reinforcement in beam-column joint cores the distribution of bars shall be in accordance with 9.5.5.5.

13.5.4.5 *Shear strength requirements.* Shear strength requirements shall be in accordance with the provisions of Section 7. At sections of beams and columns designed to provide ductility by plastic hinging, the value of P_e in eq. 7-41 shall include the prestress force, after losses, of only those tendons situated within the central third of the section depth. At sections away from potential plastic hinge regions, P_e may include the prestress force, after losses, of all tendons at the critical section.

13.5.5 *Joints in prestressed frames*

13.5.5.1 Anchorages for post-tensioned tendons shall not be placed within beam-column joint cores.

13.5.5.2 Except as provided by 13.5.5.3 the beam prestressing tendons which pass through joint cores shall be placed 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.

13.5.5.3 When partially prestressed beams are designed in which the non-prestressed reinforcement provides at least 80% of the design resisting 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 anchorage failure, or cable de-tensioning, cannot occur under seismic loads.

13.5.5.4 Ducts for post-tensioned grouted tendons through beam-column joints shall be corrugated, or shall provide equivalent bond characteristics.

13.5.5.5 Connections between precast members at beam-column joints shall be acceptable 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.

13.5.5.6 Design of joint reinforcement shall be in accordance with the provisions of 9.5.

14 SEISMIC REQUIREMENTS FOR STRUCTURES OF LIMITED DUCTILITY

14.1 Notation

A_g gross area of section, mm²

A_g^* gross area of concrete located between the compressive edge of the section and a line 0.2 h therefrom, mm²

A_s^* area of all longitudinal reinforcement within A_g^*, mm²

A_{sh} total effective area of hoop bars and supplementary cross-ties in direction under consideration within the spacing, s_h, mm²

E earthquake load specified in NZS 4203 or other appropriate loadings code

f'_c specified compressive strength of concrete, MPa

$\sqrt{f'_c}$ square root of specified compressive strength of concrete, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

f_{yh} specified yield strength of hoop or supplementary cross-tie reinforcement, MPa

h overall depth of member, mm

m $f_y/0.85f'_c$

M_e moment resulting from loading combination U, involving earthquake loads, N mm

M_e^* M_e referred to mid-depth of the section, N mm

M_{eq} moment associated with E, N mm

M_i ideal flexural strength, N mm

P_e design axial load, due to loading U, involving earthquake loads, N

R_c reduction factor for confinement

s_h spacing of hoops or supplementary cross-tie reinforcement sets, mm

S structural type factor, specified in NZS 4203 and in 14.4.1

U design load combination specified in NZS 4203 or other appropriate loadings code

For M_e, M_e^*, V_e and P_e, the combinations are those involving earthquake loads

v_c ideal shear stress provided by the concrete, MPa

V_e shear force resulting from loading combination U, involving earthquake loads, N

V_{eq} shear force associated with E, N

V_i ideal shear strength, N

γ strength parameter used for confinement criteria

ρ^* ratio of all longitudinal reinforcement within A_g^*, equal to A_s^*/A_g^*

ϕ strength reduction factor

14.2 Scope

14.2.1 Section 14 covers the design and detailing requirements for members in structures of limited ductility subjected to earthquake induced loading.

14.2.2 The requirements of this Section relate specifically to members for which design actions have been determined using the limited ductility design procedure specified in NZS 4203 or equivalent procedures specified in other appropriate loadings code.

14.2.3 Structures designed by the elastic response design procedure specified in NZS 4203 or equivalent procedures specified in other appropriate loadings code need not comply with the requirements of this Section.

14.3 Interpretation

14.3.1 *General.* Except as otherwise provided in this Section, the "General principles and requirements" and the "Additional principles and requirements for members not designed for seismic loading" specified in other Sections of this Code apply to members designed to the provisions of this Section, but the "Additional principles and requirements for members designed for seismic loading" specified in other Sections of this Code, with the exception of the relevant clauses of Section 3, do not apply.

14.3.2 *Bridge structures.* The design of bridge structures shall comply with the additional requirements of Section 14.10.

14.3.3 *Alternative design and detailing*

14.3.3.1 Notwithstanding the provisions for design and detailing specified in this Section, nothing in this Section shall prevent the adoption of the seismic requirements of other Sections of this Code, provided that the design and detailing of each structural element so designed and detailed complies consistently with those requirements.

14.3.3.2 Where the requirements of other Sections of this Code are applied in accordance with 14.3.3.1, and it is established that the design strength furnished is at least as great as is implied for structures of limited ductility as covered by this Section, then the detailing requirements for reinforcement specified in other Sections of this Code may be waived, and the detailing requirements for reinforcement specified in this Section substituted.

14.4 General design considerations

14.4.1 *Structural type factors*

14.4.1.1 The structural type factor, S, used in design shall be as specified in NZS 4203 or other appropriate loadings code.

14.4.1.2 Where not specifically identified in the appropriate loadings code, the value of S used in design shall be taken as the ratio between the equivalent lateral static force specified for structures of limited ductility, to that specified for fully ductile structures.

14.4.1.3 For bridges, the intent of 14.4.1.2 shall be deemed to be met if the loadings used in design are derived from lateral force coefficients consistent with an assumed displacement ductility factor of three or less.

14.4.1.4 In buildings of more than four storeys measured to the uppermost principal concrete floor or roof:

(a) Frames shall be the subject of a special study for the selection of the appropriate structural type factor

(b) For walls or piers with openings, the structural type factor shall be interpolated in a rational manner between that value specified for uniform walls without openings and that required for frames in accordance with (a).

14.4.1.5 When a penetrated wall or a system of coupled walls can be rationalized into two or more walls, with or without openings, through the exclusion of elements by the assumed removal of flexural continuity, then:

(a) The residual structure shall be designed to resist the entire lateral seismic load;

(b) Interpolation, as required in 14.4.1.4 (b), may be applied to each resulting wall separately; and

(c) Elements so excluded shall be designed as secondary elements in accordance with 3.5.14.

14.4.2 *Required strengths*

14.4.2.1 Shear strengths provided shall have a suitable margin over required flexural strengths, as derived from the structural type factor used for flexural design in end regions, such that

$$\phi V_i \geqslant V_e + (\frac{4}{S} - 1)\, V_{eq} \quad \text{. . . . . . (Eq. 14-1)}$$

14.4.2.2 Unless design and detailing, as specified in this Section for the end regions, is employed throughout the structure, flexural strength provided outside of the designated end regions shall be such that

$$\phi M_i \geqslant M_e + (\frac{3}{S} - 1)\, M_{eq} \quad \text{. . . . . . (Eq. 14-2)}$$

14.4.2.3 Where redistribution of moments in accordance with 3.5.3.4 is carried out, the seismic actions to which 14.4.2.1 or 14.4.2.2 apply may be assumed to be either:

(a) Those resulting from an elastic analysis of the structure, before redistribution, or

(b) Those derived from a general analysis taking the effects of redistribution into account.

14.4.3 *Capacity design and design for concurrency.* Except as specifically required by 14.3.3 in meeting the alternative requirements of other Sections of this Code, capacity design or design for concurrent earthquake effects from loadings in two principal directions is not required for structures or structural elements designed to the requirements of this Section.

14.5 End regions

14.5.1 *Designation of end regions*

14.5.1.1 Regions of potential flexural hinging, the end regions, shall be identified by the designer and suitably designed and detailed to the provisions of this Section.

14.5.1.2 Beyond the designated end regions, the additional seismic requirements of this Section, with respect to detailing only, need not be complied with.

14.5.2 *Extent of end regions*

14.5.2.1 For any member in which the provisions of 14.4.2.2 are not met, the extent of the end region shall be assumed to be the full length of the member.

14.5.2.2 For each member in which the provisions of 14.4.2.2 are met, the extent of the end region shall be as follows:

(a) For walls, including those which are analysed with all beams framing into them treated as secondary in accordance with 3.5.14 and 14.4.1.5, the height of the end region, measured from the base of the wall, shall be assumed to be equal to the greater of the horizontal length of the wall or one-sixth of the height of the wall

(b) For columns, the height of the end region, measured from the face of intersecting beams, shall be assumed to be equal to the overall depth of the column

(c) For columns complying with the provisions for walls in (a), the height of the end region may be assumed to be as specified for walls in (a)

(d) For beams, the length of the end region, measured from the face of the intersecting columns, shall be assumed to be equal to the overall depth of the beam

(e) For piles, the length of the end region, measured from the underside of the pile cap, shall be assumed to be the greater cross-sectional dimension of the pile, or the pile diameter, as appropriate.

14.6 Design for flexure and axial load

14.6.1 *General*

14.6.1.1 Design for flexure, with or without axial load, shall comply with the provisions of Section 6, within the terms of 14.3.

14.6.1.2 Within the end region, the requirements of 14.6.2 apply.

14.6.1.3 Beyond the end region, the requirements of 14.4.2.2 apply, but 14.6.2 does not apply.

14.6.2 *Confinement within the end regions*

14.6.2.1 Where the value of γ, calculated in accordance with eq. 14-3, exceeds 1.0, members with a single layer of reinforcement shall not be used, and that part of the member cross-section defined by A_g^* shall be confined in accordance with 14.6.2.2 and 14.6.2.3. In no case shall γ exceed 3.0

$$\gamma = \frac{M_e^* + 0.3 P_e h}{0.6\,\phi f_c' A_g^* h} \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{Eq. 14-3})$$

14.6.2.2 Where γ exceeds 1.0, the area of transverse reinforcement, A_{sh}, shall be at least that given by

$$A_{sh} = R_c\,(0.02\, s_h h f_c'/f_{yh}) \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{Eq. 14-4})$$

where R_c is given by

$$0 \leqslant R_c = \left[\frac{\gamma}{1 + \rho^* m} - 1\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{Eq. 14-5})$$

in which $\rho^* m$ shall be such that R_c is equal to or less than 1.0.

14.6.2.3 The reinforcement A_{sh} shall consist of hoops or supplementary cross-ties or both arranged as specified for flexural members in 6.4.4, except that the spacing of hoops or supplementary cross-ties or both along the longitudinal reinforcement placed within the area A_g^* shall not exceed ten times the diameter of such longitudinal reinforcement.

14.6.3 *Lateral tying of longitudinal reinforcement*

14.6.3.1 Notwithstanding that γ, as computed from eq. 14-3 is less than unity, all longitudinal reinforcement within A_g^* in the end region shall be tied as required by 14.6.2.3 except as provided in 14.6.3.2.

14.6.3.2 For the end regions of walls, the requirements of 14.6.3.1 need only be met when there are two layers of reinforcement and the ratio of longitudinal reinforcement exceeds $3/f_y$.

14.6.4 *Longitudinal reinforcement in the end regions of walls*

14.6.4.1 The ratio of longitudinal reinforcement over any part of the cross-section shall be not less than $0.7/f_y$, nor more than $16/f_y$.

14.6.4.2 The diameter of bars used shall not exceed one-eighth of the wall thickness at the bar locality.

14.7 Design for shear

14.7.1 *Design strength.* The shear strength provided throughout the structure, including joints and foundations, shall satisfy the requirements of 14.4.2.1.

14.7.2 *Shear stress provided by concrete*

14.7.2.1 Within the end region, v_c may be assumed to be one-half that specified in 7.3.2 or 7.3.14 as appropriate, but need not be assumed to be less than

$$v_c = 0.4\sqrt{(P_e/A_g)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{Eq. 14-6})$$

14.7.2.2 Beyond the end region the shear stress provided by the concrete, v_c, may be assumed as that specified in 7.3.2 or 7.3.14, as appropriate.

14.7.3 *Shear reinforcement within the end region*

14.7.3.1 The maximum spacing of shear reinforcement within the end regions shall not exceed one-quarter of the effective depth of the member.

14.7.3.2 Lateral reinforcement provided in accordance with 14.6.2 or 14.6.3, when satisfying the requirements for shear reinforcement, may be assumed to contribute fully to the required shear strength.

14.8 Special provisions for slabs and face loaded walls

14.8.1 Face loaded walls providing the lateral seismic resistance to the structure shall be designed as slabs or as columns as appropriate as specified herein.

14.8.2 Where the maximum design axial compressive stress on the wall cross-section exceeds $0.1 f_c'$, the wall shall be designed as a column, but otherwise may be designed as a slab.

14.8.3 Where the requirements for columns apply, in accordance with 14.8.2, or where the applied shear stress exceeds the value of v_c specified in 14.7.2.1, walls with a single layer of reinforcement shall not be used.

14.8.4 Where the requirements for slabs apply, or where they may be applied to walls in accordance with 14.8.2, the provisions relating to transverse reinforcement in the end region, as specified in 14.6.2 and 14.6.3, need not be complied with.

14.8.5 In all cases the provisions for longitudinal reinforcement specified in 14.6.4 shall be met.

14.8.6 In all other respects, slabs and face loaded walls shall comply with the requirements of this Section.

14.9 Special provisions for diaphragms

14.9.1 *Type 1 diaphragms*

14.9.1.1 Type 1 diaphragms are diaphragms intended to transfer earthquake induced horizontal floor forces to primary lateral load resisting elements. They shall be designed as provided in this clause.

14.9.1.2 Type 1 diaphragms shall be designed for a shear strength satisfying 14.4.2.1.

14.9.1.3 Type 1 diaphragms shall be designed as provided for walls in this Section, except when it can be shown that the diaphragm can introduce loads corresponding to the elastic response design procedure into the lateral load resisting system.

In applying the provisions relating to walls, the whole diaphragm shall be designed and detailed as required for end regions.

14.9.2 *Type 2 diaphragms.* Type 2 diaphragms are those required to transfer horizontal seismic shear forces from one vertical primary lateral load resisting element to another. They shall be designed for loadings consistent with the elastic reponse design procedure. In this case the provisions for walls specified in this Section need not be applied.

14.9.3 *General requirements*

14.9.3.1 These general requirements apply to type 1 and type 2 diaphragms.

14.9.3.2 The loading on a diaphragm shall not be less than that corresponding to the seismic design coefficients specified in NZS 4203 for parts and portions of buildings.

14.9.3.3 Diaphragms shall be reinforced in both directions as two-way slabs, satisfying the requirements of 5.3.32.

14.9.3.4 Where joints across diaphragms are provided, only the effective area over which interface shear transfer, in accordance with 7.3.11, can occur, shall be considered.

14.9.3.5 Where precast elements are used for floor construction, cast-in-place reinforced concrete topping, at least 50 mm thick, may be used to transfer seismic shear forces through diaphragm action, provided that:

(a) Minimum reinforcement in two directions in accordance with 5.3.32 is placed in the topping slab

(b) Composite action of the precast elements and topping for gravity load is relied on, and proper bonding of the cast *in situ* topping to the surface of the precast elements, as required by 8.4.1.4 for composite members, is assured

(c) Composite action for gravity loading is not relied on, but the surface of the precast element is clean and free from laitance and is intentionally roughened

(d) When the requirements of 14.9.3.5 (b) or (c) are satisfied and the shear stress due to diaphragm shear transfer by the topping alone exceeds $0.3 \sqrt{f_c'}$ or when the surface of the precast element is not specifically prepared for proper bonding:

(1) Ties with an effective area of 40 mm² per m² of floor area, or equivalent connectors, shall connect the topping to the precast element

(2) Spacing of connectors shall not exceed 1500 mm, and the tributary area of topping reliant on each connector shall not exceed 2.25 m²

(3) Connectors shall engage horizontal reinforcement, or shall be otherwise effectively anchored into both the topping and the precast elements.

14.10 Special requirements for bridges

14.10.1 *Form.* The designer shall choose a structural form with as predictable behaviour as is feasible.

14.10.2 *Foundations.* Except where it can be shown that deformation of foundation members occurs in a predictable and ductile manner, all primary gravity load resisting foundation members shall be designed with a suitable margin of strength over the forces corresponding to yield of the superstructure.

14.10.3 *Members resisting forces from bearings*

14.10.3.1 The dependable flexural and shear strength of members resisting the shear forces from sliding bearings shall have a suitable margin over the forces corresponding to the maximum likely coefficient of friction. Adequate account shall be taken of any additional moments which may be induced in the member as a result of earthquake actions along both major axes of the structure concurrently.

14.10.3.2 The dependable flexural and shear strength of members resisting the shear forces from elastomeric bearings shall have a suitable margin over the forces in those members derived from the elastic analysis for the design loadings.

14.10.4 *Displacements*

14.10.4.1 At points where relative movements between structural components are possible during earthquake response, such as at deck movement joints, clearances shall be provided of size not less than the relative deflection derived from the elastic analysis for the design loading.

14.10.4.2 Where appropriate, rubber buffers, or other devices, shall be used to reduce impact forces which may occur between major structural components during a severe earthquake.

14.10.5 *Structural integrity*

14.10.5.1 Positive horizontal linkage shall be provided between adjacent sections of superstructure at supports and hinges, and between superstructures and their supporting abutments.

14.10.5.2 Holding down devices shall be provided at all supports and hinges where horizontal deflections of the superstructure can cause an appreciable reduction in the gravity load reaction between superstructure and bearings.

15 STRENGTH EVALUATION OF EXISTING STRUCTURES

15.1 Notation

D dead loads, or their related internal moments and forces

h overall thickness of member, mm

ℓ_t span of member under load test (the shorter span of flat slabs and of slabs supported on four sides). The span, except as provided in 15.4.2.9, is the distance between the centres of the supports or the clear distance between the supports plus the depth of the member, whichever is the smaller, mm

L design live loads, including impact where appropriate, or their related internal moments and forces

L_R reduced live loads as defined by NZS 4203; or their related internal moments and forces

δ maximum deflection under test load of a member relative to a line joining the ends of the span, or of the free end of a cantilever relative to its support, mm.

15.2 Strength evaluation – General

15.2.1 If doubt develops concerning the safety of a structure or member, or if information is required about the load capacity of a structure in service for purposes of fixing load limits, the Engineer may order a structural strength investigation by analysis or by means of load tests, or by a combination of analyses and load tests.

15.3 Analytical investigations

15.3.1 *Field investigation.* If the strength evaluation is by analysis, a thorough field investigation shall be made of the dimensions and details of the members, properties of the materials, and other pertinent conditions of the structure as actually built.

15.3.2 *Requirements of Code to be met.* The analyses based on the investigation required by 15.3.1 shall satisfy the Engineer that the load factors or permissible stresses and design loads meet the requirements and intent of this Code or other requirements specified by the highway authority. See also 15.6.

15.4 Load tests

15.4.1 *General*

15.4.1.1 If the strength evaluation is by load tests, a qualified engineer acceptable to the Engineer shall supervise such tests.

15.4.1.2 Critical components shall be identified by analysis prior to testing. Analysis shall include, in particular, a check on shear strength of members.

15.4.1.3 A load test shall not be made until every portion of the structure to be subject to load is at least 56 days old, except that when the owner of the structure, the contractor, and all involved parties mutually agree, the test may be made at an earlier age.

15.4.1.4 When only a portion of a structure is to be load tested, the questionable portion shall be load tested in such a manner as to adequately test the suspected source of weakness.

15.4.1.5 Forty-eight hours prior to the application of the test load, a load to simulate the effect of that portion of the dead loads not already acting shall be applied and shall remain in place until all testing has been completed.

15.4.2 *Load tests of flexural members in buildings*

15.4.2.1 When flexural members in buildings, including beams and slabs, are load tested, the additional provisions of 15.4.2 shall apply.

15.4.2.2 Base readings (datum for deflection measurements) shall be made immediately prior to the application of the test load.

15.4.2.3 That portion of the building selected for loading shall be subjected to a total load, including the dead loads already acting, equivalent to 0.85 $(1.4D + 1.7L_R)$.

15.4.2.4 The test load shall be applied in not less than four approximately equal increments without shock to the structure and in such a manner as to avoid arching of the loading materials.

15.4.2.5 After the test load has been in position for twenty-four hours, initial deflection readings shall be taken.

15.4.2.6 The test load shall be removed immediately after the initial deflection readings have been taken and the final deflection readings shall be taken twenty-four hours after the removal of the test load.

15.4.2.7 If the portion of the building tested shows visible evidence of failure, the portion tested shall be considered to have failed the test and no retesting of the previously tested portion shall be permitted.

15.4.2.8 If the portion of the building tested shows no visible evidence of failure, either of the following criteria shall be taken as indication of satisfactory behaviour:

(a) The measured maximum deflection δ of a beam, floor or roof is less than $l_t^2/20{,}000h$, or

(b) The deflection recovery within twenty-four hours after removal of the test load is at least 75% of the maximum deflection.

15.4.2.9 In 15.4.2.8 (a) l_t for cantilevers shall be taken as twice the distance from the support to the cantilever end, and the deflection shall be adjusted for any support movement.

15.4.3 *Load tests of flexural members of bridges*

15.4.3.1 When flexural members of bridges, including beams and slabs, are load tested, the additional provisions of 15.4.3 shall apply.

15.4.3.2 Base readings (datum for deflection measurements) shall be made immediately prior to the application of the test load.

15.4.3.3 That portion of the structure selected for loading shall be subjected to a total load, including the dead loads already acting, equivalent to $D + 1.4L$.

15.4.3.4 The test load shall be applied in not less than four approximately equal increments without shock to the structure and in such a manner as to avoid arching of the loading materials.

15.4.3.5 Deflection readings shall be taken as soon as possible after each load increment has been added. The time interval between adding the load and reading the deflection shall be approximately the same for each increment.

15.4.3.6 The test load shall be removed immediately after the initial deflection readings have been taken and the final deflection readings shall be taken within one hour of the removal of the whole of the test load.

15.4.3.7 If the portion of the structure tested shows visible evidence of failure, the portion tested shall be considered to have failed the test and no retesting of the previously tested portion shall be permitted.

15.4.3.8 If the portion of the structure tested shows no visible evidence of failure, and the following conditions are all satisfied, the structure shall be considered satisfactory:

(a) The maximum flexural crack widths at the peak test load do not exceed 1.5 times the appropriate values listed in table 4.3

(b) The deflection at the maximum test load is not more than 20% greater than the deflection obtained by extrapolating the straight part of the load/deflection line to the maximum load

(c) The deflection recovery within one hour after removal of the total load is at least 75% of the deflection under the maximum test load.

15.4.4 *Retesting of flexural members*

15.4.4.1 Non-prestressed concrete construction failing to show 75% recovery of deflection as required by either 15.4.2.8 (b) or 15.4.3.8 (c) as applicable may be retested not earlier than 72 h after removal of the first load. The portion of the structure shall be considered satisfactory if:

(a) The portion of the structure tested shows no visible evidence of failure in retest, and

(b) Deflection recovery caused by the second test load is at least 80% of the maximum deflection in the second test.

15.4.4.2 Prestressed concrete construction shall not be retested.

15.4.5 *Subsequent inspection of tested structures.* Bridges which have been test loaded shall be inspected at regular intervals, in particular for changes in crack widths and deflection.

15.5 Members other than flexural members

15.5.1 Members other than flexural members shall preferably be investigated by analysis.

15.6 Provision for a lower load rating

15.6.1 If the structure under investigation does not satisfy the conditions or criteria of 15.3.2, 15.4.2.8 or 15.4.3.8 as applicable, the Engineer may approve a lower load rating for that structure based on the results of the load test or analysis.

15.7 Safety

15.7.1 Load tests shall be conducted in such a manner as to provide for safety of life and structure during the test.

15.7.2 No safety measures shall interfere with load test procedures or affect results.

APPENDIX A

NOTATION

a depth of equivalent rectangular stress block (see 6.3.1.7), mm – Section 6 and 13

a shear span, distance between concentrated load and face of support or as defined in 7.3.12.3 – Section 7

a larger side of rectangular contact area – Section 11

A average effective area of concrete in tension around each reinforcing bar, calculated from the effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, mm^2 – Section 4

A area of that part of the cross-section between the flexural tension face and the centre of gravity of the gross section – Section 13

A_b area of an individual bar, mm^2 – Section 5

ΣA_b sum of area of individual bars, mm^2 – Section 10

ΣA_b sum of area of longitudinal bars, mm^2 – Section 6

A_c area of concrete core of section measured to outside of peripheral spiral or hoop, mm^2 – Section 6

A_c area of concrete at the cross-section considered – Section 13

A_c^* area of concrete core extending over the outer half of the neutral axis depth which is subjected to compression, measured to outside of peripheral hoop legs, mm^2 – Section 10

A_{co} area enclosed by perimeter of section, mm^2 (see 7.3.7) – Section 7

A_{ct} area of reinforcement to resist a tensile force in corbels, mm^2 – Section 7

A_f area of flexural tension reinforcement required in corbels, mm^2 – Section 7

A_g gross area of section, mm^2 – Sections 4, 5, 6, 7, 9, 10, 12, 14, and Appendix B

A_g^* gross area of concrete section extending over outer half of the neutral axis depth which is subjected to compression, mm^2 – Section 10

A_g^* gross area of concrete located between the compressive edge of the section and a line 0.2 h therefrom, mm^2 – Section 14

A_h area of shear reinforcement in brackets and corbels parallel to flexural tension reinforcement, mm^2 – Section 7

A_{jh} total area of effective horizontal joint shear reinforcement, mm^2 – Section 9

A_{jv} total area of effective vertical joint shear reinforcement, mm^2 – Section 9

A_l total area of longitudinal reinforcement to resist torsion, mm^2 – Section 7

A_o area enclosed by the line connecting the centres of the longitudinal bars in the corners of the closed stirrups, mm^2 (see 7.3.7) – Section 7

A_{ps} area of prestressed reinforcement in tension zone, mm^2 – Sections 7 and 13

A_s area of non-prestressed tension reinforcement, mm^2 – Sections 4, 6, 7, 9, 12 and 13

A_s	area of longitudinal wall reinforcement spaced horizontally at s_v, mm^2 – Section 10
A'_s	area of compression reinforcement, mm^2 – Sections 4, 6 and 13
A'_s	area of non-prestressed compression beam reinforcement, mm^2 – Section 9
A^*_s	area of longitudinal reinforcement within A^*_g, mm^2 – Section 14
A_{sc}	area of non-prestressed tension reinforcement in one face of the column section, mm^2 – Section 9
A'_{sc}	area of non-prestressed compression reinforcement in one face of the column section, mm^2 – Section 9
A_{sh}	total effective area of hoop bars and supplementary cross-ties in direction under consideration within spacing s_h, mm^2 – Sections 6, 10 and 14
A_{sp}	area of flexural reinforcement provided, mm^2 – Section 5
A_{sr}	area of required flexural reinforcement, mm^2 – Section 5
A_{st}	total area of longitudinal reinforcement (bars or steel shapes), mm^2 – Section 6
A_t	area of bar formed into spiral reinforcement, mm^2 – Section 5
A_t	area of structural steel shape, pipe or tubing in a composite section, mm^2 – Section 6
A_t	area of one leg of a closed stirrup resisting torsion within a distance s, mm^2 – Section 7
A_{te}	area of one leg of a stirrup-tie, mm^2 – Sections 6 and 10
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 reinforcement, A_t, mm^2 – Section 5
A_v	area of shear reinforcement within a distance s, mm^2 – Section 5
A_v	area of shear reinforcement within a distance s, or area of shear reinforcement, perpendicular to flexural tension reinforcement within a distance s for deep beams, mm^2 – Section 7 and Appendix B
A_{vf}	area of shear friction reinforcement, mm^2 – Section 7
A_{vh}	area of shear reinforcement parallel to flexural reinforcement within a distance s_2, mm^2 – Section 7
A_w	area of an individual wire to be developed or spliced, mm^2 – Section 5
A_1	loaded area, mm^2 – Section 6 and Appendix B
A_2	maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm^2 – Section 6 and Appendix B
b	width of compression face of member – Sections 6, 7, 12, 13 and Appendix B
b	width or thickness of wall section, mm – Section 10
b	smaller side of rectangular contact area – Section 11
b_c	overall width of column, mm – Sections 6 and 9

b_j effective width of joint, mm – Section 9

b_o perimeter of critical section for slabs and footings, mm – Section 7 and Appendix B

b_v the width of the cross-section being investigated for horizontal shear, mm – Section 8

b_w web width, or diameter of circular section, mm – Sections 5, 6, 7 and Appendix B

b_w web width of beam, mm – Section 9

B the ratio of the reduction in moment of resistance to the numerically largest moment given anywhere by the elastic analysis for that particular member covering all appropriate combinations of design load – Section 3

B the ratio of the reduction in resistance moment to the numerically largest moment (see 13.3.10.1 (d)) – Section 13

c neutral axis depth measured from extreme compression fibre – Sections 3, 6 and 13

c computed distance of neutral axis from the compression edge of the wall section – Section 10

c the smaller of c_c or c_s, mm – Section 5

c_c distance measured from extreme tension fibre to centre of bar, mm – Section 5

c_c distance of the critical neutral axis from the compression edge of the wall section – Section 10

c_s the smaller of the distance from the façe of the concrete to the centre of bar measured along the line through the layer of bars, or half the centre-to-centre distance of bars in the layer, mm.

For splices, c_s shall be the smaller of the distance from the concrete side face to the centre of the outside bar, or one-half the clear spacing of bars spliced at the same location plus a half bar diameter, mm – Section 5

c_1 size of rectangular or equivalent rectangular column, capital or bracket measured in the direction of the span for which the moments are being determined, mm – Sections 7 and 11

c_2 size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which the moments are being determined, mm – Sections 7 and 11

C cross-sectional constant to define torsional properties. See eq. 11-12 – Section 11

C_j $\dfrac{V_{jh}}{V_{jx} + V_{jz}}$ – Section 9

C_m a factor relating actual moment diagram to an equivalent uniform moment diagram – Section 6

d distance from extreme compression fibre to centroid of tension reinforcement, mm – Sections 3, 5, 6, 8, 11, 12 and Appendix B

d distance from extreme compression fibre to centroid of longitudinal tension reinforcement but need not be less than 0.8 h for prestressed members or 0.8 ℓ_w for walls, mm.

(For circular sections, d need not be less than the distance from the extreme compression fibre to centroid of tension reinforcement in opposite half of member) – Section 7

d	distance from extreme compression fibre to centroid of prestressing steel, or to combined centroid when non-prestressing tension reinforcement is included, mm – Section 13
d'	distance from extreme compression fibre to centroid of compression reinforcement, mm – Section 4
d_b	nominal diameter of bar, wire or prestressing strand, or in a bundle, the diameter of a bar of equivalent area, mm – Section 5
d_c	distance from extreme compression fibre to centroid of prestressed reinforcement – Section 7
d_i	diameter of bend measured to inside of the bar, mm – Section 5
d_p	diameter of pile at footing base, mm – Section 12
d_s	distance from extreme tension fibre to centroid of tension reinforcement, mm – Section 4
d_s	distance from centroid of total vertical reinforcement to extreme compression fibre of wall section – Section 7
D	dead load as defined by NZS 4203 – Sections 6 and 13
D	dead loads or their related internal moments and forces – Section 15
e	eccentricity between the centre lines of the webs of a beam and a column at a joint, mm – Section 9
e	base of Napierian Logarithm – Section 13
E	design earthquake loading, specified in NZS 4203, applied to the primary elements – Section 13
E	earthquake load as defined by NZS 4203 – Sections 3, 6 and 14
E_c	modulus of elasticity of concrete, MPa (see 3.3.4.1) – Sections 3, 4, 6 and Appendix B
E_{cb}	modulus of elasticity of beam concrete – Section 11
E_{cc}	modulus of elasticity of column concrete – Section 11
E_{cs}	modulus of elasticity of slab concrete – Section 11
E_p	earthquake loads for Parts & Portions, specified in NZS 4203, applied as inertia loading to the secondary elements – Section 3
E_s	modulus of elasticity of reinforcement, MPa (see 3.3.4.2) – Sections 3, 6 and Appendix B
EI	flexural stiffness of a member – see equations 6-8 and 6-9 for columns and piers – Section 6
f'_c	specified compressive strength of concrete, MPa – Sections 3, 4, 5, 6, 7, 9, 10, 13, 14 and Appendix B
$\sqrt{f'_c}$	square root of specified compressive strength of concrete, MPa, Sections 4, 7, 12, 14 and Appendix B
f'_{ci}	compressive strength of concrete at time of initial prestress, MPa – Section 13

f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa – Sections 4, 7, and Appendix B

f_h tensile stress developed by standard hook, MPa – Section 5

f_{pc} compressive stress in concrete (after allowing for all pre-stress losses) at centroid of cross-section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, MPa.

(In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone) – Section 7

f_{pe} compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fibre of section where tensile stress is caused by externally applied loads, MPa – Section 7

f_{ps} calculated stress in prestressing steel at design load, MPa – Sections 5 and 13

f_{pu} ultimate tensile strength of prestressing tendons, MPa – Sections 7 and 13

f_{py} specified yield strength of prestressing steel, MPa or the 0.2% proof stress – Section 13

f_r modulus of rupture of concrete, MPa – Section 4

f_s steel stress at service load, MPa – Sections 4 and 5

f_s tensile stress in reinforcement – Appendix B

f_{se} effective stress in prestressing steel after losses, MPa – Sections 5 and 13

f_y specified yield strength of non-prestressed reinforcement, MPa – Sections 3, 4, 5, 6, 7, 9, 12, 13, 14 and Appendix B

f_{yh} specified yield strength of spiral, hoop or supplementary cross-tie reinforcement, MPa – Sections 6, 10 and 14

f_{yh} specified yield strength of horizontal non-prestressed shear reinforcement in a wall, MPa – Section 7

f_{yn} specified yield strength of vertical non-prestressed wall reinforcement, MPa – Section 7

f_{yt} specified yield strength of transverse reinforcement, MPa – Sections 5 and 6

h overall thickness of member, mm – Sections 4, 5, 6, 7, 11, 13, 14 and 15

h overall depth of beam – Section 10

h'' dimension of concrete core of section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop, mm – Sections 6 and 10

h_b beam depth, mm – Sections 5 and 9

h_c column depth parallel to the longitudinal beam bars being considered, mm – Section 5

h_c overall depth of column in the direction of the horizontal shear to be considered, mm – Section 9

h_v total depth of shearhead cross-section, mm – Section 7

h_w total height of wall from base to top, mm – Section 7

h_1 distance from the centroid of the tension steel to the neutral axis, mm – Section 4

h_2 distance from the extreme tension fibre to the neutral axis, mm – Section 4

I_b moment of inertia about centroidal axis of gross section of a beam as defined in 11.8.2.4 – Section 11

I_c moment of inertia of gross cross-section of column – Section 11

I_{cr} moment of inertia of cracked section – Section 4

I_e effective moment of inertia for computation of deflection – Section 4

I_g moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement, mm^4 – Sections 4 and 6

I_s moment of inertia about centroidal axis of gross section of slab
= $h^3/12$ times width of slab specified in defintions of α and β_t – Section 11

I_{se} moment of inertia of reinforcement about centroidal axis of member cross-section, mm^4 – Section 6

I_t moment of inertia of structural steel shape, pipe or tubing about centroidal axis of composite member cross-section, mm^4 – Section 6

I_x moment of inertia of section resisting externally applied factored loads – Section 7

k effective length factor for a column or pier – Section 6

k_b multiplier applied to the permitted beam bar sizes through column joints in non-yielding beams – Section 5

k_{tr} an index of the transverse reinforcement provided along the anchored bar, $A_{tr}f_{yt}/\ 10s$, expressed as mm – Section 5

K wobble friction coefficient per metre of prestressing steel – Section 13

K_b flexural stiffness of beams: moment per unit roatation – Section 11

K_c flexural stiffness of column: moment per unit rotation – Section 11

K_{cp} factor used in computing deflections allowing for long time effects – Section 4

K_{ec} flexural stiffness of equivalent column: moment per unit rotation. See eq. 11-10 – Section 11

K_s flexural stiffness of slab: moment per unit rotation – Section 11

K_t torsional stiffness of torsional member: moment per unit rotation – Section 11

ℓ span length of beam, girder or one-way slab as defined in 3.3.3.5 (a); clear projection of cantilever, mm – Section 4

ℓ length of prestressing steel element from jacking end to any point x, length of span in two way flat plates in the direction parallel to that of the reinforcement being determined, mm – Section 13

ℓ_a additional embedment length at support or at point of inflection, mm – Section 5

ℓ_b distance from critical section to start of bend, mm – Section 5

ℓ_d development length, mm – Sections 5 and 10

ℓ_{db} basic development length of a straight bar, mm – Section 5

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 – Section 5

ℓ_{hb} basic development length for a hooked bar, mm – Section 5

ℓ_n clear span for positive moment or shear and the average adjacent clear spans for negative moment – Section 3

ℓ_n length of clear span in long direction of two-way construction, measured face-to-face of columns in slabs without beams and face-to-face of beams or other supports in other cases, mm – Section 4

ℓ_n clear length of member measured from face of supports, mm – Section 6

ℓ_n the clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span – Section 10

ℓ_n length of clear span, in the direction moments are being determined, measured face-to-face of supports – Section 11

ℓ_s shortest span length of bridge deck slab, mm – Section 4

ℓ_s distance between point of zero shear and the support of a deep beam and as specified in 7.3.12.4 – Section 7

ℓ_t span of member under load test (the shorter span of flat slabs and of slabs supported on four sides). The span, except as provided in 15.4.2.9, is the distance between the centres of the supports or the clear distance between the supports plus the depth of the member, whichever is the smaller, mm – Section 15

ℓ_u unsupported length of a column or pier, mm – Section 6

ℓ_v length of shear head arm from centroid of concentrated load or reaction, mm – Section 7

ℓ_w horizontal length of wall, mm – Sections 7 and 10

ℓ_x length of clear span in short direction of rectangular slab – Section 11

ℓ_y length of clear span in long direction of rectangular slab – Section 11

ℓ_1 length of span in the direction that moments are being determined, measured centre-to-centre of supports – Section 11

ℓ_2 length of span transverse to ℓ_1 measured centre-to-centre of supports (see also 11.8.5.3 and 11.8.5.4) – Section 11

L design live loads, including impact where appropriate, or their related internal moments and forces – Section 15

L_R reduced live load as defined by NZS 4203 or their related internal moments and forces – Sections 6, 13 and 15

m $f_y/0.85f'_c$ – Section 14

M applied design moment – Appendix B

M structural material factor as defined in NZS 4203 (see 3.5.4) – Section 3

M_a maximum moment in member at stage for which deflection is being computed – Section 4

ΣM_b sum of moments at ideal strength in non-yielding beams at opposite faces of the joint, summed in the same vector sense, and related to the centre of the intersecting column, N mm – Section 5

M_c moment to be used for design of a column or pier – Section 6

ΣM_c sum of the moments at ideal strength in hinging columns at opposite faces of the joint, summed in the same vector sense, and related to the centre of the intersect-beam, N mm – Section 5

M_{cr} cracking moment – Section 4

M_e moment resulting from loading combination U, involving earthquake effects, N mm – Section 14

M_e^* M_e referred to mid-depth of the section, N mm – Section 14

M_{eq} moment associated with E, N mm

M_i ideal flexural strength of section, N mm – Section 5

M_o moment which causes zero stress at extreme fibre at which tensile stress is induced by applied load as defined by eq. 7-10 – Section 7

M_o total factored static moment – Section 11

M_p required plastic moment strength of shear-head cross-section – Section 7

M_{sx} moment midspan or the supports of strips of unit width and span ℓ_x – Section 11

M_{sy} moment at midspan or the supports of strips of unit width and span ℓ_y – Section 11

M_u design moment for a member – Section 6

M_u factored moment at section – Section 7

M_{uv} factored moment occuring at a section simultaneously with V_u – Section 7

M_v moment resistance contributed by shear head reinforcement – Section 7

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 – Section 6

M_2 value of larger design end moment on a column or pier calculated by conventional elastic frame analysis, always positive – Section 6

n modular ratio = E_s/E_c – Appendix B

n number of bars in a layer – Section 5

N design axial load normal to the cross-section occurring simultaneously with V, to be taken as positive for compression, negative for tension, and to include the effects of creep and shrinkage – Appendix B

N_c tensile force in the concrete under service load – Section 13

N_u factored tensile force on bracket or corbel acting simultaneously with V_u, to be taken positive for tension – Section 7

p_c perimeter of section, mm – Section 7

p_o perimeter of Area A_o, mm – Section 7

P_c critical load. See eq. 6-7 – Section 6

P_{cs} forces after all losses in prestressing steel passing through a joint, N – Section 9

P_e maximum design axial load in compression with given eccentricity due to gravity and seismic loading acting on the member during an earthquake – Section 4

P_e design axial load in compression with given eccentricity due to gravity and seismic loading acting on the member during an earthquake – Sections 6 and 9

P_e design axial load in compression with given eccentricity due to gravity and seismic loading acting on the member simultaneously with shear stress v_i during an earthquake – Sections 7 and 13

P_e design axial load, due to loading U, involving earthquake loads, N – Section 14

P_i ideal axial load strength at given eccentricity – Sections 4 and 13

P_{iw} ideal axial load carrying capacity of a bearing wall designed by 10.4.2 – Section 10

P_{min} minimum axial load on a column at its junction with a beam in which plastic hinges form, N – Section 5

P_o ideal axial load compressive strength when the load is applied with zero eccentricity – Sections 6 and 13

P_s prestress at jacking end – Section 13

P_u design axial load at given eccentricity – Section 6

P_u factored axial load at given eccentricity (not including any prestressing force) $\leqslant \phi P_i$ – Section 4

P_u factored axial load normal to cross-section occurring simultaneously with V_u to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage – Section 7

P_u design axial compression column load including vertical prestressing where applicable, occurring simultaneously with V_{jh}, N – Section 9

P_x prestress at any point – Section 13

r radius of gyration of cross-section of a column or pier, mm – Section 6

r the algebraic ratio at the plastic hinge section of the maximum shear force developed with positive moment hinging to the maximum shear force developed with negative moment hinging, always taken negative – Section 7

R_c reduction factor for confinement – Section 14

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 – Section 5

s centre-to-centre spacing of stirrup ties along a member, mm – Section 6

s spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, mm – Section 7 and Appendix B

s_b for a particular bar or group of bars in contact, the centre-to-centre distance, 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 d_b, mm — Section 5

s_h centre-to-centre spacing of hoop sets, mm — Section 6 and 10

s_h spacing of hoops or supplementary cross-tie reinforcement sets or both, mm — Section 14

s_v horizontal spacing of longitudinal reinforcement along the length of a wall, mm — Section 10

s_w spacing of wires to be developed or spliced, mm — Section 5

s_1 spacing of vertical reinforcement in wall, mm — Section 7

s_2 spacing of shear reinforcement in direction perpendicular to longitudinal reinforcement or spacing of horizontal reinforcement in wall, mm — Section 7

S structural type factor as defined in NZS 4203,(see also 14.4.1)— Sections 3, 7, 10 and 14

t thickness of surfacing and filling material — Section 11

t_b distance from extreme tension fibre to the centre of the adjacent bar, mm — Section 4

t_c $0.75\ A_{co}/P_c$, mm — Section 7

t_o $0.75\ A_o/P_o$ — Section 7

T_i ideal torsional strength of section — Section 7

T_u factored torsional moment at section — Section 7

u larger side of rectangular loaded area allowing for load spread — Section 11

U required strength in accordance with appropriate design loadings code — Sections 4 and 13

U design load — Section 6

U design load combination specified in NZS 4203 or other appropriate loadings code. For Me M_e^*, V_e and P_e, the combinations are those involving earthquake effects — Section 14

v total shear stress at section — Appendix B

v smaller side of rectangular load area allowing for load spread — Section 11

v_b basic shear stress, MPa — Section 7

v_c ideal shear stress provided by the concrete, MPa — Sections 3, 7 and 14

v_c allowable shear stress carried by the concrete, MPa — Appendix B

v_{ci} ideal shear stress provided by concrete when diagonal cracking results from combined shear and moment, MPa — Section 7

v_{cw} ideal shear stress provided by concrete when diagonal cracking results from excessive principal tensile stress in web, MPa — Section 7

v_{dh} design horizontal shear stress at any cross-section, MPa – Section 8

v_h permissible horizontal shear stress, MPa – Section 8 and Appendix B

v_i total shear stress, MPa – Sections 7 and 10

v_{in} shear stress sustained by members of variable depth defined by eq. 7-7, MPa – Section 7

v_{jh} nominal horizontal shear stress in joint core, MPa – Section 9

v_{ti} torsional shear stress, MPa – Section 7

V total shear force at section – Appendix B

V_{col} horizontal shear force across a column, N – Section 9

V_{ch} ideal horizontal joint shear strength provided by concrete shear resisting mechanism only, N – Section 9

V_{cv} ideal vertical joint shear strength provided by concrete shear resisting mechanism only, N – Section 9

V_e shear force resulting from loading combination U, involving earthquake loads, N

V_{eq} shear force associated with E, N

V_{di} design shear force to be resisted by diagonal shear reinforcement – Section 7

V_i ideal shear strength of section – Sections 7, 8 and 14

V_{jh} total horizontal shear force across a joint, N – Section 9

V_{jv} total vertical shear force across a joint, N – Section 9

V_{jx} total horizontal joint shear force in x direction, N – Section 9

V_{jz} total horizontal joint shear force in z direction, N – Section 9

V_p transverse component of effective longitudinal prestress force at section – Section 7

V_{sh} ideal horizontal joint shear strength provided by horizontal joint shear reinforcement, N – Section 9

V_{sv} ideal vertical joint shear strength provided by vertical joint shear reinforcement, N – Section 9

V_u factored shear force at section, N – Sections 5, 7 and 8

w density of concrete, kg/m^3 – Sections 3 and 4

w_d factored uniformity distributed dead load per unit area – Section 11

w_{max} maximum crack width at the surface of the member, mm – Section 4

w_u factored load per unit length of beam or per unit area of a slab – Section 3

w_u total factored uniformly distributed load per unit area
$= w_d + w_1$ – Section 11

w_ℓ factored uniformly distributed live load per unit area – Section 11

x shorter overall dimension of a rectangular part of cross-section – Section 11

y longer overall dimension of a rectangular part of cross-section – Section 11

y_t distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension – Sections 4 and 7

α (alpha) ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by the centre line of the adjacent panel (if any) on each side of the beam

$$= \frac{E_{cb} I_b}{E_{cs} I_s}$$ – Sections 4 and 11

α angle between inclined stirrups and longitudinal axis of member – Section 7 and Appendix B

α total angular change of prestressing steel profile in radians from jacking end to any point x – Section 13

α_c ratio of flexural stiffness of the columns above and below the slab to the combined flexural stiffness of the slabs and beams at a joint taken in the direction of the span for which moments are being determined

$$= \frac{\Sigma K_c}{(K_s + K_b)}$$ – Section 11

α_{ec} ratio of flexural stiffness of equivalent column to the combined flexural stiffness of the slabs and beams at a joint taken in the direction of the span for which moments are being determined

$$= \frac{K_{ec}}{\Sigma(K_s + K_b)}$$ – Section 11

α_m average value of α for all beams on the edges of a panel – Section 4

α_{min} minimum α_c to satisfy 11.8.13.1 (a) – Section 11

α_v ratio of stiffness of shearhead arm to surrounding composite slab section. See 7.3.15.4 – Section 7

α_1 α in the direction ℓ_1 – Section 11

α_2 α in the direction ℓ_2 – Section 11

β (beta) ratio of clear spans in long to short direction of two-way slabs – Section 4

β for reinforced concrete the angle which the tension reinforcement makes with the plane of the extreme compression fibre. For prestressed members the angle which the prestressing tendons make with the plane of the beam centroid – Section 7

β_a ratio of dead load per unit area to live load per unit area (in each case without load factors) – Section 11

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 – Section 5

β_c ratio of short side to long side of concentrated load or reaction area – Section 7 and Appendix B

β_c ratio of short side to long side of foundation – Section 12

β_d ratio of maximum design dead load moment to maximum design total load moment, always positive – Section 6

β_s ratio of length of continuous edges to total perimeter of a slab panel – Section 4

β_{sx} short span moment coefficient shown in table 11.1 – Section 11

β_{sy} long span moment coefficient shown in table 11.1 – Section 11

β_t ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of the beam, centre-to-centre of supports

$= E_{cb}C/2E_{cs}I_s$ – Section 11

β_1 factor defined in 6.3.1.7 – Section 6

γ (gamma) ratio of distance between centroids of tensile and compressive reinforcement to overall depth of the member – Section 4

γ strength parameter used for confinement criteria – Section 14

γ_f fraction of unbalanced moment transferred by flexure at slab-column connections (see 11.3.5) – Section 11

δ (delta) maximum deflection under test load of a member relative to a line joining the ends of the span, or of the free end of a cantilever relalative to its support, mm – Section 15

δ moment magnification factor (see 6.4.11.5 and 6.4.11.6) – Section 6

δ_s factor defined by eq. 11-9 (see 11.8.13) – Section 11

$\triangle$ displacement or deformation (angular or lineal) of the primary elements due to the loading E_p – Section 3

$\triangle_p$ displacement or deformation (angular or lineal) of the secondary elements due to the loading E_p – Section 3

μ (mu) coefficient of friction (see 7.3.11.5) – Section 7

μ curvature friction coefficient – Section 13

ν (nu) modification factor by which deformations are multiplied, as specified in NZS 4203 – Section 3

ρ (rho) ratio of non-prestressed tension reinforcement $= A_s/bd$ – Sections 6 and 13

ρ ratio of tension reinforcement per unit width – Section 11

ρ' ratio of non-prestressed compression reinforcement $= A'_s/bd$ – Sections 3, 6 and 13

ρ^* ratio of all longitudinal reinforcement within A^*_g equal to A^*_s/A^*_g – Section 14

ρ_b reinforcement ratio producing balanced strain conditions in a beam or slab (see 6.4.1.2) – Section 6

ρ_h ratio of horizontal shear reinforcement area to gross concrete area of vertical section – Section 7

ρ_ℓ the ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section $= A_s/b_{sv}$ – Section 10

$\rho_{max.}$, $\rho_{min.}$
maximum and minimum values of the ratio of non-prestressed tension reinforcement computed using width of web – Section 6

ρ_n ratio of total vertical reinforcement area in wall section to gross area of horizontal section of the web only – Section 7

ρ_p ratio of prestressed reinforcement = A_{ps}/bd – Section 13

ρ_s ratio of volume of spiral or circular hoop reinforcement to total volume of concrete core (out-to-out of spirals or hoops) – Section 6

ρ_t ratio of non-prestressed tension reinforcement = A_s/bd – Section 12

ρ_w $(A_s + A_{ps})/b_w d$ – Section 7

ϕ (phi) strength reduction factor (see 4.3.1 and 4.3.2) – Sections 4, 6, 7, 8, 9, 12, 13, 14 and Appendix B

ϕ_o ratio of overstrength moment of resistance to moment resulting from code specified loading, where both moments refer to the base section of wall – Sections 7 and 10

APPENDIX B

ALTERNATIVE DESIGN METHOD

B1 NOTATION

Some of the notation definitions are modified from that in the main body of the Code for specific use in the application of Appendix B.

The Design Loads refer in this Appendix to alternative method design loads and not to strength method design loads.

A_g gross area of section, mm^2

A_v area of shear reinforcement within a distance s, or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s for deep beams, mm^2

A_1 loaded area, mm^2

A_2 maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm^2

b width of compression face of member, mm

b_o periphery of critical section for slabs and footings, mm

b_w web width, or diameter of circular section, mm

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

E_c modulus of elasticity of concrete, MPa. See 3.3.4.1

E_s modulus of elasticity of steel, MPa. See 3.3.4.2

f'_c specified compressive strength of concrete, MPa

$\sqrt{f'_c}$ square root of specified compressive strength of concrete, MPa

f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa

f_s tensile stress in reinforcement, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

M applied design moment

n modular ratio $= E_s/E_c$

N design axial load normal to the cross-section occurring simultaneously with V; to be taken as positive for compression, negative for tension, and to include the effects of tension due to creep and shrinkage

s spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, mm

v total shear stress at section, MPa

v_c allowable shear stress carried by concrete, MPa

v_h permissible horizontal shear stress, MPa

V total shear force at section

α angle between inclined stirrups and longitudinal axis of member

β_c ratio of short side to long side of a concentrated load or reaction area

ρ ratio of non-prestressed tension reinforcement ratio $= A_s/bd$

ϕ capacity reduction factor. See B3.1

B2 SCOPE

B2.1 Non-prestressed reinforced concrete members may be designed using "alternative method" load combinations in accordance with NZS 4203 or other appropriate loadings code and allowable service load stresses in accordance with the provisions of this Appendix. For design of members not covered by this Appendix, the appropriate provisions of this Code shall apply.

B2.2 All provisions of this Code for non-prestressed concrete, except 3.3.3.4, shall apply to members designed by the alternative design method.

B2.3 Flexural members shall meet the requirements for deflection control in 4.4 and the requirements of 6.3.2, 6.3.3, 6.4.3 and 6.4.5 of this Code.

B3 GENERAL

B3.1 Design loadings shall be according to NZS 4203 or other appropriate loadings code and capacity reduction factors, ϕ, shall be taken as unity for members designed by the alternative design method.

B4 ALLOWABLE SERVICE LOAD STRESSES

B4.1 Stresses in concrete.

Stresses in concrete shall not exceed the following:

Flexure

Extreme fibre stress in compression . . . . $0.45 f'_c$

*Shear**

Beams and one-way slabs and footings

Shear carried by concrete, v_c . . . . $0.091 \sqrt{f'_c}$

Maximum shear carried by concrete plus shear reinforcement . . . . $v_c + 0.37 \sqrt{f'_c}$

Joists†:

Shear carried by concrete, v_c . . . . $0.10 \sqrt{f'_c}$

Two-way slabs and footings:

Peripheral shear carried by concrete, v_c ‡ . . . . $0.083 (1 + \frac{2}{\beta_c}) \sqrt{f'_c}$

but not greater than . . . . $0.17 \sqrt{f'_c}$

Bearing on loaded area § . . . . $0.30 f'_c$

B4.2 Stresses in reinforcement

Tensile stress in reinforcement f_s shall not exceed the following:

Grade 275 reinforcement . . . . 150 MPa

Grade 380 reinforcement or greater and welded wire fabric, smooth or deformed 200 MPa

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* For more detailed analysis of shear stress carried by concrete, v_c, and shear values for lightweight aggregate concrete, see B8.2

† Designed in accordance with 3.4.2 of this Code.

‡ If shear reinforcement is provided see B8.2.

§ When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be increased by A_2/A_1, but not more than 2. When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum or a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

B5 DEVELOPMENT AND SPLICES OF REINFORCEMENT

B5.1 Development and splices of reinforcement shall be as required by the relevant clauses in Section 5 of this Code.

B5.2 In satisfying the requirements of 5.3.25.3, M_i shall be taken as computed moment capacity assuming all positive moment tension reinforcement at the section to be stressed to the permissible tensile stress f_s, and V_u shall be taken as unfactored shear force at the section.

B6 FLEXURE

B6.1 For investigation of stresses at service loads, straightline theory (for flexure) shall be used with the following assumptions:

(a) Strains vary linearly as the distance from the neutral axis, except for deep flexural members with overall depth–span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a non-linear distribution of strain shall be considered. See 6.3.3 of this Code

(b) Stress–strain relationship of concrete is a straight line under service loads within allowable service load stresses

(c) In reinforced concrete members, concrete resists no tension

(d) Modular ratio, $n = E_s/E_c$, may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, value of n for lightweight concrete shall be assumed to be the same as for normal-weight concrete of the same strength

(e) In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform compression reinforcement for stress computations. Compressive stress in such reinforcement shall not exceed permissible tensile stress.

B7 COMPRESSION MEMBERS WITH OR WITHOUT FLEXURE

B7.1 Combined flexure and axial load capacity of compression members shall be taken as 40% of that computed in accordance with provisions in Section 6 of this Code.

B7.2 Slenderness effects shall be included according to the requirements of 6.4.10 and 6.4.11.

B7.3 Reinforced concrete walls shall be detailed in accordance with Section 6 of this Code.

B8 SHEAR AND TORSION

B8.1 General

B8.1.1 Design shear stress ν shall be computed by

$$\nu = \frac{V}{b_w d} \qquad \text{(Eq. B-1)}$$

where V is the design shear force at section considered.

B8.1.2 When the reaction, in direction of applied shear, introduces compression into the end regions of a member, sections located less than a distance d from face of support may be designed for the same shear ν as that computed at a distance d.

B8.1.3 Wherever applicable, effects of torsion, in accordance with provisions of Section 7 of this Code, shall be added. Shear and torsional moment strengths provided by concrete and limiting maximum strengths for torsion shall be taken as 55% of the values given in Section 7.

B8.2 Shear stress carried by concrete

B8.2.1 For members subject to shear and flexure only, shear stress carried by concrete v_c shall not exceed $0.091 \sqrt{f'_c}$ unless a more detailed calculation is made in accordance with B8.2.4.

B8.2.2 For members subject to axial compression, shear stress carried by concrete v_c shall not exceed $0.091 \sqrt{f'_c}$ unless a more detailed calculation is made in accordance with B8.2.5.

B8.2.3 For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using:

$$v_c = 0.091 \left(1 + 0.58 \frac{N}{A_g}\right) \sqrt{f'_c} \qquad \text{(Eq. B-2)}$$

where N is negative for tension. Quantity N/A_g shall be expressed in MPa.

B8.2.4 For members subject to shear and flexure only, v_c may be computed by

$$v_c = 0.083 \sqrt{f'_c} + 9.0 \frac{Vd}{M} \qquad \text{(Eq. B-3)}$$

but v_c shall not exceed $0.16 \sqrt{f'_c}$. Quantity Vd/M shall not be taken greater than 1.0, where M is design moment occurring simultaneously with V at section considered.

B8.2.5 For members subject to axial compression, v_c may be computed by

$$v_c = 0.091 \left(1 + 0.087 \frac{N}{A_g}\right) \sqrt{f'_c} \qquad \text{(Eq. B-4)}$$

Quantity N/A_g shall be expressed in MPa.

B8.2.6 Shear stresses carried by concrete v_c apply to normal weight concrete. When light-weight aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified and concrete is proportioned in accordance with ACI 318-77 Section 4.2, $1.8 f_{ct}$ shall be substituted for $\sqrt{f'_c}$ but the value of $1.8 f_{ct}$ shall not exceed $\sqrt{f'_c}$

(b) When f_{ct} is not specified, the value of $\sqrt{f'_c}$ shall be multiplied by 0.75 for "all lightweight" concrete and by 0.85 for "sand-lightweight" concrete. Linear interpolation may be applied when partial sand replacement is used.

B8.2.7 In determining shear stress carried by concrete v_c, whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable-depth members may be included.

B8.3 Shear reinforcement requirements

B8.3.1 *Types of shear reinforcement*

Shear reinforcement may consist of:

(a) Stirrups perpendicular to axis of member

(b) Welded wire fabric with wires located perpendicular to axis of member

(c) Stirrups making an angle of 45° or more with longitudinal tension reinforcement

(d) Longitudinal reinforcement with bent portion making an angle of 30° or more with longitudinal tension reinforcement

(e) Combinations of stirrups and bent longitudinal reinforcement

(f) Spirals.

B8.3.2 Design yield strength of shear reinforcement shall not exceed 415 MPa.

B8.3.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fibre and shall be anchored at both ends according to 5.4.3 of this Code to develop design yield strength of reinforcement.

B8.3.4 *Spacing limits for shear reinforcement*

B8.3.4.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$, nor 600 mm.

B8.3.4.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45° line, extending toward the reaction from mid-depth of member $d/2$ to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

B8.3.4.3 When $(v - v_c)$ exceeds $0.17\sqrt{f'_c}$ maximum spacing given in B8.3.4.1 and B8.3.4.2 shall be reduced by one-half.

B8.3.5 *Minimum shear reinforcement*

B8.3.5.1 A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members where design shear stress v is greater than half the permissible shear stress v_c carried by concrete, except:

(a) Slabs and footings

(b) Concrete joist construction defined by 3.4.2 of this Code

(c) Beams with total depth not greater than 250 mm, 2½ times thickness of flange, or one-half the width of web, whichever is greater.

B8.3.5.2 Minimum shear reinforcement requirements of B8.3.5.1 may be waived if shown by test that required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

B8.3.5.3 Where shear reinforcement is required by B8.3.5.1 or by analysis, minimum area of shear reinforcement shall be computed by

$$A_v = 0.35 \frac{b_w s}{f_y} \quad \text{(Eq. B-5)}$$

where b_w and s are in millimetres.

B8.3.6 *Design of shear reinforcement*

B8.3.6.1 Where design shear stress v exceeds shear stress carried by concrete v_c, shear reinforcement shall be provided in accordance with B8.3.6.2 to B8.3.6.8 inclusive.

B8.3.6.2 When shear reinforcement perpendicular to axis of member is used,

$$A_v = \frac{(v - v_c)\, b_w s}{f_s} \quad \text{(Eq. B-6)}$$

B8.3.6.3 When inclined stirrups are used as shear reinforcement,

$$A_v = \frac{(v = v_c)\, b_w s}{f_s\,(\sin\alpha + \cos\alpha)} \quad \text{(Eq. B-7)}$$

B8.3.6.4 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,

$$A_v = \frac{(v - v_c)\, b_w d}{f_s \sin\alpha} \quad \text{(Eq. B-8)}$$

where $(v - v_c)$ shall not exceed $0.13\sqrt{f'_c}$

B8.3.6.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, required area shall be computed by eq. B-7.

B8.3.6.6 Only the centre three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

B8.3.6.7 Where more than one type of shear reinforcement is used to reinforce the same portion of a member, required area shall be computed as the sum of the various types separately. In such computations, v_c shall be included only once.

B8.3.6.8 Values of $(v - v_c)$ shall not exceed $0.37\sqrt{f'_c}$.

B8.4 Shear friction

Where it is appropriate to consider shear transfer across a given plane such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times, shear-friction provisions of 7.3.11 of this Code may be applied, with limiting maximum stress for shear taken as 55% of that given in 7.3.11.4. Permissible stress in shear-friction reinforcement shall be that given in B4.2.

B8.5 Special provisions for slabs and footings

B8.5.1 Shear capacity 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 slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with B8.1 to B8.3 inclusive.

(b) Two-way action for slab or footing, with a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than $d/2$ to perimeter of concentrated load or reaction area. For this condition the slab or footing shall be designed in accordance with B8.5.2 and B8.5.3.

B8.5.2 Design shear stress v shall be computed by

$$v = \frac{V}{b_o d} \qquad \text{(Eq. B-9)}$$

where V and b_o shall be taken at the critical section defined in B8.5.1 (b).

B8.5.3 Design shear stress v shall not exceed v_c given by eq. B-10 unless shear reinforcement is provided.

$$v_c = 0.083\,(1 + 2\beta_c)\sqrt{f'_c} \qquad \text{(Eq. B-10)}$$

but v_c shall not exceed $0.17\sqrt{f'_c}$. β_c is the ratio of long side to short side of concentrated load or reaction area. When lightweight aggregate concrete is used, the modifications of B8.2.6 shall apply.

B8.5.4 If shear reinforcement consisting of bars or wires is provided in accordance with 7.3.15 of this Code, v_c shall not exceed $0.17\sqrt{f'_c}$ and v shall not exceed $0.25\sqrt{f'_c}$.

B8.5.5 If shear reinforcement consisting of steel I or channel shapes (shearheads) is provided in accordance with 7.3.15.4 of this Code, v on the vertical section defined in B8.5.1 (b) shall not exceed $0.29\sqrt{f'_c}$ and v on the critical section defined in 7.3.15.4 (g) shall not exceed $0.17\sqrt{f'_c}$. In equations 7-36 and 7-37, design shear force V shall be multiplied by 2 and substituted for V_i.

B8.6 Special provisions for other members

For design of deep flexural members, brackets and corbels, and walls, the special provisions of Section 7 of this Code shall be used, with shear strength provided by concrete and limiting maximum strengths for shear taken as 55% of the values given in Section 7. In 7.3.13 the design axial load shall be multiplied by 1.2 if compression and 2.0 if tension, and substituted for N_u.

B8.7 Composite concrete flexural members

For design of composite concrete flexural members, allowable horizontal shear stress v_h shall not exceed 55% of the horizontal shear strengths given in 8.3.4 and 8.4.1 of this Code.

INDEX

NEW ZEALAND STANDARD

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THE DESIGN OF CONCRETE STRUCTURES

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Part 1
Code of practice for
THE DESIGN OF CONCRETE STRUCTURES

AMENDMENT No. 1

December 1989

EXPLANATORY NOTE - Amendment No. 1 is issued to take account of the changes of steel grades in the 1989 revision of NZS 3402, *Steel bars for the reinforcement of concrete.* Other minor changes are included to correct misprints, add clarity to the wording and to make a few other minor alterations.

To ensure receiving advice of the next amendment to NZS 3101:Part 1:1982 please complete and return the amendment request form.

DECLARATION

Amendment No. 1 was declared on 8 December 1989 by the Standards Council to be an amendment to NZS 3101: Part 1:1982 pursuant to the provisions of section 10 of the Standards Act 1988.

ACKNOWLEDGEMENT

Grateful acknowledgement is made to Professor R. Park, Head of Department of Civil Engineering at the University of Canterbury for the preparation of the draft amendment.

(Amendment No. 1, December 1989)

RELATED DOCUMENTS

Delete the list of NEW ZEALAND STANDARDS and **substitute** the following:

NZS 1900:- - - -	Model building bylaw
Chapter 5:1988	Fire resisting construction and means of egress
Chapter 9:1985	Design and construction
NZS 3109:1987	Specification for concrete construction
NZS 3112:- - - -	Methods of test for concrete
Part 2:1986	Tests relating to the determination of strength of concrete
NZS 3152:1974	Manufacture and use of structural and insulating lightweight concrete
NZS 3402:1989	Steel bars for the reinforcement of concrete
NZS 3404:1989	Steel structures code
NZS 4203:1984	Code of practice for general structural design and design loadings for buildings
NZS 4702:1982*	Metal-arc welding of Grade 275 reinforcing bar

*Under amendment

(Amendment No. 1, December 1989)

FOREWORD

Delete the penultimate paragraph of page 12 and **substitute**:

"It should be noted that some provisions of this Code are based on proposed amendments to NZS 4203:1976 which have subsequently been incorporated into NZS 4203:1984".

(Amendment No. 1, December 1989)

General change

Throughout the Code, wherever the terms occur, **change** "specified yield strength" or "specified minimum yield strength" to "lower characteristic yield strength" when referring to non-prestressed reinforcement.

Refer clauses: 3.1, 4.1, 5.1 (twice), 5.3.25.2, 5.3.32.1, 6.1 (3 times), 6.3.1.4, 6.4.12.7(b), 6.4.12.8(b), 7.1 (3 times), 9.1, 10.1, 12.1, 13.1, 14.1 (twice), Appendix A (p.105) (5 times), Appendix B, B1.

(Amendment No. 1, December 1989)

1.1
In the first paragraph, **delete** "NZS 1900, Chapter 9.3" and **substitute** "NZS 1900, Chapter 9".

(Amendment No. 1, December 1989)

1.2.4
Delete the clause and the Note thereto.

(Amendment No. 1, December 1989)

2.1
In the definition of DEFORMED REINFORCEMENT, **delete** "NZS 3402P" and **substitute** "NZS 3402".

Under the definitions of "Strength", **add** the following:

"LOWER CHARACTERISTIC YIELD STRENGTH OF STEEL. The value of yield strength below which not more than 5 % of production tests in each size falls".

and:

"UPPER CHARACTERISTIC YIELD STRENGTH OF STEEL. The value of yield strength above which not more than 5 % of production tests in each size falls".

(Amendment No. 1, December 1989)

3.5.3.4(b)
Delete the first line and **substitute:** "The redistribution of beam terminal negative moments shall ..."

(Amendment No. 1, December 1989)

3.5.4.3
Delete the clause and **substitute** the following:

"Lower characteristic yield strength of reinforcement, f_y, used in potential plastic hinge regions shall not exceed 500 MPa".

(Amendment No. 1, December 1989)

3.5.4.6
Delete the clause and **substitute** the following:

"Plain round or deformed bars may be used as transverse reinforcement. Deformed bars shall not be used where significant rebending of hooks and bends is expected or possible. Plain Grade 430 bars shall carry permanent identification".

(Amendment No. 1, December 1989)

3.5.4.7
Add a new clause 3.5.4.7 as follows:

"Reinforcing bar manufactured by the hot rolled and in line continuously quenched and tempered process (see 4.1 of NZS 3402) may not retain full strength if incautious hot bending or hot rebending practices are used."

(Amendment No. 1, December 1989)

4.3.1.4
Delete the clause and **substitute** the following:

"Designs shall not be based on a lower characteristic yield strength for reinforcing steel, f_y in excess of 500 MPa".

(Amendment No. 1, December 1989)

Table 4.1
Delete the table and the notes thereto and **substitute**:

Table 4.1
MINIMUM THICKNESSES OF NON-PRESTRESSED BEAMS OR ONE-WAY SLABS. f_y = 300 MPa

Member	*Minimum thickness, h*			
	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
	Simply sup-ported	*One end contin-uous*	*Both end contin.*	*Canti-lever*
Solid one-way slabs	$\ell/25$	$\ell/30$	$\ell/35$	$\ell/13$
Beams or ribbed one-way slabs	$\ell/20$	$\ell/23$	$\ell/26$	$\ell/10$

NOTE - The values given shall be used directly for members with normal density concrete (w = 2400 kg/m³) and steel reinforcement with f_y = 300 MPa. For other conditions, the values shall be modified as follows:

(a) For structural lightweight concrete having a density in the range 1450 - 1850 kg/m³, the values shall be multiplied by (1.65 - 0.0003 w) but not less than 1.09, where w is the density in kg/m³.

(b) For f_y other than 300 MPa, the values shall be multiplied by (0.42 + f_y/520).

(Amendment No. 1, December 1989)

4.4.2.1(b)
Delete and **substitute**:

"The lower characteristic yield strength of the reinforcing steel exceeds 300 MPa".

(Amendment No. 1, December 1989)

5.3.1.2
Delete "NZS 3402P" and **substitute** "NZS 3402".

(Amendment No. 1, December 1989)

5.3.3
In line 3 after "given in table 5.1" **add** "for New Zealand manufactured steel reinforcement to NZS 3402".

Delete equation 5-1 and **substitute**:

$$\text{“}d_i \geq 0.92 \left[0.5 + \frac{d_b}{s_b} \right] \left[1 - \frac{\ell_b}{\ell_d} \right] \frac{f_y}{f'_c} d_b \quad \text{..... (Eq. 5-1)”}$$

(Amendment No. 1, December 1989)

Table 5.1
Delete the table and **substitute**:

Table 5.1
MINIMUM DIAMETERS OF BEND FOR NEW ZEALAND MANUFACTURED STEEL BARS TO NZS 3402

Steel yield strength f_y	*Bar diameter (mm)* d_b	*Minimum diameter of bend* d_i
300 or 430	6 - 20 24 - 40	5 d_b 6 d_b

(Amendment No. 1, December 1989)

Table 5.2
Delete the table and **substitute**:

Table 5.2
MINIMUM DIAMETERS OF BENDS FOR STIRRUPS AND TIES FOR NEW ZEALAND MANUFACTURED STEEL BARS TO NZS 3402

Steel yield strength	*Bar diameter* (mm)	*Minimum diameter of bend* d_i	
f_y	d_b	Plain bars	Deformed bars
300 or 430	6 - 20 24 - 40	2 d_b 3 d_b	4 d_b 6 d_b

(Amendment No. 1, December 1989)

5.3.4.1
Delete the last line, "where d_b is the stirrup or tie bar diameter" and **substitute** ", for New Zealand manufactured steel reinforcement to NZS 3402, where d_b is the stirrup or tie bar diameter".

(Amendment No. 1, December 1989)

5.3.7.2
Delete "Grade 275 reinforcement" and **substitute** "reinforcement with f_y = 300 MPa".

(Amendment No. 1, December 1989)

5.3.7.3(a)
Delete and **substitute**:

"Reinforcement having a lower characteristic yield strength other than 300 MPa .... f_y/300"

(Amendment No. 1, December 1989)

5.3.9.2
In line 2, **change** "0.24" to 0.22".

In line 3, **change** "0.044" to 0.040".

(Amendment No. 1, December 1989)

5.3.15.2
Delete the first two lines and **substitute**:

"The basic development length for hooked deformed bars with f_y = 300 MPa shall be computed by:"

(Amendment No. 1, December 1989)

5.3.15.3(a)
Delete and **substitute**:

"Reinforcement having a lower characteristic yield strength other than 300 MPa .... f_y/300"

(Amendment No. 1, December 1989)

5.3.17.1
Add the following:

"In the design and execution of welding of reinforcing bar manufactured to NZS 3402 appropriate account shall be taken of the process of manufacture."

(Amendment No. 1, December 1989)

5.3.17.2
Delete the first line and **substitute**:

"Welds shall not be made closer than 10 d_b from bends. Steel bars not conforming to NZS 3402 shall not be ..."

(Amendment No. 1, December 1989)

5.3.17.6
Add the following item.

"(e) The properties of hot rolled and in line continuously quenched and tempered types of reinforcing bar shall be taken into account in the design and execution of any welded connection".

(Amendment No. 1, December 1989)

5.3.20.1
Delete the clause and **substitute**:

"The minimum length of a lap splice in compression shall be the development length in compression ℓ_d, in accordance with 5.3.9 and 5.3.10 but not less than 0.067 $f_y d_b$ for f_y of 430 MPa or less, nor (0.12 f_y - 22) d_b for f_y greater than 430 MPa, nor 300 mm. When the specified concrete strength is less than 20 MPa the lap length shall be increased by one-third".

(Amendment No. 1, December 1989)

5.3.32.1
Delete the second paragraph (top of p.40) and **substitute**:

"Slabs where deformed bars with f_y = 300 MPa are used 0.0020

Slabs where bars with f_y = 430 MPa or welded wire fabric, deformed or plain, are used 0.0018

Slabs where reinforcement with f_y exceeding 430 MPa measured at a yield strain of 0.35 % is used .... 0.0018 x 430/f_y".

(Amendment No. 1, December 1989)

5.3.36.2
Delete the last two lines and **substitute:**

"... fabric or deformed bars with a lower characteristic yield strength of 430 MPa or greater".

(Amendment No. 1, December 1989)

5.3.36.4
Delete "Grade 275 bar" and **substitute** "bar with f_y = 300 MPa".

(Amendment No. 1, December 1989)

5.4.3.2
Add to the end of the first sentence "when the stirrups or ties are plain round bars or 6 stirrup or tie bar diameters when the stirrups or ties are deformed bars".

(Amendment No. 1, December 1989)

5.4.3.3
Add to the end of the clause: "Welds shall not be made closer than 10 d_b from bends".

(Amendment No. 1, December 1989)

5.5.1.1(a)
Delete and **substitute** the following:

"No portion of any splice shall be located within the beam/column joint region, or within one effective depth of member from the critical section of a potential plastic hinge region in a beam where stress reversal in spliced bars could occur".

(Amendment No. 1, December 1989)

5.5.1.2
Delete and **substitute** the following:

"Tensile reinforcement in beams or columns shall not be spliced by lapping in a region of tensile or reversing stress unless each spliced bar is confined by a stirrup tie so that:

$$\frac{A_{tr}}{s} > \frac{d_b f_y}{48 f_{yt}} \quad "$$

(Amendment No. 1, December 1989)

5.5.2.5(b)
Delete and **substitute** the following:

"Where beams frame into opposite sides of a column the maximum diameter of the longitudinal beam bars shall not exceed 12 h_c/f_y".

(Amendment No. 1, December 1989)

5.5.2.5(c)
Delete and **substitute** the following:

"Where the axial compressive load, $P_{min.}$ is greater than 0.4 $f'_c A_g$, the ratio of the maximum diameter of the longitudinal beam bar to the column depth parallel to the bar shall satisfy:

$$\frac{d_b}{h_c} \leq \frac{12}{f_y}\left[1 + 2\left(\frac{P_{min.}}{A_g f'_c} - 0.4\right)\right] \text{......... (Eq. 5-12)}$$

(Amendment No. 1, December 1989)

5.5.2.5(d)
Delete and **substitute** the following:

"When it can be shown that the critical section of a plastic hinge resulting from inelastic seismic displacements is at a distance from the column face of at least the depth of the beam or 500 mm, whichever is less, bar diameters up to $15h_c/f_y$ may be used".

(Amendment No. 1, December 1989)

5.5.3.3
Delete the clause and **substitute** the following:

"When columns are designed to develop plastic hinges in the end regions, the maximum diameter of column bars passing through the beams shall not be greater than $15h_b/f_y$".

(Amendment No. 1, December 1989)

5.5.3.4
Delete the clause and **substitute** the following:

"When columns are not intended to develop plastic hinges, the maximum diameter of longitudinal column bars at any level shall be $20/f_y$ times the depth, h_b, of the deepest beam framing into the column at that level. This requirement need not be met if it is shown that the stresses in the extreme column bars during an earthquake remain in tension or compression over the whole bar length contained within the joint".

(Amendment No. 1, December 1989)

5.5.6.1
In line 2, **add** the word "shall" after "members".

(Amendment No. 1, December 1989)

6.4.7.1(a)(1)
In the last line, **change** "400 MPa" to "500 MPa".

(Amendment No. 1, December 1989)

6.4.7.2(a)(1)
In the last line, **change** "400 MPa" to "500 MPa".

(Amendment No. 1, December 1989)

6.5.3.2(b)
To the end of the sentence **add**:

"except when the compression reinforcement is placed within the depth of the compression flange of a T or L beam built integrally with the web at a section subjected to positive bending moment".

(Amendment No. 1, December 1989)

6.5.3.3(e)
Add a further sentence as follows:

"The area of stirrup-ties need not satisfy equation 6-21."

(Amendment No. 1, December 1989)

6.5.4.2(b)
Delete and **substitute** the following:

"Area of longitudinal reinforcement shall not be greater than $18A_g/f_y$ except that in the region of lap splices the total area shall not exceed $24A_g/f_y$".

(Amendment No. 1, December 1989)

6.5.4.3(a)(2)
In the last line, **change** "400 MPa" to "500 MPa".

(Amendment No. 1, December 1989)

6.5.4.3(b)(2)
In the last line, **change** "400 MPa" to "500 MPa".

(Amendment No. 1, December 1989)

6.5.4.3(b)(6)
Delete and **substitute** the following:

"Each hoop bar or supplementary cross-tie shall satisfy the requirements of 6.5.3.3(b) for stirrup-ties".

(Amendment No. 1, December 1989)

7.3.1.8
Delete the clause and **substitute** the following:

"The total shear stress v_i shall not exceed 0.2 f'_c or 6 MPa".

(Amendment No. 1, December 1989)

7.3.2.2
In lines 6 and 7, **change** "effective depth less than 300 mm" to "total depth less than 300 mm".

(Amendment No. 1, December 1989)

7.3.6.1
Change "415 MPa" to "500 MPa".

(Amendment No. 1, December 1989)

7.3.9.6
Delete the clause and **substitute** the following:

"Closed stirrups shall be anchored with 135° standard hooks unless fully welded in accordance with 5.4.3.3."

(Amendment No. 1, December 1989)

7.3.11.3

Change equation 7-23 to

$$" A_{vf} = \left[\frac{V_u}{\varnothing \mu} - P_u \right] \cdot \frac{1}{f_y} "$$

(Amendment No. 1, December 1989)

7.3.11.6
Change "415 MPa" to "500 MPa".

(Amendment No. 1, December 1989)

7.5.2.2
In line 2, **change** "end regions" to "potential plastic hinge regions".

(Amendment No. 1, December 1989)

7.5.3.1
In line 3, **change** "end regions" to "potential plastic hinge regions".

(Amendment No. 1, December 1989)

7.5.3.2
Delete the clause and **substitute** the following:

"The potential plastic hinge regions of a member, where in accordance with 7.5.2, the limitations of shear stress v_c apply, shall be not less than that defined for beams in 6.5.3.1, for columns in 6.5.4.1 and for walls in the end region in 10.5.5.3".

(Amendment No. 1, December 1989)

7.5.4.4
Delete the clause and **substitute** the following:

"The requirements of 7.5.4.2(b) do not apply to members in which the minimum axial compression stress on the gross concrete area, associated with the maximum shear on the member, is more than 0.10 f'_c, or to columns subjected to an axial compression in which vertical bars are placed along all faces of the section or in circular array."

(Amendment No. 1, December 1989)

8.4.2.1
In line 3, **change** "7.3.6" to "7.3.4.3".

(Amendment No. 1, December 1989)

10.1
In the definition of ρ_ℓ , change "a_s/bs_v" to "A_s/bs_v".

(Amendment No. 1, December 1989)

13.3.6.2
In the last line, **change** "yield stress" to "lower characteristic yield strength".

(Amendment No. 1, December 1989)

13.3.8.2(b)
In line 5, **change** "410 MPa" to "430 MPa".

(Amendment No. 1, December 1989)

APPENDIX B
B4.2
Stresses in reinforcement
Delete the clause and **substitute** the following:

"Tensile stress in reinforcement, f_s shall not exceed 0.55 f_y or 200 MPa".

(Amendment No. 1, December 1989)

B8.3.2
Change "415 MPa" to "500 MPa".

(Amendment No. 1, December 1989)

REQUEST FOR ADVICE OF NEXT AMENDMENT

Please fill in the request for the next amendment to this New Zealand Standard and mail to the Standards Association of New Zealand, Private Bag, Wellington.

If this request slip has not been returned SANZ has no record that you wish to be advised of future amendments to this Standard. From 1 October 1986 a pricing system for amendments was introduced.

To confirm that the next amendment has been requested, enter details of despatch:

REQUEST FOR NEXT AMENDMENT

NZS 3101:Part 1:1982
Amendment No. 2

If more than one copy is required state quantity here . . . COPIES - please type or print clearly.

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CORRIGENDUM to Amendment No. 1 to NZS 3101:Part 1:1982

THE DESIGN OF CONCRETE STRUCTURES

Part 1
Code of practice for
THE DESIGN OF CONCRETE STRUCTURES

5.3.32.1
Change the amended clause to read:

5.3.32.1
Delete the second paragraph (top of p.40) and **substitute**:

"Slabs where deformed bars with $f_y = 300$ MPa are used
....0.0020

Slabs where bars with $f_y = 430$ MPa or welded wire fabric, deformed or plain, are used 0.0016

Slabs where reinforcement with f_y exceeding 430 MPa measured at a yield strain of 0.35 % is used
....0.0016 x $430/f_y$".

NEW ZEALAND STANDARD

NZS 3101: Part 1:1982

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THE DESIGN OF CONCRETE STRUCTURES

Part 1
Code of practice for
THE DESIGN OF CONCRETE STRUCTURES

AMENDMENT No. 2

July 1992

EXPLANATORY NOTE – This Amendment applies when this Standard is used as a Verification Method that is referenced in Approved Document B1 Structure – General, to the New Zealand Building Code. The Amendment need not apply when this Standard is used under the Model Building Bylaw system which remains in operation until 31 December 1992.

APPROVAL

Amendment No. 2 was approved in July 1992 by the Standards Council to be an amendment to NZS 3101: Part 1:1982 pursuant to the provisions of section 10 of the Standards Act 1988.

(Amendment No. 2, July 1992)

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2 DEFINITIONS

Delete the definition of "ENGINEER".

Add a new definition:

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

(Amendment No. 2, July 1992)

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3 GENERAL DESIGN REQUIREMENTS

3.5.6.1
Delete the last words "to the approval of the Engineer".

3.5.12.5
In line 2 **delete** the words "to the satisfaction of the Engineer".

(Amendment No. 2, July 1992)

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4 STRENGTH AND SERVICEABILITY

4.5.2
In line 1 **delete** the words "NZS 1900:Chapter 5" and **substitute** "the New Zealand Building Code".

(Amendment No. 2, July 1992)

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5 REINFORCEMENT – DETAILS, ANCHORAGE AND DEVELOPMENT

5.3.16.2
In line 2 **delete** the words "to the Engineer".

5.3.17.1
In line 2 **delete** the word "Engineer" and **substitute** "design engineer".

5.3.17.2
In line 2 **delete** the word "Engineer" and **substitute** "design engineer".

(Amendment No. 2, July 1992)

5.3.35
In line 1 **delete** the words "When NZS 1900:Chapter 5" and **substitute** "When the New Zealand Building Code".

(Amendment No. 2, July 1992)

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9 BEAM-COLUMN JOINTS

9.5.3.3
In item (b) line 4, **delete** the words "to the satisfaction of the Engineer".

(Amendment No. 2, July 1992)

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15 STRENGTH EVALUATION OF EXISTING STRUCTURES

15.2.1
In line 4 **delete** the words "the Engineer may order" and **substitute** "the capacity may be evaluated by".

15.3.2
Delete the clause and **substitute**:

"*Requirements of Code to be met.* The analysis based on the investigation required by 15.3.1 shall be sufficient to show that the load factors or permissible stresses and design loads meet the requirements and intent of this Code".

15.4.1.1
In lines 1 and 2 **delete** the words "a qualified engineer acceptable to the Engineer" and **substitute** "the design engineer".

15.6.1
In line 3 **delete** the words "the Engineer may approve" and **substitute** "the design engineer may nominate".

(Amendment No. 2, July 1992)

NEW ZEALAND STANDARD

NZS 3101:Part 1:1982

Pr 00

THE DESIGN OF CONCRETE STRUCTURES

Part 1
Code of practice for
THE DESIGN OF CONCRETE STRUCTURES

AMENDMENT No. 3

August 1993

EXPLANATORY NOTE – Amendment No. 3 to NZS 3101:Part 1:1982 *Code of practice for the design of concrete structures* allows use of the concrete code as a limit state design code. It will remain in force until the full, revised concrete design standard is available early in 1994. The amendment allows designs in concrete to the new *Code of practice for general structural design and design loadings for buildings* NZS 4203:1992.

APPROVAL

Amendment No. 3 was approved on 6 August 1993 by the Standards Council to be an amendment to NZS 3101: Part 1:1982 pursuant to the provisions of section 10 of the Standards Act 1988.

(Amendment No. 3, August, 1993)

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1 GENERAL

1.1 (page 15)
At the end of the first sentence following the word "structures" **add** "in accordance with the limit state design method".

(Amendment No. 3, August, 1993)

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3 GENERAL DESIGN REQUIREMENTS

3.5.1.1 (page 22)
At the end of clause 3.5.1.1(a) **add** the following sentence:

"Values of the structural ductility factor, μ, shall be given by NZS 4203:1992 but shall have a maximum value of 6".

(Amendment No. 3, August, 1993)

On page 22 **add** new clause 3.5.2.4 as follows:

"3.5.2.4 The structural performance factor, S_p, shall be 0.67 for structural systems designed in accordance with this Standard and NZS 4203, unless a different value is determined from a special study."

(Amendment No. 3, August, 1993)

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4 STRENGTH AND SERVICEABILITY

4.3.1.2 (page 28)
Delete existing clause 4.3.1.2 and **substitute** the following:

"4.3.1.2 The strength reduction factor, ϕ, shall be as follows:

(a) Flexure, with or without axial tension 0.85

(b) Axial tension 0.85

(c) Axial compression, with or without flexure:
Members with spirals, hoops or special transverse reinforcement complying with 6.4.7.1(a), 6.4.7.2(a) or 6.5.4.3 .... 0.85

Other members 0.65

except that ϕ may be increased linearly to 0.85 as P_u decreases from 0.10 $f'_c A_g$ to zero.

(d) Flexure in walls subjected to seismic forces and designed in accordance with 10.5 .... 0.85

(e) Shear and torsion 0.75

(f) Bearing on concrete 0.65

(g) Flexure in plain concrete .... 0.60 "

(Amendment No. 3, August, 1993)

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