A23.3-04

Design of concrete structures

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A23.3-04
Design of concrete structures

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Preface

This is the fifth edition of CSA A23.3, *Design of concrete structures.* It supersedes the previous editions published in 1994, 1984, 1977 (metric) and 1973 (imperial), and 1959.

This Standard is intended for use in the design of concrete structures for buildings in conjunction with CSA A23.1-04/A23.2-04, *Concrete materials and methods of concrete construction/Methods of test and standard practices for concrete,* and CSA A23.4, *Precast concrete — Materials and construction* (under preparation).

Changes in this edition include the following:

(a) This Standard is now based on the load factors and load combinations specified in the *National Building Code of Canada*, 2005 (see Annex C).

(b) Clause 2.2 contains new definitions for different types of walls; these depend on the level of axial load and the primary loading on the wall.

(c) In Clause 8.4.2, the resistance factor for concrete, ϕ_c, has been increased from 0.60 to 0.65.

(d) Clause 10 on flexure and axial loads contains a revised alternative expression for calculating the flexural stiffness, *EI*, for slenderness effects.

(e) Clause 11 on shear and torsion contains new design provisions for members such as slabs, footings, joists, and wide shallow beams. Clause 11.3.6.3 provides a new, simplified method for shear design. Clause 11.3.6.4 contains revised design provisions for the general method. These revised design provisions are based on the modified compression field theory.

(f) Clause 13 on two-way slab systems has been revised to provide different distribution factors for factored moments in column strips. New requirements for slab band construction are also specified.

(g) Clause 14 contains new provisions for reinforcement details for walls, including requirements for concentrated reinforcement and ties for vertical reinforcement. A new clause (Clause 14.4) on the structural design of shear walls has been added. It includes requirements for compression flanges for assemblies of interconnected walls.

(h) Clause 15 on foundations includes new clauses on the design requirements for pile caps and piles.

(i) In Clause 16 on precast concrete, the resistance factor for concrete, ϕ_c, has been increased from 0.65 to 0.70 for the design of elements produced in CSA-certified manufacturing plants.

(j) Clause 18 on prestressed concrete has a modified expression for the stress in unbonded prestressing tendons at factored resistance.

(k) Clause 21 on special provisions for seismic design specifies requirements that conform to the new categories for seismic force resisting systems in the *National Building Code of Canada*, 2005. Provisions for determining member stiffnesses have been added. The requirements for checking that the columns are stronger than the beams have been changed for the design of ductile frames and frames with moderate ductility. In Clause 21.4.4, the requirements for confinement reinforcement for columns have been changed to include the effects of axial load level as well as the arrangement of transverse reinforcement and longitudinal bars. Maximum spacing limits for transverse reinforcement in columns of ductile frames have been changed. New ductility requirements have been added in Clause 21.6 for individual walls, coupled walls, partially coupled walls, and coupling beams. In Clause 21.6.9 on shear strength of ductile walls, the method for determining the factored shear resistance has been changed. New requirements for the design of squat walls have been added in Clause 21.7.4. A new clause (Clause 21.8) on conventional construction (R_d = 1.5) has been added. Clause 21.9 provides new requirements for ductile moment resisting frames, ductile shear walls, and moderately ductile shear walls constructed using precast concrete. New requirements for the design of structural diaphragms have been added in Clause 21.10. Clause 21.11 provides new requirements for foundations, including footings, foundation mats, pile caps, grade beams, slabs on grade, piles, piers, and caissons. Revised requirements for the design of frame members not considered part of the seismic force resisting system are specified in Clause 21.12.

(l) Clause 22 on plain concrete contains a new clause (Clause 22.8) on the design of deep foundations.

(m) There is a new Annex D on anchorage based on the requirements specified in Appendix D of ACI (American Concrete Institute) 318M-02/318RM-02, *Building Code Requirements for Structural Concrete and Commentary*. Annex D deals with anchors cast into the concrete or post-installed into hardened concrete. It covers anchors used to transmit applied loads, including straight bolts, hooked bolts, headed studs, expansion anchors, undercut anchors, and inserts.

This Standard was prepared by the Technical Committee on Reinforced Concrete Design, under the jurisdiction of the Strategic Steering Committee on Structures (Design), and has been formally approved by the Technical Committee. It will be submitted to the Standards Council of Canada for approval as a National Standard of Canada.

December 2004

Notes:

(1) *Use of the singular does not exclude the plural (and vice versa) when the sense allows.*

(2) *Although the intended primary application of this Standard is stated in its Scope, it is important to note that it remains the responsibility of the users of the Standard to judge its suitability for their particular purpose.*

(3) *This publication was developed by consensus, which is defined by* CSA Policy governing standardization — Code of good practice for standardization *as "substantial agreement. Consensus implies much more than a simple majority, but not necessarily unanimity". It is consistent with this definition that a member may be included in the Technical Committee list and yet not be in full agreement with all clauses of this publication.*

(4) *CSA Standards are subject to periodic review, and suggestions for their improvement will be referred to the appropriate committee.*

(5) *All enquiries regarding this Standard, including requests for interpretation, should be addressed to Canadian Standards Association, 5060 Spectrum Way, Suite 100, Mississauga, Ontario, Canada L4W 5N6.*

Requests for interpretation should

(a) define the problem, making reference to the specific clause, and, where appropriate, include an illustrative sketch;

(b) provide an explanation of circumstances surrounding the actual field condition; and

(c) be phrased where possible to permit a specific "yes" or "no" answer.

Committee interpretations are processed in accordance with the CSA Directives and guidelines governing standardization *and are published in CSA's periodical* Info Update*, which is available on the CSA Web site at www.csa.ca.*

A23.3-04
Design of concrete structures

1 Scope

1.1 General
This Standard specifies requirements, in accordance with the *National Building Code of Canada,* for the design and strength evaluation of
(a) structures of reinforced and prestressed concrete;
(b) plain concrete elements; and
(c) special structures such as parking structures, arches, tanks, reservoirs, bins and silos, towers, water towers, blast-resistant structures, and chimneys.
Note: *Special requirements for parking structures are specified in CAN/CSA-S413.*

1.2 Fire resistance
This Standard requires designs to be carried out in accordance with the fire resistance requirements of the applicable building code (see Clause 8.1.2).

1.3 Alternative design procedures
Designs that use procedures which are not covered by this Standard but are carried out by a person qualified in the methods applied and provide a level of safety and performance equivalent to designs complying with this Standard are acceptable if carried out by one of the following methods:
(a) analysis based on generally established theory;
(b) evaluation of a full-scale structure or a prototype by a loading test; or
(c) studies of model analogues.

1.4 Terminology
In CSA Standards, "shall" is used to express a requirement, i.e., a provision that the user is obliged to satisfy in order to comply with the standard; "should" is used to express a recommendation or that which is advised but not required; "may" is used to express an option or that which is permissible within the limits of the standard; and "can" is used to express possibility or capability. Notes accompanying clauses do not include requirements or alternative requirements; the purpose of a note accompanying a clause is to separate from the text explanatory or informative material. Notes to tables and figures are considered part of the table or figure and may be written as requirements. Annexes are designated normative (mandatory) or informative (non-mandatory) to define their application.

1.5 Units of measurement
Equations appearing in this Standard are compatible with the following units:
(a) area: mm^2 (square millimetres);
(b) force: N (newtons);
(c) length: mm (millimetres);
(d) moment: N•mm (newton millimetres); and
(e) stress: MPa (megapascals).

Whenever the square root of the concrete strength is determined, the concrete strength and the square root of the concrete strength are both expressed in megapascals.

Other dimensionally consistent combinations of units may be used, provided that appropriate adjustments are made to constants in non-homogeneous equations.
Note: *Some examples of non-homogeneous equations are found in Clauses 12.2.2 and 12.8.*

2 Reference publications, definitions, symbols, and standard notation and calculations

2.1 Reference publications

This Standard refers to the following publications, and where such reference is made, it shall be to the edition listed below, including all amendments published thereto.

CSA (Canadian Standards Association)
A23.1-04/A23.2-04
Concrete materials and methods of concrete construction/Methods of test and standard practices for concrete
Note: *Excerpts from this Standard are presented in Annex A.*

A23.4 (under preparation)
Precast concrete — Materials and construction

CAN/CSA-G30.18-M92 (R2002)
Billet-steel bars for concrete reinforcement

G40.20-04/G40.21-04
General requirements for rolled or welded structural quality steel/Structural quality steel

CAN/CSA-S16-01
Limit states design of steel structures

CAN/CSA-S413-94 (R2000)
Parking structures

W59-03
Welded steel construction (metal arc welding)

W186-M1990 (R2002)
Welding of reinforcing bars in reinforced concrete construction

ACI (American Concrete Institute)
318M-02/318RM-02
Metric Building Code Requirements for Structural Concrete and Commentary

336.3R-93
Design and Construction of Drilled Piers

355.2-04/355.2R-04
Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary

360R-97
Design of Slabs on Grade

T1.1-01/T1.1R-01
Acceptance Criteria for Moment Frames Based on Structural Testing

ASTM International (American Society for Testing and Materials)
A 185-02
Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete

A 307-04
Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength

A 416/A 416M-02
Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

A 421/A 421M-02
Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete

A 496-02
Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement

A 497/A 497M-02
Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete

A 722/A 722M-98 (2003)
Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete

C 330-04
Standard Specification for Lightweight Aggregates for Structural Concrete

AWS (American Welding Society)
D1.1/D1.1M:2004
Structural Welding Code — Steel

NRCC (National Research Council Canada)
National Building Code of Canada, 2005

User's Guide — NBC 2005: Structural Commentaries (Part 4)

Other publications
ACI-ASCE Committee 550. 1993. Design recommendations for precast concrete structures. *ACI structural journal*. 90:115–121.

Canadian Precast/Prestressed Concrete Institute. 2005. *Design manual: Precast and prestressed concrete.* 4th ed. Ottawa: Canadian Precast/Prestressed Concrete Institute.

Cement Association of Canada. 2005. *Concrete design handbook*. 3rd ed. Ottawa: Cement Association of Canada.

Precast/Prestressed Concrete Institute. 1999. *PCI design handbook: Precast and prestressed concrete*. 5th ed. Chicago: Precast/Prestressed Concrete Institute.

2.2 Definitions

The following definitions apply in this Standard:

Auxiliary member — a rib or edge beam that serves to strengthen, stiffen, or support the shell. Auxiliary members usually act jointly with the shell.

Base (of a structure) — the level at which earthquake motions are assumed to be imparted to a structure. This level does not necessarily coincide with the ground level.

Beam — an element subjected primarily to loads and forces producing flexure.

Bell — an enlargement at the bottom of a pre-drilled cast-in-place concrete pile.

Bonded tendon — a prestressing tendon that is bonded to concrete either directly or through grouting.

Boundary elements — portions of a wall, typically at the ends, that are reinforced by vertical reinforcement and can contain transverse reinforcement. Boundary elements do not necessarily require an increase in wall thickness.

Buckling prevention ties — ties that meet the requirements of Clause 21.6.6.9 and are intended to prevent buckling of the longitudinal reinforcement under reverse cyclic loading.

Collector — an element that serves to transfer forces within a structural diaphragm to members of the seismic force resisting system.

Column — a member that has a ratio of height to least lateral dimension of 3 or greater and is used primarily to support axial compressive load.

Column capital — an enlargement of the column adjacent to the underside of a slab to improve the shear strength of the slab.
Note: *The dimensions c_1 and c_2 and the clear span ℓ_n are based on an effective support area defined by the intersection of the bottom surface of the slab, or of the drop panel if there is one, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and capital or bracket and are oriented not more than 45° to the axis of the column.*

Column strip — that portion of the design strip with a width on each side of a column centreline equal to $0.25\ell_2$ or $0.25\ell_1$, whichever is less. The column strip includes beams, if any.

Composite concrete flexural members — concrete flexural members of precast or cast-in-place concrete elements, or both, constructed in separate placements but interconnected so that all elements respond to loads as a unit.

Concrete —

> **Plain concrete** — concrete that contains no reinforcing or prestressing steel or less reinforcing or prestressing steel than the specified minimum for reinforced concrete.
>
> **Reinforced concrete** — concrete that is reinforced with not less than the minimum amount of reinforcement required by Clauses 7 to 21 and 23 and is designed on the assumption that the two materials act together in resisting forces.
>
> **Structural low-density concrete** — concrete having a 28 day compressive strength not less than 20 MPa and an air-dry density not exceeding 1850 kg/m^3.
>
> **Structural semi-low-density concrete** — concrete having a 28 day compressive strength not less than 20 MPa and an air-dry density between 1850 and 2150 kg/m^3.

Concrete cover — the distance from the concrete surface to the nearest surface of reinforcement or prestressing tendon.

Confinement ties — ties that meet the requirements of Clauses 21.4.4.2 to 21.4.4.4 and are intended to provide confinement to the enclosed concrete.

Connection — a region that joins two or more members, of which one or more is precast.

> **Ductile connection** — a connection that experiences yielding as a result of the design displacement.

Strong connection — a connection that remains elastic while adjoining members experience yielding as a result of the design displacement.

Core — that part of the member cross-section confined by the perimeter of the transverse reinforcement measured from out-to-out of the transverse reinforcement.

Cover — see **Concrete cover**.

Critical section — a section where a plastic hinge can start to form under earthquake loading.

Crosstie — a reinforcing bar that passes through the core and is anchored around reinforcing bars on opposite sides of a member.

Curvature friction — friction resulting from bends or curves in the specified prestressing tendon profile.

Deep foundation — a structural element that transfers loads from the superstructure to the deeper bearing soil or rock strata by end bearing, friction, or both. Examples of deep foundations include driven piles, drilled cast-in-place piles, and slurry walls.

Deformed reinforcement — deformed reinforcing bars, deformed wire, welded smooth wire fabric, and welded deformed wire fabric complying with Clause 3.1.3.

Design cross-section — the representative panel cross-section at the maximum moment and deflection locations of the panel for which the design forces and deflections are determined and from which the resistance and stiffness are calculated.

Design displacement — the total lateral displacement expected for the design basis earthquake calculated in accordance with Clause 4.1.8 of the *National Building Code of Canada.*

Designer — the person responsible for the design.

Design strip — the portion of a slab system that includes beams and supports along a column line and is bound by the centreline of the panels on each side.

Design width — the width of a tilt-up panel to be reinforced to withstand the factored loads tributary to it.

Development length — the length of embedded reinforcement required to develop the design strength of reinforcement.

Development length for a bar with a standard hook in tension — the length measured from the critical section to the outside end of the hook (the straight embedment length between the critical section and the start of the hook [point of tangency] plus the radius of the bend and one bar diameter).

Drilled pile — a pile cast-in-place in a pre-drilled hole.

Driven pile — a reinforced concrete, prestressed concrete, structural steel, timber, or composite pile driven into the ground.

Drop panel — thickening of the slab in the area adjacent to a column for deflection control, extra shear strength, or extra flexural depth.

Ductile coupled shear wall — a shear wall system that complies with Clauses 21.2 and 21.6 and has ductile shear walls connected by ductile coupling beam(s) where at least 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces resulting from shear in the coupling beam(s). This structural system qualifies for a force modification factor, R_d, of 4.0 in the *National Building Code of Canada.*

Ductile coupling beam — a coupling beam that complies with Clauses 21.2 and 21.6.8 and is designed to dissipate energy.

Ductile moment-resisting frame — a moment-resisting frame that complies with Clauses 21.2 and 21.5, resists seismic forces, and dissipates energy through beam flexural yielding. This structural system qualifies for a force modification factor, R_d, of 4.0 in the *National Building Code of Canada.*

Ductile partially coupled shear wall — a shear wall system that complies with Clauses 21.2 and 21.6 and has ductile shear walls connected by ductile coupling beam(s) where less than 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces resulting from shears in the coupling beam(s). This structural system qualifies for a force modification factor, R_d, of 3.5 in the *National Building Code of Canada.*

Ductile shear wall — a shear wall that complies with Clauses 21.2, 21.6.1 to 21.6.7, and 21.6.9, resists seismic forces, and dissipates energy through flexural yielding at a plastic hinge. This structural system qualifies for a force modification factor, R_d, of 3.5 in the *National Building Code of Canada.*

Effective depth of section — the distance measured from the extreme compression fibre to the centroid of the tension reinforcement.

Effective prestress — the stress remaining in prestressing tendons after all losses have occurred.

Elastic analysis — an analysis of deformations and internal forces based on equilibrium, compatibility of strains, and assumed elastic behaviour.

Embedment length — the length of embedded reinforcement provided beyond a critical section.

Experimental analysis — an analysis based on measuring deformations and strains of a structure or its model. It is based on either elastic or inelastic behaviour.

Factored load effect — the effect of factored load combinations specified in Clause 8.3 (including earthquake load effects determined in accordance with Clause 4.1.8 of the *National Building Code of Canada*).

Flat plate — a flat slab without drop panels.

Folded plate — a special class of shell structures formed by joining flat, thin slabs along their edges to create a three-dimensional spatial structure.

Footing — a shallow structural element that transfers loads from the superstructure to the bearing strata (soil or rock).

Headed bar — a bar with a welded or forged head at one or both ends, with the head dimensioned to be capable of developing the nominal tensile strength of the reinforcing bar at the head-bar interface without failure of the head or crushing failure of the concrete under the head.

Helical tie — a continuously wound reinforcement in the form of a cylindrical helix enclosing longitudinal reinforcement.

Hoop — a closed tie or continuously wound tie. A closed tie can be made up of several reinforcing elements with seismic hooks at each end. A continuously wound tie should also have seismic hooks at each end.

Jacking force — a temporary force exerted by the device that introduces tension into prestressing tendons.

Lifting stresses — stresses in a tilt-up panel during lifting.

Limit states — those conditions of a structure at which it ceases to fulfill the function for which it was designed.

Load —

Dead load — a specified dead load as defined in the *National Building Code of Canada.*

Factored load — the product of a specified load and its load factor.

Live load — a specified live load as defined in the *National Building Code of Canada*.

Specified load — a load specified by the *National Building Code of Canada* without load factors.

Sustained load — the specified dead load plus that portion of the specified live load expected to act over a period of time sufficient to cause significant long-term deflection.

Load factor — a factor applied to a specified load that, for the limit state under consideration, takes into account the variability of the loads and load patterns and analysis of their effects.

Low-density aggregate — aggregate that complies with ASTM C 330.

Middle strip — that portion of the design strip bounded by two column strips.

Moderately ductile moment-resisting frame — a moment-resisting frame that complies with Clauses 21.2 and 21.7.2, that resists seismic forces, and that dissipates energy through beam flexural yielding. This structural system qualifies for a force modification factor, R_d, of 2.5 in the *National Building Code of Canada.*

Moderately ductile shear wall — a shear wall that complies with Clauses 21.2 and 21.7.3, that resists seismic forces, and that dissipates energy through flexural yielding at a plastic hinge or through one of the two mechanisms specified in Clause 21.7.4.2. This structural system qualifies for a force modification factor, R_d, of 2.0 in the *National Building Code of Canada.*

Modulus of rupture of concrete — the flexural strength of concrete determined using the third-point loading test method specified in CSA A23.2.

Moment-resisting frame — a frame in which columns, beams, and joints resist forces through flexure, shear, and compression.

Panel — a slab area bounded by column, beam, or wall centrelines on all sides.

Partial prestressing — prestressing such that the calculated tensile stresses under specified loads exceed the limits specified in Clause 18.3.2(c).

Pedestal — an upright compression member with a ratio of unsupported height to least lateral dimension of less than 3.

Pile — an elongated structural element drilled or driven into the ground for supporting loads by end bearing, friction, or both.

Pile cap — a reinforced concrete element connected to the top of a pile or pile group that transfers loads from the superstructure to the pile or pile group.

Pile casing — a steel tube or liner used for pre-drilled cast-in-place concrete pile construction.

Pile shaft — that portion of the pile from the pile toe to the pile top, excluding any bell or cap.

Pile toe — the bottom of the pile.

Plain reinforcement — reinforcement that does not conform to the definition of deformed reinforcement.

Plastic hinge — a region of a member where inelastic flexural curvatures occur.

Post-tensioning — a method of prestressing in which the tendons are tensioned after the concrete has hardened.

Precast concrete — concrete elements cast in a location other than their final position in service.

Prestressed concrete — concrete in which internal stresses have been initially introduced so that the subsequent stresses resulting from dead load and superimposed loads are counteracted to a desired degree. This can be accomplished by post-tensioning or pretensioning.

Pretensioning — a method of prestressing in which the tendons are tensioned before the concrete is placed.

Probable moment resistance — the moment resistance of a section calculated using axial loads P_s and P_p, where applicable; $1.25f_y$ as the stress in the tension reinforcing; and the specified values of f'_c, with ϕ_c and ϕ_s taken as 1.0.

Regular two-way slab system — a slab system consisting of approximately rectangular panels and supporting primarily uniform gravity loading. Such systems meet the following geometric limitations:
(a) within a panel, the ratio of longer to shorter span, centre-to-centre of supports, is not greater than 2.0;
(b) for slab systems with beams between supports, the relative effective stiffness of beams in the two directions $(\alpha_1\ell_2^2)/(\alpha_2\ell_1^2)$ is not less than 0.2 or greater than 5.0;
(c) column offsets are not greater than 20% of the span (in the direction of offset) from either axis between centrelines of successive columns; and
(d) the reinforcement is placed in an orthogonal grid.

Reinforcement — non-prestressed steel that complies with Clauses 3.1.2 and 3.1.3.

Resistance —

Factored resistance — the resistance of a member, connection, or cross-section calculated in accordance with this Standard, including the application of appropriate resistance factors.

Nominal resistance — the resistance of a member, connection, or cross-section calculated in accordance with this Standard, without including resistance factors.

Resistance factor — the factor, specified in Clause 8.4 and applied to a specified material property or to the resistance of a member for the limit state under consideration, which takes into account the variability of dimensions, material properties, quality of work, type of failure, and uncertainty in the prediction of resistance.

Ribbed shell — a spatial structure with material placed primarily along certain preferred rib lines, with the areas between the ribs filled with thin slabs or left open.

Sandwich panel — a panel consisting of two concrete layers or wythes separated by a layer of insulation.

Seismic crosstie — a single bar having a seismic hook at one end and a hook not less than 90° with at least a six-bar-diameter extension at the other end. The hooks engage peripheral longitudinal bars. The 90° hooks of successive crossties engaging the same longitudinal bar are alternated end for end.

Seismic force resisting system — that part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects in accordance with Clause 4.1.8 of the *National Building Code of Canada.*

Seismic hook — a hook with at least a 135° bend with a six-bar-diameter extension (but not less than 100 mm) that engages the longitudinal reinforcement and is anchored in the confined core.

Slab band — a continuous extension of a drop panel between supports or between a support and another slab band.

Specified strength of concrete — the compressive strength of concrete used in the design and evaluated in accordance with Clause 4.

Spiral — a helical tie complying with Clauses 7.6.4 and 10.9.4.

Spiral column — a column in which the longitudinal reinforcement is enclosed by a spiral.

Stirrup — reinforcement used to resist shear and torsion stresses in a structural member.
Note: *The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term "ties" to lateral reinforcement in compression members.*

Structural diaphragm — a structural member, such as a floor or roof slab, that transmits forces to or between lateral-force-resisting members.

Tendon — a steel element such as a wire, bar, or strand, or a bundle of such elements, that is used to impart prestress to concrete and complies with Clause 3.1.4.

Thin shell — a three-dimensional spatial structure made up of one or more curved slabs or folded plates whose thicknesses are small compared to their other dimensions.
Note: *Thin shells are characterized by their three-dimensional load-carrying behaviour, which is determined by the geometry of their form, the manner in which they are supported, and the nature of the applied load.*

Tie — a loop of reinforcing bar or wire enclosing longitudinal reinforcement. See also **Stirrup**.

Tilt-up wall panel — a reinforced concrete panel that is site-cast on a horizontal surface and subsequently tilted to a vertical orientation to form a vertical- and lateral-load-resisting building element.

Transfer — the act of transferring force in prestressing tendons from jacks or the pretensioning anchorage to the concrete member.

Tributary width — the width of a panel attracting vertical and horizontal loads that the design width must support.

Wall — a vertical element in which the horizontal length, ℓ_w, is at least six times the thickness, t, and at least one-third the clear height of the element.

Bearing wall — a wall that supports
(a) factored in-plane vertical loads exceeding 0.10 $f'_c A_g$;
(b) weak axis moments about a horizontal axis in the plane of the wall; and
(c) the shear forces necessary to equilibrate the moments specified in Item (b).

Flexural shear wall — a shear wall that resists in-plane lateral loads by flexural action. Flexural shear walls have a height, h_w, above the section of maximum moment in the walls that is greater than $2\ell_w$.

Non-bearing wall — a wall that supports factored in-plane vertical loads less than or equal to 0.10 $f'_c A_g$ and, in some cases, moments about a horizontal axis in the plane of the wall and the shear forces necessary to equilibrate those moments.

Shear wall — a wall or an assembly of interconnected walls considered to be part of the lateral-load-resisting system of a building or structure. Shear walls support
(a) vertical loads;
(b) moments about horizontal axes perpendicular to the plane of the wall (strong axis bending); and
(c) shear forces acting parallel to the plane of the wall.
Weak axis bending can also be present.

Squat shear wall — a shear wall with a height, h_w, above the section of maximum moment in the wall that does not exceed $2\ell_w$.

Wobble friction — friction caused by the unintended deviation of prestressing sheath or duct from its specified profile.

Yield strength — the specified minimum yield strength or yield point of reinforcement.

2.3 Symbols

The following symbols apply in this Standard:

a = depth of equivalent rectangular stress block

a_g = specified nominal maximum size of coarse aggregate

A = area of that part of cross-section between flexural tension face and centroid of gross section (see Clause 18)

= effective tension area of concrete surrounding the flexural tension reinforcement and extending from the extreme tension fibre to the centroid of the flexural tension reinforcement and an equal distance past that centroid, divided by the number of bars or wires. When the flexural reinforcement consists of different bar or wire sizes, the number of bars or wires used to compute A is to be taken as the total area of reinforcement divided by the area of the largest bar or wire used (see Clause 10)

A_b = area of an individual bar

A_c = area enclosed by outside perimeter of concrete cross-section, including area of holes (if any) (see Clause 11)

= area of core of spirally reinforced compression member measured to outside diameter of spiral (see Clause 10)

A_{ch} = cross-sectional area of core of a structural member

A_{cs} = area of concrete in strips along exposed side faces of beams (see Clause 10)
= effective cross-sectional area of concrete compressive strut (see Clause 11)
A_{ct} = area of concrete on flexural tension side of member (see Figure 11.2)
A_{cv} = area of concrete section resisting shear transfer (see Clause 11)
= net area of concrete section bounded by web thickness and length of section in the direction of lateral forces considered (see Clause 21)
A_f = area of flange
A_g = gross area of section
A_{gb} = gross area of a boundary element
A_j = minimum cross-sectional area within a joint in a plane parallel to the axis of the reinforcement generating the shear in the joint, equal to the lesser of A_g of the column or $2b_wh_{col}$
A_o = area enclosed by shear flow path, including area of holes (if any)
A_{oh} = area enclosed by centreline of exterior closed transverse torsion reinforcement, including area of holes (if any)
A_p = area of prestressing tendons (see Clause 10)
= area of prestressing tendons in tension zone (see Clause 18)
= area of tendons on the flexural tension side of the member (see Clause 11)
A_s = area of longitudinal reinforcement on the flexural tension side of the member (see Clause 11)
= area of non-prestressed tension reinforcement (see Clauses 12, 13, 18, and 23)
A'_s = area of compression reinforcement
A_{sb} = minimum area of bottom reinforcement crossing one face of the periphery of a column and connecting the slab to the column or support to provide structural integrity
$A_{s,eff}$ = effective area of tension reinforcement
A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension h_c
$A_{s,min}$ = minimum area of tension reinforcement
A_{ss} = area of reinforcement in compression strut
A_{st} = area of reinforcement in tension tie (see Clause 11)
= total area of longitudinal reinforcement (see Clause 10)
A_t = area of one leg of closed transverse torsion reinforcement (see Clause 11)
= area of structural steel shape, pipe, or tubing in a composite section (see Clause 10)
A_{tr} = total cross-sectional area of reinforcement that is within spacing s and crosses the potential plane of bond splitting through the reinforcement being developed
A_v = area of shear reinforcement within a distance s
A_{ve} = effective shear cross-section area of coupling beam to be used for analysis
A_{vf} = area of shear-friction reinforcement
A_{vs} = cross-sectional area of headed shear reinforcement on a line parallel to the perimeter of the column
A_w = area of an individual wire to be developed or spliced
A_{xe} = effective axial cross-section area to be used for analysis
A_1 = loaded area
A_2 = area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support, having for its upper base the loaded area and having side slopes of 1 vertical to 2 horizontal
b = width of compression face of member (see Clauses 9, 10, and 21)
= width of compression face of panel within design width (see Clause 23)
= width of member (see Clause 22)

b_b = band width of reinforced concrete slab extending a distance $1.5h_d$ or $1.5h_s$ past the sides of the column or column capital (see Clauses 13 and 21)
= bearing width for concentrated load (see Figure 23.2)
b_d = design width (see Figure 23.2)
b_f = width of flange
b_o = perimeter of critical section for shear in slabs and footings
b_s = width of support reaction (see Figure 23.2)
b_t = tributary width (see Clause 23)
= width of tension zone of section (see Clause 10)
b_v = width of cross-section at contact surface being investigated for longitudinal shear
b_w = beam web width or diameter of circular section or wall thickness (see Clause 21)
= minimum effective web width (see Clause 11)
= width of web (see Clause 10)
b_1 = width of the critical section for shear (see Clause 13) measured in the direction of the span for which moments are determined
b_2 = width of the critical section for shear (see Clause 13) measured in the direction perpendicular to b_1
c = cohesion stress (see Clause 11)
= depth of the neutral axis, with the axial loads P_n, P_{ns}, and P_s measured from the compression edge of a wall section (see Clause 21)
= distance from extreme compression fibre to neutral axis (see Clauses 9 and 10)
= distance from extreme compression fibre to neutral axis calculated using factored material strengths and assuming a tendon force of $\phi_p A_p f_{pr}$ (see Clause 18)
= distance from extreme compression fibre to neutral axis computed for the cracked transformed section (see Clause 23)
c_t = dimension equal to the distance from the interior face of the edge column to the slab edge measured parallel to c_1, but not exceeding c_1
c_y = distance from extreme compression fibre to neutral axis calculated using factored material strengths and assuming a tendon force of $\phi_p A_p f_{py}$
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
c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1
C = cross-sectional constant used in the definition of torsional properties
C_m = factor relating actual moment diagram to an equivalent uniform moment diagram
d = distance from extreme compression fibre to centroid of longitudinal tension reinforcement, but need not be less than $0.8h$ for prestressed members and circular sections (see Clauses 11 and 18)
= distance from extreme compression fibre to centroid of tension reinforcement (see Clauses 9, 10, 12, 13, 21, and 23)
= distance from extreme compression fibre to centroid of tension reinforcement for entire composite section (see Clause 17)
d_a = depth of compression strut (see Figure 11.3)
d_b = diameter of bar, wire, or prestressing strand
d_c = distance from extreme tension fibre to centre of the longitudinal bar or wire located closest to it

d_{cs} = the smaller of
(a) the distance from the closest concrete surface to the centre of the bar being developed; or
(b) two-thirds of the centre-to-centre spacing of the bars being developed

d_p = pile shaft diameter (see Clauses 15 and 22)
= distance from extreme compression fibre to centroid of the prestressing tendons (see Clause 18)

d_v = effective shear depth, taken as the greater of $0.9d$ or $0.72h$

e = distance from centroid of section for critical shear to point where shear stress is being calculated (see Clause 13)
= eccentricity of P_{tf} parallel to axis measured from the centroid of the section (see Clause 23)

E_c = modulus of elasticity of concrete

E_p = modulus of elasticity of prestressing tendons

E_s = modulus of elasticity of non-prestressed reinforcement

EI = flexural stiffness of compression member

f'_c = specified compressive strength of concrete

f'_{cc} = specified compressive strength of concrete in columns

f_{ce} = compression stress in the concrete due to effective prestress only (after allowance for all prestress losses) at the extreme fibre of a section where tensile stresses are caused by applied loads

f'_{ce} = effective compressive strength of concrete in columns

f'_{ci} = compressive strength of concrete at time of prestress transfer

f_{cp} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses). For slabs and footings, f_{cp} is the average of f_{cp} for the two directions (see Clause 18)
= compressive stress in concrete (after allowance for all prestress losses) at the centroid of the cross-section resisting externally applied loads or at the junction of the web and flange when the centroid lies within the flange (in a composite member, f_{cp} is the resultant compressive stress at the centroid of the composite section or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and moments being resisted by the precast member acting alone) (see Clause 11)

f'_{cs} = specified compressive strength of concrete in slab

f_{cu} = limiting compressive stress in concrete strut

f'_{cw} = specified compressive strength of concrete in the wall

f_{pe} = effective stress in prestressing tendons after allowance for all prestress losses

f_{po} = stress in prestressing tendons when strain in the surrounding concrete is zero (may be taken as $0.7f_{pu}$ for bonded tendons outside the transfer length and f_{pe} for unbonded tendons)

f_{pr} = stress in prestressing tendons at factored resistance

f_{pu} = specified tensile strength of prestressing tendons

f_{py} = yield strength of prestressing tendons

f_r = modulus of rupture of concrete

f_s = calculated stress in reinforcement at specified loads

f_y = specified yield strength of non-prestressed reinforcement or anchor steel

f'_y = specified yield strength of compression non-prestressed reinforcement

f_{yh} = specified yield strength of hoop reinforcement

f_{yt} = specified yield strength of transverse reinforcement

f_{yv} = specified yield strength of headed shear reinforcement

F_a = acceleration-based site coefficient, as specified in the *National Building Code of Canada*

F_{lc} = required tension force in longitudinal reinforcement on flexural compression side of member
F_{lt} = required tension force in longitudinal reinforcement on flexural tension side of member
F_y = specified yield strength of structural steel section
h = overall thickness or height of member
h_a = height of effective embedment of tension tie (see Figure 11.3)
h_b = distance from soffit of supporting beam to soffit of supported beam (see Figure 11.1)
h_c = clear vertical distance between successive floor slabs attached to the shear wall assembly (see Clause 14)
= dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop (see Clause 21)
h_{col} = column dimension parallel to shear force in the joint
h_d = overall thickness at a drop panel
h_s = overall thickness of slab; for slabs with drop panels, the overall thickness of the slab away from the drop panel
h_u = unsupported vertical height of wall between horizontal supports
h_w = vertical height of wall (see Clause 21)
= vertical height of wall above the section of maximum moment in the wall (see Clause 14)
h_x = maximum horizontal centre-to-centre spacing between longitudinal bars on all faces of the column that are laterally supported by seismic hoops or crosstie legs
h_1 = overall height of supporting beam (see Figure 11.1)
h_2 = overall height of supported beam (see Figure 11.1)
I = moment of inertia of section about centroidal axis
I_b = moment of inertia about centroidal axis of gross section of beam
I_{cr} = moment of inertia of cracked section transformed to concrete
I_e = effective moment of inertia
I_{ec} = value of I_e at continuous end
I_{em} = value of I_e at midspan
I_{e1} = value of I_e at end 1 of a continuous beam span
I_{e2} = value of I_e at end 2 of a continuous beam span
I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
I_s = moment of inertia about centroidal axis of gross section of slab, equal to $\ell_{2a}h_s^3 / 12$
I_{st} = moment of inertia of reinforcement about centroidal axis of member cross-section
I_t = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross-section
I_E = earthquake importance factor of the structure, as specified in the *National Building Code of Canada*
J = property of the critical shear section analogous to the polar moment of inertia
k = effective length factor
k_n = factor accounting for the number of longitudinal reinforcing bars in a column
k_p = factor accounting for compression on column or wall (see Clause 21)
= factor for type of prestressing in Equation (18-1)
k_1 = bar location factor
k_2 = coating factor
k_3 = concrete density factor
k_4 = bar size factor
k_5 = welded deformed wire fabric factor

K_{bf} = panel bending stiffness at factored loads
K_{bs} = panel bending stiffness at service loads
K_c = flexural stiffness of column; moment per unit rotation
K_{ec} = flexural stiffness of equivalent column; moment per unit rotation
K_t = torsional stiffness of member; moment per unit rotation
K_{tr} = transverse reinforcement index
ℓ = effective panel height
ℓ_a = additional embedment length at support or at point of inflection (see Clause 12)
= length of effective bearing area for strut anchored by reinforcement (see Figure 11.3)
ℓ_b = length of bearing (see Figure 11.3)
ℓ_c = length of a compression member in a frame, measured from centre-to-centre of the joints in the frame (see Clause 10)
= length of the outermost compression segment of a coupled wall (see Clause 21)
= the lesser of h_c and w_c (see Clause 14)
= vertical clear distance between supports or unsupported length of the drilled pile (see Clause 22)
ℓ_{cg} = horizontal distance between centroids of walls on either side of coupling beam
ℓ_d = development length of reinforcement
ℓ_{db} = basic development length
ℓ_{dh} = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus radius of bend and one bar diameter) (see Clauses 12 and 21)
ℓ_{hb} = basic development length of standard hook in tension
ℓ_j = dimension of joint in the direction of reinforcement passing through the joint
ℓ_n = clear span (see Clauses 9 and 16)
= length of clear span in the direction that moments are being determined, measured face-to-face of supports (see Clause 13)
ℓ_o = minimum length measured from the face of the joint along the axis of the structural member, over which transverse reinforcement needs to be provided (see Clause 21)
= overall length of tendon between anchors (see Clause 18)
ℓ_t = length of attached torsional member, equal to the smaller of ℓ_{1a} or ℓ_{2a} of spans adjacent to the joint
ℓ_u = clear span or unsupported length between floors or other effective horizontal lines of lateral support (see Clause 21)
= unsupported length of compression member (see Clause 10)
ℓ_w = horizontal length of wall
ℓ_1 = length of span in the direction that moments are being determined, measured centre-to-centre of supports
ℓ_{1a} = average ℓ_1 for spans adjacent to a column
ℓ_2 = length of span transverse to ℓ_1, measured centre-to-centre of supports
ℓ_{2a} = average ℓ_2 for the adjacent spans transverse to ℓ_1
= distance from edge to panel centreline for spans along an edge
L = variable load due to intended use and occupancy, including loads due to cranes, pressure of liquids in containers, or related moments or forces
m_x = bending moment per unit length on section perpendicular to the x-axis
= total design moment per unit length on section perpendicular to the x-axis
m_{xy} = torsional moment per unit length on section

m_y = bending moment per unit length on section perpendicular to the y-axis
= total design moment per unit length on section perpendicular to the y-axis
M_a = maximum moment in member at load stage at which deflection is computed or at any previous load stage
M_b = maximum factored moment in panel at load stage at which deflection is computed, not including P-Δ effects
M_{bs} = maximum moment in panel due to service loads at load stage at which deflection is computed, not including P-Δ effects
M_c = magnified factored moment to be used for design of compression member
M_{cr} = cracking moment
M_{dc} = decompression moment, equal to the moment when the compressive stress on the tensile face of a prestressed member is zero
M_f = factored moment at interior support resisted by elements above and below the slab (see Equation (13-24))
= factored moment, including P-Δ effects (see Clause 23)
= moment due to factored loads (see Clauses 10, 11, 18, and 20)
= unbalanced moment about the centroid of the critical shear section (see Equation (13-9))
M_{fs} = factored strong axis moment acting on a shear wall
M_{fw} = factored weak axis moment acting on a shear wall
M_{nc} = nominal flexural resistance of a column
M_o = total factored static moment
M_{pb} = probable flexural resistance of a beam
M_r = factored moment resistance
M_s = factored end moment on a compression member due to loads that result in appreciable sway, calculated using a first-order elastic frame analysis (see Clause 10)
= maximum moment due to service loads, including P-Δ effects (see Clause 23)
= moment due to specified loads (see Clause 18)
= portion of slab moment balanced by support moment (see Clause 21)
M_1 = smaller factored end moment on a compression member associated with the same loading case as M_2 (positive if member is bent in single curvature, negative if bent in double curvature)
M_{1ns} = factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sway, calculated using a first-order elastic frame analysis
M_{1s} = factored end moment on a compression member at the end at which M_1 acts, due to loads that cause appreciable sway, calculated using a first-order elastic frame analysis
M_2 = larger factored end moment on a compression member (always positive)
M_{2ns} = factored end moment on a compression member at the end at which M_2 acts, due to loads that cause no appreciable sway, calculated using a first-order elastic frame analysis
M_{2s} = factored end moment on a compression member at the end at which M_2 acts, due to loads that cause appreciable sway, calculated using a first-order elastic frame analysis
n = number of bars or wires being spliced or developed along the potential plane of bond splitting
n_ℓ = total number of longitudinal bars in the column cross-section that are laterally supported by the corner of hoops or by hooks of seismic crossties
N = unfactored permanent compressive load perpendicular to the shear plane (see Clause 11)
N_c = tensile force in concrete
N_f = factored axial load normal to the cross-section occurring simultaneously with V_f, including effects of tension due to creep and shrinkage (taken as positive for tension and negative for compression)

N_r = factored resistance in tension
p_c = outside perimeter of the concrete cross-section
p_h = perimeter of the centreline of the closed transverse torsion reinforcement
P_c = critical axial load
P_f = factored axial load (see Clauses 10 and 20)
= factored load at mid-height of panel (see Clause 23)
= maximum factored axial load for earthquake loading cases (see Clause 21)
P_n = earthquake-induced transfer force resulting from interaction between elements of a linked or coupled wall system, taken as the sum of the end shears corresponding to the nominal flexural resistance in the coupling beams above the section
P_{ns} = nominal net force on a cross-section for the direction being considered due to yielding in tension or compression of concentrated and distributed reinforcement during plastic hinge formation (positive for tension)
P_o = nominal axial resistance at zero eccentricity
P_p = earthquake-induced transfer force resulting from interaction between elements of a linked or coupled wall system, taken as the sum of the end shears corresponding to the probable flexural resistance in the coupling beams above the section
P_r = factored axial load resistance of wall
$P_{r,max}$ = maximum axial load resistance calculated using Equations (10-8) and (10-9)
P_{ro} = factored axial load resistance at zero eccentricity
P_s = axial force at section resulting from factored dead load plus factored live load using earthquake load factors (see Clause 21)
= service load at mid-height of panel (see Clause 23)
P_{tf} = factored load from tributary roof or floor area
P_{ts} = service load from tributary roof or floor area
P_{wf} = factored weight of panel tributary to and above design section
P_{ws} = unfactored weight of panel tributary to and above design section
Q = stability index for a storey
r = radius of gyration of cross-section of a compression member
R_d = ductility-related force modification factor, as specified in the *National Building Code of Canada*
R_o = overstrength-related force modification factor, as specified in the *National Building Code of Canada*
R_E = reduction factor on two-way shear stress as a function of interstorey deflection
s = factor for creep deflections under sustained loads (see Clause 9)
= maximum centre-to-centre spacing of transverse reinforcement within ℓ_d (see Clause 12)
= spacing of headed shear reinforcement or stirrups measured perpendicular to b_o (see Clause 13)
= spacing of shear or torsion reinforcement measured parallel to the longitudinal axis of the member (see Clause 11)
= spacing of transverse reinforcement measured along the longitudinal axis of the structural member (see Clause 21)
s_w = spacing of wire to be developed or spliced
s_x = longitudinal spacing of transverse reinforcement
s_z = crack spacing parameter dependent on crack control characteristics of longitudinal reinforcement (see Figure 11.2)
s_{ze} = equivalent value of s_z that allows for influence of aggregate size
S = variable loads due to ice, rain, and snow (including associated rain)

$S_a(0.2)$ = damped spectral response acceleration for a period of 0.2 s, as specified in the *National Building Code of Canada*

S_p = moment, shear, or axial force at connection corresponding to development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects

S_r = factored flexural, shear, or axial resistance of a connection

t = wall thickness (see Clauses 14 and 22)

= wall thickness of box section (see Clause 11)

T = effects of imposed deformations due to moisture changes, shrinkage, creep, temperature, and ground settlement or combinations thereof

T_{cr} = pure torsional cracking resistance

T_f = factored torsional moment

T_r = factored torsional resistance provided by circulatory shear flow

v_c = factored shear stress resistance provided by the concrete

v_f = factored shear stress

v_r = factored shear stress resistance (see Clauses 13 and 18)

= factored shear stress resistance of shear plane (see Clause 11)

v_s = factored shear stress resistance provided by shear reinforcement

V_c = shear resistance attributed to the concrete factored by ϕ_c

V_f = factored horizontal shear in a storey (see Clause 10)

= factored shear force (see Clauses 11, 12, 13, 17, 20, and 22)

V_{fb} = factored shear force through a beam-column joint acting parallel to beam bars

V_p = component in the direction of the applied shear of the effective prestressing force factored by ϕ_p; for variable depth members, the sum of the component of the effective prestressing force and the components of flexural compression and tension in the direction of the applied shear, positive if resisting applied shear, factored by ϕ_p

V_r = factored shear resistance

$V_{r\ell}$ = factored longitudinal shear resistance

$V_{r,max}$ = maximum possible factored shear resistance

V_s = shear resistance provided by shear reinforcement factored by ϕ_s

V_{se} = shear transmitted to column or column capital due to specified loads, but not less than the shear corresponding to twice the self-weight of the slab

w_b = width of a bearing for a concentrated vertical load acting on a wall

w_c = clear horizontal distance between adjacent shear wall webs, if webs are present

w_{df} = factored dead load per unit area

w_f = factored load per unit area (see Clause 13)

= factored load per unit length of beam or per unit area of slab (see Clause 9)

= factored uniformly distributed lateral load (see Clause 23)

w_{lf} = factored live load per unit area

w_s = service uniformly distributed lateral load

x = anchorage length of tension tie (see Clause 11)

= centroidal x-axis of a critical section (see Clause 13)

= direction of coordinates in elastic plate theory (see Clause 13.6.4)

= shorter overall dimension of rectangular part of cross-section (see Clause 13)

x_d = dimension from face of column to edge of drop panel (see Figure 13.1)

y = centroidal y-axis of a critical section (see Clause 13.3.5.5)
= direction perpendicular to coordinate x in elastic plate theory (see Clause 13.6.4)
= longer overall dimension of rectangular part of cross-section (see Clause 13.8.2.9)

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension (see Clause 9)
= distance from centroidal axis of section to extreme fibre in tension (see Clause 18)

z = quantity limiting distribution of flexural reinforcement

α = angle between inclined stirrups or bent-up bars and the longitudinal axis of the member (see Clause 11)
= ratio of moment of inertia of beam section to moment of inertia of a width of slab bounded laterally by centrelines of adjacent panels (if any) on each side of the beam, equal to I_b/I_s (see Clause 13)

α_c = section property reduction factor used for column effective stiffness properties

α_f = angle between shear friction reinforcement and shear plane

α_m = average value of α for beams on the four sides of a panel

α_s = factor that adjusts v_c for support dimensions

α_w = section property reduction factor used for wall effective stiffness properties

α_1 = ratio of average stress in rectangular compression block to the specified concrete strength (see Clause 10)
= α in direction of ℓ_1 (see Clause 13)

α_2 = α in direction of ℓ_2

β = factor accounting for shear resistance of cracked concrete (see Clauses 11 and 21)
= ratio of clear spans in long to short directions (see Clause 13)
= ratio of long side to the short side of footing (see Clause 15)

β_b = ratio of area of cut-off reinforcement to total area of tension reinforcement at section

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

β_d = for non-sway frames and for strength and stability checks of sway frames carried out in accordance with Clauses 10.16.4 and 10.16.5, the ratio of the maximum factored sustained axial load to the maximum factored axial load associated with the same load combination
= for sway frames, except as required by Clauses 10.16.4 and 10.16.5, the ratio of the maximum factored sustained shear within a storey to the maximum factored shear in that storey

β_p = shear stress factor (see Clause 18)

β_1 = ratio of depth of rectangular compression block to depth to the neutral axis

γ_c = density of concrete

γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections

γ_w = wall overstrength factor equal to the ratio of the load corresponding to nominal moment resistance of the wall system to the factored load on the wall system, but need not be taken as less than 1.3

Δ_f = deflection of the top of a wall due to the effects of factored loads

Δ_h = additional thickness of the drop panel below the soffit of the slab

Δ_o = initial panel out-of-straightness (see Clause 23)
= relative deflection of the top and bottom of a storey, computed in accordance with Clause 10

Δ_s = panel mid-height deflection under service lateral and vertical loads

δ_b = moment magnification factor to reflect the P-Δ effect at factored loads

δ_{bs} = moment magnification factor to reflect the P-Δ effect at service loads

δ_i = interstorey deflection calculated in accordance with the *National Building Code of Canada*

δ_s = moment magnification factor accounting for second-order effects of vertical load acting on a structure in a laterally displaced configuration

ε_{cu} = maximum strain at the extreme concrete compression fibre at ultimate

ε_s = strain in reinforcement (see Clause 8)

= tensile strain in tensile tie reinforcement due to factored loads (see Clause 11)

ε_x = longitudinal strain at mid-depth of the member due to factored loads (positive when tensile)

ε_1 = principal tensile strain in cracked concrete due to factored loads

ζ_s = deflection multiplier for sustained loads

θ = angle of inclination of diagonal compressive stresses to the longitudinal axis of the member

θ_{ic} = wall or coupling beam inelastic rotational capacity

θ_{id} = wall or coupling beam inelastic rotational demand

θ_s = smallest angle between compressive strut and adjoining tensile ties

λ = factor to account for low-density concrete

μ = coefficient of friction

ρ = ratio of non-prestressed tension reinforcement, equal to A_s/bd

ρ' = reinforcement ratio for compression reinforcement, equal to A_s'/bd

ρ_h = ratio of area of horizontal distributed reinforcement to gross concrete area perpendicular to this reinforcement

ρ_n = ratio of area of distributed reinforcement parallel to the plane to A_{cv} to gross concrete area perpendicular to that reinforcement

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member

ρ_{sk} = ratio of area of skin reinforcement to A_{cs}

ρ_t = ratio of total area of reinforcing steel to gross concrete section

ρ_v = ratio of shear friction reinforcement

σ = effective normal stress

ϕ = resistance factor applied to a specified material property or to the resistance of a member, connection, or structure, which for the limit state under consideration takes into account the variability of dimensions and material properties, quality of work, type of failure, and uncertainty in the prediction of resistance

ϕ_a = resistance factor for structural steel

ϕ_c = resistance factor for concrete

ϕ_m = member resistance factor

ϕ_p = resistance factor for prestressing tendons

ϕ_s = resistance factor for non-prestressed reinforcing bars

ψ = adjustment factor for moment of inertia for prismatic modelling of columns

ω = flange buckling factor

2.4 Standard notation and calculations

2.4.1 Standard notation for loads and resistances

In this Standard, the subscript *f* denotes a load effect based on factored loads and the subscript *r* denotes a resistance calculated using factored material strengths.

2.4.2 Standard notation for reinforcing bars

In this Standard, the standard notation for metric reinforcing bars is the bar designation number followed by the letter M.

2.4.3 Bar diameter for calculations
Except for calculations involving bar areas, the diameter, d_b, of metric reinforcing bars may be taken as the bar designation number.

3 Materials

3.1 Reinforcement

3.1.1
Reinforcement and prestressing tendons shall comply with Clause 6 of CSA A23.1.

Notes:

(1) *See also Clause 8.5.*

(2) *Pretensioned epoxy-coated strands should not be used in building structures because, in the event of a fire, heat will soften the coating and reduce bond.*

3.1.2
All reinforcement shall be deformed bars, except that plain bars may be used for spirals and plain bars smaller than 10 mm in diameter may be used for stirrups or ties.

3.1.3
Deformed reinforcement shall include
(a) reinforcing bars having deformations and complying with CAN/CSA-G30.18;
(b) welded wire fabric complying with ASTM A 185, with welded intersections not farther apart than 200 mm in the direction of the principal reinforcement, and with crosswires having a cross-sectional area of not less than 35% of that of the principal reinforcement (see also Clause 11.2.4(b));
(c) welded wire fabric complying with ASTM A 497/A497M, with welded intersections not farther apart than 400 mm in the direction of the principal reinforcement, and with crosswires having a cross-sectional area of not less than 35% of that of the principal reinforcement (see also Clause 11.2.4(b)); and
(d) deformed wire for concrete reinforcement complying with ASTM A 496, and not smaller than size MD25.

3.1.4
Prestressing tendons shall comply with the applicable requirements of ASTM A 416/A 416M, ASTM A421/A 421M, and ASTM A 722/A 722M.

3.2 Concrete and other materials

3.2.1
Cast-in-place concrete and constituent materials shall comply with CSA A23.1.

3.2.2
Precast concrete and constituent materials shall comply with CSA A23.4, except as specified in Clause 16.2.2.

4 Concrete quality, mixing, and placement

4.1 Quality

4.1.1
Concrete shall be proportioned and produced in accordance with CSA A23.1 or CSA A23.4, as applicable.

4.1.2
The compressive strength of concrete, f_c', shall be determined by testing as specified in CSA A23.1, CSA A23.2, or CSA A23.4, as applicable.

4.1.3
Unless otherwise specified, f_c' shall be based on 28 day tests.

4.2 Mixing and placement
Concrete shall be mixed, placed, and cured in accordance with CSA A23.1 or CSA A23.4, as applicable.

5 Drawings and related documents
In addition to the information required by the applicable building codes, the drawings and related documents for structures designed in accordance with this Standard shall include
(a) the size and location of all structural elements, reinforcement, and prestressing tendons;
(b) provision for dimensional changes resulting from prestress, creep, shrinkage, and temperature;
(c) the locations and details of expansion or contraction joints and permissible locations and details for construction joints;
(d) the magnitude and location of prestressing forces;
(e) the specified strength of concrete in various parts of the structure at stated ages or stages of construction and the nominal maximum size and type of aggregate;
(f) the required cover;
(g) identification of the applicable reinforcing steel Standard and the specified type and grade of reinforcement;
(h) the anchorage length and the location and length of lap splices;
(i) the type and location of welded splices and mechanical connections of reinforcement;
(j) the type and grade of prestressing steel; and
(k) identification of the protective coatings for reinforcement, prestressing tendons, and hardware.

6 Formwork, falsework, embedded pipes, and construction joints

6.1 General
Formwork, falsework, construction joints, and the placement of embedded pipes and hardware shall be as specified in CSA A23.1 or CSA A23.4, as applicable.

6.2 Embedded pipes and openings
Embedded pipes and openings for mechanical and other services shall be located so as to have a negligible impact on the strength of the construction or their effects on member strength shall be considered in the design.

6.3 Construction joints

Provision shall be made for the transfer of shear and other forces through construction joints.

Note: *Construction joints in floors should generally be located near the midspan of slabs, beams, or girders unless a beam intersects a girder in that location. In such cases, the joint in the girder should be offset a distance at least equal to the depth of the beam.*

7 Details of reinforcement

Note: *The clauses of CSA A23.1 referred to in this Clause are reproduced in Annex A.*

7.1 Hooks, bends, and headed bars

7.1.1 General

Standard hooks and bends shall comply with Clause 6.6.2 of CSA A23.1. Non-standard hooks or bends shall be detailed on the drawings.

7.1.2 Stirrups and ties

Stirrups and ties shall be anchored by standard stirrup and tie hooks or by heads of headed bars. The standard stirrup and tie hooks shall have a bend of at least 135° unless the concrete cover surrounding the hook is restrained against spalling, in which case a bend of at least 90° shall be permitted. Standard tie hooks with a bend of at least 90° shall be permitted for ties in columns having a specified concrete compressive strength equal to or less than 50 MPa. Stirrups and ties of size 20M and 25M shall have inside bend diameters in accordance with Table 16 of CSA A23.1.

7.1.3 Crossties

Crossties shall be anchored by standard tie hooks or by heads of headed bars. The standard tie hooks shall have a bend of at least 135° at one end and a standard tie hook with a bend of at least 90° at the other end. The hooks shall engage peripheral longitudinal bars. The 90° hooks of successive crossties engaging the same longitudinal bar shall be alternated end for end.

7.1.4 Headed bars and studs

Headed bars and studs with a head of an area equal to ten times the bar area shall be deemed capable of developing the tensile strength of the bar without crushing of the concrete under the head provided that the specified concrete compressive strength is equal to or greater than 25 MPa and the yield strength of the bar used in the design does not exceed 500 MPa.

7.2 Placing of reinforcement

7.2.1 General

Placing of reinforcement shall be shown on the drawings and shall be as specified in CSA A23.1 or CSA A23.4, as applicable.

7.2.2 Draped fabric

When welded wire fabric with wire of 6 mm diameter or less is used for slab reinforcement in slabs not exceeding 3 m in span, the reinforcement may be curved from a point near the top of the slab over the support to a point near the bottom of the slab at midspan, provided that such reinforcement is either continuous over or securely anchored at the support.

7.3 Tolerances

7.3.1
The tolerances for placing of reinforcement shall comply with CSA A23.1 or CSA A23.4, as applicable.

7.3.2
When design requirements necessitate closer tolerances than those specified in Clause 7.3.1, such tolerances shall be clearly indicated on the construction drawings.

7.4 Spacing of reinforcement and tendons

7.4.1 Bars

7.4.1.1
The minimum clear distance between parallel bars shall comply with CSA A23.1.

7.4.1.2
In walls and one-way slabs other than concrete joist construction, the principal reinforcement shall be spaced not farther apart than the smaller of three times the wall or slab thickness or 500 mm.

7.4.1.3
The clear distance between adjacent longitudinal reinforcing bars in compression members shall not be greater than 500 mm.

7.4.2 Bundled bars

7.4.2.1
Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four bars in any one bundle. Bundled bars shall be tied, wired, or otherwise fastened together to ensure that they remain in position.

7.4.2.2
Bars larger than 35M shall not be bundled in beams or girders.

7.4.2.3
Individual bars in a bundle cut off within the span of flexural members shall terminate at different points at least 40 bar diameters apart.

7.4.2.4
Where spacing limitations and clear concrete 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.

7.4.3 Pretensioning tendons

7.4.3.1
The clear distance between pretensioning wires or strands at each end of the member shall be not less than $4d_b$ for wire and not less than $3d_b$ for strands. Closer vertical spacing and bundling of strands may be used in the middle portion of the span.

7.4.3.2
The minimum clear space between groups of bundled strands shall be not less than 1.3 times the nominal maximum size of the coarse aggregate.

7.4.4 Post-tensioning tendons
The minimum clear distance between post-tensioning tendons and the requirements for bundling of post-tensioning tendons shall comply with CSA A23.1.

7.5 Special details for columns and walls

7.5.1 Offset bars

7.5.1.1
Where offset bent bars are used, the slope of the inclined portion of the bar with respect to the axis of the column shall not exceed 1:6, and the portions of the bar above and below the offset shall be parallel to the axis of the column. These details shall be shown on the drawings.

7.5.1.2
Adequate horizontal support at the offset bends shall be treated as a design matter and shall be provided by ties, spirals, or parts of the floor construction. Horizontal thrust to be resisted shall be taken as 1.5 times the horizontal component of the factored resistance in the inclined portion of the bar. Ties or spirals, if used, shall be placed not more than 150 mm from the point of the bend.

7.5.1.3
Where a column or wall face is offset by more than 75 mm, longitudinal bars shall not be offset bent.

7.5.2 Splices and load transfer in metal cores
In composite columns,
(a) splices of structural steel cores shall be made as specified in CAN/CSA-S16; and
(b) provision shall be made at column bases to transfer the loads to the footings as specified in Clause 15.9.

7.6 Transverse reinforcement

7.6.1 General
Transverse reinforcement shall comply with Clauses 7.6.2 to 7.6.6 and, where shear or torsion reinforcement is required, with Clause 11.

7.6.2 Composite columns
Transverse reinforcement in composite columns shall comply with Clauses 10.18 and 10.19.

7.6.3 Prestressing tendons
Transverse reinforcement for prestressing tendons shall comply with Clause 18.13.

7.6.4 Spirals for compression members

7.6.4.1
Spiral reinforcement for compression members shall comply with Clause 10.9.4 and, with respect to construction and spacers, with CSA A23.1.

7.6.4.2
Spiral reinforcement shall have a minimum diameter of 6 mm.

7.6.4.3
The pitch or distance between turns of the spirals shall not exceed 1/6 of the core diameter.

7.6.4.4
The clear spacing between successive turns of a spiral shall not be less than 25 mm or greater than 75 mm.

7.6.5 Ties for compression members

7.6.5.1
In compression members, all non-prestressed longitudinal bars of sizes 30M or smaller shall be enclosed by ties having a diameter of at least 30% of that of the largest longitudinal bar. All non-prestressed longitudinal bars of sizes 35M, 45M, and 55M, and all bundled bars, shall be enclosed by ties of at least size 10M. Deformed wire or welded wire fabric of equivalent area may be used.

7.6.5.2
Tie spacing shall not exceed the smallest of
(a) 16 times the diameter of the smallest longitudinal bars or the smallest bar in a bundle;
(b) 48 tie diameters;
(c) the least dimension of the compression member; and
(d) 300 mm in compression members containing bundled bars.
For specified concrete compressive strengths exceeding 50 MPa, the tie spacing determined in accordance with Items (a) to (d) shall be multiplied by 0.75.

7.6.5.3
Ties shall be located not more than one-half of a tie spacing above the slab or footing and shall be spaced as specified in Clause 7.6.5.2 to not more than one-half of a tie spacing below the lowest reinforcement in the slab or drop panel above.

7.6.5.4
Where beams or brackets frame into a column from four directions, the ties may be terminated not more than 75 mm below the lowest reinforcement in the shallowest of such beams or brackets. (See also Clause 7.7.)

7.6.5.5
Ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie having an included angle of not more than 135°, and no bar shall be farther than 150 mm clear on either side from such a laterally supported bar.

7.6.5.6
Where the bars are located around the periphery of a circle, a complete circular tie may be used, provided that the ends of the ties are lap welded or bent at least 135° around a longitudinal bar or otherwise anchored within the core of the column.

7.6.5.7
Welded wire fabric of equivalent area may be used if spliced in accordance with Clauses 12.18 and 12.19. The required splice lengths shall be shown on the drawings.

7.6.5.8
Where anchor bolts are placed in the tops of columns or pedestals, they shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 125 mm of the top of the column or pedestal and shall consist of at least two 10M bars.

7.6.6 Beams and girders — Transverse reinforcement

7.6.6.1
Compression reinforcement in beams and girders shall be enclosed by ties or stirrups satisfying the size and spacing requirements of Clauses 7.6.5.1 and 7.6.5.2 or by welded wire fabric of an equivalent area. Such ties or stirrups shall be provided along the length where compression reinforcement is required.

7.6.6.2
Transverse reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed stirrups or spirals extending completely around all main reinforcement.

7.6.6.3
Closed ties or stirrups shall be
(a) formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar;
(b) formed in one or two pieces spliced in accordance with the requirements for a Class B splice having a lap of $1.3\ell_d$; or
(c) anchored as specified in Clause 12.13.

7.7 Special details for beam-column connections

7.7.1
At connections of principal framing elements, such as beams and columns, an enclosure shall be provided for end anchorage of reinforcement terminating in such connections.

7.7.2
The enclosure specified in Clause 7.7.1 may consist of transverse framing members, internal closed ties, spirals, or stirrups.

7.7.3
When gravity load, wind, or other lateral forces cause the transfer of moments from beams to columns, transverse reinforcement (ties) not less than that required by Equation (11-1) shall be provided within connections of framing elements to columns. Except for connections that are part of a primary seismic force resisting system, this requirement may be waived if the connection is restrained on four sides by beams or slabs of approximately equal depth. (See also Clause 12.11.2.)

7.8 Minimum reinforcement in slabs

7.8.1
A minimum area of reinforcement of $0.002A_g$ shall be provided in each direction.

7.8.2
For exposure conditions where crack control is essential, reinforcement exceeding that required by Clause 7.8.1 shall be provided.

7.8.3
Minimum reinforcement shall not be spaced farther apart than the smaller of five times the slab thickness or 500 mm.

7.8.4
At all sections where it is required, minimum reinforcement shall be developed in tension for its specified yield strength in compliance with Clause 12.

7.8.5
Prestressing tendons used as minimum reinforcement shall comply with Clause 18.12.6.

7.9 Concrete protection for reinforcement
Concrete cover for reinforcement shall comply with the cover requirements of CSA A23.1, CSA A23.4, or CAN/CSA-S413, as applicable, unless special conditions dictate otherwise. In all cases, concrete cover shall be indicated on the drawings.

8 Design — Limit states, load combinations, and material properties

8.1 Limit states

8.1.1 Durability
Concrete structures shall satisfy the durability requirements of CSA A23.1, CSA A23.4, or CAN/CSA-S413, as applicable, for the intended use and exposure conditions.

8.1.2 Fire resistance
Concrete structures shall satisfy the fire resistance requirements of the applicable building code.

8.1.3 Ultimate limit states
Structures, structural members, and connections shall be designed such that factored resistance is greater than or equal to the effect of factored loads, with the effect of factored loads being determined as specified in Clauses 8.2 and 8.3 and the factored resistance being determined as specified in Clause 8.4.

8.1.4 Serviceability limit states

8.1.4.1 Deflections
Structures and structural members shall be designed to satisfy the deflection control requirements of Clauses 9.8 and 13.2.7, with loadings as specified in Clause 8.3.3.

8.1.4.2 Local damage and cracking
Structural members and connections shall be designed to meet the minimum reinforcement area and maximum reinforcement spacing requirements of this Standard as well as the requirements of Clauses 10.6 and 18.1 to 18.4, with loadings as specified in Clause 8.3.3.
Note: *This Standard does not specifically limit crack widths.*

8.1.4.3 Vibrations
In the design of structures and structural members, consideration shall be given to controlling vibrations within acceptable limits for the intended use.

8.1.5 Structural integrity

Consideration shall be given to the robustness of the overall structural system to minimize the likelihood of a progressive type of collapse.

Notes:

(1) *Provisions for structural integrity are required for two-way slabs (Clause 13.10.6), precast concrete structures (Clause 16.5), and tilt-up structures (Clause 23.2.9).*

(2) *The requirements in this Standard generally provide a satisfactory level of structural integrity for most concrete structures for buildings. It is possible that supplementary provisions for structural integrity will be needed for mixed or unusual structural systems or for structures exposed to severe loads such as vehicle impacts or explosions. For further guidance, refer to Commentary B in the NRCC's* User's Guide *to Part 4 of the* National Building Code of Canada.

8.2 Loading

8.2.1 General

Loads shall be determined in accordance with the requirements of the applicable building code.

8.2.2 Imposed deformations

8.2.2.1 General

The short-term and long-term forces and effects resulting from the interaction of the stiffness of the structure and imposed deformations such as differential settlement, non-uniform or restrained temperature changes, and restraint of shrinkage and creep shall be considered.

Notes:

(1) *Imposed deformations produce self-equilibrating moments, reactions, and stresses.*

(2) *Imposed deformations can require considerable redistribution of internal forces, which can lead to excessive cracking at service load or to brittle failure.*

(3) *Estimates of differential settlement, creep, shrinkage, or temperature change should be based on realistic assessments of such effects occurring in service. The magnitude of the internal forces and the effects of imposed deformations depend on the magnitude of the deformation, the stiffness of the structure (cracked or uncracked) resisting the deformations, and the time necessary for the deformations to occur.*

8.2.2.2 Load factor for T-loads

When deemed necessary by the designer, imposed deformations, T, shall be included in the appropriate load combinations, with the load factor specified in the applicable building code.

8.2.3 Prestress

In statically indeterminate structures, prestress normally causes secondary moments and reactions. These shall be included in ultimate limit state design calculations, with the load factor specified in the applicable building code.

8.3 Load combinations and load factors

Note: *See Annex C.*

8.3.1 General

Structures, structural members, and connections shall be designed to resist the bending moments, axial loads, shear forces, and torsions computed from the factored loads and load combinations specified in Clauses 8.3.2 and 8.3.3 and the applicable building code.

8.3.2 Load combinations for ultimate limit states

The effect of factored loads acting on structures, structural members, and connections shall be determined in accordance with the factored load combinations specified in the applicable building code.

Note: *See Table C.1.*

8.3.3 Load combinations for serviceability limit states

A building and its structural components shall be checked for the applicable serviceability limit states specified in Clause 8.1.4 under the effects of the service loads. The applicable load combination shall be taken as the one that results in the most unfavourable effect for the limit state under consideration.

8.4 Factored resistance

Note: *See Note (3) of the preliminary Notes to Annex C.*

8.4.1 General

The factored resistance of a member, its cross-sections, and its connections shall be taken as the resistance calculated as specified in this Standard, using the material resistance factors specified in Clauses 8.4.2 and 8.4.3.

Notes:

(1) *Member resistance factors are used in Clauses 10.15.3, 10.16.3.2, and 23.3.1.3.*

(2) *In a few cases the member rigidity, EI, is multiplied by a member resistance factor, ϕ_m, specified in the applicable clauses.*

8.4.2 Factored concrete strength

The factored concrete compressive strengths used in checking ultimate limit states shall be taken as $\phi_c f'_c$. The factored concrete tensile strengths used in checking ultimate limit states are given in terms of $\phi_c \sqrt{f'_c}$, where $\phi_c = 0.65$, except as specified in Clause 16.1.3.

8.4.3 Factored reinforcement and tendon force

The factored force in reinforcing bars, tendons, and structural shapes shall be taken as the product of the appropriate resistance factor, ϕ, and the respective steel force as specified in the applicable clause of this Standard, where

(a) ϕ_s = 0.85 for reinforcing bars and embedded steel anchors;

(b) ϕ_p = 0.90 for prestressing tendons; and

(c) ϕ_a = 0.90 for structural steel.

8.5 Reinforcement and tendon properties for design

8.5.1 Design strength for reinforcement

Design calculations shall be based on the specified yield strength of reinforcement, f_y, which shall not exceed 500 MPa except for prestressing tendons.

8.5.2 Compression reinforcement

For compression reinforcement having a specified yield strength exceeding 400 MPa, the value of f_y assumed in design calculations shall not exceed the stress corresponding to a strain of 0.35%.

Note: *CAN/CSA-G30.18 defines the yield strength of Grade 500 reinforcement at a strain of 0.35%.*

8.5.3 Stress-strain curve for reinforcement

8.5.3.1 Reinforcement and tendon stress-strain curve

The force in the reinforcement shall be calculated as ϕ_s for reinforcing bars and ϕ_p for prestressing tendons, multiplied by the force determined from strain compatibility based on a stress-strain curve representative of the steel.

8.5.3.2 Simplified reinforcement stress-strain curve
For reinforcement with a specified yield strength of 500 MPa or less, the following assumptions may be used:

(a) for strains, ε_s, less than the yield strain, f_y/E_s, the force in the reinforcement shall be taken as $\phi_s A_s E_s \varepsilon_s$; and

(b) for strains, ε_s, greater than the yield strain, the force in the reinforcement shall be taken as $\phi_s A_s f_y$.

8.5.4 Modulus of elasticity of reinforcement

8.5.4.1
The modulus of elasticity of reinforcing bars, E_s, shall be taken as 200 000 MPa.

8.5.4.2
The modulus of elasticity for tendons, E_p, shall be determined by tests or supplied by the manufacturer.
Note: *Typical values of E_p range from* 190 000 *to* 200 000 MPa.

8.5.5 Coefficient of thermal expansion of reinforcement
The coefficient of thermal expansion may be taken as 10×10^{-6} / °C.

8.6 Concrete properties for design

8.6.1 Design strength of concrete

8.6.1.1
Specified concrete compressive strengths used in design shall not be less than 20 MPa or more than 80 MPa, except as allowed by Clauses 8.6.1.2, 8.6.1.3, and 22.1.1 or restricted by Clauses 11.3.6.3, 12.1.2, 18.12.3.3, and 21.2.6.
Note: *Designers planning to use specified concrete strengths exceeding* 50 MPa *should determine whether the appropriate concretes are available. Higher strengths can require prequalification of concrete suppliers and contractors and special construction techniques.*

8.6.1.2
The upper limit on the specified concrete compressive strength specified in Clause 8.6.1.1 may be waived if the structural properties and detailing requirements of reinforced concretes having a strength exceeding 80 MPa are established for concretes similar to those to be used.
Note: *High-strength concretes vary in their brittleness and need for confinement.*

8.6.1.3
Strengths lower than those specified in Clause 8.6.1.1 may be used for mass concrete, plain concrete, or strength evaluation of existing structures.

8.6.2 Modulus of elasticity

8.6.2.1
The modulus of elasticity of concrete in compression, E_c, used in design shall be taken as the average secant modulus for a stress of 0.40 f'_c determined for similar concrete in accordance with ASTM C 469. If the modulus of elasticity is critical to the design, a minimum value of E_c shall be specified and shown on the drawings.
Note: *If the modulus of elasticity is critical to the design, the designer should establish whether such concrete can be produced.*

8.6.2.2
In lieu of results from tests of similar concrete, the modulus of elasticity, E_c, for concrete with γ_c between 1500 and 2500 kg/m^3 may be taken as

$$E_c = (3300\sqrt{f'_c} + 6900)\left(\frac{\gamma_c}{2300}\right)^{1.5} \quad \textbf{(8-1)}$$

8.6.2.3
In lieu of Clauses 8.6.2.1 and 8.6.2.2, the modulus of elasticity, E_c, of normal density concrete with compressive strength between 20 and 40 MPa may be taken as

$$E_c = 4500\sqrt{f'_c} \quad \textbf{(8-2)}$$

Note: *The value of E_c is affected by the aggregate fraction in the mix, the modulus of elasticity of the aggregates, and the loading rate. The modulus of elasticity of Canadian concretes will generally be between 80 and 120% of the values specified in Clauses 8.6.2.2 and 8.6.2.3.*

8.6.3 Concrete stress-strain relationship
The concrete compressive stress-strain relationship used in design shall conform to Clause 10.1.6.

8.6.4 Modulus of rupture of concrete
The modulus of rupture, f_r, shall be taken as

$$f_r = 0.6\lambda\sqrt{f'_c} \quad \textbf{(8-3)}$$

8.6.5 Modification factors for concrete density
The effect of low-density aggregates on tensile strength and other related properties shall be accounted for by the factor λ, where
(a) λ = 1.00 for normal density concrete;
(b) λ = 0.85 for structural semi-low-density concrete in which all the fine aggregate is natural sand; and
(c) λ = 0.75 for structural low-density concrete in which none of the fine aggregate is natural sand.
Linear interpolation may be applied based on the fraction of natural sand in the mix.

8.6.6 Coefficient of thermal expansion of concrete
For the purpose of structural analysis, the coefficient of thermal expansion of concrete may be taken as 10×10^{-6} / °C.
Note: *The value of the coefficient of thermal expansion depends on the type of aggregates, the moisture state of the concrete, and the temperature of the concrete. It can vary between approximately 6×10^{-6} / °C to 13×10^{-6} / °C for concrete at temperatures between 0 and 80 °C.*

9 Structural analysis and computation of deflections

9.1 Methods of analysis

9.1.1
All members of frames or continuous construction shall be designed for the maximum effects of the factored loads as determined by an analysis carried out in accordance with one of the methods of analysis specified in Clauses 9.2 to 9.7.

9.1.2
All structural analyses shall satisfy equilibrium conditions.

9.2 Elastic frame analysis
Load effects may be determined by elastic analysis based on the assumptions specified in Clauses 9.2.1 to 9.2.4.

9.2.1 Stiffness

9.2.1.1
Assumptions for computing the relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems shall be consistent throughout the analysis.

9.2.1.2
Member stiffnesses used in analyses of the lateral deflections of frames or in second-order frame analyses shall be representative of the degree of member cracking and inelastic action at the loading stage for which the analysis is being carried out.

9.2.1.3
The effect of variable cross-sections shall be considered both in determining bending moments and in the design of the members.

9.2.2 Span length

9.2.2.1
For determining moments in continuous frames, the span length shall be taken as the distance from centre-to-centre of supports.

9.2.2.2
For beams or one-way slabs built integrally with their supports, or for columns in continuous frames, moments at the faces of the joints may be used for design.

9.2.2.3
The span length of a member that is not built integrally with its supports shall be taken as the clear span plus, at each end, half of its depth, but need not exceed the distance between centres of supports.

9.2.2.4
In the analysis of frames containing shear walls, the effect of the width of the wall on the stiffness of the beams framing into the wall shall be considered.

9.2.3 Arrangement of loads

9.2.3.1 Continuous beams and one-way slabs
For continuous beams and one-way slabs, the arrangements of live and dead loads may be limited to combinations of
(a) factored dead load of the structure and factored permanent superimposed dead load on all spans, with factored partition load and factored live load on two adjacent spans;
(b) factored dead load of the structure and factored permanent superimposed dead load on all spans, with factored partition load and factored live load on alternate spans; and
(c) factored dead and factored live load on all spans.

Note: *The superimposed dead load can (but need not) be patterned, depending on the circumstances.*

9.2.3.2 Two-way slabs
Two-way slabs analyzed using the elastic frame method shall be analyzed for the loading patterns specified in Clause 13.8.4.

9.2.4 Redistribution of moments in continuous flexural members
Except when approximate values for bending moments are used, the negative moments at the supports of continuous flexural members calculated by elastic analysis for any assumed loading arrangement may each be increased or decreased by not more than $(30 - 50c/d)$%, but not more than 20%, and the modified negative moments shall be used for calculating the moments at sections within the spans.

9.3 Approximate frame analysis

9.3.1 General
Except for prestressed concrete, approximate methods of frame analysis may be used for buildings having typical spans, storey heights, and types of construction.

9.3.2 Floor and roof loads
The moments due to floor and roof loads may be computed using an elastic analysis of a portion of the frame consisting of the floor or roof in question, with the columns above and below the floor assumed fixed at their far ends.

9.3.3 Moment and shear coefficients
In lieu of a more accurate method of frame analysis, the approximate moments and shears specified in Table 9.1 may be used in the design of continuous beams and one-way slabs, provided that
(a) there are two or more spans;
(b) the spans are approximately equal, with the longer of two adjacent spans not greater than the shorter by more than 20%;
(c) the loads are uniformly distributed;
(d) the factored live load does not exceed twice the factored dead load; and
(e) the members are prismatic.

For calculating negative moments at interior supports, ℓ_n shall be taken as the average of the adjacent clear span lengths.

Table 9.1
Approximate moments and shears
(See Clause 9.3.3.)

Moment or shear	Value
Positive moments	
End spans	
Discontinuous end unrestrained	$w_f \ell_n^2/11$
Discontinuous end integral with support	$w_f \ell_n^2/14$
Interior spans	$w_f \ell_n^2/16$
Negative moments	
Negative moment at exterior face of first interior support	
Two spans	$w_f \ell_n^2/9$
More than two spans	$w_f \ell_n^2/10$
Negative moment at other faces of interior supports	$w_f \ell_n^2/11$
Negative moment at interior face of exterior support for members built integrally with supports	
Where the support is a spandrel beam or girder	$w_f \ell_n^2/24$
Where the support is a column	$w_f \ell_n^2/16$
Shear	
Shear in end members at face of first interior support	$1.15 w_f \ell_n/2$
Shear at faces of all other supports	$w_f \ell_n/2$

9.4 Analysis by strut-and-tie models

Strut-and-tie models satisfying the requirements of Clause 11.4 may be used to determine the internal force effects, proportion the reinforcement, and confirm the concrete dimensions.
Note: *Such models are particularly appropriate in regions where plane sections do not remain plane.*

9.5 Finite element analysis

9.5.1

Finite element analysis or other numerical techniques may be used to determine load effects, provided that the differences between the behaviour of the structure and the behaviour assumed in the analysis are accounted for.
Note: *The analysis should account for the effects of cracking. If the effects of cracking are not included, the redistribution of stresses due to the anticipated cracking and the effects of this redistribution on the reinforcement layout should be explicitly considered in the design of the reinforcement.*

9.5.2

Mesh patterns and boundary conditions shall be consistent with geometry, loading, and restraint conditions. Alternative loading cases shall be considered where applicable. Care shall be taken to ensure realistic modelling of the size and stiffness of supporting elements.

9.5.3
Principal reinforcement may be concentrated in bands or tension ties. Anchorage of the reinforcement shall be explicitly considered.

9.5.4
The analysis shall be checked using independent techniques satisfying equilibrium.

9.5.5
Crack control and deflections shall be considered.

9.6 Elastic plate analysis
Analysis of planar structural elements may be based on elastic plate theory (see Clause 13.6).

9.7 Plastic analysis

9.7.1
A plastic analysis shall satisfy either the upper bound theorem or the lower bound theorem of plasticity.

9.7.2
A plastic analysis may assume either a rigid-plastic or an elastic-plastic behaviour.

9.7.3
Hinging sections shall be detailed to provide the rotational capacity assumed in the analysis.

9.7.4
Plastic analyses shall not be used for sway frames.

9.8 Control of deflections

9.8.1 General
Reinforced concrete members subject to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that could adversely affect the strength or serviceability of the structure.

9.8.2 One-way construction (non-prestressed)

9.8.2.1 Minimum thickness
The minimum thickness specified in Table 9.2 shall apply to one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness can be used without adverse effects.

Note: *It is possible that Table 9.2 will not apply to members that have high ratios of superimposed dead or live loads to the self weight.*

Table 9.2
Thicknesses below which deflections are to be computed for non-prestressed beams or one-way slabs not supporting or attached to partitions or other construction likely to be damaged by large deflections
(See Clauses 9.8.2.1, 9.8.5.1, 9.8.5.2, and 13.2.6.)

	Minimum thickness, *h*			
	Simply supported	**One end continuous**	**Both ends continuous**	**Cantilever**
Solid one-way slabs	$\ell_n/20$	$\ell_n/24$	$\ell_n/28$	$\ell_n/10$
Beams or ribbed one-way slabs	$\ell_n/16$	$\ell_n/18$	$\ell_n/21$	$\ell_n/8$

Note: *The values specified in this Table shall be used directly for members with normal-density concrete where* $\gamma_c > 2150$ kg/m^3 *and the reinforcement is Grade 400. For other conditions, the values shall be modified as follows:*
(a) *for structural low-density concrete and structural semi-low-density concrete, the values shall be multiplied by* $(1.65-0.0003\gamma_c)$*, but not less than 1.0, where* γ_c *is the density in kilograms per cubic metre; and*
(b) *for* f_y *other than* 400 MPa*, the values shall be multiplied by* $(0.4 + f_y/670)$*.*

9.8.2.2 Immediate deflections
When deflections are to be computed, deflections that occur immediately on application of load shall be computed by methods or formulas for elastic deflections, taking into consideration the effects of cracking and reinforcement on member stiffness.
Note: *Deflections may be calculated using formulas for elastic deflections based on effective moments of inertia as specified in Clauses 9.8.2.3 to 9.8.2.5, or by methods based on the integration of curvatures at sections along the span.*

9.8.2.3 E_c and I_e
Unless deflections are determined by a more comprehensive analysis, immediate deflection shall be computed using elastic deflection equations; a modulus of elasticity, E_c, for concrete as specified in Clause 8.6.2; and the effective moment of inertia, as follows:

$$I_e = I_{cr} + \left(I_g - I_{cr}\right)\left(\frac{M_{cr}}{M_a}\right)^3 \le I_g \quad \textbf{(9-1)}$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \quad \textbf{(9-2)}$$

and f_r is as given in Clause 8.6.4, except for two-way slabs (see Clause 13.2.7).
Note: *Construction loading may govern the determination of* M_a*.*

9.8.2.4 Moment of inertia for continuous spans
For continuous prismatic members, the effective moment of inertia may be taken as the weighted average of the values obtained from Equation (9-1) for the critical positive and negative moment sections, as follows:
(a) two ends continuous:

$$I_{e,avg} = 0.71_{em} + 0.15\left(I_{e1} + I_{e2}\right) \quad \textbf{(9-3)}$$

(b) one end continuous:

$$I_{e,avg} = 0.85I_{em} + 0.15I_{ec} \quad \textbf{(9-4)}$$

9.8.2.5 Sustained load deflections

Unless values are obtained by a more comprehensive analysis, the total immediate plus long-term deflection for flexural members shall be obtained by multiplying the immediate deflection caused by the sustained load considered by the factor ζ_s, as follows:

$$\zeta_s = \left(1 + \frac{s}{1+50\rho'}\right) \quad \textbf{(9-5)}$$

where

ρ' = the value at midspan for simple and continuous spans and at the support for cantilevers

The time dependent factor, s, for sustained loads shall be taken to be equal to the following values:

For loads sustained for five years or more	2.0
For loads sustained for 12 months	1.4
For loads sustained for six months	1.2
For loads sustained for three months	1.0

9.8.2.6 Deflection limits

The deflection computed in accordance with Clauses 9.8.2.2 to 9.8.2.5 shall not exceed the limits specified in Table 9.3.

9.8.3 Two-way construction (non-prestressed)

Deflection control of two-way slab systems shall be checked using Clause 13.2.

9.8.4 Prestressed concrete construction

9.8.4.1 Immediate deflection

For flexural members designed in accordance with Clause 18, immediate deflection shall be computed by methods or formulas for elastic deflection.

9.8.4.2 Moment of inertia

The moment of inertia of the gross concrete section may be used for sections that are uncracked at service loads.

9.8.4.3 Partially prestressed members

For partially prestressed members (i.e., members not satisfying Clause 18.3.2(c)), the reduction in sectional stiffness caused by cracking shall be taken into account.

9.8.4.4 Sustained load deflections

The additional long-term deflection of prestressed concrete members shall be computed by taking into account stresses in concrete and steel under sustained load, including effects of creep and shrinkage of concrete and relaxation of steel.

9.8.4.5 Deflection limits
The computed deflection shall not exceed the limits specified in Table 9.3.

9.8.5 Composite construction

9.8.5.1 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 the computation of deflection. For non-prestressed composite members containing more than one type of concrete, the portion of the member in compression shall determine whether the values specified in Table 9.2 for normal density or low-density concrete shall apply. If deflection is computed, account shall be taken of curvatures resulting from differential shrinkage of precast and cast-in-place components and of axial creep effects in a prestressed concrete member.

9.8.5.2 Unshored construction
If the thickness of a non-prestressed precast flexural member meets the requirements of Table 9.2, deflection need not be computed. If the thickness of a non-prestressed composite member meets the requirements of Table 9.2, deflection occurring after the member becomes composite need not be computed, but the long-term deflection of the precast member should be investigated for magnitude and duration of load before the beginning of effective composite action.

9.8.5.3 Deflection limits
Deflection computed in accordance with Clauses 9.8.5.1 and 9.8.5.2 shall not exceed the limits specified in Table 9.3.

Table 9.3
Maximum permissible computed deflections
(See Clauses 9.8.2.6, 9.8.4.5, 9.8.5.3, 13.2.2, and 13.2.7.)

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to specified live load, *L*, or snow load, *S*	$\ell_n/180$*
Floors not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to specified live load, *L*	$\ell_n/360$
Roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of non-structural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell_n/480$‡
Roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections	That part of the total deflection occurring after attachment of non-structural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell_n/240$§

**This limit is not intended to guard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and the long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage should be taken into consideration.*
†*Long-term deflections shall be determined in accordance with Clause 9.8.2.5 or 9.8.4.4, but may be reduced by the amount of deflection calculated to occur before the attachment of non-structural elements.*
‡*This limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.*
§*This limit shall not be greater than the tolerance provided for non-structural elements. It may be exceeded if camber is provided so that total deflection minus camber does not exceed the limit.*
Note: *For two-way slab construction, ℓ_n shall be taken as the clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.*

10 Flexure and axial loads

10.1 General principles

10.1.1 General

The factored moment and axial load resistance of members shall be based on strain compatibility and equilibrium using material resistance factors and material properties specified in Clause 8 and the additional assumptions specified in Clauses 10.1.2 to 10.1.7.

10.1.2 Plane sections assumption
The strain in reinforcement and concrete shall be assumed to be directly proportional to the distance from the neutral axis, except for unbonded tendons, deep flexural members (see Clause 10.7), and regions of discontinuities.

10.1.3 Maximum concrete strain
The maximum strain at the extreme concrete compression fibre shall be assumed to be 0.0035.

10.1.4 Balanced strain conditions
Balanced strain conditions shall exist at a cross-section when the tension reinforcement reaches its yield strain just as the concrete in compression reaches its maximum strain of 0.0035.

10.1.5 Tensile strength of concrete
The tensile strength of concrete shall be neglected in the calculation of the factored flexural resistance of reinforced and prestressed concrete members.

10.1.6 Concrete stress-strain relationship
The relationship between the compressive stress and concrete strain may be based on stress-strain curves or assumed to be any shape that results in a prediction of strength in substantial agreement with the results of comprehensive tests.

Note: *To account for differences between the in-place strength and the strength of standard cylinders, stress blocks should be based on stress-strain curves with a peak stress not greater than 0.9* f'_c. *The equations in Clause 10.1.7 include this factor.*

10.1.7 Equivalent rectangular concrete stress distribution
The requirements of Clause 10.1.6 may be satisfied by an equivalent rectangular concrete stress distribution defined by the following:

(a) a concrete stress of $\alpha_1 \phi_c f'_c$ shall be assumed to be uniformly distributed over an equivalent compression zone bounded by edges of the cross-section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain;

(b) the distance c shall be measured in a direction perpendicular to that axis; and

(c) the factors α_1 and β_1 shall be taken as follows:

$$\alpha_1 = 0.85 - 0.0015\, f'_c \text{ (but not less than 0.67)} \qquad \textbf{(10-1)}$$

$$\beta_1 = 0.97 - 0.0025\, f'_c \text{ (but not less than 0.67)} \qquad \textbf{(10-2)}$$

10.2 Flexural members — Distance between lateral supports

10.2.1
Unless a stability analysis, including the effects of torsional loading, is carried out, beams shall comply with the limits specified in Clauses 10.2.2 and 10.2.3.

10.2.2
For a simply supported or continuous beam, the distance between points at which lateral support is provided shall not exceed the smaller of $50b$ or $200b^2/d$.

10.2.3
For a cantilever beam having lateral restraint at the support, the distance between the face of the support and the end of the cantilever shall not exceed the smaller of $25b$ or $100b^2/d$.

10.3 Flexural members — T-beams

10.3.1
In T-beams, the flange and web shall be built integrally or otherwise effectively bonded together.

10.3.2
A floor topping shall not be included as part of a structural member unless it is placed monolithically with the floor slab or designed in accordance with Clause 17.

10.3.3
The effective flange width of T-beams shall be based on overhanging flange widths on each side of the web, which shall not exceed the smallest of
(a) one-fifth of the span length for a simply supported beam;
(b) one-tenth of the span length for a continuous beam;
(c) 12 times the flange thickness; or
(d) one-half of the clear distance to the next web.

10.3.4
For beams with a slab on one side only, the effective overhanging flange width shall not exceed the smallest of
(a) 1/12 of the span length of the beam;
(b) six times the flange thickness; or
(c) one-half of the clear distance to the next web.

10.4 Flexural members — Joist construction

10.4.1
Joist construction shall consist of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions. Joist construction shall meet the following limits:

Minimum rib width	100 mm
Maximum rib depth	3.5 times the minimum width of rib
Maximum clear distance between ribs	800 mm
Minimum slab thickness	1/12 of the clear distance between ribs, but not less than 50 mm

10.4.2
Construction not meeting the limitations of Clause 10.4.1 shall be designed as slabs and beams.

10.5 Flexural members — Reinforcement

10.5.1 Minimum reinforcement

10.5.1.1
At every section of a flexural member where tensile reinforcement is required by analysis, minimum reinforcement shall be proportioned so that

$$M_r \geq 1.2\ M_{cr} \quad \textbf{(10-3)}$$

where the cracking moment, M_{cr}, is calculated using the modulus of rupture, f_r, specified in Clause 8.6.4.

10.5.1.2

In lieu of Clause 10.5.1.1, minimum reinforcement may be determined as follows:

(a) for slabs and footings, as specified in Clause 7.8; and

(b) for other flexural members, as follows:

$$A_{s,min} = \frac{0.2\sqrt{f'_c}}{f_y} b_t h \quad \textbf{(10-4)}$$

where

b_t = the width of the tension zone of the section considered

For T-beams with the flange in tension, b_t need not exceed $1.5b_w$ for beams with a flange on one side of the web or $2.5b_w$ for beams with a flange on both sides of the web.

10.5.1.3

The requirements of Clauses 10.5.1.1 and 10.5.1.2 may be waived if the factored moment resistance, M_r, is at least one-third greater than the factored moment, M_f.

10.5.2 Limit of *c/d* for yielding of tension reinforcement

The tension reinforcement in flexural members shall not be assumed to reach yield unless

$$\frac{c}{d} \leq \frac{700}{700 + f_y} \quad \textbf{(10-5)}$$

For flexural members without axial loads, the area of tension reinforcement shall be limited such that Equation (10-5) is satisfied. In columns or walls, when c/d exceeds this limit the stress in the tension reinforcement shall be computed based on strain compatibility.

10.5.3 Reinforcement in T-beam flanges

10.5.3.1 Flexural tension reinforcement

Where flanges are in tension, part of the flexural tension reinforcement shall be distributed over an overhanging flange width equal to 1/20 of the beam span, or the width specified in Clause 10.3, whichever is smaller. The area of this reinforcement shall be not less than 0.004 times the gross area of the overhanging flange.

10.5.3.2 Transverse reinforcement

Where the principal reinforcement in the slab forming a T-beam flange is parallel to the beam, transverse reinforcement meeting the requirement of Equation (10-4) shall extend past the face of the web a distance of 0.3 times the clear distance between the webs of the T-beams and shall extend at least to the outer bars of the flexural tension reinforcement required by Clause 10.5.3.1.

10.6 Beams and one-way slabs — Crack control

10.6.1 Crack control parameter

Bars in flexural tension zones shall be spaced so that the quantity z given by

$$z = f_s(d_c A)^{1/3} \quad \textbf{(10-6)}$$

does not exceed 30 000 N/mm for interior exposure and 25 000 N/mm for exterior exposure. The calculated stress in reinforcement at specified load, f_s, shall be computed as the moment divided by the product of the steel area and the internal moment arm. In lieu of such computations, f_s may be taken as 60% of the specified yield strength f_y. In calculating d_c and A, the effective clear concrete cover need not be taken to be greater than 50 mm. If epoxy-coated reinforcement is used, the value of z given by Equation (10-6) shall be multiplied by a factor of 1.2.

Note: *It is possible that the requirements of this Clause will not be sufficient for structures subject to very aggressive exposure or designed to be watertight.*

10.6.2 Skin reinforcement

For reinforced members with an overall depth, h, exceeding 750 mm, longitudinal skin reinforcement shall be uniformly distributed along the exposed side faces of the member for a distance $0.5h - 2(h-d)$ nearest the principal reinforcement. The total area of such reinforcement shall be $\rho_{sk} A_{cs}$, where A_{cs} is the sum of the area of concrete in strips along each exposed side face, each strip having a height of $0.5h - 2(h-d)$ and a width of twice the distance from the side face to the centre of the skin reinforcement (but not more than half the web width), and where $\rho_{sk} = 0.008$ for interior exposure and 0.010 for exterior exposure.

The maximum spacing of the skin reinforcement shall be 200 mm. Such skin reinforcement may be included in strength calculations if a strain compatibility analysis is conducted to determine the stresses in individual bars.

10.7 Deep flexural members

10.7.1

Flexural members with clear span to overall depth ratios less than 2 shall be designed as deep flexural members, taking into account non-linear distribution of strain, lateral buckling, and the increased anchorage requirements in such members. In lieu of more accurate procedures, the strut-and-tie model of Clause 11.4 may be used. (See also Clause 12.10.5.)

10.7.2

Minimum horizontal and vertical reinforcement in the side faces of deep flexural members shall satisfy the requirements of Clauses 10.6.2 and 11.4.5.

10.8 Design of bearing zones

10.8.1

The factored bearing resistance of concrete, other than at post-tensioning anchorages, shall not exceed $0.85\phi_c f'_c A_1$, except that when the supporting surface is wider on all sides than the loaded area, the bearing resistance on the loaded area may be multiplied by $\sqrt{A_2 / A_1}$, but not more than 2.

10.8.2

Reinforcement shall be provided where required in bearing zones to resist bursting, splitting, and spalling forces.

Note: *Guidance can be found in chapter 3 of the* Design manual *published by the Canadian Precast/Prestressed Concrete Institute.*

10.9 Columns — Reinforcement limits

10.9.1

The area of longitudinal bars for compression members shall be not less than 0.01 times the gross area, A_g, of the section, except as permitted by Clause 10.10.5.

10.9.2

The area of longitudinal bars for compression members, including regions containing lap splices, shall not exceed 0.08 times the gross area of the section (see Clause 12.17.2).

Note: *The use of more than 4% of reinforcement in a column outside of the region of lap splices can involve serious practical difficulties in placing and compacting the concrete, and in placing reinforcement in beam column joints.*

10.9.3

The minimum number of longitudinal reinforcing bars in compression members shall be four for bars within rectangular or circular ties, three for bars within triangular ties, and six for bars enclosed by spirals complying with Clause 10.9.4.

10.9.4

The ratio of spiral reinforcement shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_y} \qquad \textbf{(10-7)}$$

where

f_y = the specified yield strength of spiral reinforcement (not to be taken more than 500 MPa)

10.10 Columns — Resistance

10.10.1

Columns shall be designed to have adequate factored resistance under the combinations of factored axial load and moment giving the maximum and minimum ratios of moment to axial load.

10.10.2

Columns supporting two-way slabs shall be designed to meet the additional requirements of Clause 13.

10.10.3

Slender compression members shall be designed for moments magnified in accordance with Clauses 10.13 to 10.16.

10.10.4

The maximum factored axial load resistance, $P_{r,max}$, of compression members shall be

(a) for spirally reinforced columns:

$$P_{r,\,max} = 0.85\ P_{ro} \qquad \textbf{(10-8)}$$

(b) for tied columns:

$$P_{r,\,max} = 0.80\ P_{ro} \qquad \textbf{(10-9)}$$

where

$$P_{ro} = \alpha_1 \phi_c f_c'(A_g - A_{st} - A_t - A_p) + \phi_s f_y A_{st} + \phi_a F_y A_t - f_{pr} A_p \qquad \textbf{(10-10)}$$

10.10.5

Columns with ρ_t smaller than 0.01 but larger than 0.005 may be used, provided that the factored axial and flexural resistances are multiplied by the ratio 0.5 (1 + ρ_t/0.01).

10.11 Columns — Design dimensions

10.11.1 Equivalent circular column

In lieu of using the full gross area in resistance calculations, a compression member with a square, octagonal, or other regular polygonal cross-section may be considered a circular section with a diameter equal to the least lateral dimension of the actual shape. The gross area considered, the required percentage of reinforcement, and the resistance shall be based on that circular section.

10.11.2 Column built monolithically with wall

The outer limits of the effective cross-section of a spirally reinforced or tied reinforced compression member, built monolithically with a concrete wall or pier, shall not extend a distance greater than the specified concrete cover outside of the spiral or tie reinforcement.

10.11.3 Isolated column with interlocking spirals

The outer limits of the effective cross-section of a compression member with two or more interlocking spirals shall not extend a distance greater than the specified concrete cover outside of the extreme limits of the spirals.

10.12 Columns — Transmission of loads through floor system

10.12.1

When the specified compressive strength of concrete in a column is greater than that specified for a floor system, transmission of load through the floor system shall be as specified in Clause 10.12.2 or 10.12.3.

10.12.2

Concrete of the strength specified for the column, f'_{cc}, shall be placed in the floor at the column location. The top surface of the column concrete placed in the floor shall extend at least 500 mm into the floor from the face of the column. The column concrete shall be well integrated with the floor concrete.

10.12.3

The resistance of the column in the joint region shall be based on an effective concrete compressive strength, f'_{ce}, equal to

(a) for interior columns:

$$f'_{ce} = 1.05 f'_{cs} + 0.25 f'_{cc} \le f'_{cc} \qquad \textbf{(10-11)}$$

(b) for edge columns:

$$f'_{ce} = 1.4 f'_{cs} \le f'_{cc} \qquad \textbf{(10-12)}$$

(c) for corner columns:

$$f'_{ce} = f'_{cs} \qquad \textbf{(10-13)}$$

Vertical dowels, spirals, or hoops may be added to increase the effective strength of the joint region.

10.13 Slenderness effects — General

10.13.1

Except as allowed by Clause 10.13.2, the design of compression members, restraining beams, and other supporting members shall be based on the factored forces and moments from a second-order analysis that considers material non-linearity and cracking as well as the effects of member curvature and lateral drift, the duration of the loads, shrinkage and creep, and interaction with the supporting foundation. The dimensions of the cross-sections shown on the design drawings shall be within ± 10% of the dimensions used in the analysis. The analysis procedure shall have been shown to result in predictions of strength in substantial agreement with the results of comprehensive tests of columns in indeterminate reinforced concrete structures.

10.13.2

In lieu of the procedure specified in Clause 10.13.1, the design of compression members, restraining beams, and other supporting members may be based on axial forces and moments from the analyses specified in Clauses 10.14 to 10.16, provided that $k\ell_u/r$ for all compression members is not greater than 100.

10.14 Member properties for computation of slenderness effects

10.14.1 General

10.14.1.1

The factored axial forces, P_f; the factored moments, M_1 and M_2, at the ends of the column; and, where required, the first-order lateral storey deflection, Δ_o, shall be computed using an elastic first-order frame analysis with the section properties calculated by taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and the effects of duration of the loads.

10.14.1.2

The following properties may be used to determine the section properties specified in Clause 10.14.1.1:

Modulus of elasticity	E_c from Clause 8.6.2
Moment of inertia	
Beams	$0.35I_g$
Columns	$0.70I_g$
Walls — uncracked	$0.70I_g$
Walls — cracked	$0.35I_g$
Flat plates and flat slabs	$0.25I_g$
Area	A_g

10.14.1.3

For computation of Δ_o and $\delta_s M_s$, flexural stiffness determined from Clause 10.14.1.2 shall be divided by $(1 + \beta_d)$ to account for creep due to sustained loads.

β_d shall be based on

(a) sustained axial loads (for Clauses 10.16.4 and 10.16.5); and
(b) sustained shear (for Clauses 10.14.4 and 10.16.3).

10.14.2 Radius of gyration

The radius of gyration, r, may be taken equal to $0.30h$ for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, the radius of gyration may be computed using the gross concrete section.

10.14.3 Unsupported length of compression members

10.14.3.1
The unsupported length, ℓ_u, of a compression member shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support in the direction being considered.

10.14.3.2
Where column capitals or haunches are present, the unsupported length shall be measured to the lower extremity of the capital or haunch in the plane considered.

10.14.4 Designation as non-sway
Storeys in structures shall be designated non-sway if $Q \le 0.05$ where

$$Q = \frac{\Sigma P_f \Delta_o}{V_f \ell_c} \qquad \textbf{(10-14)}$$

where
ΣP_f = the total factored vertical load in the storey in question
Δ_o = the first-order relative deflection of the top and bottom of that storey due to V_f
V_f = the factored storey shear in the storey in question

The deflection, Δ_o, shall be determined using the flexural stiffness determined from Clause 10.14.1.2, except as required by Clause 10.16.4.

10.14.5 Columns in non-sway frames or storeys
The design of columns in non-sway frames or storeys shall be based on the analysis specified in Clause 10.15.

10.14.6 Columns in sway frames or storeys
The design of columns in sway frames or storeys shall be based on the analysis specified in Clause 10.16.
Note: *If the value of Q exceeds 0.2, a more rigid structure can be required to provide stability.*

10.15 Slenderness effects — Non-sway frames

10.15.1 Effective length factor
For compression members in non-sway frames, the effective length factor, k, shall be taken as 1.0 unless analysis shows that a lower value is justified. The calculation of k shall be based on the properties specified in Clause 10.14.1.2.

10.15.2 Non-sway frames
In non-sway frames, slenderness effects may be ignored for compression members that satisfy the following equation:

$$\frac{k\ell_u}{r} \le \frac{25 - 10\,(M_1/M_2)}{\sqrt{P_f/(f_c' A_g)}} \qquad \textbf{(10-15)}$$

where M_1/M_2 is not taken less than –0.5. M_1/M_2 shall be taken as positive if the column is bent in single curvature.

10.15.3 Member stability effect

10.15.3.1
Compression members shall be designed for the factored axial load, P_f, and the moment amplified for the effects of member curvature, M_c, as follows:

$$M_c = \frac{C_m M_2}{1 - \dfrac{P_f}{\phi_m P_c}} \geq M_2 \qquad \textbf{(10-16)}$$

where
ϕ_m = 0.75

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \qquad \textbf{(10-17)}$$

where

$$EI = \frac{0.2E_c I_g + E_s I_{st}}{1+\beta_d} \qquad \textbf{(10-18)}$$

or

$$EI = \frac{0.4E_c I_g}{1+\beta_d} \qquad \textbf{(10-19)}$$

M_2 in Equation (10-16) shall not be taken as less than $P_f(15 + 0.03h)$ about each axis separately.
β_d in Equations (10-18) and (10-19) shall be based on the sustained axial load, except when used as specified in Clause 10.16.3.

10.15.3.2
For members without transverse loads between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4 \qquad \textbf{(10-20)}$$

10.15.3.3
For members with transverse loads between supports, C_m shall be taken as 1.0.

10.15.3.4
For compression members subject to bending about both principal axes, the moment about each axis shall be magnified separately based on the conditions of restraint corresponding to that axis.

10.16 Slenderness effects — Sway frames

10.16.1 Effective length factor
For compression members not braced against sway, the effective length factor, k, shall be determined based on the properties specified in Clause 10.14.1.2 and shall be greater than 1.0.

10.16.2 End moments
The moments, M_1 and M_2, at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \qquad \textbf{(10-21)}$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \qquad \textbf{(10-22)}$$

where $\delta_s M_{1s}$ and $\delta_s M_{2s}$ shall be computed as specified in Clause 10.16.3.

10.16.3 Calculation of $\delta_s M_s$

10.16.3.1

The magnified sway moments, $\delta_s M_s$, shall be taken as the column end moments calculated using a second-order analysis based on the member stiffnesses specified in Clause 10.14.1.3.

10.16.3.2

δ_s may be calculated as follows:

$$\delta_s = \frac{1}{1 - \dfrac{\sum P_f}{\phi_m \sum P_c}} \qquad \textbf{(10-23)}$$

where

$\sum P_f$ = the summation for all vertical loads in a storey

$\sum P_c$ = the summation for all sway-resisting columns in a storey

P_c shall be computed from Equation (10-17) using k from Clause 10.16.1 and EI from Equation (10-18) or (10-19), with β_d based on the sustained shear.

10.16.3.3

If $Q \leq 1/3$, δ_s may be computed as

$$\delta_s = \frac{1}{1 - Q} \qquad \textbf{(10-24)}$$

10.16.4 Slenderness limit

If an individual compression member has

$$\frac{\ell_u}{r} > \frac{35}{\sqrt{P_f / (f'_c A_g)}} \qquad \textbf{(10-25)}$$

it shall also be designed for the factored axial load, P_f, and the moment, M_c, computed using Clause 10.15.3, in which M_1 and M_2 are computed as specified in Clause 10.16.2. β_d shall be based on the sustained axial load evaluated for the factored load combination used to compute P_f, with k in Equation (10-17) as specified in Clause 10.15.1.

10.16.5 Strength and stability checks

In addition to load cases involving lateral loads, the strength and stability of the structure as a whole under factored gravity loads shall be considered using the following criteria, with β_d based on the sustained axial load:

(a) When $\delta_s M_s$ is computed as specified in Clause 10.16.3.1, the ratio of second-order lateral deflections to first-order lateral deflections under factored gravity loads plus a lateral load applied to each storey equal to 0.005 multiplied by the factored gravity load on that storey shall not exceed 2.5.

(b) When δ_s is computed as specified in Clause 10.16.3.2, δ_s computed using $\sum P_f$ and $\sum P_c$ under factored gravity load shall be positive and shall not exceed 2.5.

10.16.6 Moment magnification for flexural members

Flexural members in sway frames shall be designed for the total magnified end moments of the compression members at the joint.

10.17 Composite columns — General

10.17.1
Composite compression members shall include all such members reinforced longitudinally with bars and structural steel shapes, pipes, or hollow structural sections (HSS).

10.17.2
The resistance of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

10.17.3
In the calculation of factored capacity, the ends of a composite column shall be assumed to be hinged unless definite provisions are made to resist moment at the ends.

10.17.4
Longitudinal reinforcement shall comply with Clauses 10.9.1 and 10.9.2.

10.17.5
The total cross-section area of the metal core and reinforcement shall not exceed 20% of the gross area of the column.

10.17.6
The yield strength of the structural steel core used in the design shall be the specified minimum yield strength for the grade of structural steel used but shall not exceed 350 MPa.

10.17.7
The surface of the structural steel member in contact with concrete shall be unpainted.

10.17.8
The axial load resistance assigned to the concrete of a composite member shall be transferred to the concrete by direct bearing or shear.

10.17.9
The axial load resistance not assigned to the concrete of a composite member shall be developed by direct connection to the structural steel.

10.17.10
The structural steel elements shall be designed in accordance with CAN/CSA-S16 for any construction or other load applied before attainment of composite action.

10.17.11
The design of composite columns with a concrete core encased by structural steel shall be as specified in CAN/CSA-S16.

10.18 Composite column with spiral reinforcement

10.18.1
Spiral reinforcement shall be as specified in Clause 10.9.4.

10.18.2
For evaluation of slenderness effects, the radius of gyration of a composite section with spiral reinforcement shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t + E_s I_{st}}{(E_c A_g / 5) + E_s A_t + E_s A_{st}}} \qquad \textbf{(10-26)}$$

For computing P_c in Equation (10-17), EI of the composite section shall be not greater than

$$EI = \frac{0.2 E_c I_g}{1 + \beta_d} + E_s I_t + E_s I_{st} \qquad \textbf{(10-27)}$$

10.19 Composite column with tie reinforcement

10.19.1
Lateral ties shall extend completely around the structural steel core.

10.19.2
At least 10M ties shall be used when the greatest side dimension of a composite column is 500 mm or less.

10.19.3
At least 15M ties shall be used when the greatest side dimension of a composite column is greater than 500 mm.

10.19.4
Vertical spacing of lateral ties shall not exceed the smallest of 16 longitudinal bar diameters, one-half of the least side dimension of the composite member, or 500 mm.

10.19.5
Welded wire fabric with an area of horizontal wires per unit length of the column not less than that determined as specified in Clauses 10.19.2 to 10.19.4 may be used.

10.19.6
A longitudinal bar shall be located at every corner of a rectangular cross-section, with other longitudinal bars spaced not farther apart than one-half of the least side dimension of the composite member.

10.19.7
For evaluation of slenderness effects, the radius of gyration of a composite section with tie reinforcement shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t}{(E_c A_g / 5) + E_s A_t}} \qquad \textbf{(10-28)}$$

For computing P_c in Equation (10-17), EI of the composite section shall be not greater than

$$EI = \frac{0.2 E_c I_g}{1 + \beta_d} + E_s I_t \qquad \textbf{(10-29)}$$

11 Shear and torsion

11.1 General

11.1.1 Flexural regions
Regions of members in which it is reasonable to assume that plane sections remain plane shall be proportioned for shear and torsion using either the method specified in Clause 11.3 or the strut-and-tie model specified in Clause 11.4. In addition, the applicable requirements of Clause 11.2 shall be satisfied.

11.1.2 Regions near discontinuities
Regions of members in which the plane sections assumption of flexural theory is not applicable shall be proportioned for shear and torsion using the strut-and-tie model specified in Clause 11.4. In addition, the applicable requirements of Clause 11.2 shall be satisfied.

11.1.3 Interface regions
Interfaces between elements such as webs and flanges, between dissimilar materials, and between concretes cast at different times, or at existing or potential major cracks along which slip can occur, shall be proportioned for shear transfer as specified in Clause 11.5.

11.1.4 Slabs and footings
Slab-type regions shall be proportioned for punching shear as specified in Clause 13.

11.1.5 Alternative methods
In lieu of the methods specified in Clauses 11.1.1 to 11.1.4, the resistance of members in shear or in shear combined with torsion may be determined by satisfying the applicable conditions of equilibrium and compatibility of strains and by using appropriate stress-strain relationships for reinforcement and for diagonally cracked concrete.

11.2 Design requirements

11.2.1 Tension due to restraint
In the design for shear, the effects of axial tension due to creep, shrinkage, and thermal effects in restrained members shall be considered wherever applicable.

11.2.2 Variable depth members
For variable depth members, the components of flexural compression and tension in the direction of the applied shear shall be taken into account if their effect is unfavourable, and may be taken into account if their effect is favourable.

11.2.3 Openings
In determining shear resistance, the effect of any openings in members shall be considered.
Note: *Regions of members near openings may be designed using the strut-and-tie model (see Clause 11.4).*

11.2.4 Types of shear reinforcement
Transverse reinforcement provided for shear shall consist of the following:
(a) stirrups or ties perpendicular to the axis of the member;
(b) welded wire fabric with wires perpendicular to the axis of the member, provided that these wires can undergo a minimum elongation of 4% measured over a gauge length of at least 100 mm that includes at least one crosswire;

(c) stirrups making an angle of 45° or more with the longitudinal tension reinforcement, inclined to intercept potential diagonal cracks;
(d) for non-prestressed members, shear reinforcement consisting of 35M or smaller longitudinal bars bent to provide an inclined portion having an angle of 30° or more with the longitudinal bars and crossing potential diagonal cracks. Only the centre three-quarters of the inclined portion of these bars shall be considered effective;
(e) headed shear reinforcement that meets the requirements of Clause 7.1.4 or 13.3.8.1; or
(f) spirals.

11.2.5 Anchorage of shear reinforcement

Stirrups and other bars or wires used as shear reinforcement shall be anchored at both ends as specified in Clause 12.13 to develop the design yield strength of the reinforcement.

11.2.6 Types of torsion reinforcement

Torsion reinforcement shall consist of longitudinal reinforcement and one or more of the following types of transverse reinforcement:
(a) closed stirrups perpendicular to the axis of the member;
(b) a closed cage of welded wire fabric, with wires meeting the minimum elongation requirements of Clause 11.2.4(b) located perpendicular to the axis of the member; and
(c) spirals.

11.2.7 Anchorage of torsion reinforcement

Transverse torsion reinforcement shall be anchored
(a) by 135° standard stirrup hooks; or
(b) as specified in Item (a) or (b) of Clause 12.13.2 in regions where the concrete surrounding the anchorage is restrained against spalling.

A longitudinal reinforcing bar or bonded prestressing tendon shall be placed in each corner of closed transverse reinforcement required for torsion. The nominal diameter of the bar or tendon shall be not less than $s/16$.

Longitudinal torsion reinforcement shall be anchored as specified in Clause 12.1.

11.2.8 Minimum shear reinforcement

11.2.8.1

A minimum area of shear reinforcement shall be provided in the following regions:
(a) in regions of flexural members where the factored shear force, V_f, exceeds $V_c + V_p$;
(b) in regions of beams with an overall thickness greater than 750 mm; and
(c) in regions of flexural members where the factored torsion, T_f, exceeds $0.25T_{cr}$.

11.2.8.2

Where shear reinforcement is required by Clause 11.2.8.1 or by calculation, the minimum area of shear reinforcement shall be such that

$$A_v = 0.06\sqrt{f_c'}\,\frac{b_w s}{f_y} \tag{11-1}$$

11.2.8.3

In calculating the term A_v in Equation (11-1), inclined reinforcement and transverse reinforcement used to resist torsion may be included.

11.2.8.4
The requirement for minimum shear reinforcement specified in Clause 11.2.8.1 may be waived if it can be shown by tests that the required flexural and shear resistances can be developed when shear reinforcement is omitted. Such tests shall simulate the effects of differential settlement, creep, shrinkage, and temperature change based on a realistic assessment of such effects occurring in service.

11.2.9 Consideration of torsion

11.2.9.1
If the magnitude of the torsion, T_f, determined as specified in analysis using stiffnesses based on uncracked sections exceeds $0.25T_{cr}$, torsional effects shall be considered and torsional reinforcement designed as specified in Clause 11.3 shall be provided. Otherwise, torsional effects may be neglected.

In lieu of more detailed calculations, T_{cr} may be taken as

$$T_{cr} = \left(A_c^2 / p_c\right) 0.38\lambda\phi_c\sqrt{f_c'}\sqrt{1+\frac{\phi_p f_{cp}}{0.38\lambda\phi_c\sqrt{f_c'}}} \qquad \textbf{(11-2)}$$

For a hollow section, A_c in Equation (11-2) shall be replaced by $1.5A_g$ if the wall thickness is less than $0.75A_c/p_c$.

11.2.9.2
In a statically indeterminate structure where reduction of torsional moment in a member can occur because of redistribution of internal forces, the maximum factored torsion, T_f, at the face of the support may be reduced to $0.67T_{cr}$ provided that the corresponding adjustments to torsions, moments, and shears are made in the member and in adjoining members to account for the redistribution. For a spandrel beam where the torsion is caused by a slab, the factored torsion in the spandrel can be assumed to vary linearly from zero at midspan to $0.67T_{cr}$ at the face of the support.

11.2.10 Effective web width

11.2.10.1
Unless otherwise permitted by Clause 11.2.10.3 or 11.2.10.4, the effective web width shall be taken as the minimum concrete web width within depth d.

11.2.10.2
In determining the concrete web width at a particular level, one-half the diameters of ungrouted post-tensioning ducts or one-quarter the diameters of grouted ducts at that level shall be subtracted from the total web width.

11.2.10.3
For circular members, b_w may be taken as the diameter.

11.2.10.4
For members with tapering webs, b_w may be taken as the average web width calculated over a contiguous height that includes the minimum web width location but does not include any regions of the section where the side faces of the section slope outward at more than 20° from the direction of the applied shear.

11.2.11 Reduced prestress in transfer length

In pretensioned members, the reduction in prestress in the transfer length of prestressing tendons shall be considered when computing V_p, f_{po}, and the tensile force that can be resisted by the longitudinal reinforcement. The prestress force may be assumed to vary linearly from zero at the point at which bonding commences to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

11.2.12 Hanger reinforcement for beams supporting other beams

11.2.12.1

When a load is applied to a side face of a beam, additional transverse reinforcement shall be provided. In lieu of a strut-and-tie model design in accordance with Clause 11.4, the requirements of Clause 11.2.12.2 may be used, provided that the soffit of the supported beam is not lower than the soffit of the supporting beam.

11.2.12.2

Additional transverse reinforcement capable of transmitting a tensile force of $(1 - h_b/h_1)$ times the factored shear being transferred shall be provided, with h_b being the distance from the soffit of the supporting beam to the soffit of the supported beam and h_1 being the overall depth of the supporting beam. This additional full-depth transverse reinforcement shall be placed in the supporting beam to intercept 45° planes starting on the shear interface at one-quarter of the depth of the supported beam above its bottom face and spreading down into the supporting beam (see Figure 11.1).

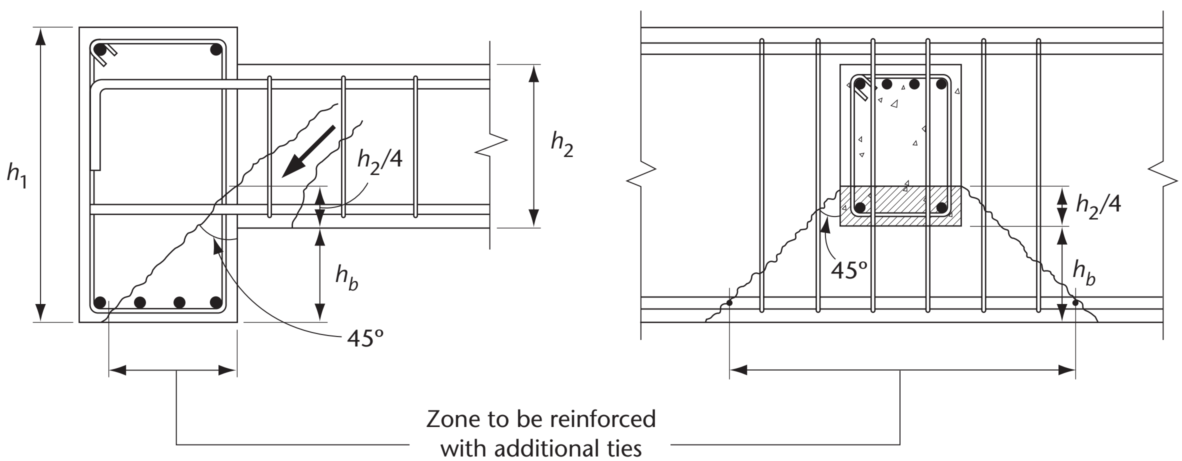


Figure 11.1
Location of additional transverse reinforcement
(See Clauses 2.3 and 11.2.12.2.)

11.2.12.3

The requirements of Clauses 11.2.12.1 and 11.2.12.2 may be waived if
(a) the interface transmitting the load extends to the top of the supporting member; and
(b) the average shear stress on this interface is not greater than $0.23\lambda\phi_c\sqrt{f_c'}$.

11.3 Design for shear and torsion in flexural regions

11.3.1 Required shear resistance

Members subjected to shear shall be proportioned so that

$$V_r \geq V_f \qquad \textbf{(11-3)}$$

11.3.2 Sections near supports

Sections located less than a distance d_v from the face of the support may be designed for the same shear, V_f, as that computed at a distance d_v, provided that

(a) the reaction force in the direction of applied shear introduces compression into the member; and

(b) no concentrated load that causes a shear force greater than $0.3\lambda\phi_c\sqrt{f'_c}\,b_w d_v$ is applied within the distance d_v from the face of the support.

11.3.3 Factored shear resistance

The factored shear resistance shall be determined by

$$V_r = V_c + V_s + V_p \qquad \textbf{(11-4)}$$

However, V_r shall not exceed

$$V_{r,max} = 0.25\phi_c f'_c\, b_w d_v + V_p \qquad \textbf{(11-5)}$$

11.3.4 Determination of V_c

The value of V_c shall be computed from

$$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v \qquad \textbf{(11-6)}$$

where β is determined as specified in Clause 11.3.6.

In the determination of V_c, the term $\sqrt{f'_c}$ shall not be taken greater than 8 MPa.

11.3.5 Determination of V_s

11.3.5.1

For members with transverse reinforcement perpendicular to the longitudinal axis, V_s shall be computed from

$$V_s = \frac{\phi_s A_v f_y d_v \cot\theta}{s} \qquad \textbf{(11-7)}$$

where θ is determined as specified in Clause 11.3.6.

11.3.5.2

For members with transverse reinforcement inclined at an angle α to the longitudinal axis, V_s shall be computed from

$$V_s = \frac{\phi_s A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad \textbf{(11-8)}$$

where θ is determined as specified in Clause 11.3.6.

11.3.6 Determination of β and θ

11.3.6.1 Members subjected to significant axial tension

For members subjected to significant axial tension, the values of β and θ shall be determined as specified in Clause 11.3.6.4.

11.3.6.2 Values for special member types

Unless otherwise permitted by Clause 11.3.6.3 or Clause 11.3.6.4, the value of β shall be taken as 0.21 and θ shall be taken as 42° for any of the following member types:

(a) slabs or footings with an overall thickness not greater than 350 mm;
(b) footings in which the distance from the point of zero shear to the face of the column, pedestal, or wall is less than three times the effective shear depth of the footing;
(c) beams with an overall thickness not greater than 250 mm;
(d) concrete joist construction defined by Clause 10.4; and
(e) beams cast integrally with slabs where the depth of the beam below the slab is not greater than one-half the width of web or 350 mm.

11.3.6.3 Simplified method

In lieu of more accurate calculations in accordance with Clause 11.3.6.4, and provided that the specified yield strength of the longitudinal steel reinforcement does not exceed 400 MPa and the specified concrete strength does not exceed 60 MPa, θ shall be taken as 35° and β shall be determined as follows:

(a) If the section contains at least the minimum transverse reinforcement as specified by Equation (11-1), β shall be taken as 0.18.
(b) If the section contains no transverse reinforcement and the specified nominal maximum size of coarse aggregate is not less than 20 mm, β shall be taken as

$$\beta = \frac{230}{(1000 + d_v)} \qquad \textbf{(11-9)}$$

(c) Alternatively, the value of β for sections containing no transverse reinforcement may be determined for all aggregate sizes by replacing the parameter d_v in Equation (11-9) by the equivalent crack spacing parameter, s_{ze}, where

$$s_{ze} = \frac{35 s_z}{15 + a_g} \qquad \textbf{(11-10)}$$

However, s_{ze} shall not be taken as less than $0.85 s_z$. The crack spacing parameter, s_z, shall be taken as d_v or as the maximum distance between layers of distributed longitudinal reinforcement, whichever is less. Each layer of such reinforcement shall have an area at least equal to $0.003 b_w s_z$ (see Figure 11.2).

When the simplified method specified in this Clause is used, all other clauses of Clause 11 shall apply, except Clause 11.3.6.4. Accordingly, this simplified method shall not be used for members subjected to significant tension, and the longitudinal reinforcement for all members shall be proportioned as specified in Clause 11.3.9.

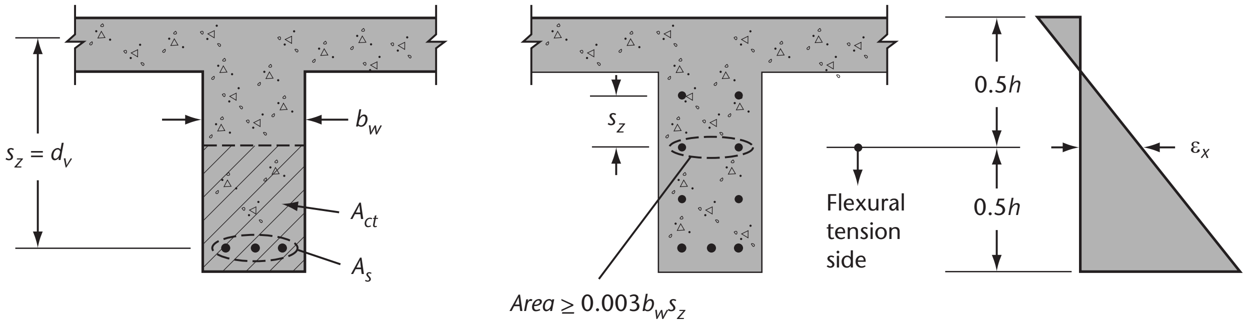


Figure 11.2
Terms in shear design equations
(See Clauses 2.3 and 11.3.6.3.)

11.3.6.4 General method

The value of β shall be determined from the following equation:

$$\beta = \frac{0.40}{(1+1500\varepsilon_x)} \bullet \frac{1300}{(1000+s_{ze})} \qquad \textbf{(11-11)}$$

For sections containing at least the minimum transverse reinforcement required by Equation (11-1), the equivalent crack spacing parameter, s_{ze}, in Equation (11-11) shall be taken as equal to 300 mm. Otherwise, s_{ze} shall be computed using Equation (11-10). If f'_c exceeds 70 MPa, the term a_g shall be taken as zero in Equation (11-10). As f'_c goes from 60 to 70 MPa, a_g shall be linearly reduced to zero.

The angle of inclination, θ, of the diagonal compressive stresses shall be calculated as

$$\theta = 29 + 7000\varepsilon_x \qquad \textbf{(11-12)}$$

In lieu of more accurate calculations, the longitudinal strain, ε_x, at mid-depth of the cross-section shall be computed from

$$\varepsilon_x = \frac{M_f/d_v + V_f - V_p + 0.5N_f - A_p f_{po}}{2\left(E_s A_s + E_p A_p\right)} \qquad \textbf{(11-13)}$$

In evaluating Equation (11-13), the following conditions shall apply:
(a) V_f and M_f shall be taken as positive quantities and M_f shall not be taken as less than $(V_f - V_p)d_v$.
(b) In calculating A_s, the area of bars that are terminated less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.
(c) If the value of ε_x calculated from Equation (11-13) is negative, it shall be taken as zero or the value shall be recalculated with the denominator of Equation (11-13) replaced by $2(E_sA_s + E_pA_p + E_cA_{ct})$. However, ε_x shall not be taken as less than -0.20×10^{-3}.
(d) For sections closer than d_v to the face of the support, the value of ε_x calculated at d_v from the face of the support may be used in evaluating β and θ.
(e) If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in ε_x shall be taken into account. In lieu of more accurate calculations, the value calculated from Equation (11-13) shall be doubled.
(f) β and θ may be determined from Equations (11-11) and (11-12), respectively, using a value of ε_x that is greater than that calculated from Equation (11-13). However, ε_x shall not be taken greater than 3.0×10^{-3}.

11.3.7 Proportioning of transverse reinforcement
Near locations where the spacing, s, of the transverse reinforcement changes, the quantity A_v/s may be assumed to vary linearly over a length h centred on the location where the spacing changes.

11.3.8 Maximum spacing of transverse reinforcement

11.3.8.1
The spacing of transverse reinforcement, s, placed perpendicular to the axis of the member shall not exceed $0.7d_v$ or 600 mm.

11.3.8.2
Inclined stirrups and bent longitudinal reinforcement shall be spaced so that every line inclined at 35° to the axis of the member and extending toward the reaction from mid-depth of the member to the longitudinal flexural tension reinforcement shall be crossed by at least one line of effective shear reinforcement. See Clause 11.2.4(d).

11.3.8.3
If V_f exceeds $0.125\lambda\phi_c f'_c b_w d_v + V_p$ or if T_f exceeds $0.25T_{cr}$, the maximum spacings specified in Clauses 11.3.8.1 and 11.3.8.2 shall be reduced by one-half.

11.3.9 Proportioning of longitudinal reinforcement

11.3.9.1 Extension of longitudinal reinforcement
At every section, the longitudinal reinforcement shall be designed to resist the additional tension forces caused by shear as specified in Clauses 11.3.9.2 and 11.3.9.3. Alternatively, for members not subjected to significant tension or significant torsion, these requirements may be satisfied by extending the flexural tension reinforcement a distance of $d_v\cot\theta$ beyond the location needed by flexure alone.

11.3.9.2 Flexural tension side
Longitudinal reinforcement on the flexural tension side shall be proportioned so that the factored resistance of the reinforcement at all sections, taking account of the stress that can be developed in this reinforcement, shall be greater than or equal to the force F_{lt}, as follows:

$$F_{lt} = \frac{M_f}{d_v} + 0.5N_f + (V_f - 0.5V_s - V_p)\cot\theta \qquad \textbf{(11-14)}$$

where M_f and V_f are taken as positive quantities and N_f is positive for axial tension and negative for axial compression. In Equation (11-14), d_v may be taken as the flexural lever arm corresponding to the factored moment resistance.

11.3.9.3 Flexural compression side
At sections where the moment term, M_f/d_v, in Equation (11-14) is less than the sum of the terms accounting for axial load and shear, longitudinal reinforcement on the flexural compression side of the section shall be proportioned so that the factored tensile resistance of this reinforcement, taking account of the stress that can be developed in this reinforcement, shall be greater than or equal to the force F_{lc}, as follows:

$$F_{lc} = 0.5N_f + (V_f - 0.5V_s - V_p)\cot\theta - \frac{M_f}{d_v} \qquad \textbf{(11-15)}$$

where M_f and V_f are taken as positive quantities and N_f is positive for axial tension and negative for axial compression.

11.3.9.4 Compression fan regions

In regions adjacent to maximum moment locations, the area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone. This provision shall apply only if the support or the load at the maximum moment location introduces direct compression into the flexural compression face of the member and the member is not subject to significant torsion.

11.3.9.5 Anchorage of longitudinal reinforcement at end supports

At exterior direct bearing supports, the longitudinal reinforcement on the flexural tension side of the member shall be capable of resisting a tensile force of $(V_f - 0.5V_s - V_p)\cot\theta + 0.5N_f$, where θ is as specified in Clause 11.3.6 and V_s is based on the transverse reinforcement provided within a length of $d_v\cot\theta$ from the face of the support. However, V_s shall not be taken as greater than V_f. The tension force in the reinforcement shall be developed at the point where a line inclined at angle θ to the longitudinal axis and extending from the inside edge of the bearing area intersects the centroid of the reinforcement. See Figure 11.3(b).

11.3.10 Sections subjected to combined shear and torsion

11.3.10.1 Transverse reinforcement for combined shear and torsion

The transverse reinforcement for combined shear and torsion shall be at least equal to the sum of that required for shear and that required for the coexisting torsion.

11.3.10.2 Transverse reinforcement for torsion

The amount of transverse reinforcement required for torsion shall be such that

$$T_r \geq T_f \qquad \textbf{(11-16)}$$

11.3.10.3 Factored torsional resistance

The value of T_r shall be computed from

$$T_r = 2A_o \frac{\phi_s A_t f_y}{s}\cot\theta \qquad \textbf{(11-17)}$$

where

A_o = $0.85A_{oh}$

and θ is as specified in Clause 11.3.6.

11.3.10.4 Cross-sectional dimensions to avoid crushing

The cross-sectional dimensions to avoid crushing shall be as follows:

(a) for box sections:

$$\frac{V_f - V_p}{b_w d_v} + \frac{T_f p_h}{1.7A_{oh}^2} \leq 0.25\phi_c f_c' \qquad \textbf{(11-18)}$$

If the wall thickness of the box section is less than A_{oh}/p_h, the second term in Equation (11-18) shall be replaced by $T_f/(1.7A_{oh}t)$ where t is the wall thickness at the location where the stresses are being checked.

(b) for other sections:

$$\sqrt{\left(\frac{V_f - V_p}{b_w d_v}\right)^2 + \left(\frac{T_f p_h}{1.7A_{oh}^2}\right)^2} \leq 0.25\phi_c f_c' \qquad \textbf{(11-19)}$$

11.3.10.5 Determination of ε_x for general method

If β and θ are being determined using Clause 11.3.6.4, the value of ε_x for a section subjected to torsion shall be determined by replacing the term $(V_f - V_p)$ in Equation (11-13) and in Clause 11.3.6.4(a) with the expression

$$\sqrt{(V_f - V_p)^2 + \left(\frac{0.9 p_h T_f}{2A_o}\right)^2} \tag{11-20}$$

11.3.10.6 Proportioning longitudinal reinforcement

The longitudinal reinforcement shall be proportioned to satisfy the requirements of Clause 11.3.9, except that the term $(V_f - 0.5V_s - V_p)$ shall be replaced by the expression

$$\sqrt{(V_f - 0.5V_s - V_p)^2 + \left(\frac{0.45 p_h T_f}{2A_o}\right)^2} \tag{11-21}$$

11.4 Strut-and-tie model

11.4.1 Structural idealization

The strength of reinforced concrete structures, members, or regions may be investigated by idealizing the reinforced concrete as a series of reinforcing steel tensile ties and concrete compressive struts interconnected at nodes to form a truss capable of carrying all of the factored loads to the supports. In determining the geometry of the truss, account shall be taken of the required dimensions of the struts and ties.

11.4.2 Proportioning of strut

11.4.2.1 Strength of strut

The dimensions of a strut shall be large enough to ensure that the calculated compressive force in the strut does not exceed $\phi_c f_{cu} A_{cs}$ where f_{cu} and A_{cs} are determined as specified in Clauses 11.4.2.2 and 11.4.2.3.

11.4.2.2 Effective cross-sectional area of strut

The value of A_{cs} shall be calculated by considering both the available concrete area and the anchorage conditions at the ends of the strut, as shown in Figure 11.3.

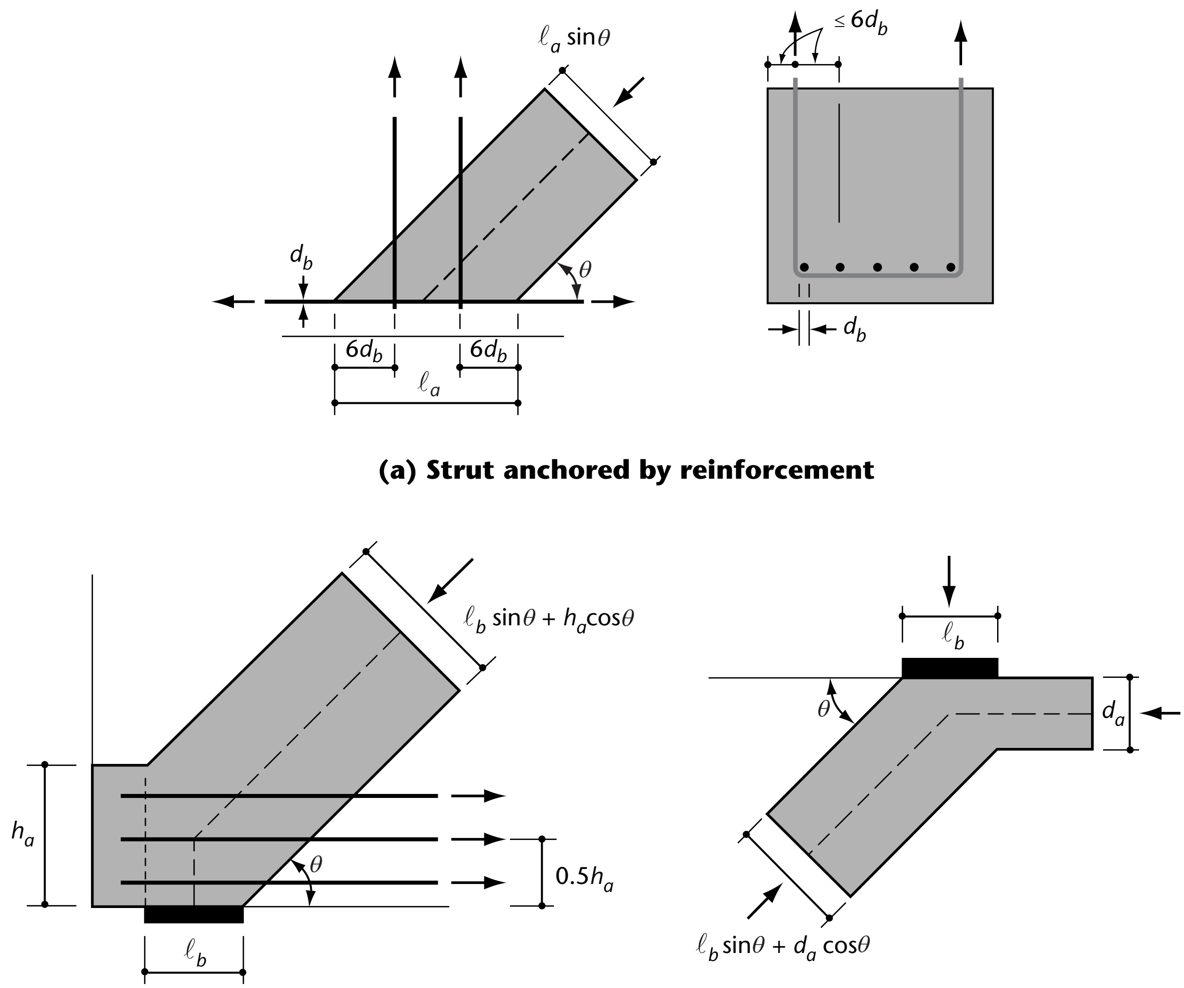


Figure 11.3
Influence of anchorage conditions on effective cross-sectional area of strut
(See Clauses 2.3, 11.3.9.5, and 11.4.2.2.)

11.4.2.3 Limiting compressive stress in struts

The value of f_{cu} shall be computed from

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \leq 0.85 f'_c \qquad \textbf{(11-22)}$$

where

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\theta_s \qquad \textbf{(11-23)}$$

and θ_s is the smallest angle between the strut and the adjoining ties and ε_s is the tensile strain in the tie inclined at θ_s to the strut. If the tensile strain in the tie changes as the tie crosses the width of the strut, ε_s may be taken as the strain in the tie at the centreline of the strut.

11.4.2.4 Reinforced struts
If a strut contains reinforcing bars that are parallel to the strut, have been detailed to develop their yield strength in compression, and are enclosed by transverse reinforcement complying with Clause 7.6.5, the calculated force in the strut shall not exceed $\phi_c f_{cu} A_{cs} + \phi_s f_y A_{ss}$.

11.4.3 Proportioning of ties

11.4.3.1 Strength of ties
The area of reinforcement in the tie, A_{st}, shall be large enough to ensure that the calculated tensile force in the tie does not exceed $\phi_s f_y A_{st}$.

11.4.3.2 Anchorage of ties in node regions
The tie reinforcement shall be anchored by appropriate development length, hook, or mechanical anchorage in accordance with Clause 12 so that it is capable of resisting the calculated tension in the reinforcement at the location where the centroid of this reinforcement crosses the edge of the adjoining strut. For straight bars extending a distance x beyond the critical location where $x < \ell_d$, the calculated stress shall not exceed $\phi_s f_y (x/\ell_d)$, where ℓ_d is computed as specified in Clause 12.

11.4.4 Proportioning of node regions

11.4.4.1 Stress limits in node regions
Unless special confinement is provided, the calculated concrete compressive stress in the node regions shall not exceed the following:
(a) $0.85\phi_c f_c'$ in node regions bounded by struts and bearing areas;
(b) $0.75\phi_c f_c'$ in node regions anchoring a tie in only one direction; and
(c) $0.65\phi_c f_c'$ in node regions anchoring ties in more than one direction.

Note: *The beneficial effects of confinement can be accounted for if substantiated by test results.*

11.4.4.2 Satisfying stress limits in node regions
The stress limits in node regions may be considered satisfied if
(a) the bearing stress on the node regions produced by concentrated loads or reactions does not exceed the stress limits specified in Clause 11.4.4.1; and
(b) the tie reinforcement is uniformly distributed over an effective area of concrete at least equal to the tie force divided by the stress limits specified in Clause 11.4.4.1.

11.4.5 Crack control reinforcement
Structures, members, or regions (other than slabs or footings) that have been designed in accordance with Clause 11.4 shall contain an orthogonal grid of reinforcing bars near each face. The ratio of reinforcement area to gross concrete area shall not be less than 0.002 in each direction. The spacing of this reinforcement shall not exceed 300 mm. If located within the tie, the crack control reinforcement may also be considered as tie reinforcement.

11.5 Interface shear transfer

11.5.1 General
A crack shall be assumed to occur along the shear plane and relative displacement shall be considered to be resisted by cohesion and friction maintained by the shear friction reinforcement crossing the crack. The factored shear stress resistance of the plane shall be computed from

$$v_r = \lambda\phi_c(c + \mu\sigma) + \phi_s \rho_v f_y \cos\alpha_f \quad \textbf{(11-24)}$$

where the expression $\lambda\phi_c(c + \mu\sigma)$ shall not exceed $0.25\phi_c f'_c$ and α_f is the angle between the shear friction reinforcement and the shear plane.

11.5.2 Values of c and μ

The following values shall be taken for c and μ:

(a) For concrete placed against hardened concrete with the surface clean but not intentionally roughened:
$c = 0.25$ MPa
$\mu = 0.60$

(b) For concrete placed against hardened concrete with the surface clean and intentionally roughened to a full amplitude of at least 5 mm:
$c = 0.50$ MPa
$\mu = 1.00$

(c) For concrete placed monolithically:
$c = 1.00$ MPa
$\mu = 1.40$

(d) For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars:
$c = 0.00$ MPa
$\mu = 0.60$

11.5.3 Alternative equation for shear stress resistance

For concrete placed monolithically or placed against hardened concrete with the surface clean and intentionally roughened to a full amplitude of at least 5 mm, the factored shear stress resistance may be computed using the following equation in lieu of Equation (11-24):

$$v_r = \lambda\phi_c k\sqrt{\sigma f'_c} + \phi_s \rho_v f_y \cos\alpha_f \qquad \textbf{(11-25)}$$

where

k = 0.5 for concrete placed against hardened concrete
= 0.6 for concrete placed monolithically

and the expression $\lambda\phi_c k\sqrt{\sigma f'_c}$ shall not exceed $0.25\phi_c f'_c$ and α_f is the angle between the shear friction reinforcement and the shear plane.

11.5.4 Values of σ and ρ_v

The value of σ shall be computed as

$$\sigma = \rho_v f_y \sin\alpha_f + \frac{N}{A_g} \qquad \textbf{(11-26)}$$

where

$$\rho_v = \frac{A_{vf}}{A_{cv}} \qquad \textbf{(11-27)}$$

and N is the unfactored permanent load perpendicular to the shear plane, positive for compression and negative for tension.

11.5.5 Inclined shear friction reinforcement

In determining the area of inclined shear friction reinforcement to be used in Equation (11-27), only that reinforcement inclined to the shear plane at an angle, α_f, such that the shear force produces tension in the inclined reinforcement, shall be included.

11.5.6 Anchorage of shear friction reinforcement

The shear friction reinforcement shall be anchored on each side of the shear plane so that the specified yield strength can be developed.

11.6 Special provisions for brackets and corbels

11.6.1
Brackets and corbels shall be designed in accordance with Clause 11.4 and Clauses 11.6.2 to 11.6.8.

11.6.2
The depth, d, at the face of a support shall be not less than the distance between the load and the face of the support.

11.6.3
The depth at the outside edge of the bearing area shall be not less than one-half of the depth at the face of the support.

11.6.4
The external tensile force, N_f, acting on the bearing area shall not be taken as less than $0.2V_f$ unless special provisions are made to avoid tensile forces.

11.6.5
In lieu of the crack control reinforcement specified in Clause 11.4.5, closed stirrups or ties parallel to the primary tensile tie reinforcement, A_{st}, and having a total area of not less than 50% of A_{st}, shall be distributed within two-thirds of the effective depth adjacent to A_{st}.

11.6.6
The ratio A_{st}/bd calculated at the face of the support shall be not less than $0.04(f_c'/f_y)$.

11.6.7
At the front face of the bracket or corbel, the primary tensile tie reinforcement, A_{st}, shall be anchored to develop the required force in the tension tie.

11.6.8
The bearing area of the load on the bracket or corbel shall not project beyond the straight portion of the tension tie bars or beyond the interior face of the transverse anchor bar, if one is provided.

11.7 Shear in joints
When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, the shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns and joints.

12 Development and splices of reinforcement

12.1 Development of reinforcement — General

12.1.1
The calculated tension or compression in reinforcement at each section of reinforced concrete members shall be developed on each side of that section by embedment length, hook, or mechanical device, or by a combination thereof. Hooks may be used in developing bars in tension only.

12.1.2
The maximum permissible value of $\sqrt{f'_c}$ in Clause 12 shall be 8 MPa.

12.2 Development of deformed bars and deformed wire in tension

12.2.1 Minimum development length
The development length, ℓ_d, of deformed bars and deformed wire in tension shall be determined from either Clause 12.2.2 or Clause 12.2.3, but ℓ_d shall be not less than 300 mm.

12.2.2 General development length equation
The development length, ℓ_d, of deformed bars and deformed wire in tension shall be

$$\ell_d = 1.15\frac{k_1 k_2 k_3 k_4}{(d_{cs} + K_{tr})}\frac{f_y}{\sqrt{f'_c}}A_b \quad \textbf{(12-1)}$$

but the term $(d_{cs} + K_{tr})$ shall not be taken greater than $2.5d_b$ where

$$K_{tr} = \frac{A_{tr} f_{yt}}{10.5sn}$$

12.2.3 Simplified development length equations
The development length, ℓ_d, of deformed bars and deformed wire in tension may be taken from Table 12.1, provided that the clear cover and clear spacing of the bars or wire being developed are at least d_b and $1.4d_b$, respectively.

Table 12.1
Development length, ℓ_d, of deformed bars and deformed wire in tension
(See Clause 12.2.3.)

Cases	Minimum development length, ℓ_d
Member containing minimum ties (Clause 7.6.5) or minimum stirrups (Clause 11.2.8.2) within ℓ_d Slabs, walls, shells, or folded plates having clear spacing of not less than $2d_b$ between bars being developed	$0.45 k_1 k_2 k_3 k_4 \frac{f_y}{\sqrt{f'_c}} d_b$
Other cases	$0.6 k_1 k_2 k_3 k_4 \frac{f_y}{\sqrt{f'_c}} d_b$

Note: *The clear cover and clear spacing requirements specified in Clause 12.2.3 meet the requirements of CSA A23.1 (see Annex A).*

12.2.4 Modification factors
The following modification factors shall be used to calculate the development length in Clauses 12.2.2 and 12.2.3:

(a) bar location factor, k_1:

k_1 = 1.3 for horizontal reinforcement placed in such a way that more than 300 mm of fresh concrete is cast in the member below the development length or splice

= 1.0 for other cases

(b) coating factor, k_2:

k_2 = 1.5 for epoxy-coated reinforcement with clear cover less than $3d_b$, or with clear spacing between bars being developed less than $6d_b$

= 1.2 for all other epoxy-coated reinforcement

= 1.0 for uncoated reinforcement

(c) concrete density factor, k_3:

k_3 = 1.3 for structural low-density concrete

= 1.2 for structural semi-low-density concrete

= 1.0 for normal-density concrete

(d) bar size factor, k_4:

k_4 = 0.8 for 20M and smaller bars and deformed wires

= 1.0 for 25M and larger bars

The product $k_1 k_2$ need not be taken greater than 1.7.

12.2.5 Excess reinforcement

The development length, ℓ_d, may be multiplied by the factor (A_s required) / (A_s provided) where reinforcement in a flexural member exceeds that required by analysis, except where anchorage or development for f_y is specifically required or the reinforcement is designed as specified in Clause 21.

12.3 Development of deformed bars in compression

12.3.1 Development length

The development length, ℓ_d, for deformed bars in compression shall be computed as the product of the basic development length, ℓ_{db}, specified in Clause 12.3.2 and the applicable modification factors specified in Clause 12.3.3, but ℓ_d shall be not less than 200 mm.

12.3.2 Basic development length

The basic compression development length, ℓ_{db}, shall be $0.24 d_b f_y / \sqrt{f'_c}$, but not less than $0.044 d_b f_y$.

12.3.3 Modification factors

The basic development length, ℓ_{db}, may be multiplied by the following factors, as applicable:

(a) for reinforcement exceeding that required by analysis: (A_s required) / (A_s provided); and

(b) for reinforcement enclosed within spiral reinforcement of not less than 6 mm diameter and not more than 100 mm pitch or within 10M ties in compliance with Clause 7.6.5 and spaced at not more than 100 mm on centre: 0.75.

12.4 Development of bundled bars

The development length of individual bars within a bundle in tension or compression shall be that for the individual bar increased by 10% for a two-bar bundle, 20% for a three-bar bundle, and 33% for a four-bar bundle.

12.5 Development of standard hooks in tension

12.5.1 Tension development length

Except as specified in Clause 12.13, the development length, ℓ_{dh}, for deformed bars in tension terminating in a standard hook as defined in Clause 6.6.2.2 of CSA A23.1 (reprinted in Annex A) shall be computed as the product of the basic development length, ℓ_{hb}, specified in Clause 12.5.2 and the applicable modification factor or factors specified in Clause 12.5.3, but shall be not less than $8d_b$ or 150 mm, whichever is greater.

12.5.2 Basic development length

The basic development length, ℓ_{hb}, for a hooked bar with f_y equal to 400 MPa shall be $100d_b/\sqrt{f'_c}$.

12.5.3 Factors modifying hook development length

The basic development length, ℓ_{hb}, shall be multiplied by the following factor(s), as applicable:

(a) For bars with f_y other than 400 MPa	$f_y/400$
(b) For 35M or smaller bars where the side cover (normal to plane of hook) is not less than 60 mm, and for 90° hooks where the cover on the bar extension beyond the hook is not less than 50 mm	0.7
(c) For 35M or smaller bars where the hook is enclosed vertically or horizontally within at least three ties or stirrup ties spaced along a length at least equal to the inside diameter of the hook at a spacing not greater than $3d_b$, where d_b is the nominal diameter of the hooked bar	0.8
(d) Where anchorage or development for f_y is not specifically required for reinforcement exceeding that required by analysis	(A_s required) / (A_s provided)
(e) For structural low-density concrete	1.3
(f) For epoxy-coated reinforcement	1.2

12.5.4 Confinement of hooks

For bars being developed by a standard hook at the ends of members where both the side cover and the top (or bottom) cover over the hook is less than 60 mm, the hook shall be enclosed within at least three ties or stirrup ties spaced along a length at least equal to the inside diameter of the hook at a spacing not greater than $3d_b$, where d_b is the nominal diameter of the hooked bar. For this case, the factor specified in Clause 12.5.3(c) shall not apply.

12.5.5 Development of bars in compression

Hooks shall not be considered effective in developing bars in compression.

12.6 Mechanical anchorage

12.6.1

Any mechanical anchorage, including heads of headed bars or headed studs, demonstrated by test to be capable of developing the strength of reinforcement without damage to the concrete, may be used.

12.6.2

The development of reinforcement may consist of a combination of mechanical anchorage and additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

12.7 Development of welded deformed wire fabric in tension

12.7.1

The development length, ℓ_d, of welded deformed wire fabric shall be computed as the product of the development length, ℓ_d, specified in Clause 12.2.2 or 12.2.3 and the applicable wire fabric factor or factors specified in Clause 12.7.2 or 12.7.3, but shall be not less than 200 mm, except in the computation of lap splices by the method specified in Clause 12.18 and in the development of web reinforcement by the method specified in Clause 12.13.

12.7.2

For welded deformed wire fabric with at least one crosswire within the development length and not less than 50 mm from the point of the critical section, the wire fabric factor, k_5, shall be the greater of

$$k_5 = \frac{f_y - 240}{f_y} \qquad \textbf{(12-2)}$$

or

$$k_5 = \frac{5d_b}{s_w} \qquad \textbf{(12-3)}$$

but need not be taken greater than 1.0.

12.7.3

For welded deformed wire fabric with no crosswires within the development length or with a single crosswire less than 50 mm from the point of the critical section, the wire fabric factor, k_5, shall be taken as 1.0.

12.8 Development of welded smooth wire fabric in tension

The yield strength of welded smooth wire fabric shall be considered to be developed by the embedment of two crosswires, with the closer crosswire not less than 50 mm from the critical section. However, the development length, ℓ_d, measured from the critical section to the outermost crosswire shall be not less than

$$\ell_d = 3.3 k_3 \frac{A_w}{s_w} \frac{f_y}{\sqrt{f'_c}} \qquad \textbf{(12-4)}$$

If excess reinforcement is present, this length may be reduced in accordance with Clause 12.2.5. ℓ_d shall be not less than 150 mm except in the computation of lap splices by the method specified in Clause 12.19.

12.9 Development of pretensioned strand

12.9.1

Three- or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length, ℓ_d, not less than

$$\ell_d = 0.145\,(f_{pr} - 0.67 f_{pe}) d_b \qquad \textbf{(12-5)}$$

12.9.2

Where bonding of a strand does not extend to the end of a member and the design includes tension at specified loads in the precompressed tensile zone as permitted by Clause 18.3.2 or 18.3.3, the development length specified in Clause 12.9.1 shall be doubled.

12.10 Development of flexural reinforcement — General

12.10.1

Tension reinforcement may be anchored into the compression zone by bending it across the web to be anchored or made continuous with the reinforcement on the opposite face of the member.

12.10.2

Critical sections for development of reinforcement in flexural members are located at points of maximum stress and at points within the span where adjacent reinforcement terminates or is bent. The location of the points of maximum stress and the points at which reinforcement is no longer required to resist flexure shall be derived from the factored bending moment diagram.

12.10.3

Reinforcement shall extend beyond the point at which it is no longer required to resist flexure as specified in Clause 11.3.9.

12.10.4

Continuing reinforcement shall have an embedment length of not less than the development length, ℓ_d, plus the longer of the effective depth of the member or $12d_b$ beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

12.10.5

Special attention shall be given to providing adequate anchorage for tension reinforcement in flexural members such as sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which the tension reinforcement is not parallel to the compression face.

12.11 Development of positive moment reinforcement

12.11.1

At least one-third of the positive moment reinforcement in simply supported members and one-quarter of the positive moment reinforcement in continuous members shall extend along the same face of the member into the support. In beams constructed monolithically with the support, such reinforcement shall extend into the support at least 150 mm, but not less than required by Clause 11.3.9.

12.11.2

When a flexural member is part of a primary lateral load resisting system, the positive moment reinforcement required by Clause 12.11.1 to be extended into the support shall be anchored to develop the specified yield strength, f_y, in tension at the face of the support.

12.11.3

At simple supports and at points of inflection, the positive moment tension reinforcement shall be limited to a diameter such that ℓ_d computed for f_y by the method specified in Clause 12.2 shall satisfy the following equation:

$$\ell_d \leq \frac{M_r}{V_f} + \ell_a \qquad \textbf{(12-6)}$$

where, at a support, ℓ_a shall be the embedment length beyond the centre of the support, and at a point of inflection, ℓ_a shall be limited to the effective depth of the member or $12d_b$, whichever is greater.

However, Equation (12-6) need not be satisfied for reinforcement terminating beyond the centreline of simple supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook.

The value of M_r / V_f may be increased by 30% when the ends of the reinforcement are confined by a compressive reaction.

12.12 Development of negative moment reinforcement

12.12.1
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.

12.12.2
At least one-third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection of not less than the effective depth of the member, $12d_b$, or 1/16 of the clear span, whichever is greater.

12.13 Anchorage of shear reinforcement

12.13.1
Web reinforcement shall be carried as close to the compression and tension surfaces of a member as cover requirements and proximity of other reinforcement will permit.

12.13.2
Transverse reinforcement provided for shear shall be anchored by one of the following means:
(a) for 15M and smaller bars, and for MD200 and smaller wire, by a standard stirrup hook (see Clause 7.1.2) around longitudinal reinforcement;
(b) for 20M and 25M stirrups, by a standard hook (see Clause 7.1.2) around longitudinal reinforcement, plus an embedment between mid-depth of the member and the outside end of the hook equal to or greater than $0.33\ell_d$;
(c) for each leg of welded smooth wire fabric forming simple U-stirrups, either
  (i) two longitudinal wires located at a 50 mm spacing along the member at the top of the U; or
  (ii) one longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first wire. The second wire may be located on the stirrup leg beyond a bend or on a bend with an inside diameter of not less than $8d_b$;
(d) for each end of a single-leg stirrup of welded smooth or deformed wire fabric, two longitudinal wires at a minimum spacing of 50 mm, with the inner wire at least $d/4$ from the mid-depth of the member. The outer longitudinal wire at the tension face shall not be farther from that face than the portion of primary flexural reinforcement closest to the face; or
(e) mechanical anchorage capable of developing the yield strength of the bar.

12.13.3
Between anchored ends, each bend in the continuous portion of a stirrup shall enclose a longitudinal bar.

12.13.4
Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall be continuous with the longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond the mid-depth, $d/2$, as specified for development length in Clause 12.2 for that part of f_y required to satisfy Clause 11.3.9.

12.13.5
Pairs of U-stirrups or ties placed so as to form a closed unit shall be considered properly spliced when the length of the laps is $1.3\ell_d$. Alternatively, in members at least 450 mm deep, where $A_b f_y$ is not more than 40 kN per leg, the splice shall be considered adequate if the stirrup legs extend the full available depth of the member.

12.14 Splices of reinforcement — General

12.14.1 Limitations on use
Splices of reinforcement shall be made only as required or permitted by design drawings or specifications, or as authorized by the designer.

12.14.2 Lap splices

12.14.2.1
Lap splices shall not be used for bars larger than 35M, except as specified in Clauses 12.16.2 and 15.9.2.4.

12.14.2.2
Lap splices of bundled bars shall be based on the lap splice length required for individual bars within a bundle, increased by 10% for a two-bar bundle, 20% for a three-bar bundle, and 33% for a four-bar bundle. Individual bar splices within a bundle shall not overlap.

12.14.2.3
Bars spliced by lap splices in flexural members shall have a transverse spacing not exceeding the lesser of one-fifth of the required lap splice length or 150 mm.

12.14.3 Welded splices and mechanical connections

12.14.3.1
Welded splices and other mechanical connections may be used.

12.14.3.2
All welding shall comply with to CSA W186.

12.14.3.3
A full welded splice shall have bars welded to develop, in tension, at least 120% of the specified yield strength, f_y, of the bar, but not less than 110% of the actual yield strength of the bar used in the test of the welded splice.

12.14.3.4
A full mechanical connection shall develop, in tension or compression as required, at least 120% of the specified yield strength, f_y, of the bar, but not less than 110% of the actual yield strength of the bar used in the test of the mechanical connection.

12.14.3.5
Welded splices and mechanical connections not meeting the requirements of Clause 12.14.3.3 or 12.14.3.4 may be used as specified in Clause 12.15.4.

12.15 Splices of deformed bars and deformed wire in tension

12.15.1

The minimum length of lap for tension lap splices shall be as required for a Class A or B splice, but not less than 300 mm, where

(a) the Class A splice length is $1.0\ell_d$; and
(b) the Class B splice length is $1.3\ell_d$.

In Items (a) and (b), ℓ_d is the tensile development length for the specified yield strength, f_y, as specified in Clause 12.2, but without the modification factor specified in Clause 12.2.5.

12.15.2

Lap splices of deformed bars and deformed wire in tension shall be Class B splices, except that Class A splices shall be permitted when

(a) the area of reinforcement provided is at least twice that required by analysis at the splice location; and
(b) less than one-half of the total reinforcement is spliced within the required lap length.

12.15.3

Welded splices or mechanical connections used where the area of reinforcement provided is less than twice that required by analysis shall meet the requirements of Clause 12.14.3.3 or 12.14.3.4.

12.15.4

Welded splices or mechanical connections used where the area of reinforcement provided is at least twice that required by analysis shall meet the following requirements:

(a) splices shall be staggered by at least 600 mm and in such a manner as to develop, at every section, at least twice the factored tensile force at that section, but not less than 140 MPa for the total area of reinforcement provided; and
(b) in computing the tensile resistance developed at each section, spliced reinforcement shall be rated at the specified splice strength. Unspliced reinforcement shall be rated at that fraction of f_y defined by the ratio of the shorter actual development length to the development length, ℓ_d, required to develop the specified yield strength, f_y.

12.15.5

Splices in tension tie members shall be made with a full welded splice or a full mechanical connection as specified in Clause 12.14.3.3 or 12.14.3.4. Splices in adjacent bars shall be staggered by at least 800 mm.

12.16 Splices of deformed bars in compression

12.16.1 Minimum lap length

The minimum lap length for compression lap splices shall be $0.073f_yd_b$ for f_y less than or equal to 400 MPa or $(0.133f_y - 24)d_b$ for f_y greater than 400 MPa, but shall not be taken less than 300 mm.

12.16.2 Lap length for bars of different sizes

When bars of different sizes are lap spliced in compression, the splice length shall be the larger of the development length of the larger bar or the splice length of the smaller bar. Bar sizes 45M and 55M may be lap spliced to 35M and smaller bars.

12.16.3 Welded splices or mechanical connections

Welded splices or mechanical connections used in compression shall meet the requirements of Clause 12.14.3.3 or 12.14.3.4.

12.16.4 End-bearing splices

12.16.4.1

In bars required for compression only, the compressive stress may be transmitted by the bearing of square cut ends held in concentric contact by a suitable device.

12.16.4.2

Bar ends shall terminate in flat surfaces within 1-1/2° of a right angle to the axis of the bars and shall be fitted to within 3° of full bearing after assembly.

12.16.4.3

End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

12.17 Special splice requirements for columns

12.17.1 General

Lap splices, butt-welded splices, mechanical connections, or end-bearing splices shall satisfy the applicable requirements of Clauses 12.17.2 to 12.17.5 for all load combinations for the column.

12.17.2 Reinforcement

Where welded splices, mechanical connections, or end-bearing splices are used, the amount of reinforcement spliced at any location shall not exceed 0.04 times the gross area of the section. Where the gross area of reinforcement exceeds 0.04 times the gross area of the section, connection or splice locations shall be spaced not less than 750 mm apart (see Clause 10.9.2).

12.17.3 Lap splices in columns

12.17.3.1

Where the bar stress due to factored loads is compressive, lap splices shall comply with Clauses 12.16.1 and 12.16.2, and, where applicable, with Clause 12.17.3.4 or 12.17.3.5.

12.17.3.2

Where the bar stress due to factored loads is tensile and does not exceed $0.5f_y$, lap splices shall be Class B if more than one-half of the bars are spliced at any section, or Class A if one-half or fewer of the bars are spliced at any section and alternate lap splices are staggered by ℓ_d.

12.17.3.3

Where the bar stress due to factored loads is greater than $0.5f_y$ in tension, lap splices shall be Class B.

12.17.3.4

In tied reinforced compression members where ties throughout the lap splice length have an effective area of not less than $0.0015hs$, the lap splice length, computed as specified in Clauses 12.16.1 and 12.16.2, may be multiplied by 0.83, but the lap splice length shall be not less than 300 mm. Tie legs perpendicular to dimension h shall be used in determining the effective area.

12.17.3.5

In spirally reinforced compression members, the lap splice length of bars within a spiral, computed as specified in Clauses 12.16.1 and 12.16.2, may be multiplied by 0.75, but the lap splice length shall be not less than 300 mm.

12.17.4 Welded splices or mechanical connections in columns

Welded splices or mechanical connections in columns shall meet the requirements of Clause 12.14.3.3 or 12.14.3.4.

12.17.5 End-bearing splices in columns

End-bearing splices meeting the requirements of Clause 12.16.4 may be used for column bars stressed in compression, provided that the splices are staggered or additional bars are provided at splice locations. The continuing vertical bars in each face of the column shall have an area of at least 0.25 of the area of the vertical reinforcement in that face.

12.18 Splices of welded deformed wire fabric in tension

12.18.1

The minimum length of lap for lap splices of welded deformed wire fabric measured between the ends of each fabric sheet shall be not less than the greater of $1.3\ell_d$ and 200 mm, and the overlap measured between the outermost crosswires of each fabric sheet shall be not less than 50 mm, where ℓ_d shall be the development length, as specified in Clause 12.7, for the specified yield strength, f_y.

12.18.2

The lap splices of welded deformed wire fabric with no crosswires within the lap splice length shall be determined as for deformed wire.

12.19 Splices of welded smooth wire fabric in tension

12.19.1

The minimum length of lap for lap splices of welded smooth wire fabric shall be as specified in Clauses 12.19.2 and 12.19.3.

12.19.2

When the area of reinforcement provided is less than twice that required at the splice location, the length of overlap measured between the outermost crosswires of each fabric sheet shall be not less than the greater of
(a) one spacing of crosswires plus 50 mm;
(b) $1.5\ell_d$; and
(c) 150 mm,
where ℓ_d shall be the development length, as specified in Clause 12.8, for the specified yield strength, f_y.

12.19.3

When the area of reinforcement provided is at least twice that required at the splice location, the length of overlap measured between the outermost crosswires of each fabric sheet shall be not less than the greater of $1.5\ell_d$ and 50 mm, where ℓ_d shall be the development length, as specified in Clause 12.8, for the specified yield strength, f_y.

13 Two-way slab systems

13.1 General

13.1.1
Clause 13 shall apply to the design of slab systems reinforced for flexure in more than one direction, with or without beams between supports.

13.1.2
A slab system may be supported on columns or walls.

13.2 Minimum slab thickness

13.2.1 General
The minimum slab thickness, h_s, shall be based on serviceability requirements but shall be not less than 120 mm.

13.2.2 Two-way slab systems
For regular two-way slab systems (see Clause 2.2), the requirement specified in Clause 9.8.2.6 to show by computation that deflections will not exceed the limits stipulated in Table 9.3 may be waived when the slab thicknesses provided are not less than the minimum thicknesses specified in Clauses 13.2.3 to 13.2.6.

Note: *It is possible that the minimum thickness specified in Clauses 13.2.3 to 13.2.6 will not be adequate for certain sequences of shoring during construction or for large live to dead load ratios.*

13.2.3 Slabs without drop panels
The minimum thickness, h_s, shall be

$$h_s \le \frac{\ell_n(0.6 + f_y/1000)}{30} \tag{13-1}$$

where

ℓ_n = the longer clear span

At discontinuous edges, an edge beam shall be provided with a stiffness ratio, α, of not less than 0.80, or the thickness required by Equation (13-1) shall be multiplied by 1.1 in the panel with the discontinuous edge or edges.

13.2.4 Slabs with drop panels
The minimum thickness, h_s, shall be

$$h_s \le \frac{\ell_n(0.6 + f_y/1000)}{30} - \frac{2x_d}{\ell_n}\ \Delta_h \tag{13-2}$$

where ℓ_n is the longer clear span and Δh is the additional thickness of the drop panel below the soffit of the slab and shall not be taken larger than h_s.

In Equation (13-2), $(2x_d/\ell_n)$ is the smaller of the values determined in the two directions and x_d shall not be taken greater than $(\ell_n/4)$.

At discontinuous edges, an edge beam shall be provided with a stiffness ratio, α, of not less than 0.80, or the thickness required by Equation (13-2) shall be multiplied by 1.1 in the panel with the discontinuous edge or edges.

13.2.5 Slabs with beams between all supports

The minimum thickness, h_s, shall be

$$h_s \leq \frac{\ell_n(0.6 + f_y/1000)}{30 + 4\beta\alpha_m} \quad \textbf{(13-3)}$$

where ℓ_n is the longer clear span, α_m shall not be taken greater than 2.0, and the value α may be determined by taking I_b equal to

$$I_b = \frac{b_w h^3}{12}\left(2.5\left(1 - \frac{h_s}{h}\right)\right) \quad \textbf{(13-4)}$$

13.2.6 Slab bands

The minimum thickness of slab bands shall be that required for beams in Table 9.2.

13.2.7 Computation of slab deflections

A slab thickness less than the minimum thickness required by Clauses 13.2.2 to 13.2.5 may be used if computations show that deflection will not exceed the limits specified in Table 9.3. Deflections shall be computed by 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 Clause 8.6.2. The effective moment of inertia shall be that specified by Equation (9-1). The moment, M_a, shall take into consideration loads, such as construction loads, that extend the cracked region. Other values of I_e may be used if the computed deflection is in reasonable agreement with the results of comprehensive tests. The effective modulus of rupture of the concrete shall be taken as one-half of the value specified by Equation (8-3). Additional long-term deflection shall be computed as specified in Clause 9.8.2.5.

13.3 Design procedures for shear for slabs without beams

13.3.1 General

In the vicinity of concentrated loads or reactions, the factored shear stress resistance, v_r, shall be equal to or greater than the maximum factored shear stress, v_f, due to the factored shear force and unbalanced moments. The stress, v_f, shall be determined for full load on all spans as well as any other patterns of loading that might result in larger stresses.

13.3.2 One-way and two-way shear

Slabs in the vicinity of columns shall be designed for two-way shear as specified in Clauses 13.3.3 to 13.3.5. Slabs shall also be designed for one-way shear as specified in Clause 13.3.6.

13.3.3 Critical shear section for two-way action

13.3.3.1

The critical section for two-way action shall be a section perpendicular to the plane of the slab and located so that its perimeter, b_0, is a minimum, but the section need not approach closer than $d/2$ to the perimeter of the concentrated load or reaction area.

13.3.3.2

At changes in slab thickness, a critical section located in the thinner portion at a distance not greater than $d/2$ from the face of the thicker portion and located such that the perimeter, b_0, is a minimum, shall also be investigated.

13.3.3.3
For square or rectangular load or reaction areas, the critical section may be assumed to have four straight sides. For edge supports, the critical section may be assumed to have three straight sides. For corner supports, the critical section may be assumed to have two straight sides. At edge and corner supports where the slab cantilevers beyond the exterior face of the support, the critical section may be assumed to extend into the cantilevered portion of the slab for a distance not exceeding d.

13.3.3.4
When openings in slabs are located at a distance of less than ten times the slab thickness from a concentrated load or reaction area, or when openings in slabs without beams are located within a column strip, that part of the perimeter of the critical section which is enclosed by straight lines projecting from the centre of the load or reaction area and tangent to the boundaries of the openings shall be considered ineffective.

13.3.4 Maximum shear stress resistance without shear reinforcement

13.3.4.1
The factored shear stress resistance, v_r, shall be the smallest of

(a) $$v_r = v_c = \left(1 + \frac{2}{\beta_c}\right)0.19\lambda\phi_c\sqrt{f'_c} \quad \textbf{(13-5)}$$

where
β_c = the ratio of long side to short side of the column, concentrated load, or reaction area

(b) $$v_r = v_c = \left(\frac{\alpha_s d}{b_o} + 0.19\right)\lambda\phi_c\sqrt{f'_c} \quad \textbf{(13-6)}$$

where
α_s = 4 for interior columns, 3 for edge columns, and 2 for corner columns

(c) $$v_r = v_c = 0.38\lambda\phi_c\sqrt{f'_c} \quad \textbf{(13-7)}$$

13.3.4.2
The value of $\sqrt{f'_c}$ used to calculate v_r in Equations (13-5) to (13-7) shall not exceed 8 MPa.

13.3.4.3
If the effective depth, d, used in two-way shear calculations exceeds 300 mm, the value of v_c obtained from Equations (13-5) to (13-7) shall be multiplied by 1300/(1000+d).

13.3.4.4
The requirements of Clause 13.3.4.3 need not be applied to the design of footings or mat foundations where the distance from the point of zero shear to the face of the column, pedestal, or wall is less than 3d.

13.3.5 Factored shear stress

13.3.5.1
For interior, edge, and corner supports, the shear forces and unbalanced moments to be transferred to the support shall be resolved into a single shear force acting at the centroid of the critical section and moments about the centroidal axes (x and y directions) of the critical section.

13.3.5.2
The shear stress due to the factored shear force acting at the centroid of the section shall be assumed to be uniformly distributed over the critical shear section.

13.3.5.3
The fraction of the unbalanced moment transferred by eccentricity of shear at interior, edge, and corner columns, γ_v, shall be

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} \qquad \textbf{(13-8)}$$

13.3.5.4
The shear stress due to moment transfer by eccentricity of shear shall be assumed to vary linearly about the centroid of the critical shear section.

13.3.5.5
The factored shear stress, v_f, shall be computed from the following equation, with the factored shear force and unbalanced moments about the x and y directions obtained from a consistent loading:

$$v_f = \frac{V_f}{b_o d} + \left(\frac{\gamma_v M_f e}{J}\right)_x + \left(\frac{\gamma_v M_f e}{J}\right)_y \qquad \textbf{(13-9)}$$

13.3.5.6
The fraction of the unbalanced moment not transferred by eccentricity of shear stress shall be transferred by flexure as specified in Clause 13.10.2.

13.3.6 One-way shear

13.3.6.1 General
One-way shear for a slab with a critical section extending in a plane across the entire width and located at a distance, d, from the face of the concentrated load or reaction area shall be as specified in Clauses 11.1 to 11.3. The one-way shear shall be distributed between the column strip and the middle strip in proportion to the design negative moments in each strip.

13.3.6.2 Corner columns
The factored shear resistance of slabs in the vicinity of corner columns, v_c, shall be taken as

$$v_c = \beta\lambda\phi_c\sqrt{f_c'} \qquad \textbf{(13-10)}$$

along a critical shear section located not farther than $d/2$ from the edge of the column or column capital, where β is as specified in Clauses 11.3.6.2 and 11.3.6.3.

Where the slab cantilevers beyond the face of the corner column or the corner column capital, the length of the critical section may be taken as extended into the cantilevered portion for a length not exceeding d.

Corner columns meeting the requirements of this Clause shall be deemed to have satisfied the requirements of Clauses 13.3.4 and 13.3.5.

Note: *For slabs greater than* 350 mm *deep, or slabs with beams exceeding the limitations specified in Clause 11.3.6.2, application of the one-way shear provisions specified in Clauses 11.3.6.3 and 11.3.6.4 should take into account the reduction of allowable concrete shear stress resulting from the effect of depth in such members.*

13.3.7 Shear reinforcement for slabs without beams

13.3.7.1
Shear reinforcement consisting of headed shear reinforcement, stirrups, or shearheads may be used to increase the shear capacity of slabs and footings. The design of shearheads shall be based on the concepts in ACI 318M-02/ACI 318RM-02.

13.3.7.2
The shear resistance shall be investigated at the section specified in Clause 13.3.3.1 and at successive sections more distant from the support.

13.3.7.3
Within the shear reinforced zone, the factored shear stress resistance, v_r, shall be computed as ($v_c + v_s$), where v_c and v_s shall be computed as specified in Clauses 13.3.8.3 and 13.3.8.5 for headed shear reinforcement and as specified in Clauses 13.3.9.3 and 13.3.9.4 for stirrups.

13.3.7.4
Shear reinforcement shall be extended to the section where v_f is not greater than $0.19\lambda\phi_c\sqrt{f'_c}$, but at least a distance $2d$ from the face of the column.

13.3.8 Headed shear reinforcement

13.3.8.1
Headed shear reinforcement shall be mechanically anchored at each end by a plate or head bearing against the concrete in such a manner that it is capable of developing the yield strength of the bar. The area of the plate or head shall be at least ten times the cross-sectional area of the bar unless a smaller area can be justified experimentally (see Clause 7.1.4).

13.3.8.2
When headed shear reinforcement is provided, the factored shear stress, v_f, shall be not greater than $0.75\lambda\phi_c\sqrt{f'_c}$.

13.3.8.3
In the zone reinforced by headed shear reinforcement, the factored shear stress resistance of the concrete, v_c, shall be $0.28\lambda\phi_c\sqrt{f'_c}$.

13.3.8.4
Headed shear reinforcement shall be located along concentric lines that parallel the perimeter of the column cross-section.

13.3.8.5
The factored shear stress resistance of headed shear reinforcement, v_s, shall be computed as

$$v_s = \frac{\phi_s A_{vs} f_{yv}}{b_o s} \tag{13-11}$$

where

A_{vs} = the cross-sectional area of the headed shear reinforcement on a concentric line parallel to the perimeter of the column

13.3.8.6
The distance between the column face and the first line of headed shear reinforcement shall be 0.4d. The upper limits for the spacing, s, between lines of headed shear reinforcement shall be based on the value of v_f at a critical section $d/2$ from the column face, as follows:

(a) $s \le 0.75d$ when $v_f \le 0.56\lambda\phi_c\sqrt{f_c'}$ **(13-12)**

(b) $s \le 0.5d$ when $v_f > 0.56\lambda\phi_c\sqrt{f_c'}$ **(13-13)**

13.3.8.7
Unless the headed shear reinforcement is otherwise protected, the minimum concrete cover over the heads shall be the same as the minimum cover for the flexural reinforcement as specified in Clause 7.9. The concrete cover shall not exceed the minimum cover plus one-half the bar diameter of the flexural reinforcement.

13.3.9 Stirrup reinforcement

13.3.9.1
Stirrups anchored as specified in Clauses 7.1.2 and 12.13 may be used as shear reinforcement provided that the overall thickness of the slab is not less than 300 mm.

13.3.9.2
When stirrups are provided, the factored shear stress, v_f, shall not be greater than $0.55\lambda\phi_c\sqrt{f_c'}$.

13.3.9.3
In the zone reinforced by stirrups, the factored shear stress resistance of the concrete, v_c, shall be $0.19\lambda\phi_c\sqrt{f_c'}$.

13.3.9.4
The factored shear stress resistance, v_s, shall be computed from Equation (13-11), with A_{vs}, the cross-sectional area of the stirrups, on a line parallel to the perimeter of the column.

13.3.9.5
The stirrup spacing, s, shall not exceed $d/2$, with the first stirrup placed at $d/4$ from the column face.

13.4 Shear in slab systems with beams

13.4.1
Beams with $(\alpha_1 \ell_{2a}/\ell_1)$ equal to or greater than 1.0 shall be designed to resist shear caused by factored loads on tributary areas bounded by 45° lines drawn from the corners of the panels and the centrelines of the adjacent panels parallel to the long sides.

13.4.2
Beams with $(\alpha_1 \ell_{2a}/\ell_1)$ less than 1.0 may be designed to resist shear obtained by linear interpolation, assuming beams carry no load at $\alpha_1 = 0$.

13.4.3
In addition to resisting shears calculated as specified in Clauses 13.4.1 and 13.4.2, beams shall be designed to resist shears caused by factored loads applied directly on the beams.

13.4.4
Slab shears may be computed on the assumption that load is distributed to supporting beams as specified in Clause 13.4.1 or 13.4.2. Resistance to total shear occurring on a panel shall be provided.

13.4.5
The shear resistance of beams shall satisfy the requirements of Clause 11.

13.5 Design procedures for flexure
Note: *See also Clauses 13.6 to 13.12 and Annex B.*

13.5.1
A slab system may be designed using any procedure satisfying conditions of equilibrium and compatibility with the supports, provided that it is shown that the factored resistance at every section is at least equal to the effects of the factored loads and that all serviceability conditions, including specified limits on deflections, are met.

13.5.2
For lateral loads, analysis of unbraced frames shall take into account the effects of cracking and reinforcement on stiffness of frame members.

13.5.3
The results of the gravity load analysis shall be combined with the results of the lateral load analysis.

13.5.4
Openings of any size may be provided in slab systems if it is shown by analysis that the factored resistance is at least equal to the effects of factored loads in accordance with Clauses 8.3, 8.4, and 13.3.3.4, and that all serviceability conditions, including the specified limits on deflections, are met.
Note: *Clause 13.10.10 provides simple rules for regular slabs without beams.*

13.6 Elastic plate theory

13.6.1
Analysis of slab systems may be based on elastic plate theory that uses either classical or numerical techniques.
Note: *The successful application of the results of analysis using elastic plate theory requires proper consideration of factors such as selection of thickness, moment redistribution due to the effects of cracking, creep, shrinkage effects, and construction loading.*

13.6.2
Care shall be taken to ensure realistic modelling of the size and effective stiffness of the supporting elements, including beams, if any.

13.6.3
Appropriate loading patterns shall be considered to ensure determination of maximum values for all stress resultants at each section.

13.6.4
When reinforcement is placed as an orthogonal mat in the x and y directions, the factored design moments shall be adjusted to account for the effects of torsion. In lieu of more detailed calculations, the design moment intensities, $m_{x,des}$ or $m_{y,des}$, in the x and y directions at any point shall be computed as follows:

(a) positive design moments:

$$m_{x,des} = m_x + |m_{xy}| \tag{13-14}$$

$$m_{y,des} = m_y + |m_{xy}| \tag{13-15}$$

If either $m_{x,des}$ or $m_{y,des}$ is negative, it shall be taken as zero.

(b) negative design moments:

$$m_{x,des} = m_x - |m_{xy}| \tag{13-16}$$

$$m_{y,des} = m_y - |m_{xy}| \tag{13-17}$$

If either $m_{x,des}$ or $m_{y,des}$ is positive, it shall be taken as zero.

13.6.5
Uniformly spaced reinforcement shall be placed in bands such that
(a) the total reinforcement provided within a band shall be sufficient to resist the total factored moment computed for that band; and
(b) the moment resistance per unit width within the band shall be at least two-thirds of the maximum factored moment intensity within the band.

Note: *Additional information on the application of finite element analysis design techniques is provided by the Cement Association of Canada's* Concrete design handbook.

13.7 Theorems of plasticity

13.7.1
Analysis of slab systems for factored loads may be based on either the lower bound or upper bound theorems of plasticity.

Note: *The successful application of the results of plastic analysis requires proper assumptions that will ensure that serviceability requirements, including creep and shrinkage effects, are satisfied.*

13.7.2
The size and effective stiffness of the supporting elements shall be considered in the analysis.

13.7.3
When strength design is based on the upper bound theorem (e.g., yield line method), the factored moments shall be obtained from calculations based on a need for a mechanism to form over the whole or part of the slab at collapse. The mechanism that is the most critical shall be used for the design of the slab.

13.7.4
Factored moments obtained using lower bound theory (e.g., strip method) shall satisfy the requirements of equilibrium and the boundary conditions applicable to the slab.

13.7.5
Reinforcement may be uniformly spaced in bands, with band widths selected to ensure that serviceability requirements are satisfied.

13.8 Slab systems as elastic frames

13.8.1 Definition of frame geometry

13.8.1.1
A regular two-way slab system (see Clause 2.2) may, for purposes of analysis, be considered a series of plane frames acting longitudinally and transversely through the building. Each frame shall be composed of equivalent line members intersecting at member centrelines, shall follow a column line, and shall include the portion of slab bounded laterally by the centreline of the panel on each side.
Note: *A floor system with beams between supports that does not satisfy the limits specified in the definition of a regular two-way slab system in Clause 2.2 may be analyzed as specified in this Clause, but the reinforcing distribution specified in Clause 13.12 will normally not be applicable.*

13.8.1.2
Each floor and roof slab with attached columns may be analyzed separately, with the far ends of the columns considered fixed.

13.8.1.3
Where slab-beams are analyzed separately, it may be assumed in determining moment at a given support that the slab-beam is fixed at any support two panels distant from the slab-beam, provided that the slab continues beyond that point.

13.8.1.4
The change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected.

13.8.1.5
Member stiffness used in the analysis of the elastic frame shall be selected to simulate the behaviour of the slab system.

13.8.2 Non-prismatic modelling of member stiffness

13.8.2.1
When members are modelled as non-prismatic elements, the member stiffness may be as specified in Clauses 13.8.2.2 to 13.8.2.10.

13.8.2.2
The moment of inertia of column and slab-beam elements at any cross-section outside of joints or column capitals shall be based on the gross area of concrete at that section.

13.8.2.3
The moment of inertia of slab-beams from the centre of the column to the face of the column, bracket, or capital shall be assumed to be equal to the moment of inertia of the slab-beam at the face of the column, bracket, or capital divided by the quantity $(1 - c_2/\ell_{2a})^2$, where c_2 and ℓ_{2a} are measured transverse to the direction of the span for which moments are being determined.

13.8.2.4
The moment of inertia of column elements from top to bottom of the slab-beam at a joint shall be assumed to be infinite.

13.8.2.5
An equivalent column shall be assumed to consist of the actual columns above and below the slab-beam plus an attached torsional member transverse to the direction of the span for which moments are being determined.

13.8.2.6
The flexibility 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 follows:

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_c} + \frac{1}{K_t} \quad \textbf{(13-18)}$$

13.8.2.7
Attached torsional members shall be assumed to have a constant cross-section throughout their length consisting of the largest of the following:
(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 Item (a) plus that part of the transverse beam above and below the slab; or
(c) a transverse beam which includes that portion of slab on each side 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.

13.8.2.8
The stiffness, K_t, of attached torsional members shall be calculated as follows:

$$K_t = \sum \frac{9E_c C}{\ell_t \left(1 - \frac{c_2}{\ell_t}\right)^3} \quad \textbf{(13-19)}$$

13.8.2.9
The section parameter, C, in Equation (13-19) may be evaluated for the cross-section by dividing it into separate rectangular parts and carrying out the following summation:

$$C = \sum \left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3} \quad \textbf{(13-20)}$$

13.8.2.10
Where beams frame into columns in the direction of the span for which moments are being determined, the value of K_t shall be multiplied by the ratio of the moment of inertia of the slab with such beam to the moment of inertia of the slab without such beam.

13.8.3 Prismatic modelling of member stiffness

13.8.3.1
When members are modelled as prismatic elements, the member stiffness may be assigned as specified in Clauses 13.8.3.2 and 13.8.3.3.

13.8.3.2
For prismatic modelling of slab-beam elements, the moment of inertia shall be based on the gross area of the concrete outside the joints or column capitals. When the moment of inertia varies outside the joint, e.g., in drop panels, the slab-beam elements may be modelled as a series of prismatic elements with moments of inertia based on the gross concrete dimensions.

13.8.3.3
For prismatic modelling of column elements, the effective moment of inertia shall be taken as equal to ψ times the moment of inertia based on the gross area outside the joints, where ψ is given as follows:

(a) for $\ell_2 / \ell_1 \leq 1.0$: $\psi = 0.3 + 0.7 \dfrac{\alpha_1 \ell_2}{\ell_1}$ **(13-21)**

(b) for $\ell_2 / \ell_1 > 1.0$: $\psi = 0.6 \left(\dfrac{\ell_2}{\ell_1} - 0.5\right) + \left(1.3 - 0.6 \dfrac{\ell_2}{\ell_1}\right) \dfrac{\alpha_1 \ell_2}{\ell_1}$ **(13-22)**

In Equations (13-21) and (13-22), ψ shall not be taken less than 0.3 or greater than 1.0, and $\alpha_1 \ell_2 / \ell_1$ shall not be taken greater than 1.0.

13.8.4 Arrangement of live load

13.8.4.1
When the loading pattern is known, the frame shall be analyzed for that load.

13.8.4.2
When the live load is uniformly distributed and does not exceed three-quarters of the specified dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum factored moments may be assumed to occur at all sections with full factored live load on the entire slab system.

13.8.4.3
For loading conditions other than those specified in Clauses 13.8.4.1 and 13.8.4.2, the maximum positive factored moment near midspan of a panel may be assumed to occur with three-quarters of the full factored uniformly distributed live load on the panel and on alternate panels, and the maximum negative factored moment in the slab at a support may be assumed to occur with three-quarters of the full factored uniformly distributed live load on adjacent panels only.

13.8.4.4
Factored moments shall not be taken as less than those occurring with full factored live loads on all panels.

13.8.5 Critical sections

13.8.5.1
Except as required by Clause 13.8.5.2, at interior and exterior supports the critical section for the negative factored moment shall be taken at the face of rectilinear supports, but not at a distance greater than $0.175\ell_1$ from the centre of the column.

13.8.5.2
In addition to the requirements specified in Clause 13.8.5.1, at exterior supports that include brackets or capitals the critical section for the negative factored moment in the span perpendicular to an edge shall be taken at a distance from the face of the supporting element not greater than one-half of the projection of the bracket or capital beyond the face of the supporting element.

13.8.5.3
When the critical section for the negative design moment is being located, circular or regular polygonal supports shall be treated as square supports with the same area.

13.8.5.4
Reinforcement to resist the moments at the critical sections shall be selected in accordance with Clauses 13.10 to 13.12.

13.8.5.5
The flexural capacity shall be checked at any change in the slab depth, e.g., at the edges of drop panels and slab bands.

13.9 Direct design method

13.9.1 Limitations

13.9.1.1
Regular two-way slab systems (see Clause 2.2) that comply with the limitations specified in Clauses 13.9.1.2 to 13.9.1.5 may be designed using the direct design method.
Note: *It is possible that in some circumstances the requirements specified in Clauses 13.9.1.2 to 13.9.1.5 will not address concerns related to slab systems with drop panels and/or beams as rigorously as the requirements specified in Clause 13.8.*

13.9.1.2
There shall be a minimum of three continuous spans in each direction.

13.9.1.3
Successive span lengths centre-to-centre of supports in each direction shall not differ by more than one-third of the longer span.

13.9.1.4
All loads shall be due to gravity only and uniformly distributed over an entire panel. The factored live load shall not exceed twice the factored dead load.

13.9.1.5
Variations from the limitations of Clauses 13.9.1.2 to 13.9.1.4 shall be acceptable if it is demonstrated by analysis that the requirements specified in Clause 13.5.1 are satisfied.

13.9.2 Total factored static moment for a span

13.9.2.1
The total factored static moment for a span shall be determined in a strip bound laterally by the centrelines of the panels on each side of the centreline of supports.

13.9.2.2
For each span of each strip, the sum of the absolute values of the positive and the average negative factored moments, in each direction, shall be not less than

$$M_o = \frac{w_f \ell_{2a} \ell_n^2}{8} \quad \textbf{(13-23)}$$

13.9.2.3
The clear span, ℓ_n, shall extend from face-to-face of columns, capitals, brackets, or walls. The value of ℓ_n used in Equation (13-23) shall be not less than $0.65\ell_1$.

13.9.3 Negative and positive factored moments

13.9.3.1
In an interior span, the total static moment, M_o, shall be distributed as follows:

Negative factored moment at the face of support	0.65
Positive factored moment at midspan	0.35

13.9.3.2
In an end span, the total factored static moment, M_o, shall be distributed as specified in Table 13.1.

Table 13.1
Distribution factors for total factored static moment
(See Clause 13.9.3.2.)

Moment	Exterior edge unrestrained	Slab with beams between all supports	Slab without beams between interior supports	Exterior edge fully restrained
Interior negative factored moment	0.75	0.70	0.70	0.65
Positive factored moment	0.66	0.59	0.52	0.35
Exterior negative factored moment	0	0.16	0.26	0.65

13.9.3.3
Negative and positive factored moments may be modified by 15% provided that the total static moment for a span in the direction considered is not less than that required by Equation (13-23).

13.9.3.4
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; however, the moments may first be modified in accordance with Clause 13.9.3.3.

13.9.4 Unbalanced factored moments in columns and walls
At an interior support, the joint and supporting elements above and below the slab shall resist the factored moment specified in the following equation in direct proportion to their stiffness:

$$M_f = 0.07\left((w_{df} + 0.5w_{lf})\ell_{2a}\ell_n^2 - w'_{df}\ell'_{2a}(\ell'_n)^2\right) \quad \textbf{(13-24)}$$

where w'_{df}, ℓ'_{2a}, and ℓ'_n refer to the shorter span.

13.9.5 Selection of reinforcement
Reinforcement to resist the moments at the critical sections shall be selected in accordance with Clauses 13.10 to 13.12.

13.10 Slab reinforcement

13.10.1 General
Reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections but shall be not less than that required by Clause 7.8.1.

Note: *Where strict crack control is a concern, slabs with drop panels, particularly in a corrosive environment, can require additional reinforcement in the negative middle strip region to limit cracking. This additional reinforcement is not included in the calculation of moment resistance. The reinforcement required to limit cracking is generally more than that required by Clause 7.8.1.*

13.10.2 Shear and moment transfer
When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, a fraction of unbalanced moment given by

$$\gamma_f = 1 - \gamma_v \qquad \textbf{(13-25)}$$

shall be transferred by flexural reinforcement placed within a width b_b.

Note: *For exterior supports, including corner columns, Clause 13.10.3 satisfies this requirement.*

13.10.3 Exterior columns
Reinforcement for the total factored negative moment transferred to the exterior columns shall be placed within a band width b_b. Temperature and shrinkage reinforcement determined as specified Clause 7.8.1 shall be provided in that section of the slab outside of the band region defined by b_b, or as required by Clause 13.10.9.

13.10.4 Spacing
Except for portions of slab area that are of cellular or ribbed construction, spacing of reinforcement at critical sections shall not exceed the following limits:

Negative reinforcement in the band defined by b_b	$1.5h_s$, but $s \le 250$ mm
Remaining negative moment reinforcement	$3h_s$, but $s \le 500$ mm
Positive moment reinforcement	$3h_s$, but $s \le 500$ mm

In the slab over cellular spaces, reinforcement shall be provided as required by Clause 7.8.

13.10.5 Anchorage

13.10.5.1
Positive moment reinforcement perpendicular to a discontinuous edge shall have embedment, straight or hooked, at least 150 mm into the spandrel beams, columns, or walls.

13.10.5.2
Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored in spandrel beams, columns, or walls, and shall be developed at the face of the support as specified in Clause 12.

13.10.5.3
Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, both the top and bottom reinforcement shall extend to the edge of the slab.

13.10.6 Structural integrity reinforcement

13.10.6.1

The summation of the area of bottom reinforcement connecting the slab, drop panel, or slab band to the column or column capital on all faces of the periphery of the column or column capital shall be

$$\sum A_{sb} = \frac{2V_{se}}{f_y} \tag{13-26}$$

Integrity reinforcement shall not be required if there are beams containing shear reinforcement in all spans framing into the column.

13.10.6.2

The reinforcement specified in Clause 13.10.6.1 shall consist of at least two bars or two tendons that extend through the column core or column capital region in each span direction.

13.10.6.3

The bottom reinforcement required by Clause 13.10.6.1 shall be provided by one or more of the following:
(a) bottom reinforcement extended such that it is lap spliced over a column or column capital, with the bottom reinforcement in adjacent spans using a Class A tension lap splice;
(b) additional bottom reinforcement passing over a column or column capital such that an overlap of $2\ell_d$ is provided, with the bottom reinforcement in adjacent spans;
(c) at discontinuous edges, bottom reinforcement extended and bent, hooked, or otherwise anchored over the supports such that the yield stress can be developed at the face of the column or column capital as specified in Clause 12; or
(d) continuous tendons draped over column capitals, with a minimum total area of prestressing steel calculated using Equation (13-26), but with f_y replaced by f_{py}.

13.10.7 Effective depth at drop panels

Where a drop panel is used to reduce the amount of negative moment reinforcement over the column, the thickness of the drop panel below the slab shall not be assumed greater than one-quarter of the distance from the edge of the drop panel to the edge of the column or column capital.

13.10.8 Curtailment of reinforcement

13.10.8.1

For regular two-way slabs (see Clause 2.2) that comply with the requirements specified in Clauses 13.9.1.2 to 13.9.1.5, minimum extensions shall be as shown in Figure 13.1.

13.10.8.2

The required extensions for slabs not complying with the requirements specified in Clauses 13.9.1.2 to 13.9.1.5 shall meet the requirements specified in Clauses 12.11 and 12.12, but shall be not less than those shown in Figure 13.1.

13.10.8.3

Where adjacent spans are unequal, the extension of negative reinforcement beyond the face of the support, as shown in Figure 13.1, shall be based on the longer span.

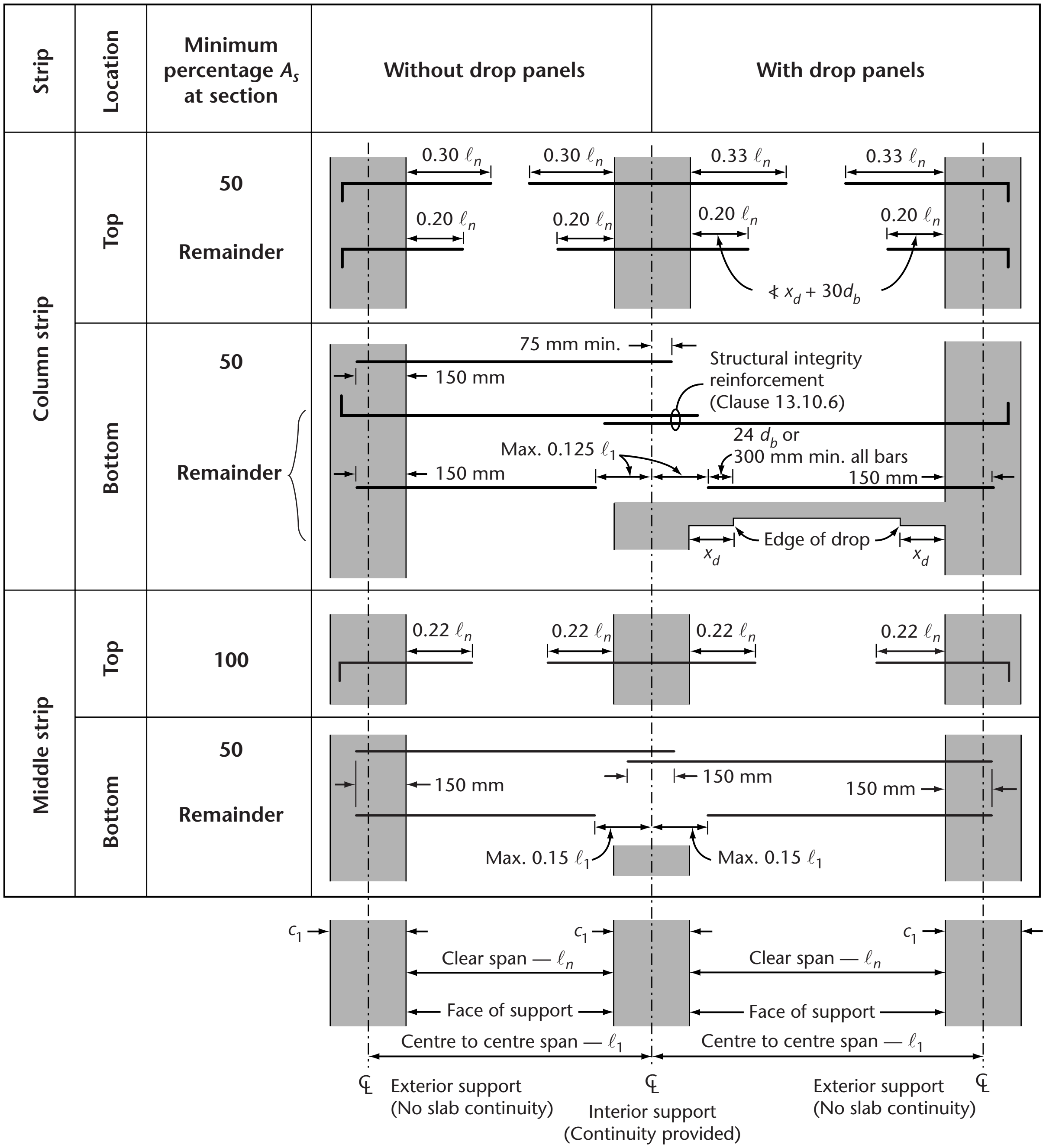


Figure 13.1
Minimum length of reinforcement for slabs without interior beams
(See Clauses 2.3 and 13.10.8.1–13.10.8.3.)

13.10.9 Top reinforcement at slab edges
Slab edges shall be reinforced with top reinforcement perpendicular to the edge to resist the factored moments caused by edge loads, but not less than that required by Clause 7.8.1.

13.10.10 Openings

13.10.10.1
Openings may be placed in regular two-way slabs without beams (see Clause 2.2) without the special analysis required by Clause 13.5.4, provided that the requirements specified in Clauses 13.10.10.2 to 13.10.10.5 are met.

13.10.10.2
Openings of any size may be located in the area common to intersecting middle strips, provided that the total amount of reinforcement required for the panel without the opening is maintained.

13.10.10.3
In the area common to intersecting column strips, not more than one-eighth of the width of the column strip in either span shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added adjacent to the sides of the opening.

13.10.10.4
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 adjacent to the sides of the opening.

13.10.10.5
The shear requirements specified in Clause 13.3.3.4 shall be satisfied.

13.11 Lateral distribution of moments for slabs without interior beams

13.11.1 General
In addition to the requirements specified in Clause 13.10, slabs without beams designed as specified in Clauses 13.8 and 13.9 shall be reinforced for flexure as specified in Clauses 13.11.2 and 13.11.3.

13.11.2 Factored moments in column strip

13.11.2.1
The column strip shall be designed to resist the total negative or positive factored moments at the critical sections multiplied by an appropriate factor as specified in Clauses 13.11.2.2 to 13.11.2.5.

13.11.2.2
The following multiplication factors shall apply to slabs without drop panels (with or without spandrel beams):

(a) Negative moment at an interior column	0.70 to 0.90
(b) Negative moment at an exterior column	1.00
(c) Positive moment at all spans	0.55 to 0.65

13.11.2.3
The following multiplication factors shall apply to slabs with drop panels (with or without spandrel beams):

(a) Negative moment at an interior column	0.75 to 0.90
(b) Negative moment at an exterior column	1.00
(c) Positive moment at all spans	0.55 to 0.65

13.11.2.4
The following multiplication factors shall apply to slabs with slab bands, in the direction of the slab band:

Negative moment at an interior column	0.80 to 1.00
Negative moment at an exterior column	1.00
Positive moment at all spans	0.80 to 1.00

13.11.2.5
The following multiplication factors shall apply to slabs with slab bands, in the direction perpendicular to the slab band:

Negative moment at an interior column in width b_b	Not less than 0.05 to 0.15, with the remaining negative moment assumed evenly distributed over the entire frame width
Negative moment at an exterior column	1.00
Positive moment at all spans where $(\ell_1/\ell_2) \geq 1.0$	0.50 to 0.60
Positive moment at all spans where $(\ell_1/\ell_2) < 1.0$	$0.5\,(\ell_1/\ell_2)$ to $0.6\,(\ell_1/\ell_2)$

13.11.2.6
For negative moment at an exterior column in slabs with spandrel beams, the requirements specified in Clause 13.11.2.2(b) or 13.11.2.3(b) shall apply.

13.11.2.7
At interior columns, the band width, b_b, shall be designed to resist at least one-third of the total factored negative moment in the entire design strip.

13.11.3 Factored moments in middle strips

13.11.3.1
That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

13.11.3.2
Each middle strip shall be proportioned to resist the sum of the factored moments assigned to its two half middle strips.

13.11.3.3
A middle strip adjacent to and parallel with an edge supported by a wall shall be proportioned to resist twice the factored moment assigned to the half middle strip corresponding to the first row of interior supports.

13.11.3.4
At slab edges, the requirements specified in Clause 13.10.9 shall be satisfied.

13.12 Reinforcement for slabs with beams between all supports

13.12.1 General
In addition to the requirements specified in Clause 13.10, slabs with beams designed as specified in Clauses 13.8 and 13.9 shall be reinforced for flexure as specified in Clauses 13.12.2 to 13.12.5.

13.12.2 Factored moments in beams

13.12.2.1
Beams shall be reinforced to resist the following fraction of the positive or interior negative factored moments determined by analysis or determined as specified in Clause 13.9.3:

$$\frac{\alpha_1}{0.3+\alpha_1}\left(1-\frac{\ell_2}{3\ell_1}\right)$$

13.12.2.2
Beams shall be proportioned for 100% of the exterior negative moment.

13.12.2.3
In addition to moments calculated for uniform loads applied to the slab as specified in Clauses 13.12.2.1 and 13.12.2.2, beams shall be proportioned to resist moments caused by concentrated or linear loads applied directly to the beams, including the weight of the beam stem.

13.12.3 Slab reinforcement for positive moment
The slab shall be reinforced to resist the factored positive moments not supported by the beams. This reinforcement may be distributed uniformly over the width of the slab.

13.12.4 Slab reinforcement for negative moment

13.12.4.1 Interior supports
The slab shall be reinforced to resist the interior negative moments not resisted by the beams. This reinforcement shall be uniformly distributed over the width of the slab.

13.12.4.2 Exterior supports
The reinforcement for the exterior factored negative moment in the beam shall be placed within a band with a width b_b unless calculations show that reinforcement placed outside this limit can develop its full capacity.

13.12.5 Corner reinforcement

13.12.5.1
In slabs with beams between supports with a value of α greater than 1.0, top and bottom slab reinforcement shall be provided at exterior corners for a distance, in each direction, equal to one-fifth of the shorter span.

13.12.5.2
The reinforcement shall be sufficient to resist a moment per unit width equal to the maximum positive moment per unit width in the slab.

13.12.5.3
The reinforcement at the top of the slab shall be provided to resist moments about axes perpendicular to the diagonal from the corner. The bottom reinforcement shall be provided to resist moments about axes parallel to the diagonal. The reinforcement may be placed in bands parallel to the sides of the slab.

14 Walls

Note: *See Clause 2.2 under "Wall" for wall type definitions.*

14.1 General requirements for all walls

14.1.1 Application
Clauses 14.1.2 to 14.4.6 shall apply to the design of walls, except where the additional requirements specified in Clauses 16 and 21 to 23 apply.

14.1.2 Lateral support of walls
Walls shall be considered laterally supported if

(a) walls or other vertical bracing elements are arranged in two directions so as to provide lateral stability to the structure as a whole; and

(b) connections between the wall and its lateral supports are designed to resist a horizontal force not less than 2% of the total factored vertical load that the wall is designed to carry at the level of the lateral support, but not less than 10 kN per metre length of the wall.

14.1.3 Design length of wall for the distribution of concentrated vertical loads

14.1.3.1
In lieu of a detailed analysis, each concentrated compressive vertical load acting on a wall shall be assumed to be uniformly distributed over a horizontal length ℓ_b of wall. At any position below the level of the concentrated load, the portion of ℓ_b on each side of the centre of the concentrated load shall be one-half of the width of the bearing plus the width enclosed by a line sloping downward at two vertical to one horizontal on each side, limited by intersection with the end of the wall. This stressed width shall not be assumed to exceed nine times the wall thickness on each side of the bearing area. For a wall subjected to more than one concentrated load, the design shall take into account the overlapping of uniformly distributed loads from each of the concentrated loads.

14.1.3.2
Cracking resulting from transverse tensile stresses caused by the spread of the concentrated loads acting on the wall shall be taken into account in the design.
Note: *Strut-and-tie models can be used to compute the amount of reinforcement required to resist the transverse force under bearing loads and to control cracking.*

14.1.4 Columns built integrally with walls
Columns built integrally with walls shall be designed as specified in Clause 10, with outside dimensions that comply with Clause 10.11.2.

14.1.5 Transfer of vertical wall loads through floor
When the specified compressive strength of the concrete in the walls, f'_{cw}, exceeds that specified for the floor, f'_{cs}, the strength of a wall-to-floor joint shall be determined using the lower of the concrete strengths in the wall and the floor. The strength of this joint can be increased by adding dowels or by increasing the strength of the concrete in the floor under and adjacent to the wall. Such concrete shall extend at least 500 mm into the floor from each face of the wall.

14.1.6 Transfer of horizontal wall forces across construction joints
Transfer of horizontal wall forces across construction joints shall be as specified in Clause 11.5. The area of the reinforcement crossing the shear plane shall be the larger of
(a) the reinforcement area provided for flexure and axial loads; and
(b) the reinforcement area required for shear friction.

In flanged walls, only the vertical reinforcement in those portions of the section assumed to resist horizontal shear shall be included in this calculation.

14.1.7 Minimum thickness of walls

14.1.7.1 Bearing walls and shear walls
The thickness of bearing walls and the webs and flanges of shear walls shall be not less than the smaller of $\ell_w/25$ or $h_u/25$, but not less than 150 mm.

14.1.7.2 Non-bearing walls
The thickness of non-bearing walls shall be not less than 1/30 of the unsupported height or length, whichever is shorter, or less than 100 mm.

14.1.8 Details of wall reinforcement

14.1.8.1 Distributed and concentrated reinforcement
Walls shall have distributed vertical and horizontal reinforcement in layers in accordance with Clauses 14.1.8.2 to 14.1.8.7. Walls shall also have concentrated vertical reinforcement in accordance with Clause 14.1.8.8.

14.1.8.2 Maximum diameter of distributed reinforcement
The diameter of bars used for distributed reinforcement shall not exceed one-tenth of the wall thickness.

14.1.8.3 Number of layers of wall reinforcement
Except for exterior basement walls or retaining walls, bearing or shear walls more than 210 mm thick shall have the reinforcement for each direction placed in two layers. Each layer shall be placed not more than $t/3$ from the surface of the wall.

14.1.8.4 Spacing of reinforcement

The vertical and horizontal reinforcement in each layer shall not be spaced farther apart than three times the wall thickness or 500 mm, whichever is less.

14.1.8.5 Distributed vertical reinforcement

The minimum area of distributed vertical reinforcement between boundary elements shall be $0.0015A_g$.

14.1.8.6 Distributed horizontal reinforcement

The minimum area of distributed horizontal reinforcement shall be $0.002A_g$. However, where crack control is critical or wall geometry or the length of the wall between joints causes significant restraint of shrinkage or thermal strains, reinforcement additional to that specified in this Clause, or other crack control measures, shall be considered.

14.1.8.7 Ties for distributed vertical compression reinforcement

Distributed vertical reinforcement, if stressed in compression, shall be tied and detailed in accordance with the requirements for column reinforcement specified in Clause 7, except that ties may be omitted if
(a) the area of vertical steel is less than $0.005A_g$; and
(b) the bar size is 20M or smaller.

14.1.8.8 Concentrated vertical reinforcement

14.1.8.8.1 Nominal concentrated vertical reinforcement

Concentrated vertical reinforcement consisting of not fewer than two 15M vertical bars shall be provided at each end of all walls.

14.1.8.8.2 Reinforcement for flexure

Concentrated vertical reinforcement shall be provided in boundary elements of shear walls to provide that part of the resistance to strong-axis bending not provided by the reinforcement specified in Clause 14.1.8.5.

14.1.8.8.3 Concentrated vertical reinforcement limits

The reinforcement within any region of concentrated vertical reinforcement shall not exceed the amounts specified in Clause 10.9, substituting A_{gb} for A_g.

14.1.8.8.4 Ties for concentrated vertical reinforcement

Concentrated vertical reinforcement in excess of two 20M bars shall be tied and detailed as specified in Clause 7.

14.1.8.9 Reinforcement at openings

In addition to the reinforcement required by Clauses 14.1.8.5, 14.1.8.6, and 14.1.8.8.1, not less than one 15M bar per layer, or reinforcement having the same area, shall be provided around all window and door or similar openings. Such bars shall extend to develop the bar, but not less than 600 mm beyond each corner of the opening.

14.2 Structural design of bearing walls

14.2.1

Except as permitted by Clause 14.2.2, bearing walls shall be designed as specified in Clauses 7, 10, and 11.

14.2.2

14.2.2.1
Subject to the requirements specified in Clause 14.2.2.2, bearing walls may be designed using the following equation:

$$P_r = \frac{2}{3}\alpha_1 \phi_c f_c' A_g \left(1 - \left(\frac{kh_u}{32t}\right)^2\right) \quad \textbf{(14-1)}$$

14.2.2.2
Clause 14.2.2.1 shall apply only if the following requirements are met:
(a) the wall has a solid rectangular cross-section that is constant over the height of the wall;
(b) the principal moments act about a horizontal axis parallel to the plane of the wall;
(c) the resultant of all factored axial loads, including the effects of the principal moment, is located within the middle third of the overall wall thickness; and
(d) the wall is supported against lateral displacement along at least the top and bottom edges.

14.2.2.3
The effective length factor, k, in Equation (14-1) shall have the following values:

For walls restrained against rotation at one or both ends (top, bottom, or both)	0.8
For walls unrestrained against rotation at both ends	1.0

14.3 Structural design of non-bearing walls
Non-bearing walls, including retaining walls and transversely loaded walls, shall be designed in accordance with the provisions of Clauses 10 and 11.

14.4 Structural design of shear walls

14.4.1 General
In addition to the requirements specified elsewhere in Clause 14, the following shall apply to shear walls:
(a) flexural shear walls shall be designed for factored axial load, factored moment about one or both axes, and factored shear as specified in Clauses 10 and 11; and
(b) squat shear walls may be designed using strut-and-tie models in accordance with Clause 11.4 and the applicable provisions of Clause 14.1.

14.4.2 Assemblies of interconnected shear walls

14.4.2.1 Shear connection
In assemblies of interconnected shear walls designed to act as a unit, reinforcement shall be provided to transmit the shear stresses necessary for the assembly of interconnected walls to act as a unit.

14.4.2.2 Compression flanges of assemblies of interconnected shear walls

If a compression flange of an assembly of interconnected shear walls has a thickness less than $h_c/15$, or less than $w_c/15$ if adjacent shear wall webs are present, the factored axial and moment resistances of the wall assembly shall both be multiplied by

$$\omega = 1.0 - 0.025(\ell_c/t - 15) \quad \textbf{(14-2)}$$

where ℓ_c is the lesser of w_c and h_c and ω shall not be less than 0.75 or greater than 1.0.

14.4.2.3 Maximum widths of overhanging flanges

The effective widths of overhanging flanges of walls shall not be assumed to extend farther from the face of the web than the smaller of

(a) half the clear distance to an adjacent shear wall web; or

(b) 25% of the total wall height above the section under consideration.

14.4.3 Horizontal reinforcement in shear walls

Horizontal reinforcement shall extend to the ends of the wall and shall be anchored at each end within the boundary elements or within the regions of concentrated reinforcement required by Clause 14.1.8.8.

14.4.4 Weak axis bending

Weak axis bending of shear walls shall be considered in conjunction with strong axis bending.

14.4.5 Diaphragms

Floor and roof diaphragms shall be designed to transfer lateral forces between walls or other lateral-load-resisting elements and floor and roof diaphragms. Connections between the diaphragms and the frames or other lateral-load-resisting elements shall be designed to resist the forces that are transferred.

14.4.6 Coupling beams

The diameter of flexural reinforcing bars in coupling beams shall be selected to provide a development length not more than one-half of the clear beam span. Alternatively, diagonal reinforcement shall be provided as specified in Clause 21.6.8.7. However, ties satisfying the requirements specified in Clause 7.6 may be provided in lieu of hoops or spirals.

15 Foundations

15.1 General

Clauses 15.2 to 15.12 shall apply to the design of isolated footings and, where applicable, to combined footings, mats, and deep foundations.

15.2 Loads and reactions

15.2.1

Footings, piles, and pile caps shall be proportioned to resist the factored loads and induced reactions.

15.2.2

The base area of the footing or the number and arrangement of piles shall be selected based on the principles of soil mechanics. Where the analysis of footings is based on other than linear distributions of soil pressure, the assumed distributions shall be based on an analysis of the interaction of the soil and the footing in accordance with the stiffness of both elements.

15.2.3
Piles and pile caps in deep foundations shall be designed on the assumption that each axial pile reaction acts at an eccentricity, in any direction, equal to the specified pile location tolerance, but not less than 50 mm.

15.3 Footings and pile caps supporting circular or regular polygonal columns or pedestals
In lieu of detailed analysis, circular or regular polygonal concrete columns or pedestals may be treated as square members, with the same area, for the location of critical sections for moment, shear, and development of reinforcement in the footings or pile caps.

15.4 Flexural design of footings

15.4.1
Design for flexure shall meet the requirements of Clause 10.

Note: *For many types of footings, the strut-and-tie method specified in Clause 11.4 can be used for design.*

15.4.2
The external moment on any section of a footing shall be determined by passing a vertical plane through the footing and computing the moment of the forces acting over the entire area of the footing on one side of that vertical plane.

15.4.3
The maximum factored moment for an isolated footing shall be computed at the critical sections located as follows:

(a) for footings supporting a concrete column, pedestal, or wall: at the face of the column, pedestal, or wall;

(b) for footings supporting a masonry wall: halfway between the middle and the edge of the wall; and

(c) for footings supporting a column with steel base plates: as determined by considering the dimensions and the stiffness of the base plate.

Note: *In many cases, the critical section can be taken halfway between the face of the column and the edge of the base plate.*

15.4.4

15.4.4.1
In two-way rectangular footings, reinforcement shall be distributed as follows:

(a) reinforcement in the long direction shall be distributed uniformly across the entire width of the footing; and

(b) for reinforcement in the short direction, a portion of the total reinforcement specified in Clause 15.4.4.2 shall be distributed uniformly over a band width (centred on the centreline of the column or pedestal) equal to the length of the short side of the footing or equal to the length of the supported wall or column, whichever is greater. The remainder of the reinforcement required in the short direction shall be distributed uniformly outside the centre band width.

15.4.4.2
The portion of the total reinforcement in the short direction distributed over the band width (see Clause 15.4.4.1(b)) is as follows:

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)} \qquad \textbf{(15-1)}$$

15.5 Shear design of footings and pile caps

15.5.1
Design for shear shall meet the requirements of Clauses 11 and 13.

Note: *For many types of footings, the strut-and-tie method specified in Clause 11.4 can be used for design.*

15.5.2
The location of the critical section for shear, as specified in Clause 13.3, shall be measured from the face of the column, pedestal, or wall for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from the location specified in Clause 15.4.3(c).

15.5.3
Shear on any section through a pile cap shall be computed in accordance with the following:
(a) the 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) the 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; and
(c) for intermediate positions of the pile centre, the portion of the pile reaction to be considered as producing shear on the section shall be based on a straight-line interpolation between the full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

15.6 Development of reinforcement in footings and pile caps

15.6.1
The development of reinforcement in footings and pile caps shall be computed in accordance with Clause 12.

15.6.2
When the strut-and-tie method is used, the development of tension reinforcement shall be as specified in Clause 11.4.3.2.

15.6.3
The critical sections for development of reinforcement shall be assumed to be at the locations specified in Clause 15.4.3 for the maximum factored moment and at all other vertical planes where changes of section or reinforcement occur. (See also Clause 12.10.5.)

15.7 Minimum depth of footings
The depth of footings above the bottom reinforcement shall be not less than 150 mm.

15.8 Piles

15.8.1 Design of piles
The moments and shears in the piles caused by lateral loads shall be calculated using procedures that account for the pile-soil interaction and non-linear soil behaviour.

15.8.2 Special requirements for piles

15.8.2.1
The stability of portions of piles without lateral restraint from soil shall be assessed as specified in Clause 10.

15.8.2.2
The outer 25 mm concrete layer of uncased drilled piles shall be neglected when the factored resistance of the pile shaft and the end-bearing resistance is determined.

15.8.2.3
For uncased drilled piles, a reduction factor of 0.90 shall be applied to the factored resistance specified in Clauses 10 and 11.

15.8.2.4
Selection of the pile bell diameter and bell side slope shall be based on the concrete shear resistance and the type of the soil (see ACI 336.3R).

15.8.2.5
For the seismic design of piles, the additional requirements specified in Clause 21.11.4 shall be met.

15.8.2.6
Where required by applicable codes, piles shall be interconnected.

15.8.3 Minimum depth of pile caps
The depth of pile caps above the bottom reinforcement and above the top of the pile shall be not less than 300 mm.

15.9 Transfer of force at base of column, pile cap, wall, or pedestal

15.9.1 General

15.9.1.1
The forces and moments at the base of a column, pile cap, wall, or pedestal shall be transferred to the supporting footing or pile.

15.9.1.2
Bearing on concrete at the contact surface between the supported and supporting members shall not exceed the factored bearing resistance of either member specified in Clause 10.8.

15.9.1.3
Reinforcement, dowels, or mechanical connectors between supported and supporting members shall be adequate to transfer
(a) all compressive force that exceeds the concrete bearing strength of either member; and
(b) any computed tensile force across the interface.

In addition, reinforcement, dowels, or mechanical connectors shall meet the requirements of Clause 12 and of Clause 15.9.2.2 or 15.9.2.3.

15.9.1.4
Lateral forces shall be transferred to the supporting pedestals, caps, piles, and footings in accordance with the interface shear transfer requirements of Clause 11.5 or by other appropriate means.

15.9.2 Cast-in-place construction

15.9.2.1
For columns, pile caps, piles, and pedestals, the area of reinforcement across the interface shall be not less than 0.005 times the gross area of the supported member.

15.9.2.2
For cast-in-place walls, the area of reinforcement across the interface shall be not less than the minimum vertical reinforcement required by Clause 14.1.8.

15.9.2.3
The size of dowels shall not exceed the size of the vertical bars by more than one bar size.

15.9.2.4
At footings, 45M and 55M longitudinal bars (in compression only) may be lap spliced with dowels to provide the reinforcement required to satisfy Clause 15.9.1. Dowels shall not be larger than 35M and shall extend into the supported member for a distance of not less than the development length of 45M or 55M bars, or the splice length of the dowels, whichever is greater, and into the footing for a distance of not less than the development length of the dowels.

15.9.2.5
If a pinned or rocker connection is provided in cast-in-place construction, the connection shall also comply with Clause 15.9.3.

15.9.3 Precast concrete construction

15.9.3.1
In precast concrete construction, the reinforcement required to satisfy Clause 15.9.1 may be provided by anchor bolts or suitable mechanical connectors.

15.9.3.2
Anchor bolts and mechanical connectors shall be designed to reach their factored resistance prior to anchorage failure of the surrounding concrete.
Note: *See Annex D for more information.*

15.10 Sloped or stepped footings

15.10.1
In sloped or stepped footings, the angle of the slope or the depth and location of the steps shall be such that design requirements are satisfied at every section.

15.10.2
Sloped or stepped footings designed as a unit shall be constructed to ensure action as a unit.

15.11 Combined footings and mats

15.11.1
Footings supporting more than one column, pedestal, or wall (combined footings or mats) shall be proportioned, in accordance with the applicable design requirements of this Standard, to resist the factored loads and induced reactions.

15.11.2
The distribution of soil pressure under combined footings and mats shall be consistent with the properties of the soil and the structure and with the established principles of soil mechanics.

15.12 Plain concrete footings and deep foundations
Plain concrete footings and deep foundations shall comply with Clause 22.

16 Precast concrete

16.1 General

16.1.1
All requirements of this Standard not specifically excluded and not in conflict with the requirements of Clauses 16.1.2 to 16.5.3.7 shall apply to structures incorporating precast concrete elements.
Note: *See CSA A23.4 for the suggested division of design responsibilities between the designer and the precast concrete manufacturer.*

16.1.2
Clauses 7.7, 7.8, 10.4, and 13 shall not apply to precast concrete.

16.1.3
For elements produced in manufacturing plants certified in accordance with Clause 16.2, the concrete material resistance factor, ϕ_c, specified in Clause 8.4.2 may be taken as 0.70.

16.2 Prequalification of manufacturer

16.2.1
All precast concrete elements covered by this Standard shall be manufactured and erected in accordance with CSA A23.4.

16.2.2
Exemptions to the requirements specified in Clause 16.2.1 may be made by the designer of the building for the following reinforced concrete elements:
(a) tilt-up walls;
(b) minor structural elements such as stair flights, stair landings, lintels, and sills; and
(c) precast slabs for lift slab construction.

The designer shall clearly indicate whether such reinforced precast elements are to be manufactured in accordance with CSA A23.4, in which case certification shall be required, or in accordance with CSA A23.1, in which case the certification requirement may be waived by the designer.

16.3 Drawings

In addition to the requirements specified in Clause 5, drawings and related documents shall include the following:
(a) sufficient dimensions to permit preparation of the shop drawings;
(b) sufficient indication of the work supporting, supported by, or attached to the precast concrete to permit preparation of the shop drawings;
(c) the class of surface finish required for structural purposes;
(d) any non-standard tolerances required for the precast concrete elements or the building structure;
(e) any superimposed loads on the precast concrete elements, the location of connections, and the factored forces to be developed at the connections to the elements;
(f) when precast elements are to act as diaphragms, the factored external forces and shears acting on the diaphragms; and
(g) the expected deformations of the structure under specified loads, insofar as they affect the design of the precast concrete elements or associated connections. Deformations due to specified earthquake loads shall be shown separately.

16.4 Design

16.4.1 General

16.4.1.1

The design shall take into account loading and restraint conditions from the initial fabrication to the intended use of the structure, including forces from stripping, storage, transportation, and erection.

16.4.1.2

The effects of initial and long-term deformations shall be considered, including the effects on interconnected elements.

16.4.2 Distribution of forces among elements

16.4.2.1

The distribution of forces that are perpendicular to the plane of the elements shall be established by analysis or test.

16.4.2.2

In-plane forces shall be transferred between the elements of a precast floor or wall system in accordance with the following:
(a) load paths for in-plane forces shall be transferred through both connections and elements;
(b) where tension forces occur, a load path of reinforcement or tendons shall be provided; and
(c) the design of joints, connections, and bearings shall include the effects of all forces to be transmitted, including the effects of specified loads, tolerances, elastic deformation, temperature, creep, and shrinkage.

16.4.3 Reinforcement of precast concrete elements

16.4.3.1

The minimum reinforcement ratio in each direction shall be not less than 0.0016 for reinforcement or 0.0004 for prestressing tendons, except as permitted by Clauses 16.4.3.2 and 16.4.3.3. Additional reinforcement shall be provided at openings and other discontinuities.

16.4.3.2
For one-way floor and roof slabs and for one-way precast, prestressed wall panels, all not exceeding 3660 mm in width, and where elements are not connected to cause restraint in the transverse direction, the minimum transverse reinforcement requirements of Clause 16.4.3.1 may be waived.

16.4.3.3
For non-prestressed walls, the minimum reinforcement ratio shall be not less than 0.001 in each direction. Spacing of reinforcement shall not exceed the smaller of five times the wall thickness or 500 mm.

16.4.4 Joints and connections

16.4.4.1
Forces shall be transferred between elements by grouted joints, shear keys, mechanical connectors, reinforcement, topping, or a combination of these means.

16.4.4.2
Precast segments, when joined and post-tensioned in accordance with CSA A23.1, may be considered homogeneous structural members.

16.4.4.3
The design of each component of a connection shall be based on the most severe combination of load eccentricities, as limited by fabrication and erection tolerances.

16.4.4.4
Special attention shall be given to the design of connections when there is a possibility of corrosion, and in particular to connections in inaccessible locations in the finished structure.

16.4.4.5
Provision for movement of elements due to earthquake shall accommodate $R_d R_o / I_E$ times the elastic deflection of the lateral force resisting system.
Note: *See the* National Building Code of Canada *for more information.*

16.4.4.6
In the design of connections that accommodate movement by deformation of the connection material, consideration shall be given to the magnitude and frequency of the movement and to the fatigue properties and ductility of the connection.

16.4.4.7
In the design of connections that accommodate movement by sliding, the increase of friction due to the tightness of the fastening, the effects of corrosion, and construction tolerances shall be taken into account.
Note: *For connections whose capacity is sensitive to erection tolerances and for connections in inaccessible locations that can be subject to corrosive conditions, connection resistance should be increased.*

16.4.5 Bearing

16.4.5.1
The allowable bearing stress at the contact surface between supported and supporting elements and between any intermediate bearing elements shall not exceed the bearing resistance for either surface as specified in Clause 10.8 or 11.4.4.

16.4.5.2
Unless tests or analysis show that performance will not be impaired, the following minimum requirements shall be met:
(a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least 1/180 of the clear span, ℓ_n, but not less than the following:

For solid or hollow-core slabs	50 mm
For beams or stemmed members	75 mm

(b) Bearing pads at unarmoured edges shall be set back a minimum of 12 mm from the face of the support, or the chamfer dimension at chamfered edges, whichever is larger.

16.5 Structural integrity

16.5.1
In buildings where precast concrete elements constitute a portion of the structural system, all structural elements shall be effectively tied together.

16.5.2

16.5.2.1
Except as specified in Clause 16.5.3, precast concrete structures shall meet the structural integrity requirements specified in Clauses 16.5.2.2 to 16.5.2.6.
Note: *Guidance on designing for structural integrity of structural systems incorporating precast elements can be obtained from the following publications:*
(a) ACI-ASCE Joint Committee 550, "Design recommendations for precast concrete structures";
(b) Canadian Precast/Prestressed Concrete Institute, Design manual;
(c) Cement Association of Canada, Concrete design handbook; *and*
(d) Precast/Prestressed Concrete Institute, PCI design handbook.

16.5.2.2
Longitudinal and transverse tensile tie reinforcement shall be incorporated so as to provide a load path to the lateral load resisting system, as specified in Clause 16.4.2.2(b).

16.5.2.3
Where precast elements form floor or roof diaphragms, the connections between the diaphragm and those elements being laterally supported shall be designed for all factored loads but shall have a factored tensile resistance of not less than 5 kN/m.

16.5.2.4
Vertical tension tie requirements shall apply to the horizontal joints in all vertical structural elements, except cladding, and shall meet the following requirements:
(a) precast columns shall have a factored tensile resistance of not less than $1.4A_g$ N;
(b) for columns with a larger cross-section than required by analysis, a reduced effective area may be substituted for A_g, but it shall not be less than $A_g/2$; and
(c) precast wall panels shall have a minimum of two ties per panel, with a factored resistance of not less than 30 kN per tie.

16.5.2.5
When factored forces and moments result in compression at the base, the ties required by Clause 16.5.2.4(c) may be anchored to the floor slab on grade.

16.5.2.6
Ties and connections shall be designed in such a manner that the resistance is governed by yielding of the steel component.

16.5.3

16.5.3.1
Structures that are three or more storeys high and are constructed with precast concrete bearing walls shall be tied together as specified in Clauses 16.5.3.2 to 16.5.3.5.

16.5.3.2
Longitudinal and transverse tension ties shall be incorporated in floor and roof systems to provide a factored resistance of not less than 14 kN per metre of width or length. Tie paths shall be provided over interior wall supports and to exterior walls. Ties shall be located in the floor or roof system or within 600 mm of the plane of the floor or roof system.

16.5.3.3
Longitudinal tension ties parallel to the floor or roof spans shall be spaced not more than 3000 mm on centres. Provisions shall be made to transfer forces around openings.

16.5.3.4
Transverse tension ties perpendicular to the span of the floor or roof shall be spaced at a distance not greater than the distance between the bearing walls.

16.5.3.5
Tension ties around the perimeter of each floor and roof, within 1500 mm of the edge, shall provide a factored tensile resistance of not less than 60 kN.

16.5.3.6
Vertical tension ties shall be provided in all walls and shall be continuous over the full height of the building. They shall provide a factored tensile resistance of not less than 40 kN per metre of wall. Not fewer than two tension ties shall be provided for each precast wall panel.

16.5.3.7
During checking for structural integrity, any beneficial effects of friction caused by gravity loads shall not be considered for the transfer of horizontal loads.

17 Composite concrete flexural members
Note: *This Clause uses the terms "transverse shear " and "longitudinal shear". For a composite beam with a horizontal axis, "transverse shear" refers to vertical shear forces and "longitudinal shear" refers to shear on a horizontal plane.*

17.1 General

17.1.1
Clauses 17.1.2 to 17.5.4 shall apply to the design of composite concrete flexural members consisting of concrete elements constructed in separate placements, but interconnected in such a manner that all elements act as a unit.

17.1.2
All of the requirements of this Standard shall apply to composite flexural members, except where modified by Clauses 17.1.3 to 17.5.4.

17.1.3
An entire composite member or portions thereof may be assumed to resist shear and moment.

17.1.4
Individual elements shall be investigated for all critical stages of loading.

17.1.5
If the specified strength, density, or other properties of the elements differ, the properties of the individual elements shall be used for the analysis.

Note: *Differential creep and shrinkage can affect the distributions of strains and deformations in the individual elements.*

17.1.6
In strength computations for composite members, no distinction shall be made between shored and unshored members.

17.1.7
All elements shall be designed to support all loads introduced prior to full development of the design strength of composite members.

17.1.8
Reinforcement shall be provided, as necessary, to control cracking and prevent separation of individual elements of composite members.

17.1.9
Composite members shall meet the requirements for control of deflections specified in Clause 9.8.

17.2 Shoring
When used, shoring shall not be removed until the supported elements have developed the design properties required to support all loads and to limit deflections and cracking at the time of shoring removal.

17.3 Transverse shear resistance

17.3.1
When an entire composite member is assumed to resist transverse shear, the design shall meet the requirements specified in Clause 11 for a monolithically cast member of the same cross-sectional shape.

17.3.2
Shear reinforcement shall be fully anchored into interconnected elements as specified in Clause 12.13.

17.4 Longitudinal shear resistance

17.4.1
For a composite member, steps shall be taken to ensure full transfer of the longitudinal shear forces at the contact surfaces of the interconnected elements.

17.4.2
Longitudinal shear shall be investigated in accordance with Clause 17.4.3 or 17.4.4.

17.4.3

17.4.3.1
Unless calculated as specified in Clause 17.4.4, the design of cross-sections subject to longitudinal shear shall be based on

$$V_{r\ell} \geq V_f \qquad \textbf{(17-1)}$$

17.4.3.2
When contact surfaces are clean, free of laitance, and intentionally roughened, the factored longitudinal shear resistance, $V_{r\ell}$, shall not be taken as greater than $0.7\phi_c b_v d$ unless ties are provided to transfer longitudinal shear.

17.4.3.3
When minimum ties are provided as specified in Clause 17.5 and contact surfaces are clean and free of laitance but not intentionally roughened, the factored longitudinal shear resistance, $V_{r\ell}$, shall not be taken as greater than $0.7\phi_c b_v d$.

17.4.3.4
When the factored shear force, V_f, at the section being considered exceeds $0.7\phi_c b_v d$, the design for longitudinal shear shall be as specified in Clause 11.5.

17.4.4
Longitudinal shear may be investigated by computing the actual compressive or tensile force in any segment and provisions shall be made to transfer that force as longitudinal shear to the supporting element. The factored longitudinal shear force shall not exceed the factored longitudinal shear resistance, $V_{r\ell}$, as specified in Clauses 17.4.3.2 to 17.4.3.4, with the area of contact surface, A_{cv}, substituted for $b_v d$.

17.4.5
When tension exists across any contact surface between interconnected elements, shear transfer by contact may be assumed only when minimum ties are provided as specified in Clause 17.5.

17.5 Ties for longitudinal shear

17.5.1
When ties are provided to transfer longitudinal shear, the tie area shall be not less than that required by Clause 11.2.8, and the tie spacing shall not exceed four times the least dimension of the supported element or 600 mm, whichever is less.

17.5.2
Ties for longitudinal shear shall consist of a single bar or wire, multiple leg stirrups, vertical legs, or welded wire fabric (smooth or deformed).

17.5.3
Ties shall be anchored into the interconnected elements as specified in Clause 12.13.

17.5.4
Reinforcement for transverse shear that is anchored into the interconnected elements as specified in Clause 12.13 may be included as ties for longitudinal shear.

18 Prestressed concrete

18.1 General

18.1.1
Clauses 18.1.2 to 18.13.4 shall apply to members prestressed with wires, strands, or bars that comply with the requirements for prestressing steels specified in Clause 3.1.4 and in CSA A23.1.
Note: *Unbonded tendons are more susceptible to corrosion than bonded tendons. The durability of structures with unbonded prestressing tendons is a function of the environment, occupancy type, and quality of work during construction. Ingress of moisture or chlorides, sulphides, nitrates, carbonates, or other industrial, food processing, or agricultural chemicals can cause corrosion or even failure of the tendons. Water, including rainwater, can enter the sheath during tendon shipping, storage, or construction, and in some cases after occupancy of the structure, if adequate protection is not provided. Materials and quality of work should meet the requirements specified in CSA A23.1.*

18.1.2
All of the requirements of this Standard not specifically excluded and not in conflict with the requirements specified in Clauses 18.1.1 and 18.1.3 to 18.13.4 shall apply to prestressed concrete.

18.1.3
The requirements specified in Clauses 10.3.3, 10.3.4, 10.4, 10.5.1, 10.5.2, 10.6.2, 10.9, 13, 14.1.7.2, and 14.2 shall not apply to prestressed concrete unless otherwise specified.

18.1.4
Prestressed members shall meet the strength requirements specified in this Standard.

18.1.5
The effects of the loads at all loading stages that could be critical during the life of the member from the time the prestress is first applied shall be considered.

18.1.6
The stresses in prestressed members at transfer and under specified loads shall satisfy the requirements of Clause 18.3.

18.1.7
Stress concentrations due to prestressing shall be considered. Adequately anchored transverse reinforcement shall be provided to control splitting.

18.1.8
The deflection of prestressed concrete members shall be determined as specified in Clause 9.8.4.

18.1.9
When adjoining parts of the structure can restrain the elastic and long-term deformations (deflections, changes in length, and rotation) of a member caused by prestressing, applied loading, foundation settlement, temperature, and shrinkage, the restraint shall be estimated and its effects on the member and on the restraining structure shall be considered.

18.1.10
The possibility of buckling in a member between points where concrete and prestressing tendons are in contact and of buckling in thin webs and flanges shall be considered.

18.1.11
In computing section properties, the loss of area due to open ducts or conduits shall be considered.

18.2 Design assumptions for flexure and axial load

18.2.1
The design of prestressed members for flexure and axial loads shall be based on the assumptions specified in Clause 10.1.

18.2.2
For investigation of the stress limits specified in Clauses 18.3 and 18.4, linear elastic material behaviour may be assumed. Concrete may be assumed to resist tension at sections that are uncracked.

18.3 Permissible stresses in concrete flexural members

18.3.1

18.3.1.1
Stresses in concrete immediately after prestress transfer due to prestress, and the specified loads present at transfer, shall not exceed the following:

(a) Extreme fibre stress in compression	$0.6\,f'_{ci}$
(b) Extreme fibre stress in tension, except as permitted by Item (c)	$0.25\lambda\sqrt{f'_{ci}}$
(c) Extreme fibre stress in tension at ends of simply supported members	$0.5\lambda\sqrt{f'_{ci}}$

18.3.1.2
The stress specified in Clause 18.3.1.1(a) may be exceeded if tests or analyses demonstrate that performance will not be impaired.

18.3.1.3
Where computed tensile stresses exceed the values specified in Items (b) and (c) of Clause 18.3.1.1, bonded reinforcement with a minimum area of $A_s = N_c/(0.5f_y)$ shall be provided in the tensile zone to resist the total tensile force, N_c, in the concrete computed on the basis of an uncracked section.

18.3.2
Stresses in concrete under specified loads and prestress (after allowance for all prestress losses) shall not exceed the following:

(a) Extreme fibre stress in compression due to sustained loads	$0.45 f'_c$
(b) Extreme fibre stress in compression due to total load	$0.60 f'_c$
(c) Extreme fibre stress in tension in precompressed tensile zone, except as specified in Clause 18.3.3	$0.50\lambda\sqrt{f'_c}$
(d) Extreme fibre stress in tension in precompressed tensile zone exposed to a corrosive environment	$0.25\lambda\sqrt{f'_c}$

18.3.3

18.3.3.1
Partially prestressed members may exceed the requirements specified in Clause 18.3.2(c) provided that tests or analyses demonstrate adequate fatigue resistance as well as adequate deflection and crack control under specified loads.

18.3.3.2
Partially prestressed members not subjected to fatigue conditions and not exposed to a corrosive environment may be deemed to have adequate deflection and crack control if the requirements of Clauses 9.8.4 and 18.8 are met.

18.4 Permissible stresses in tendons
Tensile stress in tendons shall not exceed the following:

Stress due to tendon jacking force for post-tensioning tendons	$0.85f_{pu}$, but not greater than $0.94f_{py}$
Stress due to tendon jacking force for pretensioning tendons	$0.80f_{pu}$
Stress immediately after prestress transfer	$0.82f_{py}$, but not greater than $0.74f_{pu}$
Stress in post-tensioning tendons at anchorages and couplers immediately after tendon anchorage	$0.70f_{pu}$

However, the stress due to tendon jacking force for post-tensioning and pretensioning tendons shall not exceed the maximum value recommended by the manufacturer of the prestressing tendons or anchorages. If pretensioned tendons are subjected to a temperature drop prior to concreting, the stress at the reduced temperatures shall not exceed 0.80 f_{pu}.

Note: *The specified yield strength of prestressing tendons is based on the requirements specified in ASTM A 416/A 416M, ASTM A 421/A 421M, and ASTM A 722/A 722M, which specify the following minimum values for* f_{py}:

(a) low relaxation strand or wire: $0.90f_{pu}$;

(b) stress-relieved strand or wire: $0.85f_{pu}$;

(c) plain prestressing bars: $0.85f_{pu}$; *and*

(d) deformed prestressing bars: $0.80f_{pu}$.

18.5 Loss of prestress

To determine the effective prestress, f_{pe}, allowance for the following sources of loss of prestress shall be considered:

(a) anchorage seating loss;
(b) elastic shortening of concrete;
(c) friction loss due to intended and unintended curvature in post-tensioning tendons;
(d) creep of concrete;
(e) shrinkage of concrete; and
(f) relaxation of tendon stress.

18.6 Flexural resistance

18.6.1

Strain compatibility analyses shall be based on the stress-strain curves of the steels to be used.

18.6.2

In lieu of a more accurate determination of f_{pr} based on strain compatibility, the following approximate values of f_{pr} may be used:

(a) for members with bonded tendons, provided that c/d_p is not greater than 0.5 and f_{pe} is not less than $0.6f_{py}$:

$$f_{pr} = f_{pu}\left(1 - k_p \frac{c}{d_p}\right) \qquad \textbf{(18-1)}$$

where

$k_p = 2(1.04 - f_{py}/f_{pu})$

and c shall be determined assuming a stress of f_{pr} in the tendons;

Note: *Further information can be found in the Cement Association of Canada's* Concrete design handbook.

(b) for members with unbonded tendons:

$$f_{pr} = f_{pe} + \frac{8000}{\ell_o}\sum_n (d_p - c_y) \le f_{py} \qquad \textbf{(18-2)}$$

where

$\sum_n (d_p - c_y)$ = sum of the distance $d_p - c_y$ for each of the plastic hinges in the span under consideration

and c_y shall be determined by assuming a stress of f_{py} in the tendons.

18.6.3

Tension and compression reinforcement may be considered to contribute to the flexural resistance with forces of $\phi_s A_s f_y$ and $\phi_s A_s' f_y'$, provided that they are located at least $0.75c$ from the neutral axis. Other reinforcement may be included in resistance computations if a strain compatibility analysis is conducted to determine the stress in such reinforcement.

18.7 Minimum factored flexural resistance

At every section of a flexural member, except two-way slabs, the following shall apply:

$$M_r \ge 1.2\, M_{cr} \qquad \textbf{(18-3)}$$

where

$$M_{cr} = \frac{I}{y_t}(f_{ce} + f_r)$$

where

$f_r = 0.6\lambda\sqrt{f'_c}$

unless the factored flexural resistance at the section is at least one-third greater than M_f.

18.8 Minimum bonded reinforcement

18.8.1

The minimum requirements for bonded reinforcement in beams and slabs shall be as specified in Table 18.1.

18.8.2

The bonded reinforcement required by Table 18.1 shall be uniformly distributed within the precompressed tensile zone as close to the extreme tensile fibre as the cover will permit.

Table 18.1
Minimum area of bonded reinforcement
(See Clauses 18.8.1, 18.8.2, and 18.9.2.)

	Concrete stress (see Clause 18.3.2(c))			
	Tensile stress $\leq 0.5\lambda\sqrt{f'_c}$		Tensile stress $> 0.5\lambda\sqrt{f'_c}$	
	Type of tendon		Type of tendon	
Type of member	**Bonded**	**Unbonded**	**Bonded**	**Unbonded**
Beams	0	$0.004A$	$0.003A$	$0.005A$
One-way slabs	0	$0.003A$	$0.002A$	$0.004A$
Two-way slabs				
Negative moment regions	0	$0.0006h\ell_n$	$0.00045h\ell_n$	$0.00075h\ell_n$
Positive moment regions, concrete stress $> 0.2\lambda\sqrt{f'_c}$	0	$0.004A$	$0.003A$	$0.005A$
Positive moment regions, concrete tensile stress $\leq 0.2\lambda\sqrt{f'_c}$	0	0	—	—

18.8.3

For partially prestressed beams and one-way slabs, the distribution of the bonded tendons and reinforcement shall be such that the quantity z in Equation (10-6) does not exceed 20 kN/mm for interior exposure and 15 kN/mm for exterior exposure. In lieu of more detailed analysis, the steel stress, f_s, in Equation (10-6) may be calculated as the difference between the stress in the non-prestressed reinforcement due to the specified load moment, M_s, and the stress due to the decompression moment, M_{dc}, specified in the following equation:

$$M_{dc} = f_{ce}\frac{I}{y_t} \tag{18-4}$$

Only the bonded steel shall be considered for the calculation of *A*. A bonded post-tensioned cable or a bundle of pretensioned tendons may be considered as one bar of equal area or disregarded in the calculation of *z*.

18.9 Minimum length of bonded reinforcement

18.9.1

Where bonded reinforcement is provided for flexural resistance, the minimum length shall comply with Clause 12.

18.9.2

The minimum length of bonded reinforcement required by Table 18.1 shall be as specified in Clauses 18.9.3 and 18.9.4.

18.9.3

In positive moment areas, the minimum length of bonded reinforcement shall be one-half of the clear span length and shall be centred in the positive moment area.

18.9.4

In negative moment areas, bonded reinforcement shall extend, on each side of the support, one-sixth of the longer clear span beyond the face of the support.

18.10 Frames and continuous construction

Moments for computing the required strength 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 factored loads specified in Clause 8.3. Where a minimum area of bonded reinforcement is provided as specified in Clause 18.8, negative moments may be redistributed as specified in Clause 9.2.4.

18.11 Compression members — Combined flexure and axial loads

18.11.1 General

The design of prestressed concrete members subject to combined flexure and axial loads shall be based on Clauses 10.9 to 10.16. The effects of prestress, creep, shrinkage, and temperature change shall be included.

18.11.2 Limits for reinforcement of prestressed compression members

18.11.2.1

Members with average prestress, f_{cp}, less than 1.5 MPa shall have the minimum reinforcement specified in Clauses 7.6 and 10.9 for columns or Clause 14.1.8 for walls.

18.11.2.2

Except for walls, members with average prestress, f_{cp}, equal to or greater than 1.5 MPa shall have all of their prestressing tendons enclosed by spirals or lateral ties as follows:

(a) spirals shall comply with Clause 7.6.4; and

(b) ties shall comply with Clause 7.6.5, excluding Clauses 7.6.5.2(a) and 7.6.5.5.

18.12 Two-way slab systems

18.12.1 General
Factored moments and shears in prestressed slab systems reinforced for flexure in two directions shall be determined as specified in Clause 13.8 or by more detailed design procedures.

18.12.2 Stresses under specified loads

18.12.2.1
When Clause 13.8 is used, flexural stresses due to unfactored gravity loads in column strips shall be determined by taking 75% of interior negative moments, 100% of exterior negative moments, and 60% of positive moments unless a more detailed analysis is performed.

18.12.2.2
Concrete stresses due to prestressing may be assumed to be uniformly distributed across the slab unless a more detailed analysis is performed.

18.12.2.3
The minimum average compressive stress, f_{cp}, shall be 0.8 MPa.

18.12.3 Shear resistance

18.12.3.1
In the vicinity of concentrated loads or reactions, the maximum factored shear stress, v_f, calculated as specified in Clauses 13.3.5 and 13.3.6, shall not exceed v_r.

18.12.3.2
The factored shear stress resistance, v_r, in two-way slabs shall be not greater than the factored shear stress resistance provided by the concrete, v_c, computed as specified in Clause 13.3.4 or 18.12.3.3, unless shear reinforcement is provided as specified in Clause 13.3.7, 13.3.8, or 13.3.9.

18.12.3.3
At columns supporting two-way slabs of uniform thickness, the factored shear stress resistance provided by the concrete shall be determined by

$$v_c = \beta_p \lambda \phi_c \sqrt{f_c'} \sqrt{1 + \frac{\phi_p f_{cp}}{0.33 \lambda \phi_c \sqrt{f_c'}}} + \frac{V_p}{b_o d} \qquad \textbf{(18-5)}$$

where

β_p = the smaller of 0.33 or $(\alpha_s d / b_o + 0.15)$

α_s = 4 for interior columns, 3 for edge columns, and 2 for corner columns

b_o = the perimeter of the critical section specified in Clause 13.3.3

f_{cp} = the average value of f_{cp} for the two directions and shall not be taken greater than 3.5 MPa

V_p = the factored vertical component of all prestress forces crossing the critical section

f_c' shall not be taken greater than 35 MPa and the slab shall extend at least $4h_s$ from all faces of the column. Equation (13-5), (13-6), or (13-7) shall apply to edge and corner columns when the slab extends less than $4h_s$ from a column face.

18.12.4 Shear and moment transfer
The fraction of the unbalanced moment transferred by eccentricity of shear shall comply with Clause 13.3.5.3.

18.12.5 Minimum bonded non-prestressed reinforcement

18.12.5.1
The minimum requirements for bonded reinforcement in two-way slabs shall be as specified in Clauses 18.8, 18.9, and 18.12.5.2.

18.12.5.2
In negative moment areas at column supports, the bonded reinforcement, A_s, shall be distributed within a zone equal to the column width plus 1.5 times the slab thickness beyond each side of the column. At least four bars or wires shall be provided in each direction. The spacing of the bonded reinforcement shall not exceed 300 mm.

18.12.6 Spacing of tendons

18.12.6.1
The spacing of tendons or groups of tendons in one direction shall not exceed eight times the slab thickness or 1500 mm unless adequate additional bonded reinforcement is provided so that the slab has the strength to span between tendons.

18.12.6.2
Tendon spacing shall be given special consideration in slabs supporting concentrated loads.

18.12.6.3
In slabs without beams, a minimum of two tendons or bars shall be provided in each direction over each column. These tendons or bars shall satisfy the requirements specified in Clause 13.10.6.

18.13 Tendon anchorage zones

18.13.1
Post-tensioning anchorage zones shall be designed to resist the specified tensile strength of the tendons.

18.13.2
One of the following methods shall be used for the design of anchorage zones:
(a) equilibrium based on strut-and-tie models (see Clause 11.4);
(b) elastic stress analysis (finite element methods or equivalent); or
(c) methods based on tests.

18.13.3
End blocks shall be provided where necessary for support bearing or distribution of concentrated prestressing forces.

18.13.4
Regions of stress concentrations due to abrupt changes in section or other causes shall be adequately reinforced.

19 Shells and folded plates

19.1 General

19.1.1

Clauses 19.1.2 to 19.5.2 shall apply to thin shell and folded plate concrete structures, including ribs and edge members.

19.1.2

All of the requirements of this Standard not specifically excluded and not in conflict with the requirements of Clauses 19.1 and 19.2 to 19.5.2 shall apply to thin shell structures.

19.2 Analysis and design

19.2.1

Elastic behaviour shall be an acceptable basis for determining internal forces and displacements of thin shells. This behaviour may be established by computations based on an analysis of the uncracked concrete structure in which the material is assumed to be linearly elastic, homogeneous, and isotropic. Poisson's ratio of concrete may be assumed to be equal to zero.

Note: *See Clause 13.7 for further guidance on analysis and design.*

19.2.2

Equilibrium checks of internal resistances and external loads shall be conducted to ensure consistency of results.

19.2.3

Experimental or numerical analysis procedures shall be used only when it can be shown that they provide a safe basis for design.

19.2.4

Approximate methods of analysis not satisfying compatibility of strains either within the shell or between the shell and auxiliary members shall be used only when it can be shown that they provide a safe basis for design.

19.2.5

For prestressed shells, the analysis shall also consider behaviour under loads induced during prestressing, at cracking load, and at factored load. Where prestressing tendons are draped within a shell, the design shall take into account the force components on the shell resulting from the tendon profiles not lying in one plane.

19.2.6

The thickness, h, of a thin shell and its reinforcement shall be proportioned for the required strength and serviceability.

19.2.7

The shell designer shall investigate and preclude the possibility of general or local instability.

19.2.8
Auxiliary members shall be designed in accordance with the applicable requirements of this Standard. A portion of the shell equal to the flange width specified in Clause 10.3 may be assumed to act with the auxiliary member. In such portions of the shell, the reinforcement perpendicular to the auxiliary member shall be at least equal to that required for the flange of a T-beam by Clause 10.5.3.2.

19.3 Specified yield strength of reinforcement
For non-prestressed reinforcement, the yield strength used in calculations shall not exceed 400 MPa.

19.4 Shell reinforcement

19.4.1
Shell reinforcement shall be provided to resist tensile stresses from the internal membrane forces, to resist bending and twisting moments, to control shrinkage and temperature cracking, and as special reinforcement at shell boundaries, load attachments, and shell openings.

19.4.2
Membrane reinforcement shall be provided in two or more directions in all parts of the shell.

19.4.3
The area of shell reinforcement in two orthogonal directions at any section shall be not less than the minimum slab reinforcement required by Clause 7.8, except as specified in Clause 19.4.7.

19.4.4
The reinforcement necessary for resisting shell membrane forces shall be provided so that the factored resistance in any direction shall be at least equal to the component of the principal membrane forces in the same direction due to the factored loads.

19.4.5
The area of shell tension reinforcement shall be limited so that the reinforcement will yield before crushing of concrete in compression can take place.

19.4.6
In regions of high tension, membrane reinforcement shall, if practical, be placed in the general directions of the principal tensile membrane forces. Where this is not practical, membrane reinforcement may be placed in two or more directions.

Note: *If the direction of reinforcement varies more than 15° from the direction of principal tensile membrane force, it is possible that the amount of reinforcement will have to be increased to limit the width of possible cracks under specified loads.*

19.4.7
When the magnitude of the principal tensile membrane stress within the shell varies greatly over the area of the shell surface, reinforcement resisting the total tension may be concentrated in the regions of largest tensile stress if it can be shown that this provides a safe basis for design. However, the ratio of shell reinforcement in any portion of the tensile zone shall be not less than 0.0035 based on the overall thickness of the shell.

19.4.8
Reinforcement required to resist shell bending moments shall be proportioned with due regard for the simultaneous action of membrane axial forces at the same location. When shell reinforcement is required in only one face to resist bending moments, equal amounts shall be placed near both surfaces of the shell, even if calculations do not indicate reversal of bending moments.

19.4.9
When splitting of the shell near its mid-thickness can occur because of transverse tensile stresses, transverse reinforcement shall be provided to prevent the cracks from propagating.

19.4.10
Shell reinforcement in any direction shall not be spaced farther apart than 500 mm or five times the shell thickness. Where the principal membrane tensile stress on the gross concrete area due to factored loads exceeds $0.4\lambda\phi_c\sqrt{f_c'}$, reinforcement shall not be spaced farther apart than three times the shell thickness.

19.4.11
Shell reinforcement at the junction of the shell and supporting members or edge members shall be anchored in or extended through such members in accordance with Clause 12, except that the minimum development length shall be $1.2\ell_d$ but not less than 500 mm.

19.4.12
Splice development lengths of shell reinforcement shall meet the requirements of Clause 12, except that the minimum splice length of tension bars shall be 1.2 times the value specified in Clause 12 but not less than 500 mm. The number of splices in principal tensile reinforcement shall be kept to a practical minimum. Where splices are necessary, they shall be staggered at least ℓ_d, with not more than one-third of the reinforcement spliced at any section.

19.5 Construction

19.5.1
When removal of formwork is based on a specific modulus of elasticity of concrete because of stability or deflection considerations, the value of the modulus of elasticity, E_c, shall be determined from flexural tests of field-cured beam specimens. The number and dimensions of the test beam specimens, and the test procedures, shall be specified by the designer.
Note: *For guidance see CSA A23.2-3C.*

19.5.2
If a thin shell is constructed with deviations from the shape greater than the tolerances specified by the designer, an analysis of the effect of such deviations shall be conducted and all necessary remedial actions shall be taken to ensure the shell's safe behaviour.

20 Strength evaluation procedures

Note:

(1) *This Clause specifies requirements and procedures for evaluating the strength or safe load rating of structures or structural elements where*
- *(a) doubt exists about their adequacy because of apparent or suspected deficiencies or defects;*
- *(b) the strength or load-bearing capacity is unknown;*
- *(c) a change of function creates loading characteristics different from those provided for in the design of the structure; or*
- *(d) damage that has possibility reduced the strength or load-bearing capacity has occurred.*

(2) *If the structure under investigation does not meet the requirements specified in Clause 20.2.3, 20.3.1.9, or 20.3.2.1, a lower load rating for the structure based on the results of the load test or analysis may be assigned.*

20.1 General
When the safety of a structure or structural member is in doubt and a structural strength investigation is necessary, it shall be carried out by analysis, by means of load tests, or by a combination of these methods.

20.2 Analytical investigation

20.2.1
If the strength evaluation is performed by analytical means, a thorough field investigation of the dimensions and details of the members as actually built, of the properties of the materials, and of other pertinent conditions of the existing structure shall be conducted.

20.2.2
If drawings or other documents are used in the evaluation specified in Clause 20.2.1, their completeness and any modifications of the structure not reflected on the drawings shall be considered in the evaluation.

20.2.3
The analysis based on the investigation specified in Clause 20.2.1 shall satisfy the requirements of this Standard.

20.3 Load tests
Notes:
(1) *Although load tests should be conducted in a manner that will provide for safety of life and structure, safety measures should not interfere with the load test procedures or affect results.*
(2) *Load testing prestressed systems with unbonded tendons where corrosion is suspected is generally not an acceptable method for evaluating such tendons.*

20.3.1 General

20.3.1.1
If the strength evaluation is based on load tests, an engineer experienced in such evaluations shall control the tests.

20.3.1.2
A load test shall generally not be conducted until the portion of the structure subjected to load is at least 28 d old.
Note: *When the owner of the structure, the contractor, and all other involved parties mutually agree, the test may be conducted when the structure is less than 28 d old.*

20.3.1.3
The structure or portion of the structure to be load tested shall be loaded in such a manner as to test adequately the suspected weakness and to allow for the characteristics and pattern of the expected loads.

20.3.1.4
A load to simulate the effect of the portion of the dead loads not already present shall be applied 24 h before application of the test load and shall remain in place until all testing has been completed.

20.3.1.5
The superimposed test load shall be applied in not fewer than four approximately equal increments without shock to the structure and in a manner that avoids arching of the load materials.

20.3.1.6
When an entire structural system in doubt is load tested or an entire questionable portion of a system is load tested, the test load shall be 90% of the factored loads M_f, V_f, and P_f.

20.3.1.7
When only a portion of a structural system in doubt is tested and the results of the tests are taken as representative of the structural adequacy of untested portions of the system, the test load shall be equal to the factored loads M_f, V_f, and P_f.

20.3.1.8
The test load shall be left on the structure for 24 h.

20.3.1.9
If the portion of the structure tested fails or shows visible indications of impending failure, it shall be considered to have failed the test.

20.3.2 Load tests of flexural systems or members for moment resistance

20.3.2.1
Note: *The requirements of this Clause are in addition to the requirements specified in Clause 20.3.1.9.*
When flexural systems or members, including beams and slabs, are load tested for moment resistance, they shall have a deflection recovery, within 24 h of removal of the test load, as follows:

Non-prestressed members	
First test	60%
Retest	75%
Prestressed members	80%

20.3.2.2
Deflections of beams, cantilevers, and one-way slabs shall be measured relative to the ends of the span.

20.3.2.3
In the case of two-way slabs, the central slab deflection shall be measured relative to the deflection at the supporting columns or walls.

20.3.2.4
Immediately before application of the test load, the necessary initial readings shall be made as a datum for the measurements of deflections caused by the application of the test load.

20.3.2.5
After the test load has been in position for 24 h, deflection readings shall be taken.

20.3.2.6
Following the action specified in Clause 20.3.2.5, the test load shall be removed. Deflection readings shall be taken 24 h after removal of the test load.

20.3.2.7
Retests of non-prestressed construction shall not be conducted until 72 h after removal of the first test load.

21 Special provisions for seismic design

21.1 Scope
Clauses 21.2 to 21.12 specify requirements for the design and construction of reinforced concrete members of structures for which the design earthquake forces have been determined on the basis of energy dissipation in the non-linear range of response.

21.2 General

21.2.1 Capacity design
The structures identified in Clause 21.1 shall be the subject of capacity design. In the capacity design of structures, kinematically consistent mechanisms are chosen, and the energy-dissipating elements are designed and detailed as specified in Clauses 21.2 to 21.9. All other structural elements in the seismic force resisting system (SFRS) are then provided with sufficient reserve capacity to ensure that the chosen energy-dissipating mechanisms are maintained in the selected locations without the formation of any additional mechanisms throughout the deformations that can occur.

21.2.2 Seismic force resisting systems
Clauses 21.2 to 21.9 specify requirements covering the design and detailing of six SFRSs identified in the *National Building Code of Canada*. These are ductile moment-resisting frames, moderately ductile moment-resisting frames, ductile shear walls, ductile coupled shear walls, ductile partially coupled shear walls, and moderately ductile shear walls. Additional systems are considered in Clause 21.2.3.

Clauses 21.2 to 21.9 are intended for SFRSs that are substantially regular in strength and stiffness.

Combinations of SFRSs acting in the same direction shall be permitted, provided that each system continues over the full building height. When SFRSs are not continuous over the building height or change type over the building height, when elements from two or more SFRS types are combined to create a hybrid system, or when a significant irregularity exists, an inelastic analysis such as a static pushover analysis shall be performed to

(a) verify the compatibility of the systems;
(b) confirm the assumed energy-dissipating mechanism;
(c) show that the inelastic rotational demands are less than the inelastic rotational capacities; and
(d) account for redistribution of forces.

The inelastic analysis may be waived if the performance of the system has been previously verified by experimental evidence or analysis. Systems requiring inelastic analysis shall be treated as equivalents under Section 2.5 of the *National Building Code of Canada*.

Note: *A discussion of acceptable discontinuous and combined systems can be found in the "Explanatory Notes" to CSA A23.3 in the Cement Association of Canada's* Concrete design handbook.

21.2.3 Other structural systems
A reinforced concrete structural system other than one specified in Clause 21.2.2 shall be permitted if experimental evidence and analysis demonstrate that the proposed system will have strength and toughness equal to or exceeding the strength and toughness provided by a monolithic reinforced concrete structure that meets the requirements of Clauses 21.2 to Clauses 21.12. These systems shall be treated as equivalents under Section 2.5 of the *National Building Code of Canada*.

21.2.4 Applicable clauses
The requirements of Clauses 1 to 16 and 23 shall apply to the design and detailing of structural members unless modified by the requirements of Clause 21.

21.2.5 Analysis and proportioning of structural members

21.2.5.1

The interaction of all structural and non-structural elements that materially affect the linear and non-linear response of the structure to earthquake motions shall be considered in the analysis.

21.2.5.2

21.2.5.2.1

For the purpose of determining forces in and deflections of the structure, reduced section properties shall be used. The effective property to be used as a fraction of the gross section property shall be as specified in Table 21.1.

Table 21.1
Section properties for analysis
(See Clause 21.2.5.2.)

Element type	Effective property
Beam	$I_e = 0.4I_g$
Column	$I_e = \alpha_c I_g$
Coupling beam (Clause 21.6.8.6)	$A_{ve} = 0.15A_g$; $I_e = 0.4I_g$
Coupling beam (Clause 21.6.8.7)	$A_{ve} = 0.45A_g$; $I_e = 0.25I_g$
Slab frame element	$I_e = 0.2I_g$
Wall	$A_{xe} = \alpha_w A_g$; $I_e = \alpha_w I_g$

Note: *See Clause 21.2.5.2.2 for the values of α_c and α_w.*

21.2.5.2.2

The values of α_c and α_w specified in Table 21.1 shall be determined as follows:

(a) $$\alpha_c = 0.5 + 0.6\frac{P_s}{f'_c A_g} \le 1.0 \quad \textbf{(21-1)}$$

(b) $$\alpha_w = 0.6 + \frac{P_s}{f'_c A_g} \le 1.0 \quad \textbf{(21-2)}$$

P_s in Equation (21-2) shall be determined at the base of the wall. For multiple wall segments, an average value of P_s/A_g may be used.

21.2.5.3

Structural members below the base of the structure that are intended to transmit earthquake-induced forces to the foundation shall meet the requirements of Clause 21.

21.2.5.4

In the calculation of the slenderness effects for sway frames in accordance with Clause 10.16, Q shall be calculated with Δ_0 multiplied by R_d/I_E. The value of Q shall not exceed 1/3.

21.2.5.5

All structural and non-structural members assumed not to be part of the SFRS shall comply with Clause 21.12.

21.2.6 Concrete in members resisting earthquake-induced forces

21.2.6.1

Specified concrete compressive strengths used in the SFRS shall not exceed 80 MPa.

Note: *See Clauses 10.12 and 14.1.5 for transmission of column and wall loads through floor systems.*

21.2.6.2

The specified compressive strength of structural low-density concrete used in the SFRS shall not exceed 30 MPa unless experimental evidence demonstrates that structural members made with such concrete provide strength and toughness equal to or exceeding the strength and toughness of comparable members made with normal-density concrete of the same strength.

21.2.6.3

Where the term $\sqrt{f'_c}$ is used in calculations of capacity in Clause 21, its value shall be limited to 8 MPa.

21.2.7 Reinforcement in members resisting earthquake-induced forces

21.2.7.1

Reinforcement for SFRS designed with a force modification factor, R_d, greater than 2.5 shall be weldable grade in compliance with CAN/CSA-G30.18. Reinforcement for SFRS designed with a force modification factor, R_d, of 2.5 or less shall comply with CAN/CSA-G30.18 but need not be weldable grade.

The design, detailing, and ductility requirements for structures designed using a reinforcement grade greater than 400 shall account for the increased strains.

Note: *The procedures specified in Clause 21, with the exception of those specified in Clause 21.4.4.2, were developed for Grade 400 reinforcement. The additional strains required for higher-yield-strength steel will, in general, reduce ductility.*

21.2.7.2

Clause 12.2.5 (which permits a reduction of lap splice length when the area of reinforcing steel provided exceeds the area required) shall not apply to reinforcement in members resisting earthquake-induced forces.

21.2.8 Mechanical splices

21.2.8.1

Mechanical splices shall be classified as either Type 1 or Type 2, as follows:

(a) Type 1 mechanical splices shall be those that comply with Clause 12.14.3.4.

(b) Type 2 mechanical splices shall be those that develop the specified tensile strength of the spliced bar.

21.2.8.2

Type 1 mechanical splices shall not be used within a distance equal to twice the member depth from the column or beam face, or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements. Type 2 mechanical splices may be used in any location.

21.2.9 Welded splices

21.2.9.1
Welded splices in reinforcement resisting earthquake-induced forces shall comply with Clause 12.14.3.3 and shall not be used within a distance equal to twice the member depth from the column or beam face, or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

21.2.9.2
Welding of stirrups, ties, inserts, or similar elements to longitudinal reinforcement that is required by design shall not be permitted.

21.3 Ductile moment-resisting frame members subjected to predominant flexure ($R_d = 4.0$)

21.3.1 Application

21.3.1.1
The requirements of Clause 21.3 shall apply to ductile frame members that are part of the ductile moment-resisting frame and are proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions:
(a) the axial compressive force in the member due to factored load effects shall not exceed $A_g f_c' / 10$;
(b) the clear span of the member shall be not less than four times its effective depth;
(c) the width-to-depth ratio of the cross-section shall be not less than 0.3; and
(d) the width shall be not less than 250 mm and not more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three-quarters of the depth of the flexural member.

21.3.1.2
Ductile frame members not meeting the requirements of Clause 21.3.1.1 shall be designed as specified in Clause 21.12 and shall not be considered part of the SFRS.

21.3.2 Longitudinal reinforcement

21.3.2.1
At any section of a flexural member, the areas of top reinforcement and bottom reinforcement shall each be not less than $1.4 b_w d / f_y$, and the reinforcement ratio, ρ, shall not exceed 0.025. At least two effectively continuous bars shall be provided at both top and bottom.

21.3.2.2
The positive moment resistance at the face of a joint shall be not less than one-half of the negative moment resistance provided at that face of the joint. Neither the negative nor the positive moment resistance at any section along the member length shall be less than one-quarter of the maximum moment resistance provided at the face of either end joint.

21.3.2.3

Lap splices of flexural reinforcement shall be permitted only if hoop reinforcement is provided over the lap length. The maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed $d/4$ or 100 mm. Lap splices shall not be used
(a) within the joints;
(b) within a distance of $2d$ from the face of the joint; and
(c) within a distance d from any plastic hinge caused by inelastic lateral displacements.

21.3.3 Transverse reinforcement

21.3.3.1

Hoops shall be provided in the following regions of frame members:
(a) over a length equal to $2d$, measured from the face of the joint; and
(b) over regions where plastic hinges can occur and for a distance d on either side of these hinge regions.

21.3.3.2

The first hoop shall be located not more than 50 mm from the face of a supporting member. The maximum spacing of the hoops shall not exceed
(a) $d/4$;
(b) eight times the diameter of the smallest longitudinal bars;
(c) 24 times the diameter of the hoop bars; or
(d) 300 mm.

21.3.3.3

In regions where hoops are required, longitudinal bars on the perimeter shall have lateral support complying with Clauses 7.6.5.5 and 7.6.5.6.

21.3.3.4

Hoops in flexural members may be replaced by the following two pieces of reinforcement:
(a) a U-stirrup enclosing the longitudinal reinforcement with seismic hooks at the ends; and
(b) a crosstie to make a closed hoop.
If the longitudinal reinforcing bars secured by the crossties are confined by a slab only on one side of the flexural frame member, the 90° hooks of the crossties shall all be placed on that side.

21.3.3.5

Where hoops are not required, stirrups with seismic hooks at each end shall be spaced not more than $d/2$ throughout the length of the member.

21.3.4 Shear strength requirements

21.3.4.1 Design forces

The factored shear resistance of frame members shall be at least equal to the shear determined by assuming that moments equal to the probable moment resistance act at the faces of the joint so as to produce maximum shear in the member, and that the member is then loaded with the tributary transverse load along the span. The moments corresponding to probable strength shall be calculated using the properties of the member at the faces of the joint. The factored shear need not exceed that determined from factored load combinations, with load effects calculated using R_dR_o equal to 1.0.

21.3.4.2 Shear reinforcement

Shear reinforcement shall be designed to the requirements of Clause 11, with the following exceptions:

(a) the values of $\theta = 45°$ and $\beta = 0$ shall be used in the regions specified in Clause 21.3.3.1; and

(b) transverse reinforcement required to resist shear shall be hoops over the lengths of members, as specified in Clause 21.3.3.1.

21.4 Ductile moment-resisting frame members subjected to flexure and significant axial load (R_d = 4.0)

21.4.1 Application

21.4.1.1

The requirements of Clause 21.4.2.1 to 21.4.5.2 shall apply to ductile frame members that are part of the ductile moment-resisting frame, are subject to an axial compressive force due to factored load effects that exceeds $A_g f'_c / 10$, and satisfy the following conditions:

(a) the shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 300 mm; and

(b) the ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

21.4.1.2

Ductile frame members not meeting the requirements of Clause 21.4.1.1 shall be designed as specified in Clause 21.12 and shall not be considered part of the SFRS.

21.4.2 Minimum flexural resistance of columns

21.4.2.1

The flexural resistance of any column proportioned to resist a factored axial compressive force exceeding $A_g f'_c / 10$ shall meet the requirements of Clause 21.4.2.2. Columns not meeting the requirements of Clause 21.4.2.2 shall not be considered as contributing to the resistance of the SFRS and shall meet the requirements of Clause 21.12.

21.4.2.2

The flexural resistances of the columns and the beams shall satisfy

$$\Sigma M_{nc} \geq \Sigma M_{pb} \qquad \textbf{(21-3)}$$

where

ΣM_{nc} = the sum of moments, at the centre of the joint, corresponding to the nominal resistance of the columns framing into the joint. The nominal resistance of the columns shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, that results in the lowest flexural resistance

ΣM_{pb} = the sum of moments, at the centre of the joint, corresponding to the probable resistance of the beams and girders framing into that joint. In T-beam construction where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width specified in Clause 10.3 shall be assumed to contribute to flexural strength if the slab reinforcement is developed at the critical section for flexure

Flexural resistances shall be summed such that the column moments oppose the beam moments. Equation (21-3) shall be satisfied for beam moments acting in either direction.

21.4.2.3
Axial design loads in frame columns shall account for beams yielding at levels above the level being considered. The shears from the beams shall be those given by the method specified in Clause 21.3.4.1 and using nominal rather than probable strengths. Allowance may be made for the reduction in accumulated beam shears with increasing numbers of storeys.

21.4.3 Longitudinal reinforcement

21.4.3.1
The area of longitudinal reinforcement shall be not less than 0.01 or more than 0.06 times the gross area, A_g, of the section.

21.4.3.2
Lap splices shall be permitted only within the centre half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement complying with Clauses 21.4.4.2 and 21.4.4.3.

21.4.4 Transverse reinforcement

21.4.4.1
Shear resistance shall meet the requirements of Clause 21.4.5.

21.4.4.2
Transverse reinforcement, specified as follows, shall be provided unless a larger amount is required by Clause 21.4.4.3 or 21.4.5:

(a) the volumetric ratio of circular hoop reinforcement, ρ_s, shall be not less than that given by

$$\rho_s = 0.4k_p \frac{f_c'}{f_{yh}} \quad \textbf{(21-4)}$$

where
$k_p = P_f / P_o$
and f_{yh} shall not be taken as greater than 500 MPa. However, ρ_s shall not be less than that required by Equation (10-7);

(b) the total effective area in each of the principal directions of the cross-section within spacing s of rectangular hoop reinforcement shall be not less than the larger of the amounts required by the following equations:

$$A_{sh} = 0.2k_n k_p \frac{A_g}{A_{ch}} \frac{f_c'}{f_{yh}} sh_c \quad \textbf{(21-5)}$$

$$A_{sh} = 0.09 \frac{f_c'}{f_{yh}} sh_c \quad \textbf{(21-6)}$$

where
$k_n = n_\ell / (n_\ell - 2)$
$k_p = P_f / P_o$
and f_{yh} shall not be taken as greater than 500 MPa;

(c) transverse reinforcement may be provided by single or overlapping hoops. Crossties of the same bar size and spacing as the hoops may be used. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar; and

(d) if the thickness of the concrete outside the confining transverse reinforcement exceeds 100 mm, additional transverse reinforcement shall be provided within the cover at a spacing not exceeding 300 mm.

21.4.4.3
Transverse reinforcement shall be spaced at distances not exceeding the smallest of the following:
(a) one-quarter of the minimum member dimension;
(b) six times the diameter of the smallest longitudinal bar; or
(c) s_x, as follows:

$$s_x = 100 + \left(\frac{350 - h_x}{3}\right) \qquad \textbf{(21-7)}$$

21.4.4.4
On each face of a column, the distance h_x shall not exceed the greater of 200 mm or one-third of the core dimension in that direction, and shall not be more than 350 mm.

21.4.4.5
Transverse reinforcement in the amount specified in Clauses 21.4.4.1 to 21.4.4.3 shall be provided over a length, ℓ_o, from the face of each joint and on both sides of any section where flexural yielding can occur as a result of inelastic lateral displacement of the frame. The length, ℓ_o, shall be determined as follows:
(a) where $P_f \leq 0.5\phi_c f_c' A_g$, ℓ_o shall be not less than 1.5 times the largest member cross-section dimension or one-sixth of the clear span of the member; and
(b) where $P_f > 0.5\phi_c f_c' A_g$, ℓ_o shall be not less than twice the largest member cross-section dimension or one-sixth of the clear span of the member.

21.4.4.6
Columns that can develop plastic hinges because of their connection to rigid members such as foundations or discontinued walls or because of their position at the base of the structure shall be provided with transverse reinforcement as specified in Clauses 21.4.4.1 to 21.4.4.3 over their clear height. This transverse reinforcement shall continue into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column. If the column terminates on a footing or mat, this transverse reinforcement shall extend into the footing or mat as required by Clause 21.11.

21.4.4.7
Where transverse reinforcement, as specified in Clauses 21.4.4.2 to 21.4.4.4, is not provided throughout the length of the column, the remainder of the column length shall contain hoop reinforcement with centre-to-centre spacing not exceeding the smaller of six times the diameter of the longitudinal column bars or 150 mm.

21.4.5 Shear strength

21.4.5.1
A column shall have a factored shear resistance that exceeds the greater of
(a) shear forces due to the factored load effects; or
(b) the design shear force determined from consideration of the maximum forces that can be generated at the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths of the member associated with the range of factored axial loads on the member. The member shears need not exceed those determined from strengths based on the probable moment strength of the transverse members framing into the joint.

The factored shear resistance of the column need not be greater than the factored load effect calculated using $R_d R_o$ equal to 1.0.

21.4.5.2
Shear reinforcement shall be designed to the requirements of Clause 11, with the following exceptions:
(a) values of $\beta \leq 0.10$ and $\theta \geq 45°$ shall be used in the region specified in Clause 21.4.4.5; and
(b) the transverse reinforcement required to resist shear shall be hoops.

21.5 Joints of ductile moment-resisting frames (R_d = 4.0)

21.5.1 General

21.5.1.1
The requirements of Clauses 21.5.1.2 to 21.5.5.7 shall apply to joints of ductile frames serving as parts of the SFRS.

21.5.1.2
Factored forces in joints shall be determined by assuming that the tensile stress in the longitudinal beam reinforcement at the joint is $1.25f_y$, except that they need not exceed the forces determined from the factored load combinations with factored load effects calculated using R_dR_o equal to 1.0.

21.5.1.3
Longitudinal beam reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension as specified in Clause 21.5.5 and in compression as specified in Clauses 12.3 and 12.5.5.

21.5.2 Transverse reinforcement in joints

21.5.2.1
Transverse hoop reinforcement, as specified in Clause 21.4.4 and calculated using the larger of f_c' for the column or the joint, shall be provided within the joint unless the joint is confined by structural members as specified in Clause 21.5.2.2.

21.5.2.2
Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half of the amount required by Clause 21.5.2.1 shall be provided where members frame into all four sides of the joint and each member width is at least three-quarters of the column width. At these locations, the spacing, s_x, specified in Clause 21.4.4.3 may be taken as 150 mm.

21.5.2.3
The transverse reinforcement required by Clause 21.4.4 shall be provided through the joint to provide confinement for longitudinal beam reinforcement located outside the column core, if such confinement is not provided by a beam framing into the joint.

21.5.3 Longitudinal column reinforcement

21.5.3.1
Longitudinal column reinforcement in round column cores shall be uniformly distributed around the column core with a centre-to-centre spacing not exceeding the larger of
(a) 200 mm; or
(b) one-third of the column core diameter.

21.5.3.2
Longitudinal column reinforcement in rectangular column cores shall have the reinforcing in each face uniformly distributed along that face, with a centre-to-centre spacing corresponding to the tie spacing specified in Clause 21.4.4.4.

21.5.4 Shear resistance of joints

21.5.4.1
The factored shear resistance of the joint shall not exceed the following, where f_c' is the strength of the concrete in the joint:
(a) for confined joints: $2.2\lambda\phi_c\sqrt{f_c'}A_j$;
(b) for joints confined on three faces or on two opposite faces: $1.6\lambda\phi_c\sqrt{f_c'}A_j$; and
(c) for other joints: $1.3\lambda\phi_c\sqrt{f_c'}A_j$.

A member that frames into a face shall be considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint shall be considered confined if such confining members frame into all faces of the joint.

21.5.4.2
The shear force, V_{fb}, in the joint determined using the forces specified in Clause 21.5.1.2, and accounting for other forces on the joint, shall not exceed the factored resistance specified in Clause 21.5.4.1.

21.5.5 Development length for tension reinforcement in joints

21.5.5.1
Hooks, if used, shall be standard 90° hooks and shall be located within the confined column core.

21.5.5.2
For normal-density concrete, the development length, ℓ_{dh}, for a bar with a standard 90° hook shall not be less than the greatest of
(a) $8d_b$;
(b) 150 mm; or
(c) for bar sizes of 35M and smaller, the length given by

$$\ell_{dh} = 0.2\frac{f_y}{\sqrt{f_c'}}d_b \tag{21-8}$$

21.5.5.3
For structural low-density concrete, the development length for a bar with a standard hook shall be not less than 1.25 times that required by Clause 21.5.5.2.

21.5.5.4
For bar sizes of 35M and smaller, the development length, ℓ_d, for a straight bar in a joint shall be not less than
(a) 2.5 times the length required by Clause 21.5.5.2 or 21.5.5.3, if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm; and
(b) 3.5 times the length required by Clause 21.5.5.2 or 21.5.5.3, if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.

21.5.5.5
Straight bars terminated at a joint shall pass through the confined core of a column. Any portion of the straight embedment length not within the confined core shall be considered 60% effective.

21.5.5.6
The diameter of straight beam and column bars passing through the joint shall satisfy the following equation:

$$d_b \leq \lambda \frac{\ell_j}{24k_2} \qquad \textbf{(21-9)}$$

21.5.5.7
If epoxy-coated reinforcement is used, the development lengths specified in Clauses 21.5.5.2 to 21.5.5.5 shall be multiplied by the applicable factor specified in Clause 12.2.4 or 12.5.3.

21.6 Ductile walls (R_d = 3.5 or 4.0)

21.6.1 Application

21.6.1.1
The requirements specified in Clauses 21.6.1.2 to 21.6.9.7 shall apply to ductile shear walls and ductile coupled or partially coupled shear walls serving as parts of the SFRS. Walls with h_w/ℓ_w of 2.0 or less shall be designed for R_d = 2.0 in accordance with Clause 21.7.4.

21.6.1.2
A ductile wall with openings shall be designed as
(a) a ductile shear wall with a single plastic hinge in accordance with Clauses 21.6.2 to 21.6.7 and Clause 21.6.9; or
(b) a ductile coupled or partially coupled shear wall in accordance with Clauses 21.6.2 to 21.6.9.

To be classified as a ductile shear wall, a wall with openings shall be proportioned in such a manner that at every vertical section the strength and stiffness of the elements connecting the wall segments shall be sufficient for the wall to resist the total overturning and gravity forces through a single compression stress block at the wall end.

Note: *The intent of this Clause is to restrict ductile shear walls to walls where plane sections remain essentially plane.*

21.6.2 General requirements

21.6.2.1
Each wall shall be detailed for plastic hinges to occur at all locations over its height, except as specified in Clauses 21.6.2.2 and 21.6.2.3.

21.6.2.2
The following shall apply to buildings where the SFRS does not contain structural irregularity types 1, 3, 4, 5, or 6 as defined in Article 4.1.8.6 of the *National Building Code of Canada* over the building height:
(a) the walls shall be detailed for plastic hinges over a height equal to at least 1.5 times the length of the longest wall above the design critical section. In the case of walls designed to the requirements of Clause 21.6.8, the height to be taken shall be at least 1.5 times the length of the longest individual element in the direction under consideration;
(b) the flexural and shear reinforcement required for the critical section shall be maintained over the height specified in Item (a);
(c) for all elevations above the plastic hinge region, the design overturning moments and shears shall be increased by the ratio of the factored moment resistance to the factored moment, both calculated at the top of the plastic hinge region; and
(d) detailing for plastic hinging shall extend below the critical section to the footing unless there is a significant increase in strength and stiffness below the critical section, in which case the detailing shall extend down the distance specified in Item (a) or to the footing, whichever is less.

21.6.2.3
For buildings containing structural irregularity types 1 or 3 over their height, the detailing specified in Clauses 21.6.2.2(a) and 21.6.2.2(b) shall be applied at each irregularity and shall continue for the distance specified in Clause 21.6.2.2(a) above and below each irregularity.

21.6.3 Dimensional limitations

21.6.3.1
The effective flange widths to be used in the design of L-, C-, or T-shaped sections shall not be assumed to extend farther from the face of the web than
(a) one-half of the distance to an adjacent structural wall web; or
(b) 25% of the wall height above the section under consideration.

21.6.3.2
The wall thickness within a plastic hinge shall be not less than $\ell_u/10$, except as permitted by Clauses 21.6.3.3 to 21.6.3.5, but shall not be less than $\ell_u/14$.

21.6.3.3
Clause 21.6.3.2 shall be required to apply only to those parts of a wall that under factored vertical and lateral loads are more than halfway from the neutral axis to the compression face of the wall section.

21.6.3.4
Clause 21.6.3.2 shall not be required to apply to simple rectangular walls where the distance from the neutral axis to the compression face, calculated for factored load effects, is located within a distance of the lesser of $4b_w$ or $0.3\ell_w$ from the compression face of the wall section.

21.6.3.5
Clause 21.6.3.2 shall not be required to apply to any part of a wall that lies within a distance of $3b_w$ from a continuous line of lateral support provided by a flange or cross wall. The width of the flange providing effective lateral support shall be not less than $\ell_u/5$.

21.6.4 Reinforcement

21.6.4.1
Unless otherwise specified, all reinforcement in walls shall be anchored, spliced, or embedded in accordance with the requirements for reinforcement in tension specified in Clause 12 and modified by Clause 21.6.4.2. All lap splices shall have a minimum length of $1.5\ell_d$.

21.6.4.2
Where Type 2 mechanical splices are used, not more than alternate bars in each layer of distributed and concentrated longitudinal reinforcement shall be spliced at any section, and the centre-to-centre distance between splices of adjacent bars shall be not less than $40d_b$, measured along the longitudinal axis of the wall.

21.6.4.3
The reinforcement ratio within any region of concentrated reinforcement, including regions containing lap splices, shall be not more than 0.06.

21.6.4.4
The diameter of the bars used in a wall shall not exceed one-tenth of the thickness of the wall at the bar location.

21.6.5 Distributed reinforcement

21.6.5.1
Both vertical and horizontal distributed reinforcement shall be provided in such a manner that the reinforcement ratio for this distributed reinforcement is not less than 0.0025 in each direction. The reinforcement spacing in each direction shall not exceed 450 mm. Splices shall comply with Clause 21.6.4.1 or 21.6.4.2. Vertical distributed reinforcement shall be tied as specified in Clause 7.6.5. Ties may be omitted if
(a) the area of vertical steel is less than $0.005A_g$; and
(b) the maximum bar size is 20M or smaller.

21.6.5.2
In regions of plastic hinging, the spacing of distributed reinforcement in each direction shall not exceed 300 mm, and if the area of vertical distributed reinforcement is greater than $0.005A_g$ or the maximum bar size is greater than 15M, the vertical distributed reinforcement shall be tied as specified in Clause 21.6.6.9.

21.6.5.3
At least two curtains of reinforcement shall be used if, in regions of plastic hinging, the in-plane factored shear force assigned to the wall exceeds $0.18\lambda\phi_c\sqrt{f'_c}A_{cv}$.

21.6.5.4
Horizontal reinforcement shall extend to the ends of the wall and shall be contained at each end of the wall within a region of concentrated reinforcement as specified in Clause 21.6.6.

21.6.5.5
In regions of plastic hinging, horizontal reinforcement shall be anchored within a region of concentrated reinforcement to develop $1.25f_y$.

21.6.6 Concentrated vertical reinforcement

21.6.6.1
Concentrated vertical reinforcement shall be provided at each end of the wall. Each concentration shall be a minimum of four bars placed in at least two layers.

21.6.6.2
The concentrated reinforcement shall be proportioned to resist that portion of factored load effects, including earthquake, not resisted by distributed vertical reinforcement.

21.6.6.3
The minimum concentrated reinforcement shall be not less than $0.001b_w\ell_w$ at each end of the wall.

21.6.6.4
The minimum area of concentrated reinforcement in regions of plastic hinging shall be at least $0.0015b_w\ell_w$ at each end of the wall.

21.6.6.5
In the case of flanged walls, concentrated reinforcement at the end(s) of the effective flanges may supply up to one-half of the required minimum wall web concentrated reinforcement, with the remainder placed at the end of the wall web.

21.6.6.6
The concentrated reinforcement shall consist of straight bars without cranks.

21.6.6.7
In regions of plastic hinging, not more than 50% of the reinforcement at each end of the walls shall be spliced at the same location. In such walls, a total of at least one-half of the height of each storey shall be completely clear of lap splices in the concentrated reinforcement.

21.6.6.8
The concentrated reinforcement shall be at least tied as a column as specified in Clause 7.6, and the ties shall be detailed as hoops. In regions of plastic hinging, the concentrated reinforcement shall be tied with buckling prevention ties as specified in Clause 21.6.6.9.

21.6.6.9
Buckling prevention ties shall comply with Clause 7.6.5.5 or 7.6.5.6 and be detailed as hoops. The tie spacing shall not exceed the smallest of
(a) six longitudinal bar diameters;
(b) 24 tie diameters;
(c) one-half of the least dimension of the member; or
(d) the tie spacing required by Clause 21.6.7, if applicable.

21.6.7 Ductility of ductile shear walls
Note: *This Clause applies to individual walls that are effectively continuous in cross-section from the base of the structure to the top of the wall and are designed to have a single plastic hinge at the base.*

21.6.7.1
To ensure ductility in the hinge region, the inelastic rotational capacity of the wall, θ_{ic}, shall be greater than the inelastic rotational demand, θ_{id}.

21.6.7.2
The inelastic rotational demand on a wall, θ_{id}, may be taken as

$$\theta_{id} = \frac{\left(\Delta_f R_o R_d - \Delta_f \gamma_w\right)}{\left(h_w - \frac{\ell_w}{2}\right)} \geq 0.004 \qquad \textbf{(21-10)}$$

where
$\Delta_f R_o R_d$ = the design displacement
$\Delta_f \gamma_w$ = the elastic portion of the displacement
ℓ_w = the length of the longest wall in the direction being considered

The value of 0.004 is a minimum rotational demand.

21.6.7.3
The inelastic rotational capacity of a wall, θ_{ic}, may be taken as

$$\theta_{ic} = \left(\frac{\varepsilon_{cu}\ell_w}{2c} - 0.002\right) \leq 0.025 \qquad \textbf{(21-11)}$$

where
ℓ_w = the length of the individual wall

ε_{cu} shall be taken as 0.0035 unless the compression region of the wall is confined as a column in accordance with Clause 21.6.7.4. The value of 0.025 is the upper limit on inelastic rotation capacity governed by tension steel strain. The distance to the neutral axis, c, shall be determined by plane section analysis or as follows:

$$c = \frac{P_s + P_n + P_{ns} - \alpha_1 \phi_c f'_c A_f}{\alpha_1 \beta_1 \phi_c f'_c b_w} \qquad \textbf{(21-12)}$$

21.6.7.4

When ε_{cu} is taken greater than 0.0035 but less than or equal to 0.014, the compression region of the wall shall be confined as a column. The transverse reinforcement shall be determined from Equation (21-5), with k_p taken as $(0.1 + 30\varepsilon_{cu})$. This reinforcement shall be provided over a distance of not less than $c(\varepsilon_{cu} - 0.0035)/\varepsilon_{cu}$ from the compression face of the wall. The minimum vertical reinforcement ratio in any part of this confined region shall be 0.005.

21.6.8 Additional requirements for ductile coupled and partially coupled shear walls

Note: *This Clause applies to an assembly of wall segments that are joined together by ductile coupling beams designed to dissipate energy.*

21.6.8.1

To ensure ductility of coupled systems, the inelastic rotational capacity of both the walls and the coupling beams shall be greater than their respective inelastic rotational demands.

21.6.8.2

The inelastic rotational demand on ductile coupled and partially coupled walls shall be taken as

$$\theta_{id} = \frac{\Delta_f R_o R_d}{h_w} \geq 0.004 \qquad \textbf{(21-13)}$$

where

$\Delta_f R_o R_d =$ the design displacement

21.6.8.3

The inelastic rotational capacity of the wall segment shall be calculated using the methods specified in Clause 21.6.7.3, except that ℓ_w shall be taken as the lengths of the individual wall segments for partially coupled walls and as the length of the system for coupled walls.

21.6.8.4

The inelastic rotational demand on coupling beams shall be taken as

$$\theta_{id} = \left(\frac{\Delta_f R_o R_d}{h_w}\right)\frac{\ell_{cg}}{\ell_u} \qquad \textbf{(21-14)}$$

The inelastic rotational capacity of coupling beams, θ_{ic}, shall be taken as

(a) 0.04 for coupling beams designed with diagonal reinforcement in accordance with Clause 21.6.8.7; and

(b) 0.02 for coupling beams designed in accordance with Clause 21.6.8.6.

21.6.8.5
Ductile coupled and partially coupled shear walls shall have ductile coupling beams. Ductile coupling beams shall have a depth not greater than twice the clear span of the beam. Diagonally reinforced coupling beams meeting the requirements of Clause 21.6.8.7 shall be provided unless the shear stress resulting from factored load effects is less than $0.1(\ell_u / d)\sqrt{f'_c}$, in which case conventionally reinforced coupling beams meeting the requirements of Clause 21.6.8.6 or diagonally reinforced coupling beams meeting the requirements of Clause 21.6.8.7 shall be provided.

21.6.8.6
Coupling beams without diagonal reinforcement shall meet the requirements of Clause 21.3 and the following requirements:
(a) the clear span, ℓ_u, shall be not less than $2\ell_d$;
(b) the anchorage of the flexural reinforcement into the wall shall meet the requirements of Clause 21.5 or 21.6.8.7;
(c) beams wider than the wall shall have
    (i) the front interface between the beam and the wall designed for the beam forces within the wall width; and
    (ii) the side interface(s) between the beam and the wall designed to transfer all of the forces in the beam overhang(s) to the wall; and
(d) beams not centred on the wall shall have
    (i) the beams and the walls designed for the eccentricities; and
    (ii) the beam stiffness reduced to account for the out-of-plane deformations.

21.6.8.7
Coupling beams with diagonal reinforcement shall resist the entire in-plane factored shear and flexure by diagonal reinforcement in both directions. The width of the coupling beam shall be less than or equal to the width of the wall. Where beams are not centred on the width of the wall, the resultant eccentricities shall be considered in the design of the beams and the walls. The diagonal reinforcement in each direction shall be interlocked and enclosed by hoops or spirals whose spacing shall not exceed the smallest of
(a) six diagonal bar diameters;
(b) 24 tie diameters; or
(c) 100 mm.

The centroid of each diagonal group shall be centred in the beam. Reinforcement shall be anchored into the wall with a minimum embedment of $1.5\ell_d$, where the 1.5 factor includes top bar effects but not bundled bar effects. Bars terminating with a standard hook contained within confinement ties shall have a minimum straight embedment into the wall of $1.0\ell_d$.

Note: *When the dimensions of a coupling beam are such that V_f/bh exceeds $1.0\sqrt{f'_c}$, it can be difficult to construct the beam on account of reinforcement congestion, particularly at the intersection of the coupling beam reinforcement with the concentrated wall reinforcement.*

21.6.8.8
Except as permitted by Clause 21.6.8.9, walls at each end of a coupling beam shall be designed so that the factored moment resistance of the wall about its centroid, calculated using axial loads P_s and P_n, exceeds the moment at its centroid resulting from the nominal resistance of the coupling beams framing into the wall and the factored moment in the wall.

21.6.8.9
If the wall at one end of the coupling beam has a factored resistance less than the nominal coupling beam resistance, the following requirements shall apply:
(a) the coupling beam shall meet the shear stress requirement specified in Clause 21.6.8.5 and the requirements specified in Clause 21.6.8.6;
(b) the wall shall be designed to the requirements specified in Clauses 21.4.4.1 to 21.4.4.3, 21.4.4.6, and 21.4.5; and

(c) the joint between the wall and the coupling beam shall meet the requirements specified in Clause 21.5.

21.6.8.10
Concentrated vertical reinforcement, as specified in Clause 21.6.6, shall be provided in the walls at both ends of coupling beams over the full height of the building and shall be tied as specified in Clause 21.6.6.9.

21.6.8.11
Coupled and partially coupled shear walls shall be designed with that portion of the overturning moment resisted by axial forces in the walls increased at each level by the ratio of the sum of the nominal capacities of coupling beams to the sum of the factored forces in the coupling beams above the level under consideration.

21.6.8.12
Assemblies of coupled and partially coupled shear walls connected together by coupling beams that function as a closed tube or tubes shall be designed with

(a) that portion of the overturning moment due to lateral loads resisted by axial forces in the walls, increased at each level by the ratio of the sum of the nominal capacities of coupling beams to the sum of the factored forces in the coupling beams required to resist lateral loads above the level under consideration; and

(b) an additional increase in overturning moment resisted by axial forces in the walls at each level corresponding to the increase in the sum of the nominal capacities of the coupling beams required to resist the accidental torsion above the level under consideration.

21.6.8.13
In lieu of a more detailed assessment, wall segments that act as tension flanges in the flexural mode shall be assumed to have no shear resistance over the height of the plastic hinge. For wall assemblies carrying torsion as a tube, the shear forces in the tension flange shall be redistributed.

21.6.9 Shear strength of ductile walls

21.6.9.1
Walls shall have a factored shear resistance greater than the shear due to the effects of factored loads. The shear due to the effects of factored loads shall account for the magnification of the shear due to the inelastic effects of higher modes. In addition, the factored shear resistance shall not be less than the smaller of

(a) the shear corresponding to the development of the probable moment capacity of the wall system at its plastic hinge locations; or

(b) the shear resulting from design load combinations that include earthquake, with load effects calculated using $R_d R_o$ equal to 1.0.

21.6.9.2
The shear design of ductile walls shall meet the requirements specified in Clauses 11 and 21.6.9.3 to 21.6.9.7.

21.6.9.3
The effective shear depth, d_v, of a wall shall be as specified in Clause 2.3, but need not be taken as less than $0.8\ell_w$.

21.6.9.4
All construction joints in walls shall be clean and free of laitance and shall meet the requirements specified in Clause 11.5.

21.6.9.5
The effect of openings in walls shall be accounted for using the procedures specified in Clause 11.4.

21.6.9.6
For regions of plastic hinging, the following additional requirements shall apply:

(a) The factored shear demand on the wall shall not exceed $0.10\phi_c f_c' b_w d_v$ unless it is shown that the inelastic rotational demand on the wall, θ_{id}, given by Equation (21-10) or (21-13) is less than 0.015. When $\theta_{id} = 0.005$, the factored shear demand shall not exceed $0.15\phi_c f_c' b_w d_v$. For inelastic rotational demands between these limits, linear interpolation may be used.

(b) The value of β specified in Clause 11.3.4 shall be taken as zero unless it is shown that the inelastic rotational demand on the wall, θ_{id}, given by Equation (21-10) or (21-13) is less than 0.015. When $\theta_{id} \leq 0.005$, the value of β shall not be taken greater than 0.18. For inelastic rotational demands between these limits, linear interpolation may be used.

(c) The value of θ in Clause 11.3.5 shall be taken as 45° unless the axial compression ($P_s + P_p$) acting on the wall is greater than $0.1f_c' A_g$. When $(P_s + P_p) \geq 0.2f_c' A_g$, the value of θ shall not be taken less than 35°. For axial compressions between these limits, linear interpolation may be used.

21.6.9.7
When the strut-and-tie model of Clause 11.4 is used for shear design, the following exceptions shall apply in regions of plastic hinging:

(a) The limiting compressive stress in the strut shall be taken as 0.8 times the value determined from Equation (11-22).

(b) A compression strut that, during the reverse direction of seismic loading, is a tension tie designed to yield, shall contain a minimum of four bars placed in at least two layers. This reinforcement shall be tied as a column in accordance with Clause 7.6, and the ties shall be detailed as hoops. In addition, the spacing of the ties shall not exceed the smallest of six longitudinal bar diameters, 24 tie diameters, or 100 mm.

21.7 Building members designed for moderate ductility (R_d = 2.0 or 2.5)

21.7.1 Application

21.7.1.1
The requirements specified in Clauses 21.7.1.2 to 21.7.4.8 shall apply to SFRSs designed using a force modification factor, R_d, of 2.0 for walls and 2.5 for frames.

21.7.1.2
Tilt-up wall panels shall be designed to the requirements of Clause 23, except that the requirements of Clause 21.7.2 shall apply to wall panels with openings when the maximum inelastic rotational demand on any part of the panel exceeds 0.02 radians. However, the inelastic rotational demand shall not exceed 0.04 radians. The requirements specified in Clause 21.7.4 shall apply to solid wall panels when the maximum in-plane shear stress exceeds $0.1\phi_c\sqrt{f_c'}$.

Note: *Methods for calculating rotational demand on elements of tilt-up panels with openings can be found in the "Explanatory Notes" to CSA A23.3 in the Cement Association of Canada's* Concrete design handbook. *The seismic performance of tilt-up buildings depends not only on the performance of the concrete wall panels but also on the performance of the roof structure and the connection between the wall panels and the roof. This Standard covers only the design of the concrete wall panels.*

21.7.2 Moderately ductile moment-resisting frames

21.7.2.1 Detailing of beams

21.7.2.1.1
The positive moment resistance at the face of the joint shall be not less than one-third of the negative moment resistance provided at that face of the joint. Neither the negative nor the positive moment resistance at any section along the length of the member shall be less than one-fifth of the maximum moment resistance provided at the face of either joint.

21.7.2.1.2
At both ends of the member, 10M or larger stirrups detailed as hoops shall be provided over lengths equal to twice the member depth measured from the face of the supporting member toward midspan. The first stirrup shall be located not more than 50 mm from the face of the supporting member and the spacing shall not exceed the smallest of
(a) $d/4$;
(b) eight times the diameter of the smallest longitudinal bar enclosed;
(c) 24 times the diameter of the stirrup bar; or
(d) 300 mm.

21.7.2.1.3
Stirrups shall be spaced not more than $d/2$ throughout the length of the member.

21.7.2.2 Detailing of columns

21.7.2.2.1
Transverse reinforcement shall be detailed as hoops and crossties or spirals.

21.7.2.2.2
The sum of the factored flexural resistances of the column sections framing into a joint, accounting for axial loads, shall exceed the sum of the nominal flexural resistances of the beams framing into the same joint. In T-beam construction where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width as specified in Clause 10.3 shall be assumed to contribute to flexural strength if the slab reinforcement is developed at the critical section for flexure.

Flexural resistances shall be summed in such a manner that the column moments oppose the beam moments. This requirement shall be satisfied for beam moments acting in either direction. The design column forces need not exceed those determined from the factored load combinations, with factored load effects calculated using R_dR_o equal to 1.0.

21.7.2.2.3
Transverse reinforcement shall be provided at both ends of the columns over a length equal to the largest of one-sixth of the clear height, the maximum cross-sectional dimension, or 450 mm, with a spacing not exceeding the smallest of
(a) eight longitudinal bar diameters;
(b) 24 tie diameters; or
(c) one-half of the minimum column dimension.

21.7.2.2.4
In the direction perpendicular to the longitudinal axis of the column, crossties or legs of overlapping hoops shall have centre-to-centre spacings not exceeding 350 mm.

21.7.2.2.5
Columns that can develop plastic hinges because of their connection to rigid members such as foundations or discontinued walls, or because of their position at the base of the structure, shall be provided with transverse reinforcement over their clear height, as follows:

(a) the volumetric ratio of circular hoop reinforcement, ρ_s, shall be not less than

$$\rho_s = 0.3 k_p \frac{f'_c}{f_{yh}} \qquad \textbf{(21-15)}$$

where

$k_p = P_f / P_o$

and f_{yh} shall not be taken as greater than 500 MPa. However, ρ_s shall not be less than that required by Equation (10-7);

(b) the total effective area in each of the principal directions of the cross-section within spacing s of rectangular hoop reinforcement, A_{sh}, shall be not less than the larger of the following:

$$A_{sh} = 0.15 k_n k_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} s h_c \qquad \textbf{(21-16)}$$

$$A_{sh} = 0.09 \frac{f'_c}{f_{yh}} s h_c \qquad \textbf{(21-17)}$$

where

$$k_n = \frac{n_\ell}{(n_\ell - 2)}$$

$k_p = P_f / P_o$

and f_{yh} shall not be taken as greater than 500 MPa;

(c) transverse reinforcement may be provided by single or overlapping hoops. Crossties of the same bar size and spacing as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar; and

(d) if the thickness of the concrete outside the confining transverse reinforcement exceeds 100 mm, additional transverse reinforcement shall be provided within the cover at a spacing not exceeding 300 mm.

21.7.2.3 Shear in frames
The factored shear resistance of beams and columns resisting earthquake effects shall be not less than the lesser of

(a) the sum of the maximum shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated using earthquake load combinations for gravity loads; or

(b) the maximum shear obtained from factored load combinations, with factored load effects calculated using $R_d R_o$ equal to 1.0.

21.7.2.4 Joints in frames

21.7.2.4.1
The design shear forces acting in a beam column joint shall be those induced by the nominal resistance of the beams or the columns framing into the joint, whichever is less, except that they need not exceed those determined from the factored load combinations, with factored load effects calculated using $R_d R_o$ equal to 1.0. Where beams frame into the joint from two directions, each direction may be considered independently.

21.7.2.4.2
The factored shear resistance of the joint shall not exceed the following, where f_c' is the strength of the concrete in the joint:
(a) for confined joints: $2.2\lambda\phi_c\sqrt{f_c'}A_j$;
(b) for joints confined on three faces or on two opposite faces: $1.6\lambda\phi_c\sqrt{f_c'}A_j$; and
(c) for other joints: $1.3\lambda\phi_c\sqrt{f_c'}A_j$.

A member that frames into a face shall be considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint shall be considered to be confined if such confining members frame into all faces of the joint.

21.7.2.4.3
Horizontal joint reinforcement shall be designed for shear in accordance with Clause 11.

21.7.2.4.4
Transverse hoop reinforcement shall be provided over the depth of the joint and spaced a maximum distance of 150 mm. Longitudinal column reinforcement shall have a centre-to-centre spacing not exceeding 300 mm and shall not be cranked within the joint.

21.7.2.4.5
The diameter of straight beam and column bars passing through the joint, d_b, shall satisfy the following equation:

$$d_b \le \lambda \frac{\ell_j}{20\,k_2} \quad \textbf{(21-18)}$$

21.7.3 Moderately ductile shear walls

21.7.3.1 General
The requirements specified in Clauses 21.7.3.2 to 21.7.3.4 shall apply to walls with h_w/ℓ_w greater than 2.0. The dimensional limitations specified in Clause 21.6.3 shall apply, except that in Clause 21.6.3.2 the limit $\ell_u/10$ shall be taken as $\ell_u/14$ and $\ell_u/14$ shall be taken as $\ell_u/20$.

21.7.3.2 Ductility of walls
Ductility of walls shall be determined in accordance with the methods specified in Clause 21.6.7, except that the minimum rotational demand may be taken as 0.003. The requirements specified in Clause 21.6.7 shall be considered satisfied if
(a) the depth to the neutral axis, c, determined in accordance with Equation (21-12) is less than 0.15 ℓ_w; or
(b) the depth to the neutral axis, c, determined in accordance with Equation (21-12) is less than 0.33 ℓ_w and the displacement, Δ_f, of the wall does not exceed $h_w/350$.

21.7.3.3 Detailing of walls

21.7.3.3.1
The distributed reinforcement ratio for walls shall be not less than 0.0025 in the vertical and horizontal directions and shall be tied in accordance with Clause 14.1.8.7.

21.7.3.3.2
Concentrated longitudinal reinforcement in plastic hinge regions shall meet the requirements of Clauses 21.6.4.1, 21.6.6.6, and 21.6.6.9 and shall be tied as columns in accordance with Clause 7 outside these regions.

21.7.3.3.3
Corners and junctions of intersecting walls shall be adequately tied to ensure unity of action. All horizontal bars shall extend to the far face of the joining wall and be bent around a vertical bar with a standard 90° hook.

21.7.3.4 Shear strength of moderately ductile shear walls

21.7.3.4.1
Shear walls shall have a factored shear resistance greater than the shear due to the effects of factored loads, but not less than the smaller of
(a) the shear corresponding to the development of the nominal moment capacity of the wall system at its plastic hinge locations; or
(b) the shear resulting from design load combinations that include earthquake effect, with loads calculated using $R_d R_o$ equal to 1.0.

21.7.3.4.2
Walls with moderate ductility shall be designed as specified in Clauses 21.6.9.2 to 21.6.9.7. The requirements of these clauses shall be considered satisfied if
(a) the factored shear force in the wall does not exceed $0.1\phi_c f'_c b_w d_v$;
(b) the value of β in Clause 11.3.4 is taken as 0.1; and
(c) the value of θ in Clause 11.3.5 is taken as 45°.

21.7.4 Squat shear walls

21.7.4.1
The requirements specified in Clauses 21.7.4.2 to 21.7.4.8 shall apply to walls with h_w/ℓ_w of 2.0 or less. Squat walls shall be designed using an R_d of 2.0.

21.7.4.2
The foundation and diaphragm components of the SFRS shall have factored resistances that are greater than the nominal wall capacity, which shall be based on one of the following energy-dissipation mechanisms:
(a) a flexural mechanism where the shear corresponding to the development of the nominal moment capacity of the wall is less than the nominal shear resistance; or
(b) a shear mechanism where the shear corresponding to the development of the nominal moment capacity of the wall is greater than the nominal shear resistance. The minimum shear resistance shall be taken as not less than $0.2\sqrt{f'_c}\, b_w d_v$.

However, the nominal wall capacity need not be taken larger than the load calculated with design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 1.0.

Note: *Squat walls can develop either a flexural or a shear mechanism to absorb seismic energy; however, the factored resistance can be significantly higher than the factored forces as a result of minimum reinforcement requirements and wall geometries. Consequently, the selection of an overstrength to use when applying capacity design principles to other components of the SFRS is very important.*

21.7.4.3
The dimensional limitations specified in Clause 21.6.3.1 shall apply.

21.7.4.4
The reinforcement requirements specified in Clause 21.6.4 shall apply.

21.7.4.5
The distributed reinforcement shall satisfy the following requirements:
(a) Both vertical and horizontal distributed reinforcement shall be provided in such a manner that the reinforcement ratio for this distributed reinforcement is not less than 0.003 in each direction. The reinforcement spacing in each direction shall not exceed 300 mm.
(b) At least two curtains of reinforcement shall be used if the in-plane factored shear force assigned to the wall exceeds $0.18\lambda\phi_c\sqrt{f'_c}b_w d_v$.
(c) Horizontal reinforcement shall extend to the ends of the wall and shall be contained at each end within a region of tied vertical reinforcement as specified in Clause 21.7.4.6.

21.7.4.6
Tied vertical reinforcement shall be provided at each end of the wall and at junctions of intersecting walls. The minimum reinforcement ratio of 0.005 shall be provided over a minimum wall length of 300 mm. The tied vertical reinforcement shall consist of a minimum of four bars and shall be tied as a column in accordance with Clause 7.6. The ties shall be detailed as hoops.

21.7.4.7
The vertical tension force required to resist overturning at the base of the wall shall be provided by concentrated reinforcement and vertical distributed reinforcement in addition to the amount required by Clause 21.7.4.8 to resist the shear corresponding to the applied bending moment. Plane sections analysis may be used for these calculations. All vertical reinforcement required at the base of the wall shall be extended the full height of the wall.

21.7.4.8
The shear design of squat walls shall meet the requirements specified in Clauses 11 and 21.6.9.3 to 21.6.9.5, and the following additional requirements:
(a) The factored shear demand on the wall shall not exceed $0.15\phi_c f'_c b_w d_v$.
(b) The value of β in Clause 11.3.4 shall be taken as zero. The value of θ may be freely chosen between a maximum value of 45° and a minimum value of 30°. The same value of θ shall be used to determine the required amount of distributed horizontal reinforcement in accordance with Clause 11.3.5 and the corresponding amount of distributed vertical reinforcement for shear in accordance with Clause 21.7.4.8(c).
(c) The amount of distributed vertical reinforcement required to resist shear shall be determined from the following equation as a function of the required amount of distributed horizontal reinforcement, ρ_h:

$$\rho_v = \rho_h \cot^2\theta - \frac{P_s}{\phi_s f_y A_g} \qquad \textbf{(21-19)}$$

(d) When the strut-and-tie model of Clause 11.4 is applied to short walls, the additional requirements of Clause 21.6.9.7 shall apply.

21.8 Conventional construction (R_d = 1.5)

21.8.1 General
The requirements specified in Clauses 21.8.2 to 21.8.4 shall apply to elements of the SFRS.

Structures designed with an R_d of 1.5 need only comply with Clauses 21.8.2 to 21.8.4, i.e., not with Clauses 21.1 to 21.7 and 21.9 to 21.12.

21.8.2 Frames

Frame members shall comply with the requirements of Clauses 7.7, 11.7, and 12.11.2. The columns shall contain ties that comply with Clause 21.6.6.9 unless

(a) the sum of the factored resistances of columns framing into a joint is greater than the factored resistance of the beams framing into the joint;

(b) the factored resistances of the columns are greater than the effects of factored loads calculated using $R_d R_o$ equal to 1.0; or

(c) $I_E F_a S_a(0.2)$ is less than 0.2.

21.8.3 Walls

21.8.3.1

Walls shall be designed to the requirements of Clauses 14 and 21.2.7.2.

21.8.3.2

Walls shall have a factored shear resistance greater than the shear due to the effects of factored loads, but not less than the smaller of

(a) the shear corresponding to the development of the factored moment capacity of the wall system at its critical sections; or

(b) the shear resulting from design load combinations that include earthquake effect, with loads calculated using $R_d R_o$ equal to 1.0.

21.8.4 Two-way slabs without beams

21.8.4.1

The factored slab moment at support related to earthquake effect shall be determined for factored load combinations including earthquake effects. All reinforcement provided to resist M_s, the portion of slab moment balanced by support moment, shall be placed within the column strip (see Clause 2.2).

21.8.4.2

The fraction of the moment, M_s, determined using Equation (13-25) shall be resisted by reinforcement placed within the effective width b_b. The effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than c_t, measured perpendicular to the slab span.

21.8.4.3

Not less than one-half of the reinforcement in the column strip at a support shall be placed within the effective slab width b_b.

21.8.4.4

Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.

21.8.4.5

Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.

21.8.4.6

Not less than one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement shall be continuous and shall develop its yield strength at the face of support as specified in Clause 13.8.5.1.

21.8.4.7
At discontinuous edges of the slab, all top and bottom reinforcement at a support shall be developed at the face of support as specified in Clause 13.8.5.1.

21.8.4.8
At the critical section for columns specified in Clause 13.3.3.1, two-way shear caused by factored gravity loads shall not exceed $0.4V_c$, where V_c shall be calculated as specified in Clause 13.3.4 for non-prestressed slabs and in Clause 18.12.3.3 for prestressed slabs. This requirement may be waived if the contribution of the earthquake-induced factored two-way shear stress transferred by eccentricity of shear in accordance with Clauses 13.3.5.3 to 13.3.5.5 at the point of maximum stress does not exceed one-half of the stress, v_c, permitted by Clause 13.3.4.

21.9 Precast concrete

21.9.1 General
The seismic design of ductile moment-resisting frames, ductile flexural walls, and moderately ductile flexural walls constructed using precast concrete shall comply with Clause 21.9.

21.9.2 Ductile moment-resisting frames constructed using precast concrete (R_d = 4.0)

21.9.2.1
Ductile moment-resisting frames with ductile connections constructed using precast concrete shall satisfy the following requirements, as well as all requirements for ductile moment-resisting frames constructed with cast-in-place concrete:

(a) the factored shear resistance for connections computed as specified in Clause 11.5.1 shall be greater than or equal to 150% of the shear calculated as specified in Clause 21.3.4.1 or 21.4.5.1; and

(b) mechanical splices of beam reinforcement shall be located not closer than $h/2$ from the joint face and shall meet the requirements of Clause 21.2.8.

21.9.2.2
Ductile moment-resisting frames with strong connections constructed using precast concrete shall satisfy the following requirements as well as all requirements for ductile moment-resisting frames constructed with cast-in-place concrete:

(a) the requirements of Clause 21.3.1.1(b) shall apply to segments between locations where flexural yielding is intended to occur as a result of design displacements;

(b) the factored resistance of the strong connection, S_r, shall be not less than S_p;

(c) primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region; and

(d) column-to-column connections shall have a factored resistance not less than $1.4S_p$. At column-to-column connections, the factored resistance shall be not less than 0.4 times the maximum probable flexural strength for the column within the storey height, and the factored shear resistance of the connection shall be not less than that determined in accordance with Clause 21.4.5.1.

21.9.2.3
Ductile moment-resisting frames constructed using precast concrete and not meeting the requirements of Clause 21.9.2.1 or 21.9.2.2 shall comply with ACI T1.1/T1.1R and the following requirements:
(a) the details and materials used for the test specimens shall be representative of those used in the structure; and
(b) the design procedure used to proportion the test specimens shall define the mechanism by which the frame resists gravity and earthquake effects and shall establish acceptance values for sustaining that mechanism. Portions of the mechanism that deviate from code requirements shall be contained in the test specimens and shall be tested to determine upper bounds for acceptance values.

21.9.3 Ductile shear walls constructed using precast concrete (R_d = 3.5 or 4.0)
Ductile shear walls constructed using precast concrete shall meet all of the requirements of Clause 21.6 for cast-in-place ductile shear walls and shall contain strong connections. The factored resistance of the strong connection, S_r, shall be not less than S_p.

21.9.4 Moderately ductile shear walls constructed using precast concrete (R_d = 2.0)

21.9.4.1
Moderately ductile shear walls constructed using precast concrete shall meet all of the requirements of Clause 21.7.3 or 21.7.4 for cast-in-place moderately ductile walls unless they are designed in accordance with Clause 23, in which case the requirements of Clause 21.7.1.2 shall apply.

21.9.4.2
In connections between wall panels, yielding shall be restricted to steel elements or reinforcement. If connections between the wall panels and the foundations are relied on for energy dissipation, the reinforcement shall be adequately anchored to both the wall panel and the foundation to develop the probable strength of reinforcement, in accordance with Clause 12.

21.9.4.3
Elements of the connection that are not designed to yield shall develop at least 150% of the specified yield strength of the yielding element.

21.10 Structural diaphragms (R_d = 2.0, 2.5, 3.5, or 4.0)

21.10.1 General
Floor and roof systems acting as structural diaphragms to transmit and transfer forces induced by earthquake ground motions shall be designed in accordance with Clauses 21.10.2 to 21.10.8.

21.10.2 Design forces
Design forces for diaphragms and their connections shall comply with Article 4.1.8.15 of the *National Building Code of Canada.*

21.10.3 Diaphragm systems

21.10.3.1
A diaphragm shall be idealized as a system consisting of the following components arranged to provide a complete load path for the forces:
(a) chords proportioned to resist diaphragm moments as tensions and compression forces;
(b) collectors arranged to transfer the forces to, from, and between the vertical SFRSs; and
(c) either
  (i) shear panels to transfer forces to, from, and between the chords and collectors; or
  (ii) continuous strut-and-tie in-plane shear trusses.

21.10.3.2
Diaphragm elements shall be made effectively continuous by the provisions for force transfer at all edges and ends. Embedment, tying, and anchorage shall be provided at all edges of shear panels to transfer shears to adjacent chords, collectors, and shear panels. Collectors shall be anchored to the vertical SFRSs.

21.10.4 Reinforcement

21.10.4.1
The minimum reinforcement ratio for structural diaphragms shall comply with Clause 7.8. Reinforcement spacing in each direction in non-post-tensioned floor and roof systems shall not exceed 500 mm. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

21.10.4.2
The diameter of the bars used in diaphragm struts, ties, chords, and collector elements shall not exceed one-sixth of the minimum element dimension at the bar location.

21.10.4.3
All continuous reinforcement in struts, ties, chords, and collector elements shall be anchored or spliced as specified in Clauses 12 and 21.2.8. Anchorage and splice lengths for reinforcement not contained within confinement or buckling prevention ties shall be increased by 50%, or, for splices, laps shall be staggered with at least one lap length from the end of one lap to the start of the next.

21.10.4.4
Splices of tensile reinforcement in the chords and collector elements of diaphragms shall develop the yield strength of the reinforcement. Mechanical and welded splices shall comply with Clauses 21.2.8 and 21.2.9, respectively. Type 2 splices shall be required where mechanical splices are used to transfer forces between collectors and the vertical components of the SFRS.

21.10.4.5
Bonded prestressing tendons used as primary reinforcement in diaphragm chords or collectors shall be proportioned in such a manner that the stress due to design seismic forces does not exceed 400 MPa. Precompression from unbonded tendons shall be permitted to resist diaphragm design forces if a complete load path is provided.

21.10.5 Monolithic concrete systems

21.10.5.1
Slabs serving as shear panels shall be not less than 50 mm thick for joist and waffle systems and 100 mm for all other systems.

21.10.5.2
The factored shear strength of a shear panel shall be taken as

$$V_r = A_{cv}(0.2\phi_c\sqrt{f_c'} + \phi_s\rho_n f_y) \le 0.8\phi_c A_{cv}\sqrt{f_c'} \quad \textbf{(21-20)}$$

21.10.5.3
Chords, collectors, struts, and ties shall be proportioned to have compressive stresses less than $0.2f_c'$ or shall be provided with buckling prevention ties. The dimensions of the section shall provide for a minimum cover of 2-1/2 bar diameters, but not less than 50 mm for all longitudinal reinforcement, and a minimum clear spacing of three diameters, but not less than 40 mm at splices and anchorage zones.

21.10.6 Precast systems

21.10.6.1
Cast-in-place composite and non-composite toppings shall be permitted to serve as shear panels. Composite toppings shall be not less than 50 mm thick and non-composite toppings not less than 65 mm thick. The surface of the previously hardened concrete on which composite topping slabs are placed shall be clean, free of laitance, and intentionally roughened.

21.10.6.2
The factored shear strength of a shear panel shall be taken as

$$V_r = \phi_s A_{cv}\rho_n f_y \le 0.6\phi_c A_{cv}\sqrt{f_c'} \quad \textbf{(21-21)}$$

where A_{cv} is calculated based on the thickness of the topping slab. The required web reinforcement shall be distributed uniformly in both directions. Where welded wire fabric is used as the distributed reinforcement, the wires parallel to the span of the precast elements shall be spaced not less than 250 mm on centre.

21.10.6.3
Chords, collectors, struts, and ties shall comply with Clause 21.10.5.3.

21.10.7 Composite systems

21.10.7.1
Composite concrete toppings on steel decks may be used as shear panels. The composite toppings shall be not less than 60 mm thick above the top of the flutes.

21.10.7.2
The factored shear strength of composite toppings on steel decks may be taken from manufacturer's data, with appropriate modifications of the published data to account for the effects of reverse cyclic loading.

21.10.7.3
For decks bounded by steel beams and girders designed as full composite members with headed stud shear connectors, the shear strength of a reinforced topping slab shall be taken as

$$V_r = \phi_s A_{cv}\rho_n f_y \le 0.6\phi_c A_{cv}\sqrt{f_c'} \quad \textbf{(21-22)}$$

where A_{cv} is calculated based on the topping thickness above the flutes.

21.10.7.4
Chords, collectors, struts, and ties may be structural steel and/or reinforced concrete sections. Structural steel members used for this purpose shall have headed stud shear connectors designed to transfer the shear forces from the topping. Reinforced concrete sections shall comply with Clause 21.10.5.3.

21.10.8 Construction joints
All construction joints in diaphragms shall comply with Clause 6.3. Contact surfaces shall be treated as specified in Clause 11.5.

21.11 Foundations (R_d = 2.0, 2.5, 3.5, or 4.0)

21.11.1 General

21.11.1.1
Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground shall comply with Clauses 21.11.1.2 to 21.11.4.6.

21.11.1.2
The factored resistance of the foundation system and the supports of frames or walls shall be sufficient to develop the nominal moment capacity of the frames or walls and the corresponding shears. Where the factored moment resistance of any wall or frame exceeds the required factored moment, the following shall apply:
(a) the factored resistance of unanchored footings supporting those walls or frames need not exceed the maximum factored load effects determined with loads calculated using R_dR_o equal to 2.0; and
(b) where frames or walls are supported by anchored footings or elements other than foundations, the factored resistance of those elements need not exceed the maximum factored load effects determined with loads calculated using R_dR_o equal to 1.0.

21.11.1.3
The requirements of Clause 21.11 for piles, drilled piers, caissons, and slabs on grade shall be in addition to the requirements specified in Clause 15.

21.11.2 Footings, foundation mats, and pile caps

21.11.2.1
Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap and shall be fully developed for tension at the interface.

21.11.2.2
Columns designed assuming fixed-end conditions at the foundation shall comply with Clause 21.11.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90° hooks near the bottom of the foundation, with the free end of the bars oriented toward the centre of the column.

21.11.2.3
Concentrated wall reinforcement shall extend to the bottom of the footing, mat, or pile cap and terminate with a 90° hook.

21.11.2.4
Columns or areas of concentrated wall reinforcement that have an edge within one-half of the footing depth from an edge of the footing shall have the same transverse reinforcement provided below the top of the footing as provided above the footing. This transverse reinforcement shall extend into the footing a distance not less than the smaller of the depth of the footing, mat, or pile cap or the development length in tension of the longitudinal reinforcement.

21.11.2.5
Where earthquake effects create uplift forces in columns or concentrated reinforcement of flexural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat, or pile cap. Such reinforcement shall be not less than 0.001 times the gross sectional area in each direction or 120% of the required factored capacity calculated using the nominal resistance of the wall or column tension reinforcement, whichever is less.

21.11.3 Grade beams and slabs on grade

21.11.3.1
Grade beams and slabs designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supporting column or anchored within the pile cap or footing at all discontinuities.

21.11.3.2
Grade beams not connected to a slab designed to act as horizontal ties between pile caps or footings shall be proportioned in such a manner that the smallest cross-sectional dimension shall be equal to or greater than the clear spacing between connected columns divided by 20, but need not be greater than 450 mm. Closed ties shall be provided at a spacing not exceeding one-half of the smallest cross-sectional dimension or 300 mm, whichever is smaller.

21.11.3.3
Grade beams and beams that are part of a mat foundation subject to flexure from columns that are part of the SFRS shall comply with Clause 21.3. Joints between these columns and grade beams shall comply with Clause 21.5.

21.11.3.4
Slabs on grade that resist seismic forces from walls or columns that are part of the SFRS shall be designed as structural diaphragms in accordance with Clause 21.10. The design drawings shall clearly state that the slab on grade is a structural diaphragm and part of the SFRS.

21.11.4 Piles and piers

21.11.4.1
The requirements of Clauses 21.11.4.2 to 21.11.4.6 shall apply to concrete piles and piers supporting structures designed for earthquake resistance.

21.11.4.2
Piles or piers resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members.

21.11.4.3
Piles or piers shall have transverse reinforcement as specified in Clause 21.4.4 at the following locations:
(a) at the top of the member for at least five times the largest member's cross-sectional dimension, but not less than 2000 mm below the bottom of the pile cap;
(b) along the entire unsupported length plus the length specified in Clause 21.11.4.3(a) for piles in air, in water, or in soil incapable of providing lateral support; and
(c) within five pile diameters of the interface between soils of different strength or stiffness.

21.11.4.4
For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation of pile tips.

21.11.4.5
The slenderness effects of piles shall be considered for the portion of the piles in air, in water, or in soil incapable of providing lateral support.

21.11.4.6
Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns.
Note: *Batter pile systems should be used with extreme caution because subsoil deformations caused by earthquake effects can cause pile loads far in excess of those due to the seismic forces in the superstructure.*

21.12 Frame members not considered part of the seismic force resisting systems (R_d = 2.0, 2.5, 3.5, or 4.0)
Note: *The intent of this Clause is to provide a minimum level of ductility and strength for all structural members subject to seismically induced deformations but not considered part of the SFRSs.*

21.12.1 General

21.12.1.1
Elements not required to resist either gravity or lateral loading shall be considered non-structural elements. These elements need not be detailed to the requirements of Clauses 21.12.1.2 to 21.12.3.3, provided that
(a) the effects of their stiffness and strength on forces and deformations in all structural elements at the design displacement are calculated;
(b) the factored capacity of the structural elements is sufficient to resist these forces and deformations; and
(c) the non-structural elements are anchored to the building in accordance with Article 4.1.8.17 of the *National Building Code of Canada.*

21.12.1.2
Unless it can be shown that factored moments in structural members will not exceed their nominal resistance when the complete structure is deformed laterally to the design displacement, these members shall be designed to accommodate lateral deflection through the formation of a plastic hinge mechanism. When the effects of design displacement are not explicitly checked, the requirements of Clauses 21.12.2.2 and 21.12.3 shall be applied.

21.12.1.3
The resistance of each structural member shall be sufficient to carry all forces due to factored gravity loads as well as the axial and shear forces induced in the member when subject to the design displacement.

21.12.2 Plastic hinges in members

21.12.2.1
Where Clause 21.12.1.2 indicates the formation of plastic hinges under the effects of factored gravity loads and the design displacement, the plastic hinge regions shall be detailed as specified in Clauses 21.12.2.2 to 21.12.2.4 to ensure adequate rotational capacity.

21.12.2.2
Where the member forces induced at 1/2.5 of the design displacement are greater than their nominal resistances, the members shall meet the following requirements:
(a) Flexural members (beams) shall meet the requirements Clauses 21.3.2 and 21.3.4, and stirrups shall be spaced not more than $d/2$ throughout the length of the member.
(b) Flexural members with factored axial loads exceeding $A_g f_c'/10$ but less than $0.35P_0$ shall meet the requirements of Clauses 21.4.4, 21.4.5, and 21.5.2.1.
(c) Flexural members with factored axial loads in excess of $0.35P_0$ shall have a nominal resistance greater than the induced member force or, where member forces are not explicitly calculated, shall not be permitted.

21.12.2.3
Where the member forces induced at 1/1.3 of the design displacement are less than their nominal resistances, the members shall be required to meet the requirements of Clauses 3 to 18 only.

21.12.2.4
Members not meeting the requirements of Clause 21.12.2.2 or 21.12.2.3 shall be detailed to the requirements of Clause 21.7.

21.12.3 Slab column connections

21.12.3.1
Where the maximum gravity load two-way shear stresses, excluding shear stresses from unbalanced moment and determined using seismic load combinations, exceed R_E times the limiting stresses in Clause 13.3.4 or 18.12.3.3, shear reinforcement shall be provided as specified in Clause 21.12.3.2, with R_E calculated as follows:

$$R_E = \left(\frac{0.005}{\delta_i}\right)^{0.85} \le 1.0 \qquad \textbf{(21-23)}$$

where δ_i is ≤ 0.025.

21.12.3.2
When shear reinforcement is required by Clause 21.12.3.1, the following requirements shall be satisfied:
(a) Shear reinforcement shall be provided in such a manner that the maximum gravity load two-way shear stresses, excluding shear stresses from unbalanced moment and determined using seismic load combinations, do not exceed R_E times the limiting shear resistance calculated using 50% of v_c, calculated in accordance with Clause 13.3.8.3, 13.3.9.3, or 18.12.3.3, and acting in combination with v_s, calculated in accordance with Clause 13.3.8.5.
(b) The factored shear stress resistance of the shear reinforcement, calculated in accordance with Clause 13.3.8.5, shall be not less than $0.3\sqrt{f_c'}$.
(c) The factored shear stress resistance of the shear reinforcement shall be not less than that required by Clause 13.3.
(d) Shear reinforcement shall be detailed in accordance with Clause 13.3.
(e) Shear reinforcement shall extend a minimum of $4d$ beyond the face of the column.

21.12.3.3
For post-tensioned slabs, mild steel bottom reinforcement meeting the requirements of Clause 13.10.6 shall be provided in accordance with Item (a), (b), or (c) of Clause 13.10.6.3. The minimum total prestressing steel need not satisfy the requirements of Clause 13.10.6.3(d).

22 Plain concrete

22.1 General

22.1.1
Clause 22 specifies requirements for the design of concrete members containing less reinforcement than the minimum amount specified for reinforced concrete members elsewhere in this Standard. The requirements of Clause 22 shall be limited to pedestals with $\ell_c/h \leq 3$, walls not exceeding 3 m in total height that have continuous vertical support, pad footings, spread footings, drilled piles, and slabs on grade. The requirements of Clause 22 shall be further limited to concretes with compressive strengths not less than 15 MPa.

22.1.2
Plain concrete shall not be used for structural members where ductility is required, such as for earthquake or blast resistance.

22.1.3
Plain concrete shall not be used for pile caps.

22.1.4
Plain concrete shall not be used for members relied on to transmit tension force.

22.2 Control joints

22.2.1
In plain concrete construction, control joints shall be provided to divide a structural member into discontinuous elements. The size of each element shall be limited to control stresses caused by restraint to movements from creep, shrinkage, temperature effects, and differential settlement.

22.2.2
In determining the number and location of control joints, consideration shall be given to the influence of climatic conditions; selection and proportioning of materials; mixing, placing, and curing of concrete; the degree of restraint to movement; and stresses due to load.

22.2.3
The locations and details of control joints shall be indicated on the drawings or in the specifications.

22.2.4
Concrete placement shall be interrupted only at control joints.

22.3 Design

22.3.1
The strength design of plain concrete members for factored flexural and axial loads shall be based on a linear stress-strain relationship in both tension and compression.

22.3.2
The flexural tensile strength of concrete may be considered in the design.

22.3.3
No strength shall be assigned to reinforcement that might be present.

22.3.4
The bearing stress on the concrete at the contact surface between supporting and supported members shall not exceed the permissible bearing stress for each surface as specified in Clause 10.8.

22.3.5
The entire cross-section of a member shall be considered in the design, except for footings cast against soil (see Clause 22.6.3).

22.4 Walls

22.4.1

22.4.1.1
The effective length factor, *k*, for walls braced at the top and bottom against lateral translation shall be as follows:

If restrained against rotation at one or both ends (top, bottom, or both)	0.8
If unrestrained against rotation at both ends	1.0

22.4.1.2
Except as specified in Clause 22.4.1.3, walls subject to combined flexure and axial load shall be proportioned so that the maximum compressive stress under factored loads is limited to

$$0.75\phi_c f'_c\left(1-\left(\frac{k\ell_c}{32t}\right)^2\right)$$

and the maximum tensile stress shall not exceed $0.37\lambda\phi_c\sqrt{f'_c}$. The minimum eccentricity shall be $0.1t$.

22.4.1.3
Plain concrete walls of solid rectangular cross-section may be designed in accordance with the following equation if the resultant of all factored loads, including the effects of lateral loads applied to the wall, is located within the middle third of the overall thickness of the wall:

$$P_r = 0.37\phi_c f'_c A_g\left(1-\left(\frac{k\ell_c}{32t}\right)^2\right) \qquad \textbf{(22-1)}$$

22.4.2
The horizontal length of wall to be considered effective for each concentrated load or reaction shall not exceed the centre-to-centre distance between loads or the width of bearing plus four times the wall thickness.

22.4.3
Plain concrete bearing walls shall have a thickness of not less than 1/20 of the unsupported height or length, whichever is shorter.

22.4.4
Foundation walls and exterior basement walls shall be not less than 190 mm thick.

22.4.5
Walls shall be braced against lateral translation and keyed or dowelled to other intersecting members as required for lateral stability.

22.4.6
Not less than two 15M bars shall be provided around all window and door openings. Such bars shall extend at least 600 mm beyond the corners of the openings.

22.5 Pedestals
Pedestals subject to combined flexural and axial load shall be proportioned so that the maximum compression stress under factored loads does not exceed $0.75\phi_c f_c'$ and the maximum tension stress does not exceed $0.37\lambda\phi_c\sqrt{f_c'}$. The minimum eccentricity shall be $0.1h$.

22.6 Footings

22.6.1 Base area of footing
The base area of the footing shall be determined from forces and moments transmitted by the footing to the soil. The soil pressure shall be selected in accordance with the principles of soil mechanics.

Note: *See the* National Building Code of Canada *for information on limit states design of foundations.*

22.6.2 Minimum thickness
The specified thickness of plain concrete footings shall be not less than 200 mm.

22.6.3 Minimum thickness for calculations
For footings cast against soil, the overall thickness, h, used in calculations shall be taken as 50 mm less than the specified thickness.

22.6.4 Critical sections

22.6.4.1
The critical sections for moment and shear shall be as specified in Clause 15.4.3.

22.6.4.2
For the location of critical sections for moment and shear, circular or regular polygonal concrete columns or pedestals may be treated as square members with the same area.

22.6.5 Strength in bending

The factored resistance in bending shall be based on a maximum stress in tension of $0.37\lambda\phi_c\sqrt{f_c'}$ and a maximum stress in compression of $0.75\phi_c f_c'$.

22.6.6 Shear resistance

22.6.6.1 One-way action

22.6.6.1.1

The maximum factored shear, V_f, shall be computed at a distance h from the face of the support. Sections located closer to the support may be designed for the same shear.

22.6.6.1.2

The factored shear resistance for rectangular sections, V_r, shall be

$$V_r = \frac{2}{3}(0.18\lambda\phi_c\sqrt{f_c'}\,bh) \qquad \textbf{(22-2)}$$

22.6.6.2 Two-way shear

22.6.6.2.1

The maximum factored shear, V_f, shall be computed at a critical section perpendicular to the plane of the footing and located so that its perimeter, b_o, is a minimum, but not closer than $h/2$ to the perimeter of the concentrated load or reaction area.

22.6.6.2.2

The factored shear resistance, V_r, shall be

$$V_r = \frac{2}{3}\left(\left(1+\frac{2}{\beta_c}\right)0.18\lambda\phi_c\sqrt{f_c'}\,b_o h\right) \qquad \textbf{(22-3)}$$

but

$$V_r \le \frac{2}{3}(0.37\lambda\phi_c\sqrt{f_c'}\,b_o h) \qquad \textbf{(22-4)}$$

22.7 Slabs on grade

Plain concrete slabs shall be designed with due regard to loading and foundation conditions.

Note: *For information on the design of slabs on grade, see ACI 360R.*

22.8 Drilled piles

22.8.1

In addition to meeting the design eccentricity requirement specified in Clause 15.2, the cross-sections of uncased drilled piles shall be designed for a minimum eccentricity of $0.1d_p$.

22.8.2

The outer 25 mm layer of uncased drilled piles shall be neglected when the pile resistance and the stresses in the pile shaft due to factored loads are determined.

22.8.3

For uncased drilled piles, a reduction factor of 0.8 shall be applied to the maximum factored stresses specified in Clause 22.8.5.

22.8.4
The stability of portions of piles without lateral restraint from soil shall be considered.

22.8.5
Drilled piles subjected to combined factored bending moments, shears, and compression loads shall be proportioned so that stresses do not exceed the following limits:
(a) flexure and axial loads:
 (i) extreme fibre stress in compression:

$$0.75\phi_c f_c'\left(1-\left(\frac{k\ell_c}{28d_p}\right)^2\right)$$

 (ii) extreme fibre stress in tension:

$$0.37\lambda\phi_c\sqrt{f_c'}$$

(b) shear:

$$V_r = \frac{3}{4}(0.18\lambda\phi_c\sqrt{f_c'}) \qquad \textbf{(22-5)}$$

22.8.6
For proportioning of the pile bell, see Clause 15.8.2.4.

23 Tilt-up wall panels

23.1 General

23.1.1
The requirements of Clauses 23.1.2 to 23.7.2 shall apply to tilt-up wall panels.

23.1.2
The requirements of Clauses 3 to 15 and 21 shall apply to tilt-up wall panels, except as modified by the requirements of Clauses 23.1.1 and 23.1.3 to 23.7.2.

23.1.3
Tilt-up panels are slender vertical flexural slabs that resist lateral wind or seismic loads and are subject to very low axial stresses. Because of their high slenderness ratios, they shall be designed for second-order P-Δ effects to ensure structural stability and satisfactory performance under specified loads.

23.2 Design requirements

23.2.1 Effective panel height
The effective panel height, ℓ, shall be the centre-to-centre distance between lateral supports.

23.2.2 Minimum panel thickness
The minimum panel thickness for a prismatic load-bearing panel without stiffening elements shall be 140 mm.

23.2.3 Maximum height-to-thickness ratio
The maximum effective panel height-to-thickness ratio shall be
(a) 50 for panels with a single mat of reinforcement at mid-depth; or
(b) 65 for panels with a mat of reinforcement near each face.

23.2.4 Minimum reinforcement
Minimum panel reinforcement shall comply with Clauses 10.5, 14.1.8, and 21.7.3.3, as applicable.

23.2.5 Concrete cover and tolerances

23.2.5.1
If quality control procedures are followed so that the designer can be assured that the tilt-up contractor meets the requirements in CSA A23.4 with respect to dimensional control, reinforcement placement, aggregate size, concrete quality, and curing, the cover requirements specified in CSA A23.4 may be used, except as required by Clause 23.2.5.2. Otherwise, the design shall comply with CSA A23.1.

23.2.5.2
The cover and quality of concrete in tilt-up panels that have to withstand the effects of aggressive or corrosive environments shall comply with CSA A23.1.

23.2.5.3
Tilt-up panels requiring a fire resistance rating or forming part of a firewall shall meet the thickness and cover requirements of the applicable building codes.

23.2.6 Thermal effects
The design of tilt-up panels shall take into account the effects of any thermal gradients that could occur through the panel.

23.2.7 Sandwich panels
Sandwich wall panels, in addition to resisting applied loads, shall be designed to resist effects such as composite or non-composite action between wythes, thermal effects between wythes where composite action is assumed, thermal bridging, lifting stresses imposed on one wythe by the other, and vertical and torsional support of one wythe by the other.

23.2.8 Connections

23.2.8.1
The design of connections shall take into account in-plane and out-of-plane forces; the additional effects of shrinkage, creep, temperature, and movement; and the applicable requirements of Clause 21.

23.2.8.2
The resistances of connections between the tilt-up panels and any adjoining elements shall be greater than the effects of factored loads.

23.2.9 Structural integrity

23.2.9.1
Tension ties shall be provided in the transverse and longitudinal directions of the structure and around the perimeter of the structure to effectively tie the elements together.

23.2.9.2
Panels shall have connections top and bottom to resist a minimum factored force of 5 kN/m perpendicular to the panel.

23.2.10 Effective reinforcement
Where vertical reinforcement is placed in two layers, the effect of compression reinforcement shall be ignored.

23.3 Analysis and design

23.3.1 Flexure and axial load interaction and slenderness effects

23.3.1.1
All moment and deflection calculations specified in Clauses 23.3.1.2 to 23.3.2 are based on simple support conditions top and bottom. For other support and fixity conditions, moments and deflections shall be calculated using established principles of structural mechanics.

23.3.1.2
In lieu of a more accurate analysis, the procedures specified in Clauses 23.3.1.3 to 23.3.1.5 shall be used when the stress due to factored vertical loads at the cross-section under consideration meets the following requirement:

$$\frac{P_{wf}+P_{tf}}{A_g} < 0.09\phi_c f_c' \qquad \textbf{(23-1)}$$

23.3.1.3
The factored moment, M_f, shall be determined at the mid-height of the panel and shall be equal to

$$M_f = M_b\delta_b \qquad \textbf{(23-2)}$$

where

$$M_b = \frac{w_f\ell^2}{8} + P_{tf}\frac{e}{2} + (P_{wf}+P_{tf})\Delta_o$$

$$\delta_b = \frac{1}{1-\dfrac{P_f}{\phi_m K_{bf}}} \geq 1.0$$

where

$P_f = P_{wf} + P_{tf}$

$$K_{bf} = \frac{48E_c I_{cr}}{5\ell^2}$$

where

$$I_{cr} = \frac{bc^3}{3} + \frac{E_s}{E_c}A_s(d-c)^2$$

and the member resistance factor, ϕ_m, is taken as 0.75.

23.3.1.4

The initial out-of-straightness, Δ_o, at mid-height of the panel shall take into account the effects of non-planar and flexible casting beds, deformations caused by the tilting process, thermal gradients through the panel, and creep, and shall not be taken less than $\ell/400$.

23.3.1.5

The factored resisting moment, M_r, provided by the panel cross-section shall be such that

$$M_r \geq M_f \qquad \textbf{(23-3)}$$

The resisting moment may be calculated using an effective area of reinforcement, $A_{s,eff}$, as follows:

$$A_{s,eff} = \frac{\phi_s A_s f_y + P_f}{\phi_s f_y} \qquad \textbf{(23-4)}$$

23.3.2 Deflection limitations

Unless serviceability requirements lead to the conclusion that a larger deflection is acceptable, the horizontal mid-height deflection, Δ_s, under specified lateral and vertical loads shall not exceed $\ell/100$, but it shall not be greater than can be tolerated by attached structural or non-structural elements. The horizontal mid-height deflection may be computed as follows:

$$\Delta_s = \frac{5M_s\ell^2}{48E_c I_e} = \frac{M_s}{K_{bs}} \qquad \textbf{(23-5)}$$

where

$M_s = M_{bs}\delta_{bs}$

where

$$M_{bs} = \frac{w_s\ell^2}{8} + P_{ts}\frac{e}{2} + (P_{ws} + P_{ts})\Delta_o$$

$$\delta_{bs} = \frac{1}{1 - \dfrac{P_s}{K_{bs}}} \geq 1.0$$

where

$P_s = P_{ws} + P_{ts}$

$$K_{bs} = \frac{48E_c I_e}{5\ell^2}$$

and where I_e is as specified in Clause 9.8.2.3, substituting M_s for M_a.

Note: *Because I_e depends on M_s, iteration is necessary.*

23.4 Effects of openings

23.4.1 Design width

23.4.1.1

A design width on each side of an opening shall support the combined factored axial and lateral loads from its tributary width. This design width shall be used over the full height of the panel.

23.4.1.2
The design width shall be limited to
(a) 12 times the thickness of a solid panel; or
(b) 12 times the thickness of the structural wythe of a sandwich panel.

23.4.2 Tributary width
The tributary width for design shall be the design width plus one-half the width of adjacent openings (see Figure 23.1).

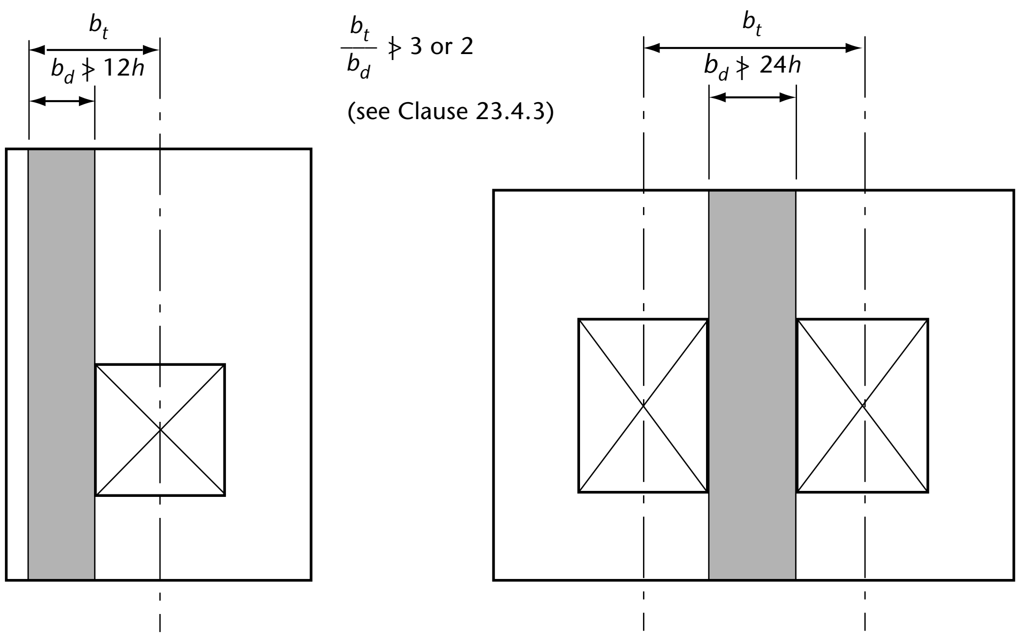


Figure 23.1
Effect of openings on design width, b_d
(See Clause 23.4.2.)

23.4.3 Ratio of tributary width to design width
Unless a more detailed analysis, accounting for the internal force effects, indicates otherwise, the ratio of tributary width to design width shall not exceed 3. For panels with a single layer of reinforcement and $\ell/h > 40$, the ratio shall not exceed 2.

23.5 Concentrated loads or reactions

23.5.1 Design width
The design width, b_d, for a panel subjected to concentrated loads or concentrated reactions shall be determined from Figure 23.2.

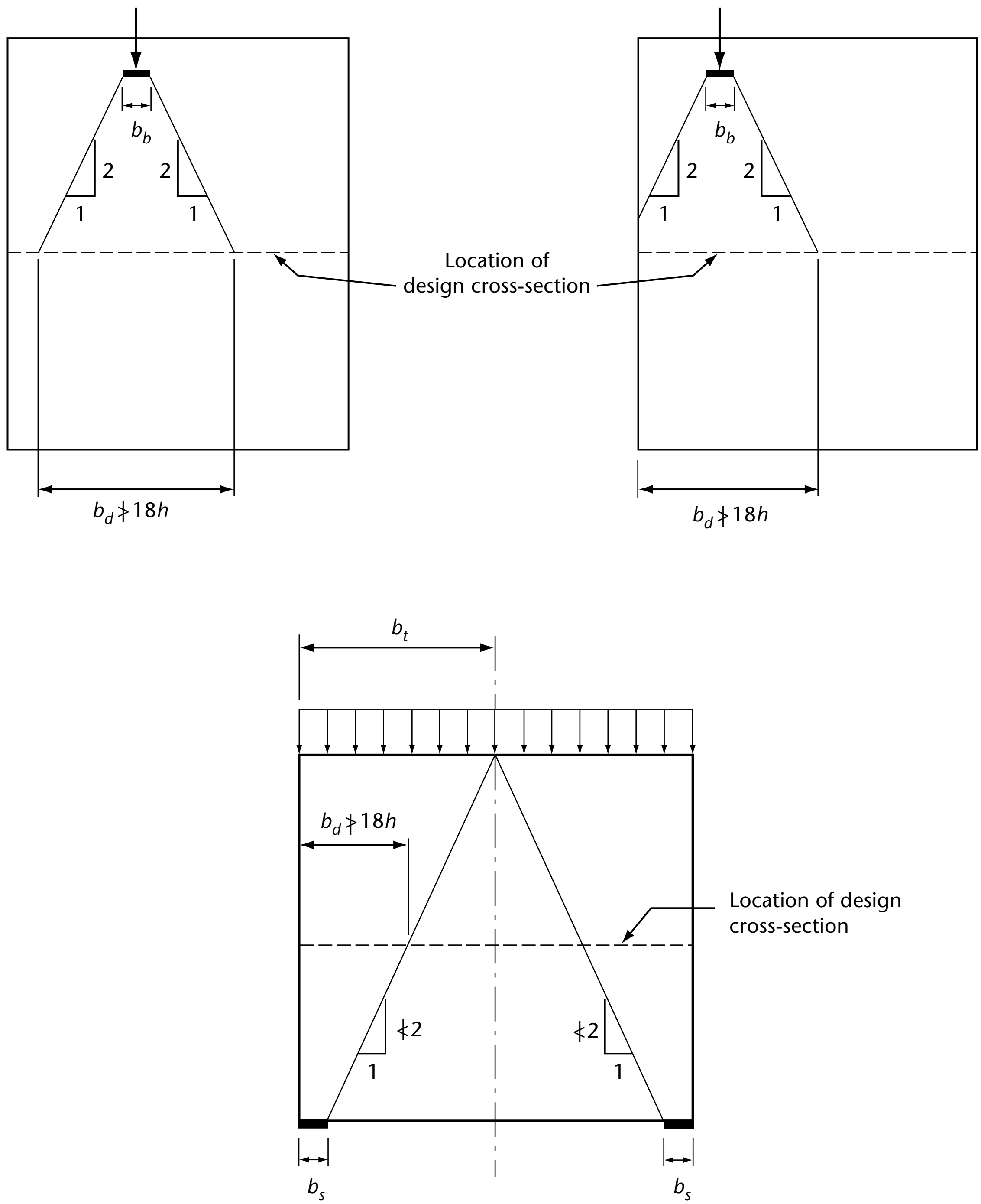


Figure 23.2
Effect of concentrated loads or reactions on design width, b_d
(See Clauses 2.3, 23.5.1, and 23.5.4.)

23.5.2 Bearing
The allowable bearing stress at the contact surface between supported and supporting elements and between any intermediate bearing elements shall not exceed the bearing resistance of either element, as specified in Clause 10.8 or 11.4.4.

23.5.3 Lateral and vertical components
The design of connections to panels for concentrated loads or reactions shall take into account lateral and vertical components in accordance with Clause 11.6.4.

23.5.4 Tributary width for vertical and lateral loads
For panels with concentrated vertical and lateral reactions at the bottom of the panel, the tributary width assumed for vertical and lateral loading shown in Figure 23.2 and the total factored axial load and moment shall be carried only by the design width. For panels with continuous lateral support at the top and bottom of the panel, the factored moment at the design cross-section shall be assumed to be uniformly distributed across the full panel width.

23.5.5 Concentrated loads or reactions
Panels subjected to concentrated loads or reactions shall be designed in accordance with Clause 11.4.

23.6 Shear

23.6.1 In-plane shear

23.6.1.1
Where tilt-up panels are used as shear walls, analysis of the panels shall include the effects of in-plane stresses, local buckling, roof diaphragm connections, and panel stability. The connections between panels shall be designed to provide nominal ductility.

23.6.1.2
The design for factored shear forces in the plane of the panel shall meet the requirements of Clause 11.3.

23.6.2 Out-of-plane shear
The design for shear forces due to loads acting perpendicular to the face of the panel shall meet the requirements of Clause 11.3.

23.7 Lifting stresses

23.7.1 General
The stresses imposed on a panel during lifting shall be limited to ensure that the performance of the erected panel is not impaired.

23.7.2 Elastic — Uncracked analysis
Analysis of tilt-up panels during the lifting operation shall be based on elastic uncracked section properties using specified loads. The effects of suction between the panel and the floor and impact loads from crane equipment shall be considered.

Annex A (informative) ***Excerpts from CSA A23.1-04,*** **Concrete materials and methods of concrete construction**

Notes:

(1) *This Annex is not a mandatory part of this Standard.*

(2) *A number of clauses from CSA A23.1 that are especially important to design engineers are reprinted in this Annex with their original numbering. It is expected that CSA A23.1 will be revised during the life of this Standard, resulting in minor changes to or renumbering of clauses. If this occurs, users should refer to the revised Standard. Users should also check CSA A23.4 and CAN/CSA-S413 to determine whether the requirements of those two Standards affect the applicability of the clauses reprinted in this Annex.*

(3) *This Annex reprints only those portions of Clause 2 of CSA A23.1 applicable to the other clauses reprinted in this Annex.*

2 Reference publications

This Standard and CSA A23.2 refer to the following publications, and where such reference is made, it shall be to the edition listed below, including all amendments published thereto.

CSA (Canadian Standards Association)

A23.3-94 (R2000)
Design of Concrete Structures

CAN/CSA-S6-00
Canadian Highway Bridge Design Code

CAN/CSA-S413-94 (R2000)
Parking Structures

S478-95 (R2001)
Guideline on Durability in Buildings

S806-02
Design and Construction of Building Components with Fibre-Reinforced Polymers

W59-03
Welded Steel Construction (Metal-Arc Welding)

W186-M1990 (R2002)
Welding of Reinforcing Bars in Reinforced Concrete Construction

ANSI/AWS (American National Standards Institute/American Welding Society)

D1.1:2004
Structural Welding Code — Steel

NRCC (National Research Council Canada)

National Building Code of Canada, 1995

PCA (Portland Cement Association)

IS001.08T, 2001
Effects of Substances on Concrete and Guide to Protective Treatments

4 Materials and concrete properties

4.1 Requirements for concrete and alternative methods for specifying concrete

4.1.1 Durability requirements

4.1.1.1 General

4.1.1.1.1
Concrete that will be subjected in service to weathering, sulphate attack, a corrosive environment, or any other process of deterioration covered by this Standard shall meet the requirements of Clauses 4.1.1.1 to 4.1.1.9 and 7.4, and Tables 1 to 4 and 20 as appropriate.

6.6 Fabrication and placement of reinforcement

6.6.1 General
The sizes and spacing of the reinforcement and its concrete cover shall be as shown on the construction drawings.

6.6.2 Hooks and bends

6.6.2.1 General
Unless otherwise stated on the construction drawings, fabrication and detailing of hooks shall be as specified in Clauses 6.6.2.2 to 6.6.2.5.

6.6.2.2 Standard hooks
The term "standard hook" as used herein shall mean
(a) a semicircular bend plus an extension of at least four bar diameters but not less than 60 mm at the free end of the bar;
(b) a 90° bend plus an extension of at least 12 bar diameters at the free end of the bar; or
(c) for stirrup and tie anchorage only, either a 90° or 135° bend plus an extension of at least six bar diameters but not less than 60 mm at the free end of the bar. Hooks for stirrups or ties shall have a 135° bend, unless the concrete surrounding the hook is restrained from spalling. (See CSA A23.3.)

6.6.2.3 Minimum bend diameter
The diameter of the bend measured on the inside of the bar for standard hooks, except stirrup and tie hooks, shall be not less than the values set out in Table 16.

6.6.2.4 Stirrup and tie hooks

6.6.2.4.1
The inside diameter of bends and 90° hooks for stirrups and ties shall be not less than four bar diameters.

6.6.2.4.2
The inside diameter of 135° hooks shall be not less than 20 mm, four bar diameters, or the diameter of the bar enclosed by the hook, whichever is the greatest.

6.6.2.4.3
The inside diameter of bends in welded wire fabric for stirrups or ties shall be not less than four wire diameters. Bends with an inside diameter less than eight wire diameters shall be not less than four wire diameters from the nearest welded intersection.

6.6.2.5 Bending

6.6.2.5.1
All bars shall be bent at temperatures between 10 °C and 100 °C, unless otherwise permitted by the owner.

6.6.2.5.2
No bars partially embedded in concrete shall be field bent except as shown on the drawings or as permitted by the owner.

6.6.2.5.3
The bending tolerances shall be sufficiently accurate to comply with the placing and protection tolerances stipulated in Clause 6.6.7.

6.6.3 Spirals

6.6.3.1
The size and spacing of spirals shall be as shown on the construction drawings.

6.6.3.5
Anchorage of spiral reinforcement shall be provided by 1-1/2 extra turns of spiral rod or wire at each end of the spiral unit.

6.6.3.6
Splices in spirals shall have a minimum 50 bar diameter lap plus a 90° hook around a longitudinal bar at the free end or shall be welded in accordance with CSA W186.

6.6.3.7
The reinforcing spiral shall extend from the floor level in any storey or from the top of the footing to the level of the lowest horizontal reinforcement in the slab, drop panel, or beam above.

6.6.3.8
Where beams or brackets are not present on all sides of a column, ties shall extend above the termination of the spiral to the bottom of the slab or drop panel.

6.6.3.9
In a column with a capital, the spiral shall extend to a plane at which the diameter or width of the capital is twice that of the column.

6.6.4 Ties

6.6.4.1
The size, spacing, and arrangement of ties shall be as shown on the construction drawings. When welded wire mesh of random length is used as tie reinforcement, the required splice length shall be indicated on the drawings.

6.6.5 Spacing of reinforcement

6.6.5.1
The spacing of bars shall be as shown on the construction drawings.

6.6.5.2
The clear distance between parallel bars or parallel bundles of bars shall be not less than 1.4 times the bar diameter, not less than 1.4 times the nominal maximum size of the coarse aggregate, and not less than 30 mm. This clear distance shall apply to the distance between a contact lap splice and adjacent splices or bars.

6.6.5.3
Where parallel reinforcement is placed in two or more layers, the bars in the upper layer shall be placed directly above those in the bottom layer.

6.6.5.6
Spacing of post-tensioning ducts shall be as specified in Clause 6.8.

6.6.6 Concrete cover

6.6.6.1 General
Concrete cover shall be measured from the concrete surface to the nearest deformation (or surface, for smooth bars or wires) of the reinforcement. Reinforcement includes ties, stirrups, and main reinforcement. For textured architectural surfaces, concrete cover shall be measured from the deepest point of the textured surface.

6.6.6.2 Specified cover for reinforced and prestressed concrete

6.6.6.2.1
The specified cover for reinforcement shall be based on consideration of life expectancy, exposure conditions, protective systems, maintenance, and the consequences of corrosion.

Notes:

(1) *The desired service life should be established early in the design process. See CSA S478.*

(2) *Requirements for corrosion protection can be influenced by the ease of access for inspection and repair and the feasibility and cost of repair or replacement.*

(3) *Service life can be improved by*
- *(a) increasing the cover and the duration of moist curing;*
- *(b) reducing the water-to-cementing materials ratio;*
- *(c) adding supplementary cementing materials, corrosion inhibitors, or membranes; and*
- *(d) improving drainage.*

(4) *As the positioning of reinforcement is not exact, in some cases it is advisable to increase the specified cover to ensure adequate protection. Service life can be extended by reducing the variability in placement of reinforcement.*

6.6.6.2.2
The specified cover for fibre-reinforced polymer bars, grids, and tendons in prestressed and reinforced concrete shall be in accordance with CSA S806.

6.6.6.2.3
The specified cover for steel reinforcement, tendon sheaths, and ducts in prestressed and reinforced concrete shall be not less than the larger of the values in Table 17.

Note: *See Clause 6.6.8 for tolerances of concrete cover and Clauses 6.8.2.4 and 6.8.2.13 for additional cover requirements for prestressing elements.*

6.6.6.3 Cover for fire resistance
Where a structural concrete member is required to have a fire-resistant rating, the minimum cover for reinforcement shall be specified by the owner.
Note: *Information can be found in the NRCC* National Building Code of Canada, *Annex D.*

6.6.8 Tolerances for location of reinforcement
Unless otherwise specified by the owner, reinforcement, prestressing steel, and post-tensioning ducts shall be placed within the following tolerances:
(a) concrete cover: ±12 mm (but the concrete cover shall in no case be reduced by more than 1/3 of the specified cover);
(b) where the depth of a flexural member, the thickness of a wall, or the smallest dimension of a column is
    (i) 200 mm or less: ±8 mm;
    (ii) larger than 200 mm but less than 600 mm: ±12 mm; and
    (iii) 600 mm or larger: ±20 mm;
(c) lateral spacing of bars: ±30 mm;
(d) longitudinal location of bends and ends of bars: ±50 mm; and
(e) longitudinal location of bends and ends of bars at discontinuous ends of members: ±20 mm.
Note: *Where reinforcement is added to help provide a more rigid reinforcing mat or cage, as for instance in prefabricated reinforcing cages, such additional reinforcement is not subject to the tolerances of this Clause, except for the minimum cover requirements.*

6.6.10 Welding of reinforcement

6.6.10.1
Welding of reinforcement shall conform to the requirements of CSA W186.

6.6.10.2
Tack welding of reinforcing bars shall be performed in accordance with CSA W186.

6.7 Fabrication and placement of hardware and other embedded items

6.7.1 General
Clause 6.7 covers the fabrication and placement of hardware for concrete building structures that have been designed in accordance with CSA A23.3. The details and location of this hardware shall be shown on the construction drawings.
Note: *For reinforced concrete structures other than buildings, the owner should show clearly on the drawings and specifications any departures from the requirements of Clauses 6.7.2 to 6.7.5.*

6.7.3 Tolerances for placing anchor bolts and hardware

6.7.3.1
Unless otherwise specified by the owner, the location of anchor bolts and embedded items shall not vary from the dimensions shown on the erection drawings by more than the following (see also Figure 3):
(a) 3 mm centre-to-centre of any two bolts within an anchor bolt group, where an anchor bolt group is defined as the set of anchor bolts that receives a single fabricated steel or precast concrete member;
(b) 8 mm centre-to-centre of adjacent anchor bolt groups;
(c) a maximum accumulation of 8 mm per 30 m along the established column line of multiple anchor bolt groups, but not to exceed a total of 30 mm. The established column line is the actual field line most representative of the centres of the as-built anchor bolt groups along a line of columns; and
(d) 8 mm from the centre of any anchor bolt group to the established column line through that group.

The tolerances of Items (b), (c), and (d) apply to offset dimensions, as shown on the construction drawings and measured perpendicular to the nearest column line.

6.7.3.2
Vertical alignment variations for anchor bolts shall not exceed 3 mm or 1 mm in 40 mm, whichever is larger.

6.7.3.3
Slope variations for hardware serving as bearing plates shall not exceed 1 mm in 40 mm, with a maximum of 3 mm for plates having side dimensions less than 300 mm, and a maximum of 5 mm for plates having side dimensions of 300 mm or larger.

6.7.4 Welding of hardware

6.7.4.1
Welding of steel hardware shall conform to the requirements of CSA W59.
Note: *Welding procedures should be such that no damage to the concrete will result.*

6.7.4.2
Welding of reinforcing bars to hardware shall conform to the requirements of CSA W186.

6.7.4.3
Material and equipment for stud welding of bars and anchors shall be compatible and shall be used in accordance with the recommendations of the manufacturers of the material and equipment.
Note: *See the Supplement to ANSI/AWS D1.1.*

6.8 Post-tensioning

6.8.2 Unbonded tendons

6.8.2.4.1
In corrosive environments, the concrete cover to the sheath shall be not less than 50 mm.

6.8.2.4.2
The concrete cover to the anchorage measured in a direction perpendicular to the tendon shall be not less than 40 mm.

Table 1
Definitions of C, F, N, A, and S classes of exposure
(See Clauses 4.1.1.1.1, 4.1.1.5, 4.4.4.1.1.1, 4.4.4.1.1.2, 6.6.7.5.1, and 8.4.1.2, and Table 2 [of CSA A23.1].)

C-XL	Structurally reinforced concrete exposed to chlorides or other severe environments with or without freezing and thawing conditions, with higher durability performance expectations than the C-1, A-1, or S-1 classes.
C-1	Structurally reinforced concrete exposed to chlorides with or without freezing and thawing conditions. Examples: bridge decks, parking decks and ramps, portions of marine structures located within the tidal and splash zones, concrete exposed to seawater spray, and salt water pools.
C-2	Non-structurally reinforced (i.e., plain) concrete exposed to chlorides and freezing and thawing. Examples: garage floors, porches, steps, pavements, sidewalks, curbs, and gutters.
C-3	Continuously submerged concrete exposed to chlorides but not to freezing and thawing. Examples: underwater portions of marine structures.
C-4	Non-structurally reinforced concrete exposed to chlorides but not to freezing and thawing. Examples: underground parking slabs on grade.
F-1	Concrete exposed to freezing and thawing in a saturated condition but not to chlorides. Examples: pool decks, patios, tennis courts, freshwater pools, and freshwater control structures.
F-2	Concrete in an unsaturated condition exposed to freezing and thawing but not to chlorides. Examples: exterior walls and columns.
N	Concrete not exposed to chlorides nor to freezing and thawing. Examples: footings and interior slabs, walls, and columns.
A-1	Structurally reinforced concrete exposed to severe manure and/or silage gases, with or without freeze-thaw exposure. Concrete exposed to the vapour above municipal sewage or industrial effluent, where hydrogen sulphide gas may be generated. Examples: reinforced beams, slabs, and columns over manure pits and silos, canals, and pig slats; and access holes, enclosed chambers, and pipes that are partially filled with effluents.
A-2	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure. Examples: reinforced walls in exterior manure tanks, silos, and feed bunkers, and exterior slabs.
A-3	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure in a continuously submerged condition. Concrete continuously submerged in municipal or industrial effluents. Examples: interior gutter walls, beams, slabs, and columns; sewage pipes that are continuously full (e.g., forcemains); and submerged portions of sewage treatment structures.
A-4	Non-structurally reinforced concrete exposed to moderate manure and/or silage gases and liquids, without freeze-thaw exposure. Examples: interior slabs on grade.
S-1	Concrete subjected to very severe sulphate exposures (Tables 2 and 3).
S-2	Concrete subjected to severe sulphate exposure (Tables 2 and 3).
S-3	Concrete subjected to moderate sulphate exposure (Tables 2 and 3).

Notes:
(1) *"C" classes pertain to chloride exposure.*
(2) *"F" classes pertain to freezing and thawing exposure without chlorides.*
(3) *"N" class is exposed to neither chlorides nor freezing and thawing.*
(4) *All classes of concrete shall comply with the minimum requirements of "S" class noted in Tables 2 and 3.*

Table 16
Bend diameter for standard hooks
(See Clause 6.6.2.3 [of CSA A23.1].)

	Minimum bend diameter,* mm		
	Steel grade		
Bar size	300 R	400 R or 500 R	400 W or 500 W
10	60	70	60
15	90	100	90
20	—	120	100
25	—	150	150
30	—	250	200
35	—	300	250
45	—	450†	400
55	—	600†	550

*Bend diameters shall not be reduced by more than 10% from those listed unless otherwise permitted by the owner.

†Special fabrication is required for bends exceeding 90° for bars of these sizes and grades.

Table 17
Concrete cover
(See Clauses 4.3.2.2.1 and 6.6.6.2.3 [of CSA A23.1].)

	Exposure class (see Tables 1 and 2)		
Exposure condition	N*	F-1, F-2, S-1, S-2	C-XL, C-1, C-3, A-1, A-2, A-3
Cast against and permanently exposed to earth	—	75 mm	75 mm
Beams, girders, columns, and piles	30 mm	40 mm	60 mm
Slabs, walls, joists, shells, and folded plates	20 mm	40 mm	60 mm
Ratio of cover to nominal bar diameter†	1.0	1.5	2.0
Ratio of cover to nominal maximum aggregate size	1.0‡	1.5	2.0

*This refers only to concrete that will be continually dry within the conditioned space, i.e., members entirely within the vapour barrier of the building envelope.

†The cover for a bundle of bars shall be the same as that for a single bar with an equivalent area.

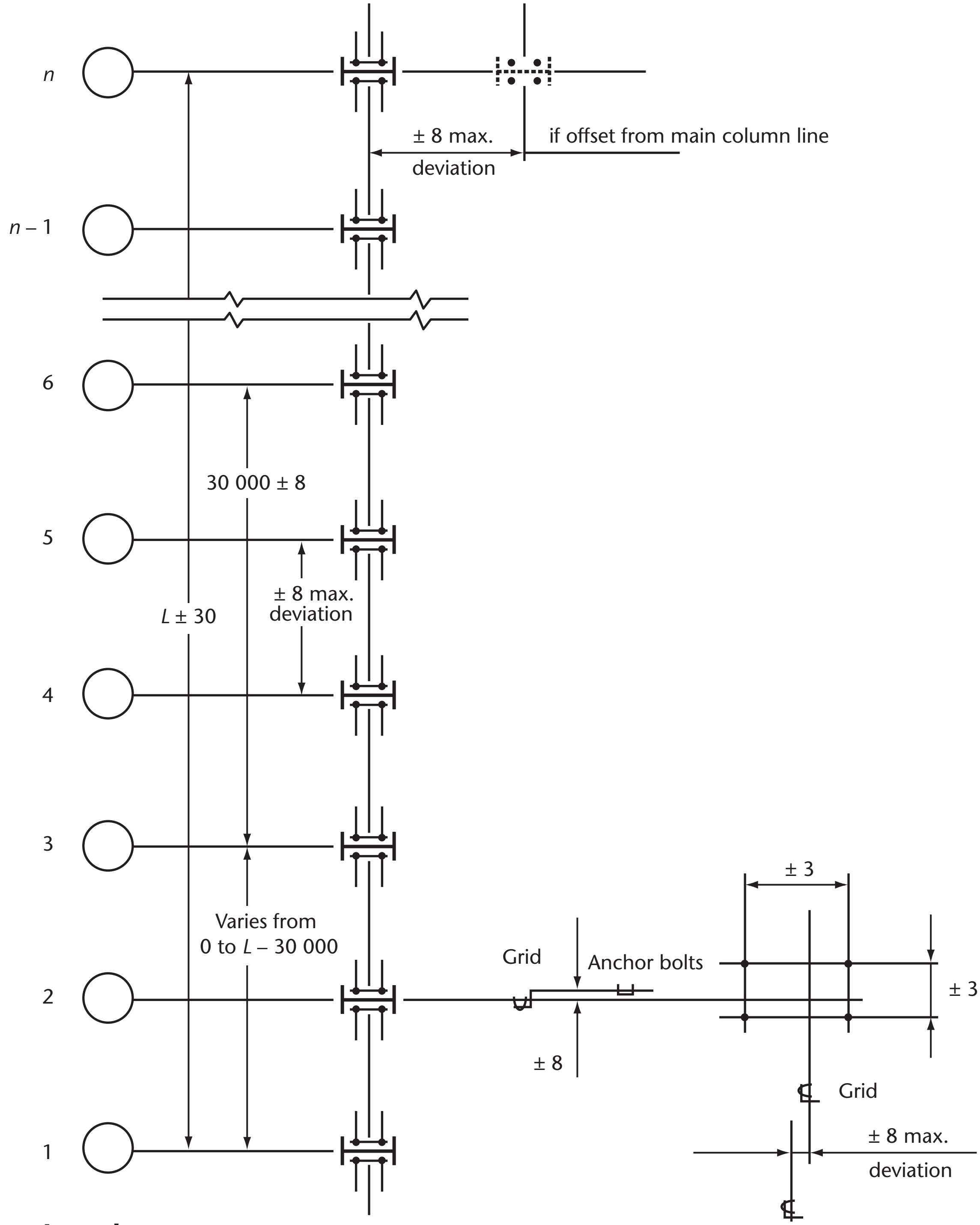
‡The specified cover from screeded surfaces shall be at least 1.5 times the nominal maximum aggregate size to reduce interference between aggregate and reinforcement where variations in bar placement result in a cover smaller than specified.

Notes:

(1) *Greater cover or protective coatings may be required for exposure to industrial chemicals, food processing, and other corrosive materials. See PCA IS001.08T.*

(2) *For information on the additional protective measures and requirements for parking structures, see CAN/CSA-S413.*

(3) *For information on the additional protective measures and requirements for bridges, see CAN/CSA-S6.*



Legend:
n = total number of columns
L = specified length between outermost anchor bolts

Figure 3
Tolerances on anchor bolt placement
(See Clause 6.7.3.1 [of CSA A23.1].)

Annex B (informative)
Rectangular two-way slab systems with stiff supports on four sides

Note: *This Annex is not a mandatory part of this Standard.*

B.1 Introduction

B.1.1
This Annex applies to rectangular two-way systems where the slab is reinforced in two directions and supported on four sides by walls or stiff beams. It may be used to determine slab thicknesses and loads on supporting beams or walls and to determine the moments and shears in slabs.

B.1.2
In this Annex, a stiff supporting beam is one in which $b_w h_b^3 / \ell_n h_s^3$ is not less than 2.0.

B.2 Symbols
The following symbols apply in this Annex:

b_w = width of beam web
C_{ad} = moment coefficient for positive dead load moment in short span
C_{al} = moment coefficient for positive live load moment in short span
$C_a neg$ = moment coefficient for negative moment in short span
C_{bd} = moment coefficient for positive dead load moment in long span
C_{bl} = moment coefficient for positive live load moment in long span
$C_b neg$ = moment coefficient for negative moment in long span
h_b = overall depth of supporting beam
h_s = overall depth of slab
ℓ_a = clear span of a two-way slab in the short direction
ℓ_b = clear span of a two-way slab in the long direction
ℓ_n = clear span of supporting beam
m = ratio of short to long span of a two-way slab, equal to ℓ_a / ℓ_b
$M_{ad} pos$ = positive dead load moment in short span
$M_{al} pos$ = positive live load moment in short span
$M_a neg$ = negative moment in short span
$M_{bd} pos$ = positive dead load moment in long span
$M_{bl} pos$ = positive live load moment in long span
$M_b neg$ = negative moment in long span
w_{df} = factored dead load per unit area
w_f = factored load per unit area
w_{lf} = factored live load per unit area

B.3 Design method

B.3.1
The minimum slab thickness should be determined in accordance with Clause 13.2, but should not be less than
(a) 100 mm;
(b) the perimeter of the slab divided by 140, in the case of slabs discontinuous on one or more edges; or
(c) the perimeter of the slab divided by 160, in the case of fully continuous slabs.

B.3.2
A two-way slab should be considered as consisting of strips in each direction, as follows:
(a) a middle strip, one-half of a panel in width, symmetrical about the panel centreline and extending through the panel in the direction in which moments are considered; and
(b) a column strip, one-half of a panel in width, occupying the two quarter-panel areas outside the middle strip.

B.3.3
Critical sections for moment should be assumed to be as follows:
(a) for negative moment, along the edges of the panel at the faces of the supports; and
(b) for positive moment, along the centrelines of the panels.

B.3.4
Negative bending moments per unit width for the middle strips should be computed in accordance with the following equations and the coefficients specified in Table B.1:

(a) $M_a neg = C_a neg\, w_f \ell_a^2$ **(B-1)**

(b) $M_b neg = C_b neg\, w_f \ell_b^2$ **(B-2)**

B.3.5
Positive bending moments per unit width should be computed as the sum of Equations (B-3) and (B-4) for the middle strip in the short direction and Equations (B-5) and (B-6) for the middle strip in the long direction, using the coefficients specified in Table B.2. These equations are as follows:

(a) $M_{al}\, pos = C_{al}\, w_{lf} \ell_a^2$ **(B-3)**

(b) $M_{ad}\, pos = C_{ad}\, w_{df} \ell_a^2$ **(B-4)**

(c) $M_{bl}\, pos = C_{bl}\, w_{lf} \ell_b^2$ **(B-5)**

(d) $M_{bd}\, pos = C_{bd}\, w_{df} \ell_b^2$ **(B-6)**

B.3.6
The bending moments in the column strips should be two-thirds of the bending moments in the middle strip.

B.3.7
Where the ratio, *m*, of short to long span is less than 0.5, the slab should be considered a one-way slab in the short direction, but reinforcement for negative moments required for *m* equal to 0.5 should be provided in the long direction.

B.3.8
At discontinuous edges of two-way slabs, a negative moment of three-quarters of the positive moment should be assumed.

B.3.9
In all cases, special reinforcement should be provided at exterior corners in accordance with Clause 13.12.5.

B.3.10
Where the negative moment on one side of a support is less than 80% of that on the other side, the difference should be distributed between the two slabs in proportion to their relative stiffnesses.

B.3.11
The shear stresses in the slabs should be computed on the assumption that the load, w_f, is distributed to the supports in accordance with Clause B.4.

B.4 Loads on slab supports

B.4.1
The loads on the supporting beams of a two-way rectangular panel may be assumed to be the load within the tributary areas of the panel bounded by the intersection of 45° lines from the corners and the median line of the panel parallel to the long side.

B.4.2
The bending moments in the supporting beams may be determined for design purposes by using an equivalent uniform load per unit length of beam for each panel supported, as follows:

(a) for the short span:

$$\frac{w_f \ell_a}{3} \tag{B-7}$$

(b) for the long span:

$$\frac{w_f \ell_a}{3} \times \frac{(3 - m^2)}{2} \tag{B-8}$$

Table B.1
Coefficients for negative moments
(See Clause B.3.4.)

$m = \ell_a/\ell_b$	Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_a neg$	—	0.045	—	0.050	0.075	0.071	—	0.033	0.061
	$C_b neg$	—	0.045	0.076	0.050	—	—	0.071	0.061	0.033
0.95	$C_a neg$	—	0.050	—	0.055	0.079	0.075	—	0.038	0.065
	$C_b neg$	—	0.041	0.072	0.045	—	—	0.067	0.056	0.029
0.90	$C_a neg$	—	0.055	—	0.060	0.080	0.079	—	0.043	0.068
	$C_b neg$	—	0.036	0.070	0.040	—	—	0.062	0.052	0.025
0.85	$C_a neg$	—	0.060	—	0.066	0.082	0.083	—	0.049	0.072
	$C_b neg$	—	0.031	0.065	0.034	—	—	0.057	0.046	0.021
0.80	$C_a neg$	—	0.065	—	0.071	0.084	0.086	—	0.055	0.075
	$C_b neg$	—	0.026	0.061	0.029	—	—	0.051	0.041	0.017
0.75	$C_a neg$	—	0.069	—	0.076	0.085	0.088	—	0.061	0.078
	$C_b neg$	—	0.022	0.056	0.024	—	—	0.044	0.036	0.014
0.70	$C_a neg$	—	0.074	—	0.081	0.086	0.091	—	0.068	0.081
	$C_b neg$	—	0.017	0.050	0.019	—	—	0.038	0.029	0.011
0.65	$C_a neg$	—	0.077	—	0.085	0.087	0.093	—	0.074	0.083
	$C_b neg$	—	0.014	0.043	0.015	—	—	0.031	0.025	0.008
0.60	$C_a neg$	—	0.081	—	0.089	0.088	0.095	—	0.080	0.085
	$C_b neg$	—	0.010	0.035	0.011	—	—	0.024	0.018	0.006
0.55	$C_a neg$	—	0.084	—	0.092	0.089	0.096	—	0.085	0.086
	$C_b neg$	—	0.007	0.028	0.008	—	—	0.019	0.014	0.005
0.50	$C_a neg$	—	0.086	—	0.094	0.090	0.097	—	0.089	0.088
	$C_b neg$	—	0.006	0.022	0.006	—	—	0.014	0.010	0.003

Notes:

(1) —— *means that supports are free to rotate.*

(2) ═══ *means that supports are fixed against rotation.*

Table B.2
Coefficients for live and dead load positive moments

(See Clause B.3.5.)

$m = \ell_a/\ell_b$	Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	C_{al}	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	C_{ad}	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	C_{bl}	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
	C_{bd}	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
0.95	C_{al}	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	C_{ad}	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	C_{bl}	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
	C_{bd}	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
0.90	C_{al}	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	C_{ad}	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	C_{bl}	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
	C_{bd}	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	C_{al}	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	C_{ad}	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	C_{bl}	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
	C_{bd}	0.026	0.012	0.023	0.019	0.011	0.017	0.025	0.017	0.013
0.80	C_{al}	0.055	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	C_{ad}	0.055	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	C_{bl}	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
	C_{bd}	0.023	0.011	0.020	0.016	0.009	0.014	0.022	0.025	0.010
0.75	C_{al}	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	C_{ad}	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	C_{bl}	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.014
	C_{bd}	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	C_{al}	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	C_{ad}	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	C_{bl}	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.012
	C_{bd}	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.65	C_{al}	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	C_{ad}	0.074	0.032	0.054	0.050	0.036	0.053	0.065	0.044	0.034
	C_{bl}	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
	C_{bd}	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	C_{al}	0.081	0.058	0.072	0.067	0.059	0.068	0.077	0.065	0.059
	C_{ad}	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	C_{bl}	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
	C_{bd}	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	C_{al}	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
	C_{ad}	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	C_{bl}	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
	C_{bd}	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	C_{al}	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	C_{ad}	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	C_{bl}	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004
	C_{bd}	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

Notes:

(1) —— *means that supports are free to rotate.*

(2) ═══ *means that supports are fixed against rotation.*

Annex C (informative) ***Load combinations and load factors in the National Building Code of Canada, 2005***

Notes:

(1) *This Annex is not a mandatory part of this Standard.*

(2) *This Annex provides an adapted version of portions of Subsection 4.1.3 from an unpublished draft of the* National Building Code of Canada*, 2005 (NBC). Those portions deal with load factors and load combinations. This material has been adapted in accordance with CSA editorial requirements and is included for information only.*

(3) *The load factors and load combinations presented in this Annex are discussed in Part 4 of the NBC* User's Guide *and should be used in conjunction with the resistance factors specified in Clause 8.4.*

(4) *The NBC defines the following classes of loads:*
- *(a) permanent loads such as dead loads, D, and effects of prestress, P;*
- *(b) variable loads due to use and occupancy, L, wind loads, W, and snow loads, S;*
- *(c) rare loads such as earthquake loads, E; and*
- *(d) imposed deformations, T (see Clause 8.2.2).*

(5) *The equations used to compute the loads S, W, and E for snow, wind, and earthquake in the NBC include importance factors I_s, I_w, and I_e, which are a function of the use and occupancy of the building.*

(6) *The following symbols are used in this Annex:*

D = *permanent loads due to dead load, or related internal moments and forces*

E = *earthquake loads, or related internal moments and forces*

H = *load due to lateral earth pressure, including groundwater, and related internal moments and forces*

L = *variable load due to intended use and occupancy, including loads due to cranes and pressure of liquids in containers, or related moments or forces*

P = *effects of prestress, including secondary moments due to prestress*

R = *nominal resistance of a member, connection, or structure based on the dimensions and on the specified properties of the structural materials*

S = *variable loads due to ice, rain, and snow (including associated rain)*

T = *effects of imposed deformations due to moisture changes, shrinkage, creep, temperature, and ground settlement, or combinations thereof*

W = *variable loads due to wind, or related internal moments and forces*

ϕ = *resistance factor applied to a specified material property or to the resistance of a member, connection, or structure, which for the limit state under consideration takes into account the variability of dimensions and material properties, quality of work, type of failure, and uncertainty in the prediction of resistance*

C.1 Limit states design

Note: *See Appendix A of the NBC.*

C.1.1 Definitions

The following definitions apply in this Annex:

Companion load — a specified variable load that accompanies the principal load in a given load combination.

Companion-load factor — a factor that, when applied to a companion load in the load combination, gives the probable magnitude of a companion load acting simultaneously with the factored principal load.

Effects — forces, moments, deformations, or vibrations that occur in the structure.

Factored load — the product of a specified load and its principal-load factor or companion-load factor.

Factored resistance — the product of nominal resistance, *R*, and the applicable resistance factor, ϕ.

Importance factor — a factor applied in Subsections 4.1.6 to 4.1.8 of the NBC to obtain the specified load, to account for the consequences of failure as related to the limit state and the use and occupancy of the building.

Limit states — those conditions of a building structure in which the building ceases to fulfill the function for which it was designed.
Note: *Those states concerning safety are called ultimate limit states (ULS) and include exceeding the load-carrying capacity, overturning, sliding, and fracture. Those states that restrict the intended use and occupancy of the building are called serviceability limit states (SLS) and include deflection, vibration, permanent deformation, and local structural damage such as cracking. Those limit states that represent failure under repeated loading are called fatigue limit states.*

Principal load — the specified variable load or rare load that dominates in a given load combination.

Principal-load factor — a factor applied to the principal load in the load combination to account for the variability of the load and load pattern and analysis of its effects.

Specified loads (*D, E, H, L, P, S, T,* and *W*) — the loads specified in Note (6) of the preliminary Notes to this Annex.

C.1.2 Strength and stability

C.1.2.1

A building and its structural components shall be designed to have sufficient strength and stability so that the factored resistance, ϕR, is greater than or equal to the effect of factored loads, where the effect of factored loads shall be determined in accordance with Clause C.1.2.2.

C.1.2.2

The effect of factored loads for a building or structural component shall be determined in accordance with the load combinations specified in Table C.1 and the provisions of Clause C.1.2, the applicable combination being that which results in the most critical effect. (See Appendix A of the NBC.)

C.1.2.3

Where the effects due to lateral earth pressure, H, restraint effects from prestress, P, and imposed deformation, T, affect the structural safety, they shall be taken into account in the calculations, i.e., H with a load factor of 1.5, P with a load factor of 1.0, and T with a load factor of 1.25. (See Appendix A of the NBC.)

Table C.1
Load combinations for ultimate limit states
(See Clauses 8.3.2, C.1.2.2, and C.1.2.4 to C.1.2.8.)

	Load combination*†	
Case	Principal loads	Companion loads
1	$1.4D$	—
2	($1.25D$‡ or $0.9D$§) + $1.5L$**	$0.5S$†† or $0.4W$
3	($1.25D$‡ or $0.9D$§) + $1.5S$	$0.5L$††,‡‡ or $0.4W$
4	($1.25D$‡ or $0.9D$§) + $1.4W$	$0.5L$‡‡ or $0.5S$
5	$1.0D$§ + $1.0E$§§	$0.5L$††,‡‡ + $0.25S$††

**See Clause C.1.2.2.*
†*See Clause C.1.2.3.*
‡*See Clause C.1.2.7.*
§*See Clause C.1.2.4.*
***See Clause C.1.2.5.*
††*See Article 4.1.5.5 of the NBC.*
‡‡*See Clause C.1.2.6.*
§§*See Clause C.1.2.8.*

Notes:
(1) *This Table corresponds to Table 4.1.3.2 of the NBC.*
(2) *The factored load combinations in this Table each include one or more permanent loads, one principal variable load that dominates a given load combination, and one or more companion variable loads that have a magnitude likely to occur in combination with the given principal variable load when that principal variable load acts on the structure.*

C.1.2.4
Except as provided in Sentence 4.1.8.16.(1) of the NBC, the counteracting factored dead load, $0.9D$ in the load combinations specified in Cases 2 to 4 of Table C.1 and $1.0D$ in the load combination specified in Case 5 of Table C.1, shall be used when dead load acts to resist overturning, uplift, sliding, and failure due to stress reversal, and to determine anchorage requirements and factored member resistances. (See Appendix A of the NBC.)

C.1.2.5
The principal-load factor 1.5 for live load, L, in Table C.1 may be reduced to 1.25 for liquids in tanks.

C.1.2.6
The companion-load factor 0.5 for live load, L, in Table C.1 shall be increased to 1.0 for storage occupancies, and for equipment areas and service rooms in Table 4.1.5.3 of the NBC.

C.1.2.7
The load factor 1.25 for dead load, D, for soil, superimposed earth, plants, and trees in Table C.1 shall be increased to 1.5, except that when the soil depth exceeds 1.2 m, the factor may be reduced to $1 + 0.6/h_s$, but not less than 1.25, where h_s is the depth of soil in metres supported by the structure.

C.1.2.8
Earthquake load, E, in the load combination specified in Case 5 of Table C.1 includes horizontal earth pressure due to earthquake determined in accordance with Sentence 4.1.8.16.(4) of the NBC.

C.1.2.9
Provision shall be made to ensure adequate stability of a structure as a whole and adequate lateral, torsional, and local stability of all structural parts.

C.1.2.10
Sway effects produced by vertical loads acting on the structure in its displaced configuration shall be taken into account in the design of buildings and their structural members.

C.1.3 Serviceability
A building and its structural components shall be checked for serviceability limit states as defined in Clause 4.1.3.1.(1)(a) of the NBC under the effect of service loads for serviceability criteria specified or recommended in Articles 4.1.3.5 and 4.1.3.6 of the NBC and in the Standards listed in Section 4.3 of the NBC.

Annex D (informative) ***Anchorage***

Note: *This informative (non-mandatory) Annex has been written in normative (mandatory) language to facilitate adoption where users of the Standard or regulatory authorities wish to adopt it formally as additional requirements to this Standard.*

D.1 Introduction

D.1.1
This Annex specifies design requirements for anchors in concrete used to transmit forces to concrete elements by tension, shear, or a combination of tension and shear. The specified safety levels are intended for in-service conditions rather than for short-term handling and construction conditions.

D.1.2
This Annex applies to both cast-in anchors and post-installed anchors (see Figure D.1). Specialty inserts, through-bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, adhesive or grouted anchors, and direct anchors such as powder- or pneumatic-actuated nails or bolts are not covered by this Annex. Reinforcement used as part of the embedment shall be designed in accordance with the applicable clauses of this Standard.

D.1.3
Headed studs and headed bolts that have a geometry that has been demonstrated to result in a pullout resistance in uncracked concrete equal to or exceeding 1.4 N_{pr} (where N_{pr} is as specified in Equation (D-17)) are covered by this Annex. Hooked bolts that have a geometry that has been demonstrated to result in a pullout resistance without the benefit of friction in uncracked concrete equal or to exceeding 1.4 N_{pr} (where N_{pr} is as specified in Equation (D-18)) are also covered by this Annex, as are post-installed anchors that meet the assessment requirements of ACI 355.2/355.2R. The suitability of post-installed anchors for use in concrete shall be demonstrated by the ACI 355.2/355.2R prequalification tests.

D.1.4
Load applications that are predominantly high cycle, fatigue, or impact are not covered by this Annex.

D.2 Definitions
The following definitions apply in this Annex:

5% fractile — 90% confidence that there is a 95% probability of the actual strength exceeding the nominal strength.

Anchor — a steel element cast into concrete or post-installed into a hardened concrete member and used to transmit applied forces. Examples include straight bolts, hooked bolts (J- or L-bolts), headed studs, expansion anchors, undercut anchors, and inserts.

Anchor group — a number of anchors of approximately equal effective embedment depth, with each anchor spaced less than three times its embedment depth from one or more adjacent anchors.

Anchor pullout strength — the strength corresponding to the anchoring device or a major component of the device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete (see Figure D.2).

Attachment — the structural assembly, external to the surface of the concrete, that transmits loads to the anchor.

Brittle steel element — an element with a tensile test elongation of less than 14% over a 50 mm gauge length.

Cast-in anchor — a headed bolt or hooked bolt installed before concrete is placed.

Concrete breakout strength — the strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member (see Figures D.2 and D.3).

Concrete pryout strength — the strength corresponding to formation of a concrete spall behind a short, stiff anchor with an embedded base that is displaced in the direction opposite to the applied shear force (see Figures D.2 and D.3).

Distance sleeve — a sleeve that encases the centre part of an undercut anchor, a torque-controlled expansion anchor, or a displacement-controlled expansion anchor but does not expand.

Ductile steel element — an element with a tensile test elongation of at least 14% over a 50 mm gauge length. A steel element meeting the requirements of CSA G40.21 or ASTM A 307 can be considered ductile.

Edge distance — the distance from the edge of the concrete surface to the centre of the nearest anchor.

Effective embedment depth — the overall depth through which the anchor transfers force to the surrounding concrete. The effective embedment depth is normally the depth of the failure surface in tension applications. For cast-in headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head (see Figure D.1).

Expansion anchor — a post-installed anchor inserted into hardened concrete that transfers loads into the concrete by direct bearing, friction, or both. Expansion anchors can be torque controlled (where the expansion is achieved by a torque acting on the screw or bolt) or displacement controlled (where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug).

Expansion sleeve — the outer part of an expansion anchor that is forced outward by the centre part by applied torque or impact to bear against the sides of the predrilled hole.

Headed stud — a headed steel anchor that meets the requirements of CSA W59 or AWS D1.1/D1.1M and is affixed to a plate or similar steel attachment by stud arc welding. The underside of the plate or steel attachment is assumed to be cast flush with the concrete surface.

Hooked bolt — a cast-in anchor that is anchored mainly by mechanical interlock from the 90° bend (L-bolt) or 180° bend (J-bolt) at its lower end and has a minimum e_h of $3d_o$.

Post-installed anchor — an anchor installed in hardened concrete. Expansion anchors and undercut anchors are examples of post-installed anchors.

Projected area — the area on the free surface of a concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

Side-face blowout strength — the strength of anchors with deeper embedment but thinner side cover corresponding to concrete spalling on the side face around the embedded head while no major breakout occurs at the top concrete surface (see Figures D.2 and D.3).

Specialty insert — a predesigned and prefabricated cast-in anchor specifically designed for attachment of bolted or slotted connections. Specialty inserts are often used for handling, transportation, and erection, but also for anchoring structural elements. They are not covered by this Annex.

Supplementary reinforcement — reinforcement designed to tie a potential concrete failure prism to a structural member.

Undercut anchor — a post-installed anchor that derives its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. The undercutting is achieved with a special drill before installation of the anchor or by the anchor itself during its installation.

D.3 Symbols

The following symbols apply in this Annex:

A_{bh} = bearing area of the head of stud or anchor bolt

A_N = projected concrete failure area of an anchor or group of anchors, for calculation of resistance in tension, as defined in Clause D.6.2.1. A_N is not taken greater than nA_{No} (see Figure D.6)

A_{No} = projected concrete failure area of one anchor, for calculation of resistance in tension, when not limited by edge distance or spacing, as specified in Clause D.6.2.1 (see Figure D.5)

A_{se} = effective cross-sectional area of anchor

A_V = projected concrete failure area of an anchor or group of anchors, for calculation of resistance in shear, as defined in Clause D.7.2.1. A_V is not taken greater than nA_{Vo} (see Figure D.10)

A_{Vo} = projected concrete failure area of one anchor, for calculation of resistance in shear, when not limited by corner influences, spacing, or member thickness, as specified in Clause D.7.2.1 (see Figure D.9)

c = distance from centre of an anchor shaft to the edge of concrete

c_{ac} = critical edge distance specified in Clause D.9.7

$c_{a,min}$ = minimum edge distance to preclude premature splitting failure of post-installed anchors, as determined from ACI 355.2/355.2R

c_{max} = the largest edge distance

c_{min} = the smallest edge distance

c_1 = distance from the centre of an anchor shaft to the edge of concrete in one direction. Where shear force is applied to anchor, c_1 is in the direction of the shear force (see Figure D.9)

c_2 = distance from centre of an anchor shaft to the edge of concrete in the direction orthogonal to c_1

d_o = outside diameter of anchor or shaft diameter of headed stud, headed anchor bolt, or hooked anchor

d_o' = value substituted for d_o when an oversized anchor is used

e_h = distance from the inner surface of the shaft of a J-bolt or L-bolt to the outer tip of the J- or L-bolt

e_N = actual eccentricity of a normal force on an attachment

e_N' = eccentricity of normal force on a group of anchors. The distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension is always positive (see Figure D.8)

e_V = actual eccentricity of a shear force on an attachment

e_V' = eccentricity of shear force on a group of anchors; the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear (see Figure D.12)

f_c' = specified compressive strength of concrete

f_r = modulus of rupture of concrete

f_t = calculated tensile stress in a region of a member

f_{ut} = specified tensile strength of anchor steel

f_y = specified yield strength of anchor steel

F_a = acceleration-based site coefficient, as specified in the *National Building Code of Canada*

h = thickness of member in which an anchor is anchored, measured parallel to anchor axis

h_{ef} = effective anchor embedment depth (see Figure D.1)

I_E = earthquake importance factor of the structure, as specified in the *National Building Code of Canada*

k = coefficient for factored concrete breakout resistance in tension

k_{cp} = coefficient for pryout resistance

ℓ = load-bearing length of anchor for shear, not to exceed $8d_o$

= h_{ef} for anchors with a constant stiffness over the full length of the embedded section, such as headed studs or post-installed anchors with one tubular shell over the full length of the embedment depth

= $2d_o$ for torque-controlled expansion anchors with a distance sleeve separated from the expansion sleeve

n = number of anchors in a group

N_{br} = factored concrete breakout resistance in tension of a single anchor in cracked concrete, as defined in Clause D.6.2.2

N_{cbgr} = factored concrete breakout resistance in tension of a group of anchors, as specified in Clause D.6.2.1

N_{cbr} = factored concrete breakout resistance in tension of a single anchor, as specified in Clause D.6.2.1

N_{cpr} = factored pullout resistance in tension of a single anchor, as specified in Clause D.6.3.1

N_f = factored tensile load

N_{pr} = factored pullout resistance in tension of a single anchor in cracked concrete, as specified in Clause D.6.3.4 or D.6.3.5

N_r = factored resistance in tension

N_{sbgr} = factored side-face blowout resistance of a group of anchors

N_{sbr} = factored side-face blowout resistance of a single anchor

N_{sr} = factored resistance of a single anchor or group of anchors in tension as governed by the steel resistance, as specified in Clauses D.6.1.1 and D.6.1.2

R = resistance modification factor

s = anchor centre-to-centre spacing

s_o = centre-to-centre spacing of the outer anchors along the edge in a group

$S_a(0.2)$ = damped spectral response acceleration for a period of 0.2 s, as specified in the *National Building Code of Canada*

t = thickness of washer or plate

V_{br} = factored concrete breakout resistance in shear of a single anchor in cracked concrete, as specified in Clause D.7.2.2 or D.7.2.3

V_{cbgr} = factored concrete breakout resistance in shear of a group of anchors, as specified in Clause D.7.2.1

V_{cbr} = factored concrete breakout resistance in shear of a single anchor, as specified in Clause D.7.2.1

V_{cpgr} = factored concrete pryout resistance of a group of anchors, as specified in Clause D.7.3

V_{cpr} = factored concrete pryout resistance of a single anchor, as specified in Clause D.7.3

V_f = factored shear force

V_r = factored shear resistance

V_{sr} = factored resistance in shear of a single anchor as governed by the steel resistance, as specified in Clauses D.7.1.1 and D.7.1.2

λ = factor to account for low-density concrete

ϕ_c = concrete material resistance factor for concrete

ϕ_s = steel embedment material resistance factor for reinforcement

$\psi_{c,N}$ = modification factor for resistance in tension to account for cracking, as specified in Clause D.6.2.6

$\psi_{cp,N}$ = modification factor for concrete breakout resistance to account for premature splitting failure, as specified in Clause D.6.2.7

$\psi_{c,P}$ = modification factor for pullout resistance to account for cracking, as specified in Clause D.6.3.6

$\psi_{c,V}$ = modification factor for resistance in shear to account for cracking, as specified in Clause D.7.2.7

$\psi_{ec,N}$ = modification factor for resistance in tension to account for anchor groups loaded eccentrically, as specified in Clause D.6.2.4

$\psi_{ec,V}$ = modification factor for resistance in shear to account for anchor groups loaded eccentrically, as specified in Clause D.7.2.5

$\psi_{ed,N}$ = modification factor for resistance in tension to account for edge distances smaller than $1.5h_{ef}$, as specified in Clause D.6.2.5

$\psi_{ed,V}$ = modification factor for resistance in shear to account for edge distances smaller than $1.5c_1$, as specified in Clause D.7.2.6

D.4 General requirements

D.4.1 Analysis

Anchors and anchor groups shall be designed for critical effects of factored loads as determined by elastic analysis. Plastic analysis approaches may be used where factored resistance is controlled by ductile steel elements, provided that deformational compatibility is taken into account.

D.4.2 Load combinations

Anchors shall be designed for all of the factored load combinations specified in Clause 8.

D.4.3 Seismic considerations

D.4.3.1

Where load combinations include earthquake effects, the applicable additional requirements of Clauses D.4.3.2 to D.4.3.6 shall apply.

D.4.3.2

This Annex shall not apply to the design of anchors in plastic hinge zones of concrete structures under seismic loads.

D.4.3.3

In regions where $I_E F_a S_a(0.2) \geq 0.35$ and the load combinations include earthquake effects, the additional requirements of Clauses D.4.3.4 to D.4.3.6 shall apply.

D.4.3.4
Post-installed structural anchors for use as specified in Clause D.1.3 shall pass the simulated seismic tests specified in ACI 355.2/355-2R.

D.4.3.5
The factored design resistance of anchors shall be taken as 75% of the values determined in accordance with Clause D.5.1.1.

D.4.3.6
Anchors shall be designed to be governed by the tensile or shear strength of a ductile steel element unless the attachment connected to the structure by the anchor is designed so that it will undergo ductile yielding at a load level not greater than 75% of the minimum anchor design resistance.

D.4.4 Concrete density
All requirements for anchor axial tension and shear resistance shall apply to normal-density concrete. When low-density aggregate concrete is used, N_r and V_r shall be modified by multiplying all values of $\sqrt{f'_c}$ affecting N_r and V_r by λ. Linear interpolation based on the fraction of natural sand in the mix may be applied.

D.4.5 Concrete strength limit
The values of f'_c used for calculations in this Annex shall not exceed 70 MPa for cast-in anchors and 55 MPa for post-installed anchors. Testing shall be required for post-installed anchors used in concrete with f'_c greater than 55 MPa.

D.5 Resistance of structural anchors

D.5.1

D.5.1.1
The design of structural anchors shall be based on computations that satisfy the requirements of Clause D.5.2 or on test evaluation using the 5% fractile of test results for the following:
(a) steel strength of anchor in tension (Clause D.6.1);
(b) steel strength of anchor in shear (Clause D.7.1);
(c) concrete breakout resistance of anchor in tension (Clause D.6.2);
(d) concrete breakout resistance of anchor in shear (Clause D.7.2);
(e) pullout resistance of anchor in tension (Clause D.6.3);
(f) concrete side-face blowout resistance of anchor in tension (Clause D.6.4); and
(g) concrete pryout resistance of anchor in shear (Clause D.7.3).
In addition, anchors shall have the edge distances, spacings, and thicknesses for precluding splitting failure required by Clause D.9.

D.5.1.2
The following shall apply to the design of anchors, except as required by Clause D.4.3:

$$N_r \geq N_f \quad \textbf{(D-1)}$$

$$V_r \geq V_f \quad \textbf{(D-2)}$$

N_r and V_r are the lowest design resistances determined from all applicable failure modes. N_r is the lowest design resistance in tension of an anchor or anchor group, as determined from a consideration of N_{sr}, N_{cpr}, either N_{sbr} or N_{sbgr}, and either N_{cbr} or N_{cbgr}. V_r is the lowest design resistance in shear of an anchor or anchor group, as determined from a consideration of V_{sr}, either V_{cbr} or V_{cbgr}, and either V_{cpr} or V_{cpgr}.

D.5.2

D.5.2.1

The nominal resistance for any anchor or anchor group shall be based on design models that result in predictions of resistance in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal resistance shall be based on the 5% fractile of the basic individual anchor resistance. For nominal resistances related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall be accounted for. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

D.5.2.2

The effect of supplementary reinforcement provided to confine or restrain the concrete breakout may be included in the design models specified in Clause D.5.2.1.

D.5.2.3

For anchors with diameters not exceeding 50 mm and tensile embedments not exceeding 625 mm in depth, the concrete breakout resistance requirements specified in Clause D.5.2.1 shall be considered satisfied by the design procedure specified in Clauses D.6.2 and D.7.2.

D.5.3

Resistance to combined tensile and shear loads shall be considered in the design by use of an interaction expression that results in a computation of resistance in substantial agreement with the results of comprehensive tests. This requirement shall be considered satisfied by Clause D.8.

D.5.4

The resistance modification factor, *R*, specified in Clauses D.6 and D.7 shall be as follows:

(a) for an anchor governed by strength of a ductile steel element:

Tension loads	0.80
Shear loads	0.75

(b) for an anchor governed by strength of a brittle steel element:

Tension loads	0.70
Shear loads	0.65

(c) for an anchor governed by concrete breakout, blowout, pullout, or pryout strength:

	Condition A*	Condition B*
Shear loads	1.15	1.00
Tension loads		
Cast-in headed studs, headed bolts, or hooked bolts	1.15	1.00
Post-installed anchors (category determined in accordance with ACI 355.2/355.2R)		
Category 1 (low sensitivity to installation and high reliability)	1.15	1.00
Category 2 (medium sensitivity to installation and medium reliability)	1.00	0.85
Category 3 (high sensitivity to installation and lower reliability)	0.85	0.70

**Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided or where pullout or pryout strength governs.*

D.6 Design requirements for tensile loading

D.6.1 Steel resistance of anchor in tension

D.6.1.1

The factored resistance of an anchor in tension as governed by the steel, N_{sr}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.6.1.2

The factored resistance of an anchor or anchor group in tension shall not exceed

$$N_{sr} = nA_{se}\phi_s f_{ut} R \quad \textbf{(D-3)}$$

where f_{ut} shall not be taken greater than the smaller of $1.9f_y$ or 860 MPa.

In this equation the effective area, A_{se}, of a threaded anchor may be assumed to be 70% of the gross area.

D.6.2 Concrete breakout resistance of anchor in tension

D.6.2.1

The factored concrete breakout resistance of an anchor or anchor group in tension shall not exceed

(a) for a single anchor:

$$N_{cbr} = \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_{br} \quad \textbf{(D-4)}$$

(b) for an anchor group:

$$N_{cbgr} = \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_{br} \quad \textbf{(D-5)}$$

In these equations, N_{br} is the factored concrete breakout resistance value for a single anchor in tension in cracked concrete. In these equations, A_N is the projected area of the failure surface for the anchor or anchor group and shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centrelines of the anchor, or in the case of an anchor group, from a line through a row of adjacent anchors (see Figure D.4). A_N shall not exceed n times the A_N of the single anchor nearest an edge or corner considered alone, where n is the number of tensioned anchors in the group. A_{No} is the projected area of the failure surface of a single anchor remote from edges, as follows:

$$A_{No} = 9h_{ef}^2 \tag{D-6}$$

D.6.2.2

The factored concrete breakout resistance of a single anchor in tension in cracked concrete shall not exceed

$$N_{br} = k\phi_c\sqrt{f_c'}h_{ef}^{1.5}R \tag{D-7}$$

where

k = 10 for cast-in headed studs, headed bolts, and hooked bolts
 = 7.0 for post-installed anchors

The k factor for post-installed anchors may be increased in accordance with ACI 355.2/355.2R product-specific tests, but shall not exceed 10.

Alternatively, for cast-in headed studs and headed bolts with 275 mm < h_{ef} < 625 mm, the factored concrete breakout resistance of a single anchor in tension in cracked concrete shall not exceed

$$N_{br} = 3.9\phi_c\sqrt{f_c'}h_{ef}^{5/3}R \tag{D-8}$$

D.6.2.3

For the special case of anchors in an application with three or four edges with the largest edge distance, c_{max}, less than or equal to $1.5h_{ef}$, the embedment depth h_{ef} used in Equations (D-6) to (D-9), (D-11), and (D-12) shall be limited to $c_{max}/1.5$ but be not less than one-third of the maximum spacing between anchors or anchor groups (see Figure D.7).

D.6.2.4

The modification factor for eccentrically loaded anchor groups shall be

$$\psi_{ec,N} = \frac{1}{\left(1+\dfrac{2e_N'}{3h_{ef}}\right)} \tag{D-9}$$

This equation shall be valid for

$$e_N' \le \frac{s_o}{2} \tag{D-10}$$

where

s_o = the centre-to-centre spacing of the outer anchors in tension

If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity, e_N', for use in Equation (D-9).

In the case where eccentric loading exists about two axes, the modification factor, $\psi_{ec,N}$, shall be computed for each axis individually, and the product of these factors used as $\psi_{ec,N}$ in Equation (D-5).

D.6.2.5
The modification factor for edge effects shall be

$$\psi_{ed,N} = 1 \text{ if } c_{min} \geq 1.5h_{ef} \tag{D-11}$$

$$= 0.7 + 0.3\frac{c_{min}}{1.5h_{ef}} \text{ if } c_{min} < 1.5h_{ef} \tag{D-12}$$

D.6.2.6
When an anchor is located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, the following modification factor may be used:

$\psi_{c,N} = 1.25$ for cast-in headed studs, headed bolts, and hooked bolts
= 1.4 for post-installed anchors when k = 7.0 is used in Equation (D-7)

Where k used in Equation (D-7) is taken from an ACI 355.2/355.2R product evaluation report for post-installed anchors approved for use in both cracked and uncracked concrete, the value of both k and $\psi_{c,N}$ shall be based on the product evaluation report.

For post-installed anchors approved for use only in uncracked concrete in accordance with ACI 355.2/355.2R, the value of k in the ACI 355.2/355.2R product evaluation report shall be used in Equation (D-7) and $\psi_{c,N}$ shall be 1.0.

Where analysis indicates cracking at service load levels, $\psi_{c,N}$ shall be taken as 1.0 for both cast-in and post-installed anchors.

D.6.2.7
The modification factor for post-installed anchors designed for uncracked concrete in accordance with Clause D.6.2.6 without supplementary reinforcement to control splitting shall be

$$\psi_{cp,N} = 1 \text{ if } c_{a,min} \geq c_{ac} \tag{D-13}$$

$$= \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}} \text{ if } c_{a,min} < c_{ac} \tag{D-14}$$

where the critical distance, c_{ac}, is as specified in Clause D.9.7.

D.6.2.8
When an additional plate or washer is added at the head of the anchor, the projected area of the failure surface may be calculated by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward a distance t from the outer edge of the head of the anchor, where t is the thickness of the washer or plate.

D.6.3 Pullout resistance of anchor in tension

D.6.3.1
The factored pullout resistance of an anchor in tension shall not exceed

$$N_{cpr} = \psi_{c,P} N_{pr} \tag{D-15}$$

D.6.3.2
For post-installed expansion and undercut anchors, the pullout strength shall not be calculated in tension. Values of N_{pr} shall be based on the 5% fractile of results of tests performed and evaluated in accordance with ACI 355.2/355.2R.

D.6.3.3
For single cast-in headed studs and headed bolts, the pullout resistance in tension may be calculated using Clause D.6.3.4. For single J-bolts or L-bolts, the pullout resistance in tension may be calculated using Clause D.6.3.5. Alternatively, values of N_{pr} based on the 5% fractile of tensile tests performed in the same manner as the ACI 355.2/355.2R procedures but without the benefit of friction may be used.

D.6.3.4
The pullout resistance in tension of a single headed stud or headed bolt, N_{pr}, for use in Equation (D-15) shall not exceed

$$N_{pr} = 8A_{bh}\phi_c f_c' R \qquad \textbf{(D-16)}$$

D.6.3.5
The pullout resistance in tension of a single J-bolt or L-bolt, N_{pr}, for use in Equation (D-15) shall not exceed

$$N_{pr} = 0.9\phi_c f_c' e_h d_o R \qquad \textbf{(D-17)}$$

where $3d_o \le e_h \le 4.5d_o$.

D.6.3.6
For an anchor located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, a modification factor of $\psi_{c,P} = 1.4$ may be used. Otherwise, $\psi_{c,P}$ shall be taken as 1.0.

D.6.4 Concrete side-face blowout resistance of a headed anchor in tension

D.6.4.1
For a single headed anchor with deep embedment close to an edge ($c < 0.4h_{ef}$), the factored side-face blowout resistance, N_{sbr}, shall not exceed

$$N_{sbr} = 13.3c\sqrt{A_{bh}}\phi_c\sqrt{f_c'}R \qquad \textbf{(D-18)}$$

If the single anchor is located at a perpendicular distance, c_2, less than $3c$ from an edge, the value of N_{sbr} shall be modified by multiplying it by the factor $(1 + c_2/c)/4$, where $1 \le c_2/c \le 3$.

D.6.4.2
For multiple headed anchors with deep embedment close to an edge ($c < 0.4h_{ef}$) and spacing between anchors less than $6c$, the factored resistance of the anchor group for a side-face blowout failure, N_{sbgr}, shall not exceed

$$N_{sbgr} = \left(1 + \frac{s_o}{6c}\right)N_{sbr} \qquad \textbf{(D-19)}$$

where s_o = distance between the outer anchors along the edge in the group
and N_{sbr} is obtained from Equation (D-18) without modification for a perpendicular edge distance.

D.7 Design requirements for shear loading

D.7.1 Steel resistance of anchor in shear

D.7.1.1
The factored resistance of an anchor in shear as governed by steel, V_{sr}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.7.1.2
The factored resistance of an anchor or anchor group in shear shall not exceed the following:

(a) for cast-in headed stud anchors:

$$V_{sr} = nA_{se}\phi_s f_{ut} R \quad \textbf{(D-20)}$$

where f_{ut} shall not be taken greater than the smaller of $1.9f_y$ or 860 MPa;

(b) for cast-in headed bolts, hooked bolt anchors, and post-installed anchors without sleeves extending through the shear plane:

$$V_{sr} = nA_{se}\phi_s 0.6f_{ut} R \quad \textbf{(D-21)}$$

where f_{ut} shall not be taken greater than the smaller of $1.9f_y$ or 860 MPa; and

(c) for post-installed anchors with sleeves extending through the shear plane, V_{sr} shall be based on the 5% fractile of results of tests performed and evaluated in accordance with to ACI 355.2/355.2R. Alternatively, Equation (D-21) may be used if the area of the sleeve is neglected.

D.7.1.3
Where anchors are used with built-up grout pads, the factored resistances specified in Clause D.7.1.2 shall be reduced by 20%.

D.7.2 Concrete breakout resistance of anchor in shear

D.7.2.1
The factored concrete breakout resistance in shear of an anchor or anchor group shall not exceed the following:

(a) For shear force perpendicular to the edge on a single anchor:

$$V_{cbr} = \frac{A_V}{A_{Vo}}\psi_{ed,V}\psi_{c,V}V_{br} \quad \textbf{(D-22)}$$

(b) For shear force perpendicular to the edge on an anchor group:

$$V_{cbgr} = \frac{A_V}{A_{Vo}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_{br} \quad \textbf{(D-23)}$$

(c) For shear force parallel to an edge, V_{cbr} or V_{cbgr} may be twice the value for shear force determined from Equation (D-22) or (D-23), respectively, with the shear force assumed to act perpendicular to the free edge and with $\psi_{ed,V}$ taken to be equal to 1 (see Figure D.11).

(d) For anchors located at a corner, the limiting factored concrete breakout resistance shall be determined for each edge and the minimum value shall be used (see Figure D.11).

In these equations, V_{br} is the factored concrete breakout resistance value for a single anchor. A_V is the projected area of the failure surface on the side of the concrete member at its edge for a single anchor or anchor group. This area may be evaluated as the base of a truncated half-pyramid projected on the side face of the member where the top of the half-pyramid is given by the axis of the anchor row selected as critical. The value of c_1 shall be taken as the distance from the edge to this axis. A_V shall not exceed nA_{Vo}, where n is the number of anchors in the group. A_{Vo} is the projected area for a single anchor in a deep member and remote from edges in the direction perpendicular to the shear force. This area may be evaluated as the base of a half-pyramid with a side length parallel to the edge of $3c_1$ and a depth of $1.5c_1$, as follows (see Figure D.4):

$$A_{Vo} = 4.5c_1^2 \tag{D-24}$$

Where anchors are located at varying distances from the edge and are welded to the attachment so as to distribute the force to all anchors, the strength may be evaluated based on the distance to the farthest row of anchors from the edge. In this case, the value of c_1 may be based on the distance from the edge to the axis of the farthest anchor row that is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

D.7.2.2

The factored concrete breakout resistance in shear of a single anchor in cracked concrete shall not exceed

$$V_{br} = 0.58\left(\frac{\ell}{d_o}\right)^{0.2}\sqrt{d_o}\,\phi_c\sqrt{f_c'}\,c_1^{1.5}R \tag{D-25}$$

D.7.2.3

For cast-in headed studs, headed bolts, or hooked bolts that are rigidly welded to steel attachments having a minimum thickness equal to the greater of 10 mm or half of the anchor diameter, the factored concrete breakout resistance in shear of a single anchor in cracked concrete shall not exceed

$$V_{br} = 0.66\left(\frac{\ell}{d_o}\right)^{0.2}\sqrt{d_o}\,\phi_c\sqrt{f_c'}\,c_1^{1.5}R \tag{D-26}$$

provided that
(a) for an anchor group, the resistance is determined based on the resistance of the row of anchors farthest from the edge;
(b) the centre-to-centre spacing of the anchors is not less than 65 mm; and
(c) supplementary reinforcement is provided at the corners if $c_2 \le 1.5h_{ef}$.

D.7.2.4

For the special case of anchors in a narrow ($c_2 < 1.5c_1$), thin ($h < 1.5c_1$) member, the edge distance, c_1, used in Equations (D-24) to (D-27) and (D-30) shall be limited to

$$\frac{c_2}{1.5} \quad \text{if} \quad c_2 > h$$

$$\frac{h}{1.5} \quad \text{if} \quad c_2 < h$$

D.7.2.5
The modification factor for eccentrically loaded anchor groups shall be

$$\psi_{ec,V} = \frac{1}{1 + \frac{2e'_V}{3c_1}} \qquad \textbf{(D-27)}$$

This equation shall be valid for

$$e'_V \leq \frac{s_o}{2} \qquad \textbf{(D-28)}$$

where
s_o = the centre-to-centre spacing of the outer anchors in shear

D.7.2.6
The modification factor for edge effects shall be

$$\psi_{ed,V} = 1.0 \quad \text{if } c_2 \geq 1.5c_1 \qquad \textbf{(D-29)}$$

$$= 0.7 + 0.3\frac{c_2}{1.5c_1} \quad \text{if } c_2 < 1.5c_1 \qquad \textbf{(D-30)}$$

D.7.2.7
For anchors located in a region of a concrete member where an analysis that includes temperature and shrinkage effects indicates no tension ($f_t < f_r$) at service loads, a modification factor of $\psi_{c,V} = 1.4$ may be used.

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels, the following modification factors may be used:

For anchors in cracked concrete with no edge reinforcement or edge reinforcement smaller than a 15M bar	$\psi_{c,V} = 1.0$
For anchors in cracked concrete with edge reinforcement of a 15M bar or greater between the anchor and the edge	$\psi_{c,V} = 1.2$
For anchors in cracked concrete with edge reinforcement of a 15M bar or greater between the anchor and the edge and with the edge reinforcement enclosed within stirrups spaced not more than 100 mm apart	$\psi_{c,V} = 1.4$

D.7.3 Concrete pryout resistance of an anchor in shear

The factored pryout resistance, V_{cpr} or V_{cpgr}, shall not exceed

$$V_{cpr} = k_{cp} N_{cbr} \tag{D-31}$$

$$V_{cpgr} = k_{cp} N_{cbgr} \tag{D-32}$$

where

$k_{cp} = 1.0$ for $h_{ef} < 65$ mm

$= 2.0$ for $h_{ef} \geq 65$ mm

N_{cbr} and N_{cbgr} shall be determined from Equations (D-4) and (D-5), respectively.

D.8 Interaction of tensile and shear forces

D.8.1

Unless determined in accordance with Clause D.5.3, anchors or anchor groups that are subjected to both shear and axial loads shall be designed to satisfy the requirements of Clauses D.8.2 through D.8.4. The value of N_r shall be the smallest of the steel resistance of the anchor in tension, concrete breakout resistance of the anchor in tension, pullout resistance of the anchor in tension, and side-face blowout resistance. The value of V_r shall be the smallest of the steel resistance of the anchor in shear, the concrete breakout resistance of the anchor in shear, and the pryout resistance.

D.8.2

If $V_f \leq 0.2V_r$, full resistance in tension shall be permitted, as follows (see Figure D.13):

$$N_r \geq N_f \tag{D-33}$$

D.8.3

If $N_f \leq 0.2N_r$, full resistance in shear shall be permitted, as follows (see Figure D.13):

$$V_r \geq V_f \tag{D-34}$$

D.8.4

If $V_f > 0.2V_r$ and $N_f > 0.2N_r$, the following shall apply (see Figure D.13):

$$\frac{N_f}{N_r} + \frac{V_f}{V_r} \leq 1.2 \tag{D-35}$$

D.9 Required edge distances, spacings, and thicknesses to preclude splitting failure

D.9.1

Minimum spacings and edge distances for anchors and minimum thicknesses of members shall comply with Clauses D.9.2 to D.9.7 unless reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI 355.2/355.2R shall be permitted.

D.9.2

Unless determined in accordance with Clause D.9.5, the minimum centre-to-centre spacing of anchors shall be $4d_o$ for untorqued cast-in anchors and $6d_o$ for torqued cast-in anchors and post-installed anchors.

D.9.3

Unless determined in accordance with Clause D.9.5, minimum edge distances for cast-in headed anchors that will not be torqued shall be based on the minimum cover requirements for reinforcement specified in Clause 7.9. For cast-in headed anchors that will be torqued, the minimum edge distances shall be $6d_o$.

D.9.4

Unless determined in accordance with Clause D.9.5, minimum edge distances for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement specified in Clause 7.9 and the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2/355.2R, and shall be not less than 2.0 times the nominal maximum aggregate size. In the absence of such product-specific ACI 355.2/355.2R test information, the minimum edge distance shall be taken as not less than the following:

Undercut anchors	$6d_o$
Torque-controlled anchors	$8d_o$
Displacement-controlled anchors	$10d_o$

D.9.5

For anchors where installation does not produce a splitting force and the anchors will remain untorqued, if the edge distance or spacing is less than that specified in Clauses D.9.2 to D.9.4, calculations shall be performed by substituting for d_o a smaller value, d_o', that meets the requirements of Clauses D.9.2 to D.9.4. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having that fictitious diameter.

D.9.6

The value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of two-thirds of the member thickness or the member thickness less 100 mm.

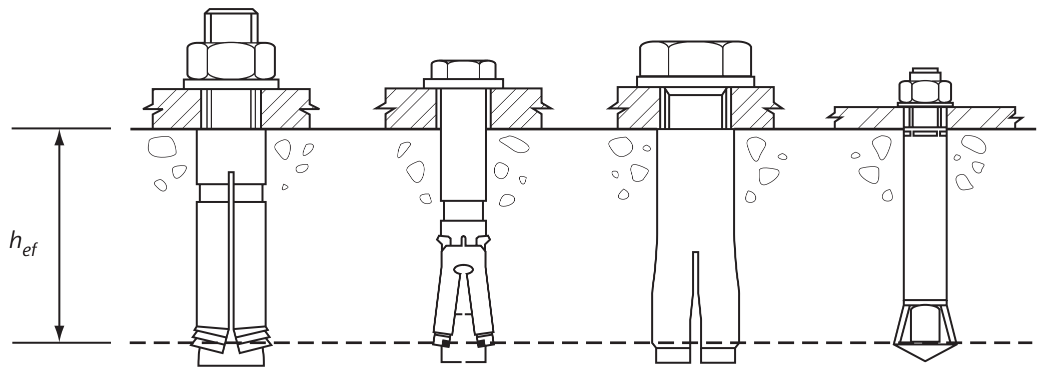
D.9.7

Unless determined from tension tests in accordance with ACI 355.2/355.2R, the critical edge distance, c_{ac}, shall not be taken less than the following:

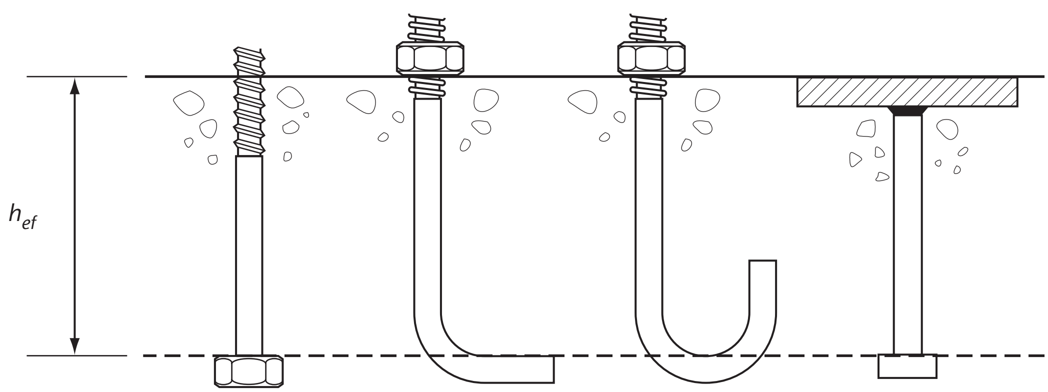
Undercut anchors	$2.5h_{ef}$
Torque-controlled anchors	$4h_{ef}$
Displacement-controlled anchors	$4h_{ef}$

D.9.8
Project drawings and project specifications shall specify use of anchors with the minimum edge distance assumed in the design.

D.10 Installation of anchors
Anchors shall be installed in accordance with the project drawings and project specifications.



(a) Post-installed anchors



(b) Cast-in anchors

Figure D.1
Types of anchors
(See Clauses D.1.2, D.2, and D.3.)

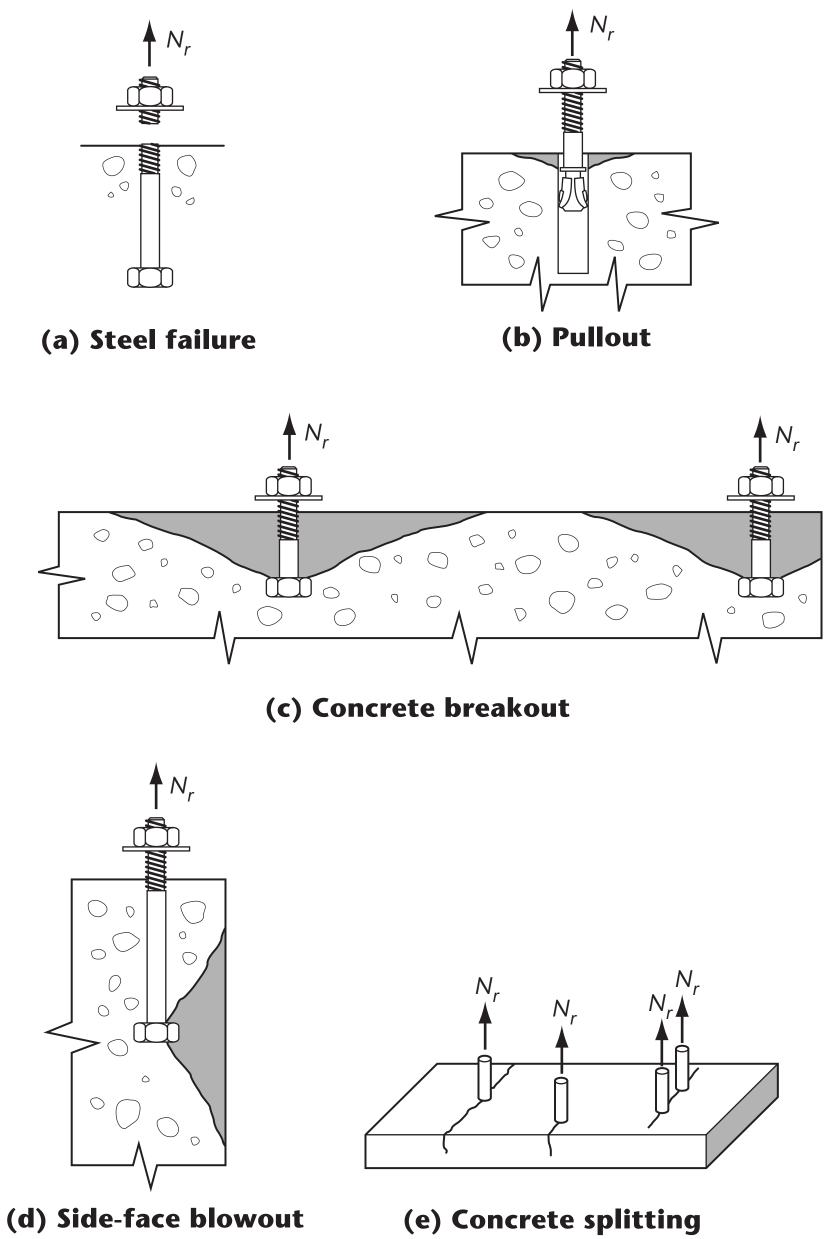
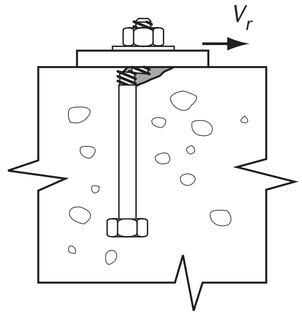
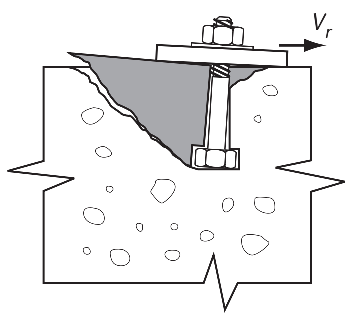


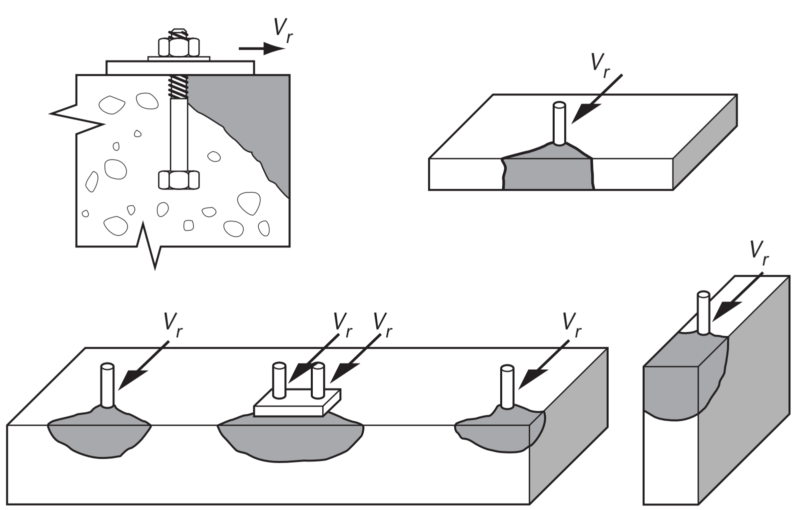
Figure D.2
Failure modes for anchors under tensile loading
(See Clause D.2.)



(a) Steel failure preceded by concrete spall

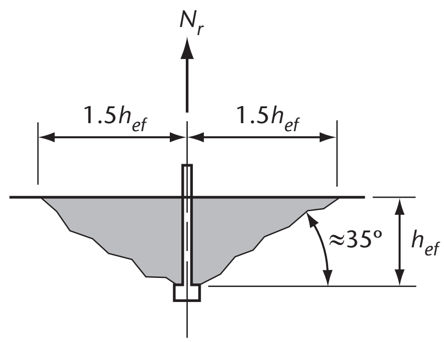


(b) Concrete pryout for anchors far from a free edge

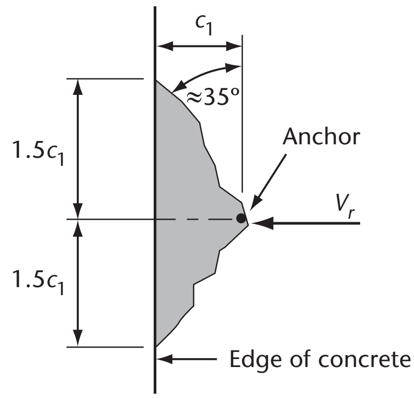


(c) Concrete breakout

Figure D.3
Failure modes for anchors under shear loading
(See Clause D.2.)

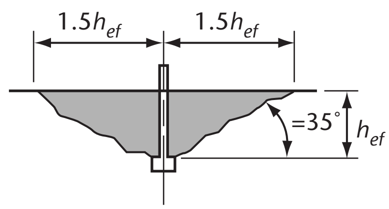


(a) Breakout cone for tension

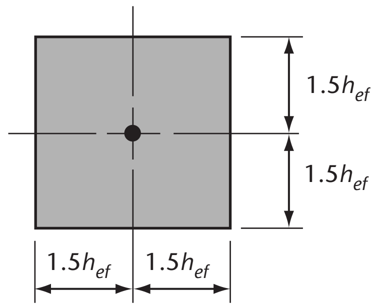


(b) Breakout cone for shear

Figure D.4
Breakout cones
(See Clauses D.6.2.1 and D.7.2.1.)



(a) Section through failure cone



(b) Plan view

Notes:

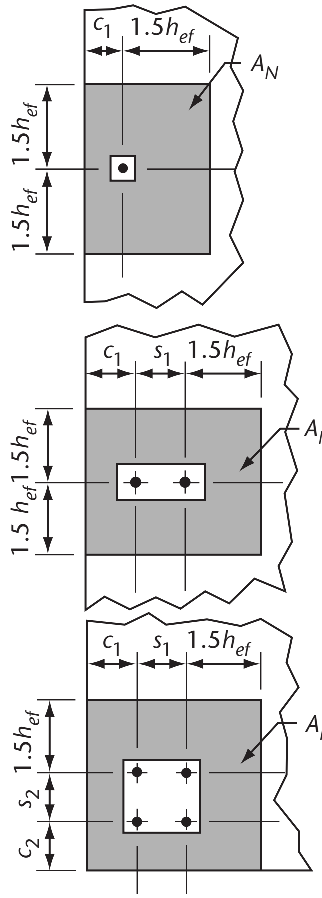
(1) *The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is 1.5 h_{ef}.*

(2) $A_{No} = (2 \times 1.5h_{ef}) \times (2 \times 1.5h_{ef})$

$= 3h_{ef} \times 3h_{ef}$

$= 9\,h_{ef}^2$

Figure D.5
Calculation of A_{No}
(See Clause D.3.)



$A_N = (c_1 + 1.5h_{ef})(2 \times 1.5h_{ef})$
if $c_1 < 1.5h_{ef}$

$A_N = (c_1 + s_1 + 1.5h_{ef})(2 \times 1.5h_{ef})$
if $c_1 < 1.5h_{ef}$ and $s_1 < 3h_{ef}$

$A_N = (c_1 + s_1 + 1.5h_{ef})(c_2 + s_2 + 1.5h_{ef})$
if c_1 and $c_2 < 1.5h_{ef}$
and s_1 and $s_2 < 3h_{ef}$

Figure D.6
Projected areas for single anchors and groups of anchors
(See Clause D.3.)

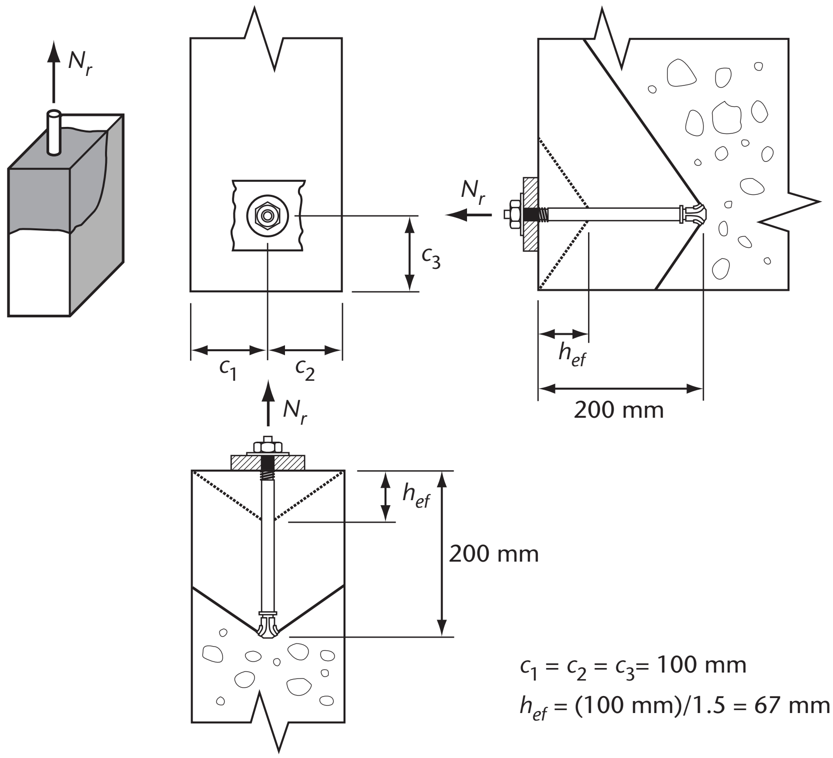
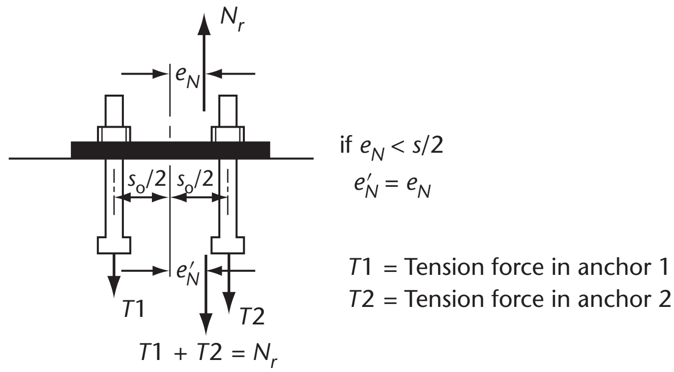
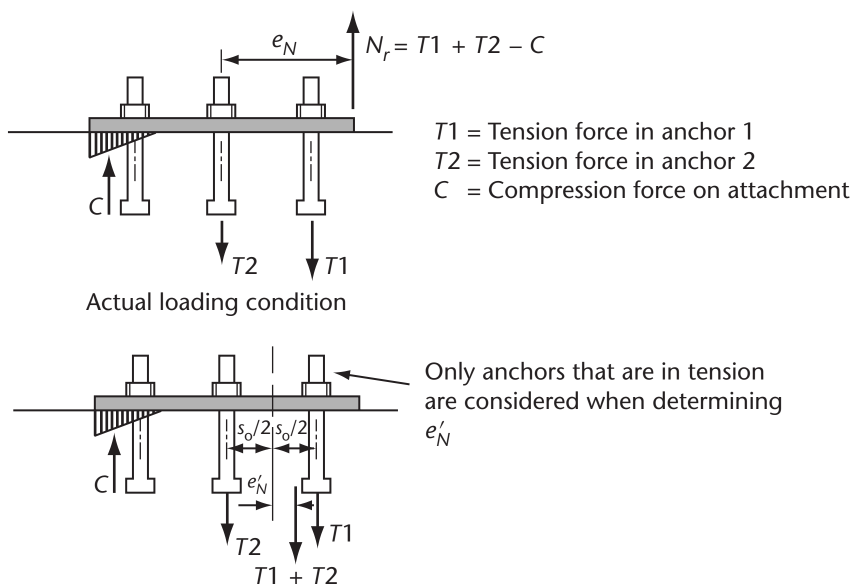


Figure D.7
Failure surfaces in narrow members for different embedment depths
(See Clauses D.6.2.3.)

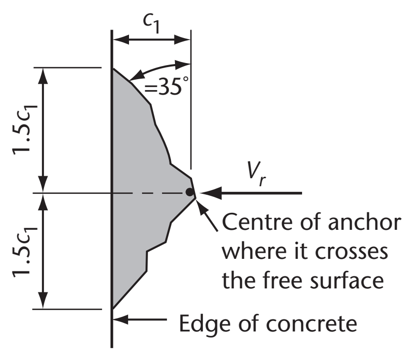


(a) Determination of e'_N when all anchors in a group are in tension

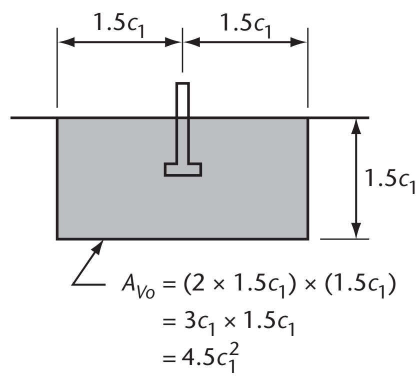


(b) Determination of e'_N with only some anchors in tension

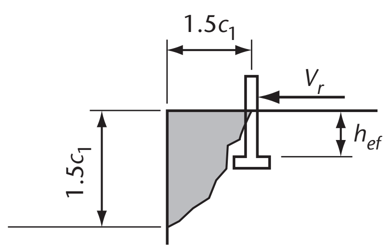
Figure D.8
Definition of dimension e'_N
(See Clause D.3.)



(a) Plan view



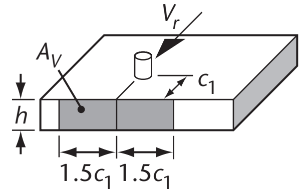
(b) Front view



(c) Side section

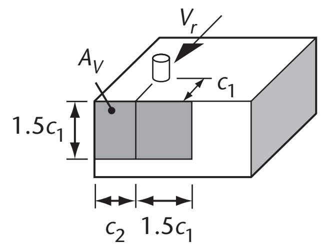
Note: *The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is $1.5c_1$.*

Figure D.9
Calculation of A_{Vo}
(See Clause D.3.)



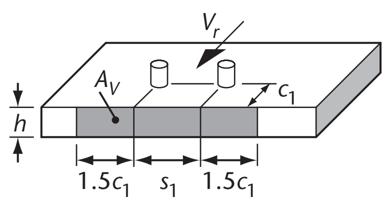
$A_V = 2 \times 1.5c_1 \times h$

(a) A single anchor, if $h < 1.5c_1$



$A_V = 1.5c_1\ (1.5c_1 + c_2)$

(b) A single anchor, if $c_2 < 1.5c_1$

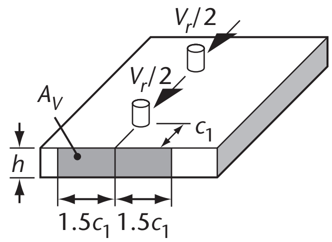


$A_V = (2 \times 1.5c_1 + s_1) \times h$

(c) Two loaded anchors aligned parallel to edge, if $h < 1.5c_1$ and $s_1 < 3c_1$

Figure D.10
Projected areas for single anchors and anchor groups
(See Clause D.3.)

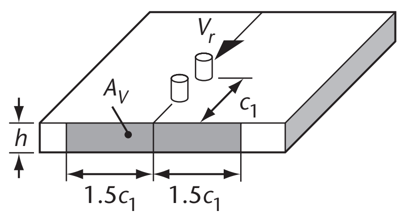
(Continued)



$A_V = 2 \times 1.5c_1 \times h$

(d) Two loaded anchors aligned perpendicular to edge, if $h < 1.5c_1$*

**One assumption of the distribution of forces indicates that half the shear would be critical on the front anchor and its projected area.*

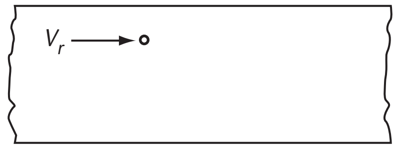


$A_V = 2 \times 1.5c_1 \times h$

(e) Two loaded anchors aligned perpendicular to edge and rigidly connected, if $h < 1.5c_1$†

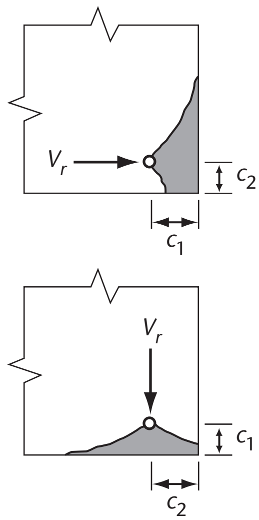
†*Another assumption of the distribution of forces that applies only where anchors are rigidly connected to the attachment indicates that the total shear would be critical on the rear anchor and its projected area.*

Figure D.10 (Concluded)



V_r (parallel to edge) = 2 × V_r (perpendicular to edge)

(a) Shear force parallel to edge



(b) Anchors near a corner

Figure D.11
Shear loading parallel and perpendicular to edge
(See Clause D.7.2.1.)

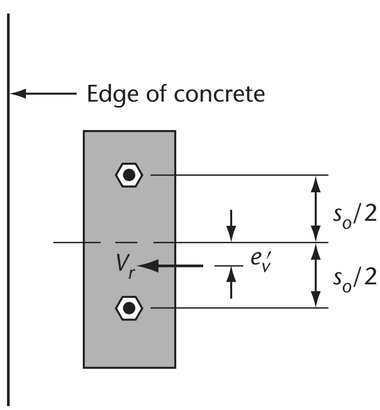


Figure D.12
Definition of dimension e'_V
(See Clause D.3.)

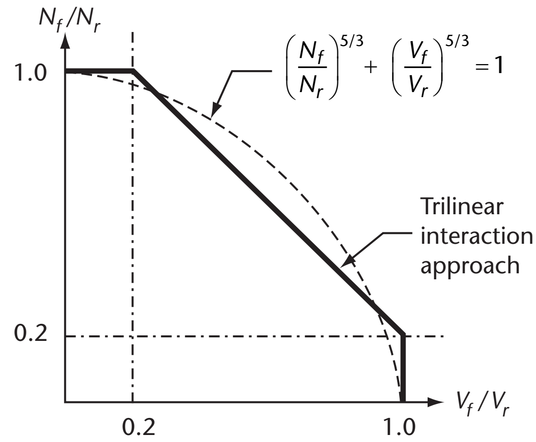


Figure D.13
Shear and tensile load interaction equation
(See Clauses D.8.2–D.8.4.)

Proposition de modification

N'hésitez pas à nous faire part de vos suggestions et de vos commentaires. Au moment de soumettre des propositions de modification aux normes CSA et autres publications CSA prière de fournir les renseignements demandés ci-dessous et de formuler les propositions sur une feuille volante. Il est recommandé d'inclure

- le numéro de la norme/publication
- le numéro de l'article, du tableau ou de la figure visé
- la formulation proposée
- la raison de cette modification.

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- Standard/publication number
- relevant Clause, Table, and/or Figure number(s)
- wording of the proposed change
- rationale for the change.

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