A23.3-14

Design of concrete structures

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Design of concrete structures

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Contents

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In addition, the members of the Structural Engineers Association of BC Technical Committee on Concrete Design made contributions to the development of this Standard.

Preface

This is the sixth edition of CSA A23.3, *Design of concrete structures.* It supersedes the previous editions published in 2004, 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/A23.2, *Concrete materials and methods of concrete construction/Methods of test and standard practices for concrete,* and CSA A23.4, *Precast concrete — Materials and construction*.

Changes in this edition include the following:

a) Clause 3.1 contains new definitions for conventional construction, moderately ductile wall systems, different types of tilt-up construction, and gravity-load resisting frames.

b) Clause 7.4.3.1 contains new requirements for the clear distance between pretensioning wires or strands at the ends of members. Clause 7.6.5 contains new requirements for additional column ties in column-slab connections over the slab depth where the slab is discontinuous. In Clause 7.6.4, the minimum diameter of spiral reinforced has been changed to 10 mm and the limit of one-sixth of the core diameter for the clear spacing between successive turns in a spiral has been removed. Clause 7.7.3 has new requirements for column ties in beam-column joints.

c) Clause 9.2.1.2 gives guidance on stiffnesses to be used in members of lateral load resisting systems for wind loading. Clause 9.8 provides cautionary notes on member minimum thickness requirements and accounting for construction stages and early loading in computing deflections.

d) Clause 10.9.4 contains a new requirement for the required ratio of spiral reinforcement. Clause 10.10.4 has increased the maximum factored axial load resistance of spirally reinforced columns and contains new provisions for the resistance of compression members as a function of wall thickness. Clause 10.16.3 provides a new factor for determining the amplitude of sway moments.

e) Changes to the shear design provisions in Clause 11 include the following: the need to account for cover spalling for members subjected to high shear stress; new requirement for sections near supports; definition of special member types; accounting for effect of bars terminated in the flexural tension zone; and increased spacing limit for transverse reinforcement for special cases. Changes to the strut-and-tie design provisions of Clause 11.4 include the following: introduction of refined strut-and-tie models; modelling of members subjected to uniform loads; revised strut dimensions for struts anchored by reinforcement and for struts in narrow part of fanning compression regions; simplified expression for limiting compressive stress in struts; new detailing requirements for anchorage of ties; and provisions accounting for confinement of bearing in nodal regions.

f) Clause 13 on two-way slab systems has been revised to include the following: the use of d_v in determining the one-way shear resistance; new details for bottom bars in column strips of slabs with drop panels (see Figure 13.1); and a change in the definition of V_{se} for the design of structural integrity reinforcement (see Clauses 13.10.6.1 and 3.2).

g) Clause 14 contains a new requirement to account for strong axis bending in bearing walls and new wall thickness requirements and slenderness requirements for flexural shear walls.

h) Clause 18.3.1 permits a higher compressive stress limit in the concrete at transfer at the ends of simply supported members.

i) Clause 21 on special provisions for seismic design has a number of significant changes. This Clause has been reorganized so that all the requirements for ductile frames are in Clause 21.3, while all the requirements for moderately ductile frames are in Clause 21.4. New dimensional limitations for moderately ductile moment-resisting frames have been added in Clause 21.4.2. The requirements for moderately ductile shear walls have been spelled out in greater detail, and because of the significant overlap with the requirements for ductile shear walls, the requirements for moderately ductile and ductile shear walls are presented together in Clause 21.5. All shear wall design requirements that were redundant with Clause 14 have been removed from Clause 21. Thus, the designer of seismic shear walls must look to Clause 14 for important requirements such as dimensional limitations, transfer of forces across construction joints, and many other requirements. The requirements for strength and ductility over the height of shear walls in Clause 21.5.2 have been expanded. New requirements have been added for the design for bending moment and shear force below the plastic hinge at the base, and for the increased shear force in walls due to the inelastic effects of higher modes. New requirements have been added in Clause 21.5.5 for the anchorage of horizontal reinforcement at the ends of walls depending on the level of ductility. New requirements have been added in Clause 21.5.7 to ensure that walls have adequate ductility to tolerate some yielding near mid-height due to higher mode bending moments. The design requirements for two new types of reinforced concrete SFRS — moderately ductile coupled walls and moderately ductile partially coupled walls — have been added in Clause 21.5.8. The requirements for squat shear walls in Clause 21.5.10 have been relaxed where the walls are longer than needed. The requirements for conventional construction shear walls in Clause 21.6.3 have been expanded. New requirements for the design and detailing of tilt-up construction, including moderately ductile and limited ductility tilt-up walls and frames, are presented in Clause 21.7. New requirements for the design of foundations are presented in Clause 21.10, including the requirement to consider foundation movements. New requirements are presented in Clause 21.11 to ensure that all members not considered part of the seismic-force-resisting system have adequate displacement capacity.

j) Clause 23.2.9 provides revised design provisions for structural integrity of tilt-up construction. The effective area of reinforcement used to calculate the factored resisting moment has been modified.

k) Annex D on anchorage has been modified to include changes to the requirements specified in Appendix D of ACI 318M-11/318RM-11, *Building Code Requirements for Structural Concrete* and *Commentary*. Annex D provides new provisions for the bond strength of adhesive anchors in tension; installation of horizontal and upwardly inclined adhesive anchors; the bond strength of adhesive anchors in tension; the resistance of anchors for load cases involving earthquake effects; revised breakout resistance in shear for an anchor in cracked concrete; and new requirements for the installation of anchors.

This Standard was prepared by the Technical Committee on Reinforced Concrete Design, under the jurisdiction of the Strategic Steering Committee on Construction and Civil Infrastructure, and has been formally approved by the Technical Committee.

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 Standard 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 Standard.*

4) *To submit a request for interpretation of this Standard, please send the following information to* ***inquiries@csagroup.org*** *and include "Request for interpretation" in the subject line:*

- 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) *where possible, phrase the request in such a way that a specific "yes" or "no" answer will address the issue.*

Committee interpretations are processed in accordance with the CSA Directives and guidelines governing standardization and are available on the Current Standards Activities page at standardsactivities.csa.ca.

5) *This Standard is subject to review five years from the date of publication and suggestions for its improvement will be referred to the appropriate committee. To submit a proposal for change, please send the following information to* ***inquiries@csagroup.org*** *and include "Proposal for change" in the subject line:*

a) *Standard designation (number);*
b) *relevant clause, table, and/or figure number;*
c) *wording of the proposed change; and*
d) *rationale for the change.*

A23.3-14
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 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 that 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 this Standard, “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; and “may” is used to express an option or that which is permissible within the limits of the standard.

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

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)

A23.1-14/A23.2-14
Concrete materials and methods of concrete construction/Test Methods and standard practices for concrete

A23.4-09
Precast concrete — Materials and construction

G30.18-09
Billet-steel bars for concrete reinforcement

G40.20-04/G40.21-13
General requirements for rolled or welded structural quality steel/Structural quality steel

S16-14
Design of steel structures

S413-07 (R2012)
Parking structures

W59-13
Welded steel construction (metal arc welding)

W186-M1990 (R2012)
Welding of reinforcing bars in reinforced concrete construction

ACI (American Concrete Institute)

302.1R-04
Guide for Concrete Floor and Slab Construction

302.2R-06
Guide for Concrete Slabs that Receive Moisture – Sensitive Flooring Materials

318M-11/318RM-11
Metric Building Code Requirements for Structural Concrete and Commentary

336.3R-93 (R2006)
Design and Construction of Drilled Piers

355.2-07/355.2R-07
Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary

355.4-11
Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary

360R-06
Design of Slabs on Grade

374.1-05
Acceptance criteria for moment frames based on structural testing and commentary

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

ASTM International (American Society for Testing and Materials)
A307-12
Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength

A416/A416M-12a
Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

A421/A421M-10
Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete

A496-07/A496M-07
Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement

A497/A497M-07
Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete

A722/A722M-12
Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete

A1064/A1064M-13
Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete

C330/C330M-09
Standard Specification for Lightweight Aggregates for Structural Concrete

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

LATBSDC (Los Angeles Tall Buildings Structural Design Council)
Alternate Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Area (2011)

NRCC (National Research Council Canada)
National Building Code of Canada, 2015

User's Guide — NBC 2015: 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. 2016. *Concrete design handbook*. 4th ed. Ottawa: Cement Association of Canada.

Precast/Prestressed Concrete Institute. 2010. *PCI design handbook: Precast and prestressed concrete*. 7th ed. Chicago: Precast/Prestressed Concrete Institute.

3 Definitions and symbols

3.1 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.

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 21.2.8.1 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 reinforcement — reinforcement that meets the requirements of Clause 21.2.8.2 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.

Conventional construction — a seismic-force-resisting system with low ductility capacity designed in accordance with Clauses 1 to 18 or Clause 23 with the additional requirements of Clause 21.6.

Conventional tilt-up construction — a seismic-force-resisting system with tilt-up wall panels having low ductility capacity designed in accordance with Clause 23. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 1.3 in the *National Building Code of Canada*.

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.

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.

Note: *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 4.1.3.

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.

Drift ratio — the ratio of lateral displacement to height over which the displacement occurs.

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 21.5 and has ductile shear walls connected by ductile coupling beams 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 seismic-force-resisting system qualifies for a force modification factor, R_d, of 4.0 in accordance with the *National Building Code of Canada.*

Ductile coupling beam — a coupling beam designed to dissipate energy that complies with Clauses 21.2 and 21.5.

Ductile moment-resisting frame — a moment-resisting frame that dissipates energy primarily through beam flexural yielding and complies with Clauses 21.2 and 21.3. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 4.0 in accordance with the *National Building Code of Canada.*

Ductile partially coupled shear wall — a coupled wall system that has ductile wall piers connected by ductile coupling beams where less than 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces in the wall piers resulting from shears in the coupling beams and complies with Clauses 21.2 and 21.5. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 3.5 in accordance with the *National Building Code of Canada*.

Ductile shear wall — a cantilever wall that dissipates energy through flexural yielding in a plastic hinge near the base and complies with Clauses 21.2 and 21.5. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 3.5 in accordance with the *National Building Code of Canada.*

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.

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).

Gravity-load resisting frame — a frame consisting of slabs and/or beams supported by columns and/or walls all of which are not considered to be part of the seismic-force-resisting-system.
Note: *See Clause 21.11 for the design of gravity-load resisting frame members for seismic displacement.*

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. Seismic crossties within a hoop may be considered to provide effective hoop reinforcement.

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.

Limited ductility tilt-up construction — a seismic force resisting system with tilt-up wall panels designed using a force-based approach in accordance with Clauses 21.2 to 21.7. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 1.5 in accordance with the *National Building Code of Canada*.

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 C330.

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

Moderately ductile coupled shear wall — a coupled wall system that has moderately ductile wall piers connected by moderately ductile coupling beams where at least 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces in the wall piers resulting from shear in the coupling beams and complies with Clauses 21.2 and 21.5. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 2.5 in the *National Building Code of Canada.*

Moderately ductile coupling beam — a coupling beam that dissipates energy by yielding of longitudinal or diagonal reinforcement and complies with Clauses 21.2 and 21.5.8.

Moderately ductile moment-resisting frame — a moment-resisting frame that complies with Clauses 21.2 and 21.4, that resists seismic forces, and that dissipates energy through beam flexural yielding. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 2.5 in the *National Building Code of Canada.*

Moderately ductile partially coupled shear wall — a coupled wall system that has moderately ductile wall piers connected by moderately ductile coupling beams where less than 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces in the wall piers resulting from shear in the coupling beams and complies with Clauses 21.2 and 21.5. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 2.0 in the *National Building Code of Canada.*

Moderately ductile shear wall — a cantilever wall that dissipates energy through flexural yielding in a plastic hinge near the base and complies with Clauses 21.2 and 21.5. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 2.0 in accordance with the *National Building Code of Canada.*

Moderately ductile tilt-up construction — a seismic-force-resisting system with tilt-up wall panels having moderate ductility capacity that complies with Clauses 21.2 to 21.7. The design of the system includes the explicit consideration of the inelastic displacement demands on the tilt-up wall panels. This seismic-force-resisting system qualifies for a force modification factor, R_d, of 2.0 in accordance with 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.

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 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.

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.
Note: *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 reinforcement; 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 4.1.2 and 4.1.3.

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 — resistance calculated using axial loads P_s and P_n where applicable and the specified values of f'_c and f_y, with ϕ_c and ϕ_s taken as 1.0.

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.

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 (SFRS) — 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.

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 4.1.4.

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 $0.04\, f_c' A_g$;
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 $0.04 f_c' A_g$ 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 $2\ell_w$.

Wall pier — a vertical wall adjacent to an opening between coupling beams in a coupled or partially coupled wall.

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

3.2 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_s = length of uniform bearing stress in soil or rock required to resist the applied loads (see Clause 21)

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 11.1)

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_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

$A_{s,\mathrm{eff}}$ = 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

$A_{s,\mathrm{min}}$ = 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; width of footing parallel to axis of footing rotation (see Clause 21)

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_b = diameter of bar, wire, or prestressing strand

d_{ba} = diameter of longitudinal bar anchoring compression strut

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 and for the case of walls need not be taken less than 0.8ℓ_w

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} = compressive stress in concrete due to prestress (after allowance for all prestress losses) at the centroid of the cross section (see Clause 11.2.9.1)

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 = 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 (see Clause 11.3.9.3)

F_{lt} = required tension force in longitudinal reinforcement on flexural tension side of member (see Clause 11.3.9.2)

F_y = specified yield strength of structural steel section

G_0 = initial shear modulus of soil or rock immediately below the foundation (see Clause 21)

h = overall thickness or height of member

h_a = height of effective embedment of tension tie (see Figure 11.5)

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 effectiveness of transverse reinforcement, equal to $n_\ell/(n_\ell - 2)$

k_p = axial load ratio, equal to $P_f / A_g \alpha_1 f_c'$

= 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 11.5)

ℓ_b = length of bearing (see Figure 11.5)

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_f = length of footing perpendicular to axis of footing rotation (see Clause 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 = confinement factor (see Clause 11.4.4.1)

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, 20, and 21)

= 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 Clauses 7.4.3.1 and 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)

P_f = 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 in accordance with Equation 10.10

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_s = uniform bearing stress in soil or rock resisting a vertical load and overturning moment on a footing (see Clause 21)

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)

$S(T)$ = design spectral response acceleration for a period of T, as specified in the *National Building Code of Canada*

$S_a(0.2)$ = 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 (see Clause 9); period in seconds (see Clause 21)

T_a = fundamental lateral period of vibration of the building in the direction under consideration, determined in accordance with the *National Building Code of Canada*

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
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	= lateral deflection at top of wall calculated using the modified section properties in Clause 21.2 and the factored seismic loads calculated in accordance with the *National Building Code of Canada*
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 drift ratio equal to interstorey deflection divided by the interstorey height 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) (see Clause 11)

ε_1 = principal tensile strain in cracked concrete due to factored loads (see Clause 11)

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 longitudinal 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

$\psi_{h,V}$ = factor used to modify shear strength of anchors located in concrete members with $h < 1.5_{c1}$, as specified in Clause D.7.2.8

ω = flange buckling factor

3.3 Standard notation and calculations

3.3.1 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.

3.3.2 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.

3.3.3 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.

4 General requirements

4.1 Materials — Reinforcement

4.1.1

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.*

4.1.2

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.

4.1.3

Deformed reinforcement shall include

a) reinforcing bars having deformations and complying with CSA G30.18;

b) welded wire fabric complying with ASTM A1064/A1064M, 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 A497/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 A496 and not smaller than size MD25.

4.1.4

Prestressing tendons shall comply with the applicable requirements of ASTM A416/A416M, ASTM A421/A421M, and ASTM A722/A722M.

4.2 Concrete and other materials

4.2.1

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

4.2.2
Precast concrete and constituent materials shall comply with CSA A23.4, except as specified in Clause 16.2.2.

4.3 Concrete quality, mixing, and placement

4.3.1 Quality

4.3.1.1
Concrete shall be proportioned and produced in accordance with CSA A23.1 or CSA A23.4, as applicable.

4.3.1.2
The compressive strength of concrete, f_c', shall be determined by testing as specified in CSA A23.1/A23.2 or CSA A23.4, as applicable.

4.3.1.3
Unless otherwise specified, f_c' shall be based on 28 day tests.

4.3.2 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 there may be a bend of at least 90°. Standard tie hooks with a bend of at least 90° may be used 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, except that if specified compressive strength of concrete at time of prestress transfer, f'_{ci} is 27.5 MPa or greater, minimum centre-to-centre spacing of strands shall be 45 mm for 12.7 mm nominal diameter or smaller and 50 mm for strands of 15.2 mm nominal diameter. 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 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 10 mm.

7.6.4.3

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, except as noted in Clause 14.1.8.7, 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. Where a slab is not continuous beyond one or more faces of the column, the ties that engage the vertical steel on any face not confined by a slab shall continue within the depth of the slab.

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. Where a beam or bracket is not continuous beyond one or more faces of the column, the ties which engage the vertical steel on any face not confined by a beam or bracket shall continue within the depth of the beam or bracket. 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, column ties in accordance with Clause 7.6.5, having a maximum spacing of 150 mm but not less than two column ties, shall be provided within beam column connections. 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 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 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.1a.*

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 ϕ_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: *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 strength, 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 C469. 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

$$E_c = (3300\sqrt{f'_c} + 6900)\left(\frac{\gamma_c}{2300}\right)^{1.5}$$ **Equation 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

$$E_c = 4500\sqrt{f'_c}$$ **Equation 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

$$f_r = 0.6\lambda\sqrt{f_c'} \quad \textbf{Equation 8.3}$$

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 for lateral deflection or in second-order frame analyses shall be representative of the degree of member cracking and inelastic action at the limit state for which the analysis is being carried out.

Note: *For wind effects, to obtain reasonable predictions of the deformations and the period of vibration, approximate member stiffnesses are provided in the explanatory notes. For second-order frame analyses at Ultimate Limit States, Clause 10.14.1.2 provides suitable member stiffness properties. For seismic design, Clause 21.2.5 specifies member stiffnesses for members of the seismic force resisting system.*

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	$\ell_n/16$	$\ell_n/18$	$\ell_n/21$	$\ell_n/8$

Notes:
1) *This Table gives traditional values that provide guidance for preliminary proportioning but are insufficient for beams or one-way slabs supporting partitions or other construction likely to be damaged by large deflections.*
2) *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:

$$I_e = I_{cr} + \left(I_g - I_{cr}\right)\left[\frac{M_{cr}}{M_a}\right]^3 \le I_g$$ **Equation 9.1**

where

$$M_{cr} = \frac{f_r I_g}{Y_t}$$ **Equation 9.2**

and f_r shall be taken as half the value given in Equation 8.3.

Note: *Depending on the shoring and reshoring schedule adopted, the maximum M_a may occur during construction. If construction loadings are unknown, M_a may be computed as the maximum moment due to specified dead and live loads for the computation of instantaneous and long-term deflections.*

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:

$$I_{e,avg} = 0.7\, I_{em} + 0.15(I_{e1} + I_{e2})$$ **Equation 9.3**

b) one end continuous:

$$I_{e,avg} = 0.85\, I_{em} + 0.15 I_{ec}$$ **Equation 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:

Equation 9.5

$$\zeta_s = \left[1 + \frac{s}{1 + 50\rho'}\right]$$

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, applied when the concrete reaches an age of 28 days or greater, 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

Note: *When a sustained load is applied when the concrete is less than 28 days old, the immediate plus long-term deflection may exceed that computed using Equation 9.5. For concrete loaded at an age between 7 and 28 days, the additional long-term deflection may be estimated by multiplying ζ_s by the factor [1.6 – 0.6 (t – 7)/21] where t is the concrete age, in days, at the time of loading.*

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.*

Notes:

1) *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.*

2) *In some instances, the total deflection may be critical, not the incremental deflection. For example, the installation of long solid millwork units and the performance of some flooring depend on total deflection.*

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:

$$\alpha_1 = 0.85 - 0.0015 f_c' \text{ (but not less than 0.67)}$$ **Equation 10.1**

$$\beta_1 = 0.97 - 0.0025 f_c' \text{ (but not less than 0.67)}$$ **Equation 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

$$M_r \geq 1.2 M_{cr} \qquad \textbf{Equation 10.3}$$

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:

$$A_{s,\min} = \frac{0.2\sqrt{f_c'}}{f_y} b_t h \qquad \textbf{Equation 10.4}$$

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

$$\frac{c}{d} \leq \frac{700}{700 + f_y} \qquad \textbf{Equation 10.5}$$

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

$$z = f_s (d_c A)^{1/3}$$ **Equation 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: Precast and prestressed concrete *published by the Canadian Precast/Prestressed Concrete Institute.*

10.9 Compression members — Reinforcement limits

10.9.1
The area of longitudinal bars for columns 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 A_g 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

Equation 10.7

$$\rho_s = 0.5\left(\frac{A_g}{A_c} - 1\right)^{1.4} \frac{f'_c}{f_y}$$

where

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

10.10 Compression members — Resistance

10.10.1
Compression members 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
Compression members 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:

$$P_{r,\max} = 0.90 P_{ro} \quad \textbf{Equation 10.8}$$

b) for tied columns and walls tied along the full length in accordance with Clause 7.6.5:

$$P_{r,\max} = (0.2 + 0.002h) P_{ro} \le 0.80 P_{ro} \quad \textbf{Equation 10.9}$$

c) for other walls:

$$P_{r,\max} = (0.15 + 0.002h) P_{ro} \le 0.75 P_{ro} \quad \textbf{Equation 10.10}$$

where

h is the wall thickness or the minimum column dimension, and

$$P_{ro} = \alpha_1 \phi_c f'_c \left(A_g - A_{st} - A_t - A_p\right) + \phi_s f_y A_{st} + \phi_a F_y A_t - f_{pr} A_p \quad \textbf{Equation 10.11}$$

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, including $P_{r,max}$, are multiplied by the ratio 0.5 $(1 + \rho_t / 0.01)$.

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:

$$f'_{ce} = 1.05 f'_{cs} + 0.25 f'_{cc} \le f'_{cc} \qquad \textbf{Equation 10.12}$$

b) for edge columns:

$$f'_{ce} = 1.4 f'_{cs} \le f'_{cc} \qquad \textbf{Equation 10.13}$$

c) for corner columns:

$$f'_{ce} = f'_{cs} \qquad \textbf{Equation 10.14}$$

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.

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. For walls, the unsupported vertical height, h_u, shall be used in place of ℓ_u wherever it appears.

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 $Q \leq 0.05$ where

$$Q = \frac{\Sigma P_f \Delta_o}{V_f \ell_c}$$ **Equation 10.15**

where
$\sum P_f$ = 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:

$$\frac{k\ell_u}{r} \leq \frac{25 - 10(M_1/M_2)}{\sqrt{P_f/(f'_c A_g)}}$$ **Equation 10.16**

where M_1/M_2 is not taken less than –0.5. M_1/M_2 shall be taken as positive if the member 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:

$$M_c = \frac{C_m M_2}{1 - \frac{P_f}{\phi_m P_c}} \geq M_2$$ **Equation 10.17**

where

$\phi_m = 0.75$

Equation 10.18

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2}$$

where

$$EI = \frac{0.2E_c I_g + E_s I_{st}}{1+\beta_d}$$ **Equation 10.19**

or

$$EI = \frac{0.4E_c I_g}{1+\beta_d}$$ **Equation 10.20**

M_2 in Equation 10.17 shall not be taken as less than P_f (15 + 0.03h) about each axis separately with the member bent in single curvature with C_m taken as 1.0. β_d in Equations 10.19 and 10.20 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

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4$$ **Equation 10.21**

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

$$M_1 = M_{1ns} + \delta_s M_{1s}$$ **Equation 10.22**

$$M_2 = M_{2ns} + \delta_s M_{2s}$$ **Equation 10.23**

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:

$$\delta_s = \frac{1}{1 - \frac{\sum P_f}{\phi_m \sum P_c}} \qquad \textbf{Equation 10.24}$$

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 10.18 using k from Clause 10.16.1 and EI from Equation 10.19 or 10.20, with β_d based on the sustained shear.

10.16.3.3

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

$$\delta_s = \frac{1}{1 - 1.2Q} \qquad \textbf{Equation 10.25}$$

10.16.4 Slenderness limit

If an individual compression member has

$$\frac{\ell_u}{r} > \frac{35}{\sqrt{P_f / (f'_c A_g)}} \qquad \textbf{Equation 10.26}$$

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 10.18 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 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 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

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t + E_s I_{st}}{(E_c A_g / 5) + E_s A_t + E_s A_{st}}}$$ **Equation 10.27**

For computing P_c in Equation 10.18, EI of the composite section shall be not greater than

$$EI = \frac{0.2\, E_c I_g}{1 + \beta_d} + E_s I_t + E_s I_{st}$$ **Equation 10.28**

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

Equation 10.29

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t}{(E_c A_g / 5) + E_s A_t}}$$

For computing P_c in Equation 10.18, EI of the composite section shall be not greater than

$$EI = \frac{0.2 E_c I_g}{1+\beta_d} + E_s I_t$$ **Equation 10.30**

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 $0.25T_{cr}$.

Note: *Footings and pile caps designed using strut-and-tie models in accordance with Clause 11.4 need not satisfy the minimum shear reinforcement requirements of Clause 11.2.8.*

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

$$A_v \geq 0.06\sqrt{f'_c}\,\frac{b_w s}{f_y}$$ **Equation 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 from 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

$$T_{cr} = (A_c^2 / p_c)0.38\lambda\phi_c\sqrt{f'_c}\sqrt{1 + \frac{\phi_p f_{cp}}{0.38\lambda\phi_c\sqrt{f'_c}}}$$ **Equation 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.

11.2.10.5
The minimum area of shear reinforcement specified by Equation 11.1 is calculated using the unspalled web width. When the area of shear reinforcement exceeds eleven times this calculated minimum, spalling of the concrete cover shall be accounted for in determining b_w; otherwise spalling may be neglected. If spalling is to be accounted for, the concrete cover down to the centerline of the outermost reinforcement shall be assumed to have spalled off unless the ends of the compression diagonals are restrained against spalling.

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.

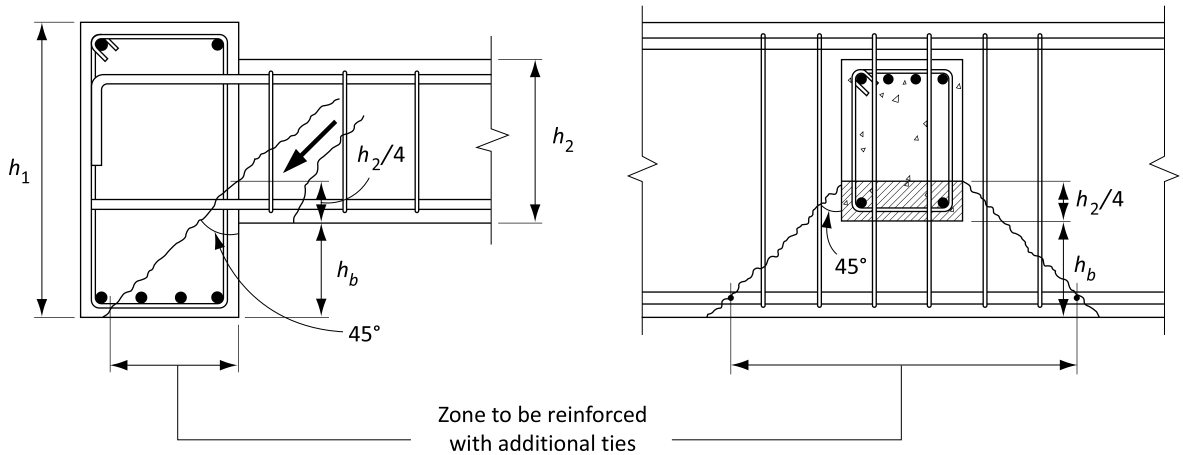
11.2.12 Hanger reinforcement for beams supporting other beams

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). The bottom longitudinal reinforcing bars in the supported beam shall be placed above the bottom bars in the supporting beam.

Figure 11.1
Location of additional transverse reinforcement
(See Clauses 3.2 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 $0.23\lambda\phi_c\sqrt{f_c'}$.

11.2.13 Termination of longitudinal reinforcement in flexural tension zones

11.2.13.1

The reductions of shear resistance caused by terminating longitudinal reinforcement in flexural tension zones shall be taken into account. It can be assumed that the reductions in shear capacity occur over a length d_v centred upon the termination point.

11.2.13.2

If the factored shear resistance has been calculated using the simplified method of either Clause 11.3.6.2 or Clause 11.3.6.3 then the calculated shear resistance within the length specified in Clause 11.2.13.1 shall be reduced by 15%.

11.2.13.3

If transverse shear reinforcement is required near the termination point, the spacing, s, of this reinforcement shall not exceed $0.35d_v$.

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

$V_r \geq V_f$ **Equation 11.3**

11.3.2 Sections near supports

11.3.2.1

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;
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; and
c) loads applied within distance d_v from the face of the support do not increase the absolute magnitude of the shear at the face by more than 20%.

11.3.2.2

For support regions satisfying Clause 11.3.2.1 a) and b) but not satisfying c), sections located less than $0.5d_v$ from the face of the support may be designed for the same shear, V_f, as that computed at a distance of $0.5d_v$ from the support face.

11.3.3 Factored shear resistance

The factored shear resistance shall be determined by

$V_r = V_c + V_s + V_p$ **Equation 11.4**

However, V_r shall not exceed

$V_{r,\max} = 0.25\phi_c f'_c b_w d_v + V_p$ **Equation 11.5**

11.3.4 Determination of V_c

The value of V_c shall be computed from

$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v$ **Equation 11.6**

where β is determined as specified in Clause 11.3.6, however β need not be taken less than 0.05.

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

$V_s = \dfrac{\phi_s A_v f_y d_v \cot\theta}{s}$ **Equation 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

$$V_s = \frac{\phi_s A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha}{s}$$ **Equation 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 two times the effective shear depth, d_v, 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 overall depth is not greater than one-half the width of web or 550 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

$$\beta = \frac{230}{(1000 + d_v)}$$ **Equation 11.9**

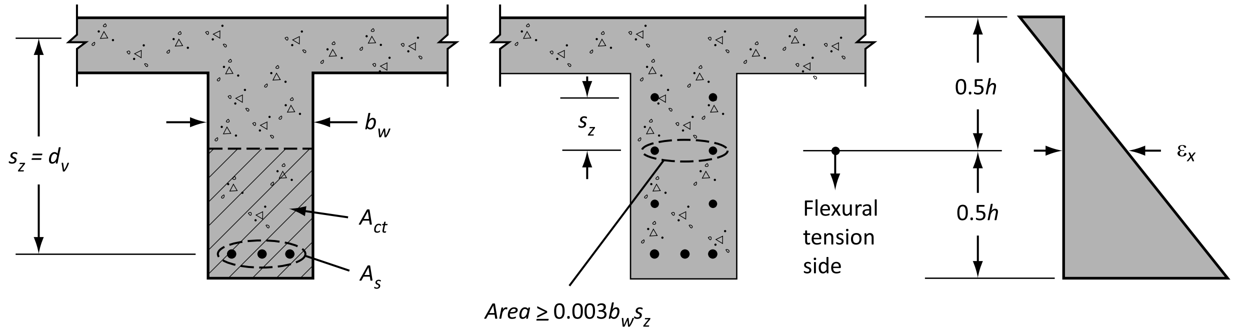
c) Alternatively, the value of β for sections containing no transverse reinforcement may be determined for all aggregate sizes by replacing the parameter d_v in Equation 11.9 by the equivalent crack spacing parameter, s_{ze}, where

$$s_{ze} = \frac{35 s_z}{15 + a_g}$$ **Equation 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 3.2 and 11.3.6.3.)



11.3.6.4 General method

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

$$\beta = \frac{0.40}{(1+1500\varepsilon_x)} \bullet \frac{1300}{(1000+s_{ze})}$$ **Equation 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

$$\theta = 29 + 7000\varepsilon_x$$ **Equation 11.12**

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

$$\varepsilon_x = \frac{M_f / d_v + V_f - V_p + 0.5N_f - A_p f_{po}}{2\left(E_s A_s + E_p A_p\right)}$$ **Equation 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. If longitudinal bars are terminated in a flexural tension zone, the value of ε_x as given by Equation 11.13 at the cutoff location shall be increased by 50%.
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.

11.3.8.4

The maximum spacing of 600 mm specified in Clause 11.3.8.1 can be waived if β is determined from Equation 11.11 and in this equation, s_{ze} is taken as (s–300).

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:

Equation 11.14

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

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, V_s shall not be taken greater than V_f and 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:

$$F_{lc} = 0.5N_f + (V_f - 0.5V_s - V_p)\cot\theta - \frac{M_f}{d_v}$$ **Equation 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. In Equation 11.15, V_s shall not be taken greater than V_f.

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 11.5 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

$$T_r \geq T_f$$ **Equation 11.16**

11.3.10.3 Factored torsional resistance

The value of T_r shall be computed from

$$T_r = 2A_o \frac{\phi_s A_t f_y}{s}\cot\theta$$ **Equation 11.17**

where
A_o = $0.85A_{oh}$
θ = 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:

$$\frac{V_f - V_p}{b_w d_v} + \frac{T_f p_h}{1.7A_{oh}^2} \le 0.25\phi_c f'_c$$

Equation 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:

$$\sqrt{\left(\frac{V_f - V_p}{b_w d_v}\right)^2 + \left(\frac{T_f p_h}{1.7A_{oh}^2}\right)^2} \le 0.25\phi_c f'_c$$

Equation 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

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

Equation 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

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

Equation 11.21

11.4 Strut-and-tie model

11.4.1 Structural idealization

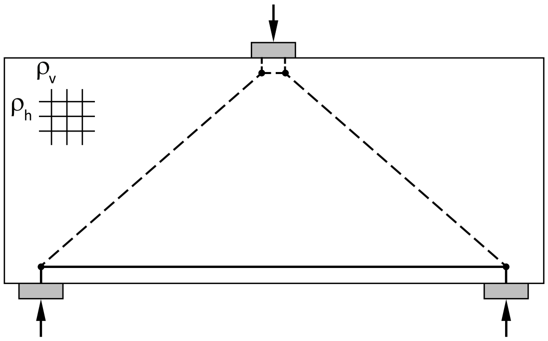
11.4.1.1 General

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.

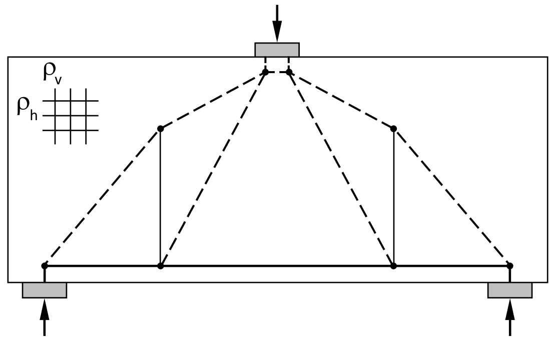
11.4.1.2 Simplified and refined strut-and-tie models

The crack control reinforcement may also be used as tension ties in the strut-and-tie model provided this reinforcement is well anchored in accordance with Clause 11.4.3.3 (see Figure 11.3).

Figure 11.3
Simplified and refined strut-and-tie models for deep beam
(See Clause 11.4.1.2.)



a) Simplified strut-and-tie model



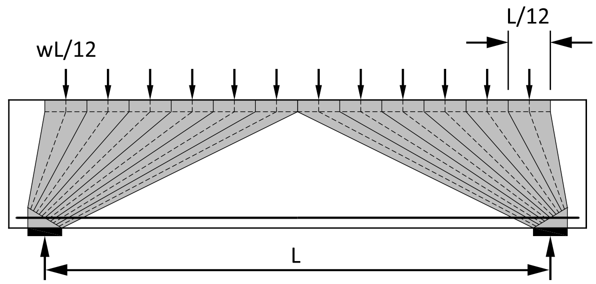
b) Refined strut-and-tie model using crack control reinforcement as additional tension ties

11.4.1.3 Modelling members subjected to uniform loads

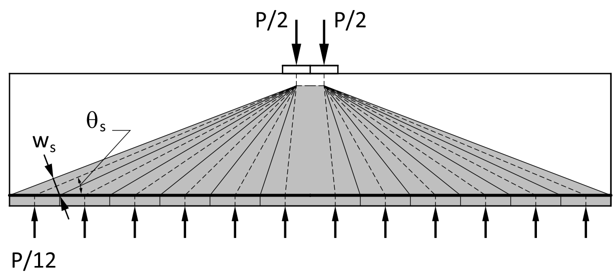
In modelling members subjected to uniform loads, a series of struts shall be used to represent the areas where the compressive stresses fan out from the bearing areas. [see Figure 11.4 a) and b)]. However, where the uniform load is applied to the flexural compression face and the member contains crack control reinforcement the fanning area can be modelled by a single strut located at the resultant of the load [see Figure 11.4 c)].

Figure 11.4
Modelling fanning regions in slabs and footings subjected to uniform loads
(See Clause 11.4.1.3.)

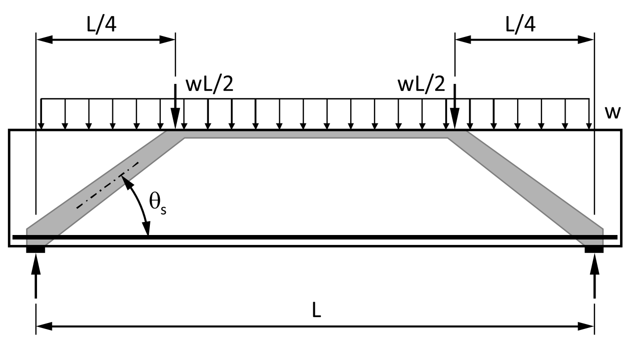
a) Modelling of fanning compression with a series of struts — Tension tie at narrow part of fan.



b) Modelling of fanning compression with a series of struts — Tension tie at wide part of fan.



c) Modelling each fanning area with a single strut for members containing crack control reinforcement.



11.4.2 Proportioning of strut

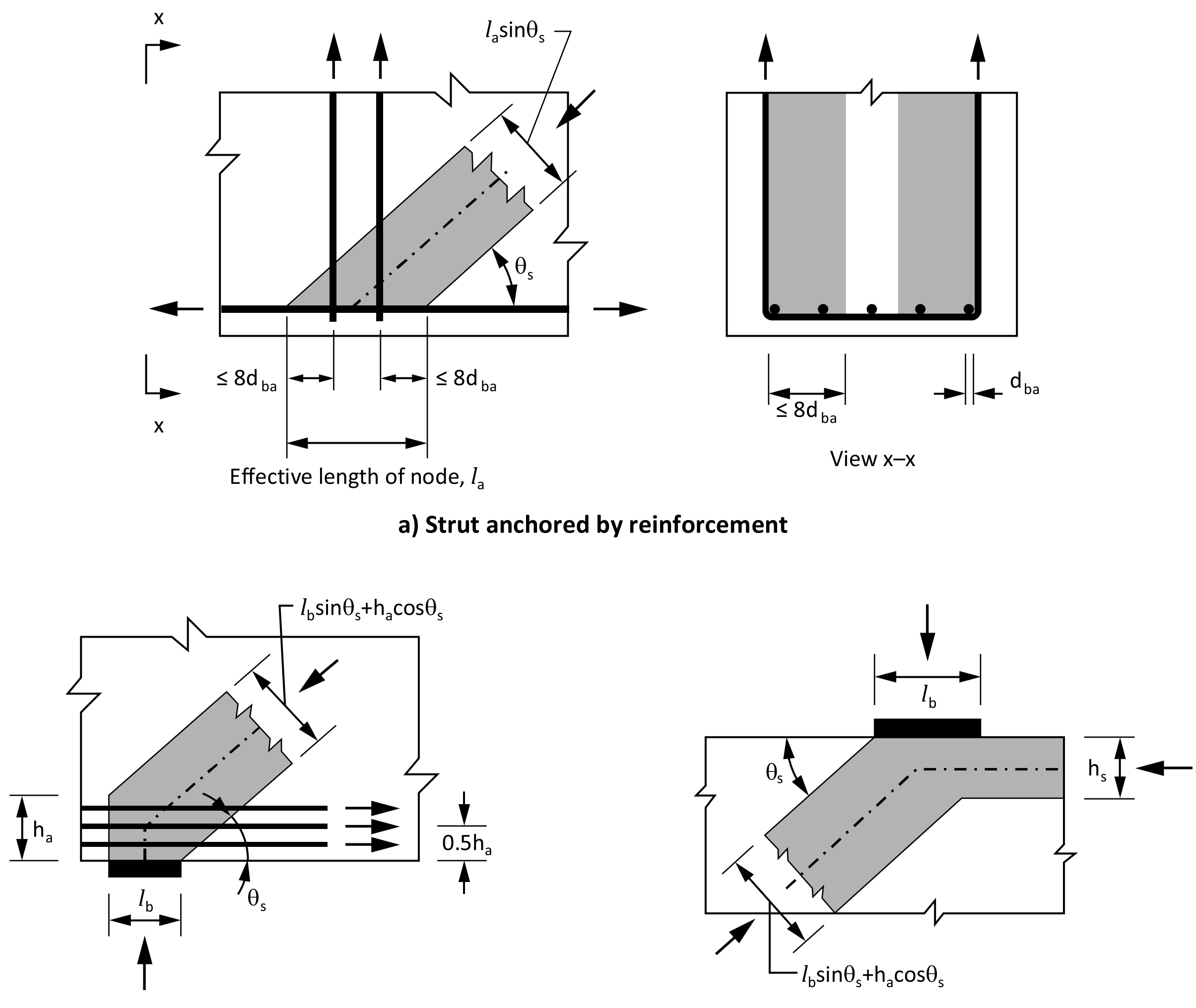
11.4.2.1 Strength of strut

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 11.5. When a strut is anchored only by reinforcement, the effective concrete area may be considered to extend a distance of up to eight bar diameters from the bar anchoring the closed stirrups and the concrete cover should be neglected as shown in Figure 11.5 a).

Figure 11.5
Influence of anchorage conditions on effective cross-sectional area of strut
(See Clauses 3.2, 11.3.9.5, and 11.4.2.2.)

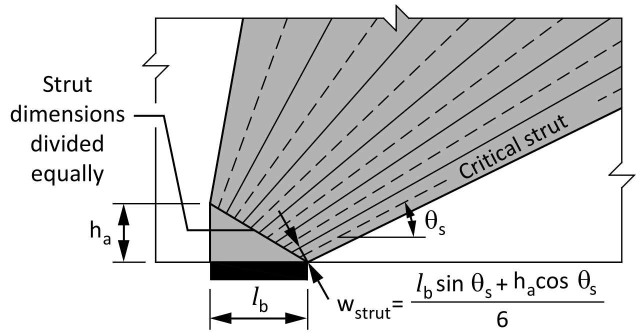


b) Strut anchored by bearing plate and reinforcement

c) Strut anchored by bearing plate and strut

When a fanning region, with the tension tie at the narrow part of fan, is modeled by a series of struts the area A_{cs} shall be taken as being equal for each strut and shall be determined at the location where the strut connects to the nodal region as shown in Figure 11.6.

Figure 11.6
Determining area A_{cs} for critical strut in fan region — Tension tie at narrow part of fan
(See Clause 11.4.2.2.)



11.4.2.3 Limiting compressive stress in struts

The value of f_{cu} shall be computed from

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \le 0.85 f'_c$$ **Equation 11.22**

where

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\theta_s$$ **Equation 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.

In lieu of using Equation 11.22, the limiting compressive stress, f_{cu}, may be calculated using Equation 11.24 provided that the specified yield strength of the reinforcing steel does not exceed 400 MPa,

$$f_{cu} = \frac{f'_c}{1.14 + 0.68\cot^2\theta_s} \le 0.85 f'_c$$ **Equation 11.24**

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 shall be large enough to ensure that the calculated tensile force in the tie does not exceed $\phi_s f_y A_{st} + \phi_p (f_{po} + 400) A_p$.

11.4.3.2 Anchorage of ties in node regions

The tie reinforcement shall be anchored to the nodal zones by straight bar embedment or standard hooks in accordance with Clause 12, or by headed bars or by prestressing anchorages 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.

11.4.3.3 Anchorage of ties anchoring compressive struts

Tension ties anchoring compressive struts in regions away from bearing regions shall be detailed as closed stirrups with each bend enclosing a longitudinal bar. Crack control reinforcement used as tension ties shall consist of closed stirrups with each bend enclosing a longitudinal bar.

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) $0.85\phi_c m f_c'$ in node regions bounded by struts and bearing areas;
b) $0.75\phi_c m f_c'$ in node regions anchoring a tie in only one direction; and
c) $0.65\phi_c m f_c'$ in node regions anchoring ties in more than one direction.

where m is the confinement modification factor taken as $\sqrt{A_2 / A_1}$ but not more than 2.0 as defined in Clause 10.8.1. The factor m shall be taken as 1.0 unless reinforcement capable of controlling cracking is provided.

Note: *Other 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 be not 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

$$v_r = \lambda\phi_c(c + \mu\sigma) + \phi_s \rho_v f_y \cos\alpha_f \qquad \textbf{Equation 11.25}$$

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
μ = 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
μ = 1.00

c) For concrete placed monolithically:
c = 1.00 MPa
μ = 1.40

d) For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars:
c = 0.00 MPa
μ = 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 11.25:

$$v_r = \lambda\phi_c k\sqrt{\sigma f'_c} + \phi_s \rho_v f_y \cos\alpha_f$$ **Equation 11.26**

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

$$\sigma = \rho_v f_y \sin\alpha_f + \frac{N}{A_g}$$ **Equation 11.27**

where

$$\rho_v = \frac{A_{vf}}{A_{cv}}$$ **Equation 11.28**

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 11.28, 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, joint shear reinforcement satisfying the requirements of Clause 7.7 shall be provided.

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

Equation 12.1

$$\ell_d = 1.15\frac{k_1 k_2 k_3 k_4}{(d_{cs} + K_{tr})}\frac{f_y}{\sqrt{f_c'}}A_b$$

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_d, 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 $100 d_b / \sqrt{f'_c}$.

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, k_5, 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

$$k_5 = \frac{f_y - 240}{f_y}$$ **Equation 12.2**

or

$$k_5 = \frac{5d_b}{s_w}$$ **Equation 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

$$\ell_d = 3.3 k_3 \frac{A_w}{s_w} \frac{f_y}{\sqrt{f'_c}}$$

Equation 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

$$\ell_d = 0.145\left(f_{pr} - 0.67 f_{pe}\right) d_b$$

Equation 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:

$$\ell_d \le \frac{M_r}{V_f} + \ell_a$$ **Equation 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 $0.073f_y d_b$ 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 3.1), 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 thickness, h_s, shall satisfy

$$h_s \geq \frac{\ell_n\left(0.6 + f_y / 1000\right)}{30}$$ **Equation 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 thickness, h_s, shall satisfy

$$h_s \geq \frac{\ell_n\left(0.6 + f_y / 1000\right)}{30} - \frac{2x_d}{\ell_n}\Delta_h$$ **Equation 13.2**

where ℓ_n is the longer clear span and Δ_h 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

$$h_s \geq \frac{\ell_n\left(0.6 + f_y / 1000\right)}{30 + 4\beta\alpha_m}$$ **Equation 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

$$I_b = \frac{b_w h^3}{12}\left(2.5\left(1 - \frac{h_s}{h}\right)\right)$$ **Equation 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 construction loads, where known and the loading conditions assumed in Clause 9.8.2.3. 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)

$$v_r = v_c = \left(1 + \frac{2}{\beta_c}\right)0.19\lambda\phi_c\sqrt{f'_c}$$ **Equation 13.5**

where

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

b)

$$v_r = v_c = \left(\frac{\alpha_s d}{b_o} + 0.19\right)\lambda\phi_c\sqrt{f'_c}$$ **Equation 13.6**

where
α_s = 4 for interior columns, 3 for edge columns, and 2 for corner columns

c)

$$v_r = v_c = 0.38\lambda\phi_c\sqrt{f'_c}$$ **Equation 13.7**

13.3.4.2
The value of $\sqrt{f'_c}$ used to calculate v_c in Equations 13.5 to 13.7 and 13.10 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 2d.

13.3.5 Factored shear stress

13.3.5.1
For corner supports not meeting the requirements of Clause 13.3.6.2, and interior and edge 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

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$ **Equation 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:

$$v_f = \frac{V_f}{b_o d} + \left(\frac{\gamma_v M_f e}{J}\right)_x + \left(\frac{\gamma_v M_f e}{J}\right)_y$$ **Equation 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_v, 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, V_c, of slabs in the vicinity of corner columns shall be taken as

$$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_o d_v$$ **Equation 13.10**

where β is as specified in Clauses 11.3.6.2 and 11.3.6.3 and b_o is determined for a critical shear section located not farther than $d/2$ from the edge of the column or column capital. The value of $\sqrt{f'_c}$ used to calculate V_c in Equation 13.10 shall not exceed 8 MPa.

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:

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/318RM.

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

$$v_s = \frac{\phi_s A_{vs} f_{yv}}{b_o s}$$ **Equation 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 0.35*d* to 0.4*d*. 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

$$v_f \le 0.56\lambda\phi_c\sqrt{f'_c}$$ **Equation 13.12**

b) $s \le 0.5d$ when

$$v_f > 0.56\lambda\phi_c\sqrt{f'_c}$$ **Equation 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:

$$m_{x,des} = m_x + |m_{xy}| \qquad \textbf{Equation 13.14}$$

$$m_{y,des} = m_y + |m_{xy}| \quad \textbf{Equation 13.15}$$

If either $m_{x,des}$ or $m_{y,des}$ is negative, it shall be taken as zero.

b) negative design moments:

$$m_{x,des} = m_x - |m_{xy}| \quad \textbf{Equation 13.16}$$

$$m_{y,des} = m_y - |m_{xy}| \quad \textbf{Equation 13.17}$$

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 3.1) 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 3.1 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:

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_c} + \frac{1}{K_t}$$ **Equation 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:

$$K_t = \sum \frac{9E_cC}{\ell_t\left(1 - \frac{c_2}{\ell_t}\right)^3}$$ **Equation 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:

$$C = \sum\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3}$$ **Equation 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 $\ell_2 /_1 \leq 1.0$: $\psi = 0.3 + 0.7\dfrac{\alpha_1 \ell_2}{\ell_1}$ **Equation 13.21**

b) for $\ell_2 / \ell_1 > 1.0$: $\psi = 0.6\left(\dfrac{\ell_2}{\ell_1} - 0.5\right) + \left(1.3 - 0.6\dfrac{\ell_2}{\ell_1}\right)\dfrac{\alpha_1 \ell_2}{\ell_1}$ **Equation 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 3.1) 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

$$M_o = \frac{w_f \ell_{2a} \ell_n^2}{8}$$

Equation 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 unre-strained	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:

$$M_f = 0.07\left(\left(w_{df} + 0.5w_{lf}\right)\ell_{2a}\ell_n^2 - w'_{df}\ell'_{2a}(\ell'_n)^2\right)$$ **Equation 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

$$\gamma_f = 1 - \gamma_v$$ **Equation 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 in 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 $s \leq 250$ mm
Remaining negative moment reinforcement:	$3h_s$, but $s \leq 500$ mm
Positive moment reinforcement:	$3h_s$, but $s \leq 500$ 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

$$\sum A_{sb} = \frac{2V_{se}}{f_y}$$ **Equation 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 3.1) 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.

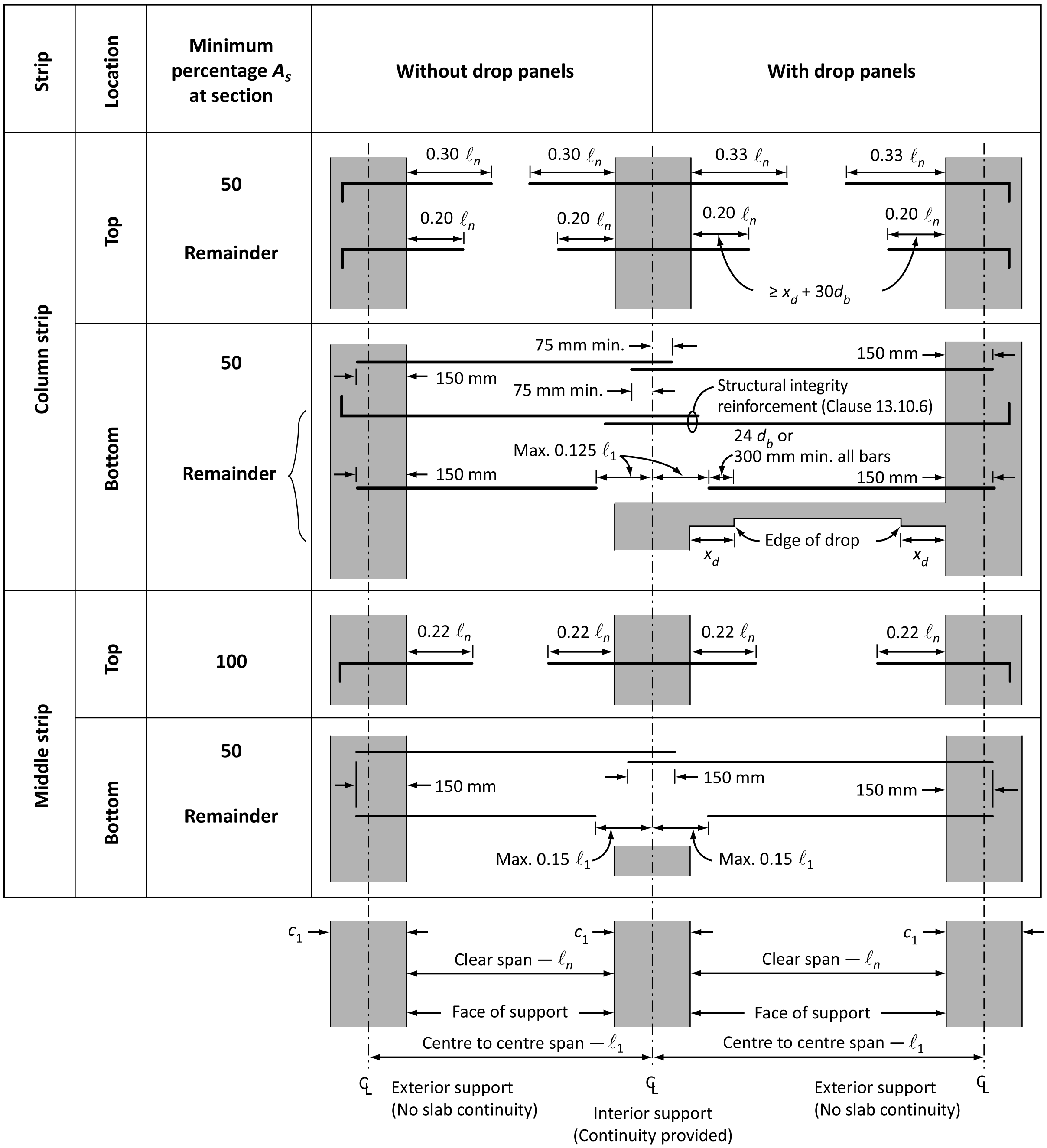
13.10.8.2

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 3.2 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 3.1) 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
Except as permitted in Clause 13.11.2.5, 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 3.1 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 5 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 required as compression reinforcement 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 ratio within any region of concentrated reinforcement, including regions containing lap splices, shall be not more than 0.08.

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:

$$P_r = \frac{2}{3}\alpha_1 \phi_c f'_c A_g \left(1 - \left(\frac{kh_u}{32t}\right)^2\right)$$

Equation 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

14.2.3

If present, the calculation of the factored resistance of bearing walls shall account for significant strong axis bending moments in accordance with Clause 10.

Note: *Strong axis bending moments may be applied to bearing walls due to the resultant of the axial load not being at the centroid of the section due to in-plane offset of the wall or may be induced by deformation of the lateral force resisting system subjected to the factored lateral loads such as wind and seismic loads.*

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 Design of flexural shear walls

14.4.2.1 General

The design of flexural shear walls shall satisfy either Clause 14.4.2.2 or Clause 14.4.2.3.

14.4.2.2 Low axial compression

When the factored axial compression applied to a wall is such that the tension reinforcement will reach yield in accordance with Clause 10.5.2, the compression end of the wall shall have a wall thickness of at least $h_u/20$ unless

a) the compression strain depth, c, is smaller than the lesser of $4b_w$ or $0.3\ell_w$ or;
b) a continuous line of lateral support is provided to the compression end of the wall by a cross wall or wall flange having a width not less than $h_u/5$.

14.4.2.3 High axial compression

When the factored axial compression applied to a wall is such that the tension reinforcement will not reach yield in accordance with Clause 10.5.2, the design of the wall shall consider slenderness effects. However, the influence of wall slenderness need not 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 cross wall or a flange having a width not less than $h_u/5$.

The required thickness of a wall subjected to the factored axial compression and moment about the strong axis may be determined using a reduced length of wall. The reduced length of wall shall be such that the resultant axial force representing axial load and moment acts at the centre of the reduced length. The thickness of the wall shall satisfy the slenderness requirements of Clause 10.15 for the factored axial load applied to the reduced length of wall.

14.4.3 Assemblies of interconnected shear walls

14.4.3.1 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.

14.4.3.2 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

$$\omega = 1.0 - 0.025(\ell_c / t - 15) \qquad \textbf{Equation 14.2}$$

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.

14.4.3.3 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.

14.4.4 Horizontal reinforcement in shear walls

Horizontal reinforcement shall extend to the ends of the wall with horizontal reinforcement required for shear to be anchored in accordance with Clause 12.13.2 or anchored within regions of concentrated reinforcement at the ends of the wall.

14.4.5 Weak axis bending

Weak axis bending of shear walls shall be considered in conjunction with strong axis bending.

14.4.6 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.

14.4.7 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 21.5.8.2. 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:

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)} \qquad \textbf{Equation 15.1}$$

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 prequalified in accordance with CSA A23.4, the concrete material resistance factor, ϕ_c, specified in Clause 8.4.2 of this Standard 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) minor structural elements such as stair flights, stair landings, lintels, and sills; and
b) 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 be not 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
Tension ties shall be incorporated in floor and roof systems to provide a factored resistance of not less than 14 kN per metre of horizontal wall length for longitudinal ties and 14 kN per metre of floor or roof span for transverse ties. 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 horizontal wall length. 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

$V_{r\ell} \geq V_f$ **Equation 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 4.1.4 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 except as permitted by Item b)	$0.6f'_{ci}$
b) Extreme fibre stress in compression at ends of simply supported members	$0.67f'_{ci}$
c) Extreme fibre stress in tension, except as permitted by Item d)	$0.25\lambda\sqrt{f'_{ci}}$
d) 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	$0.45f'_c$
b) Extreme fibre stress in compression due to total load	$0.60f'_c$
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 $0.6f_{py}$:

$$f_{pr} = f_{pu}\left(1 - k_p \frac{c}{d_p}\right)$$ **Equation 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:

$$f_{pr} = f_{pe} + \frac{8000}{\ell_o}\sum_n \left(d_p - c_y\right) \le f_{py}$$ **Equation 18.2**

where
$\sum_n \left(d_p - c_y\right)$ = 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:

$$M_r \ge 1.2M_{cr}$$ **Equation 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:

$$M_{dc} = f_{ce}\frac{I}{y_t}$$ **Equation 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

Equation 18.5

$$v_c = \beta_p \lambda \phi_c \sqrt{f_c'} \sqrt{1 + \frac{\phi_p f_{cp}}{0.33 \lambda \phi_c \sqrt{f_c'}}} + \frac{V_p}{b_o d}$$

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);
c) methods based on tests; or
d) simplified equations where applicable.

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.

18.13.5
Three dimensional effects shall be considered in design and analyzed using three-dimensional procedures or approximated by considering the summation of effects for two orthogonal planes.

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

Notes:

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, 20.3.1.10, 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
Before conducting a load test in accordance with this Clause, calculations shall be made to ensure that the expected failure mode is ductile. If the expected failure mode is brittle, load testing using the provisions of this Clause shall not be carried out.

Note: *The provisions of Clause 20 are based on deflection recovery of systems or members with capacities that are governed by ductile flexural failure. A load test in accordance with Clause 20 is inappropriate for assessing the reliability of a member with a brittle failure mode.*

20.3.1.2
If the strength evaluation is based on load tests, an engineer experienced in such evaluations shall control the tests.

20.3.1.3
A load test shall generally not be conducted until the portion of the structure subjected to load is at least 28 days 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 days old.*

20.3.1.4
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.

20.3.1.5
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.

20.3.1.6
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.

20.3.1.7
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.

20.3.1.8
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.

20.3.1.9
The test load shall be left on the structure for 24 h.

20.3.1.10
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 20.3.1.10.*
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 21.11 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 from the non-linear response of the seismic-force-resisting system (SFRS).

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 21.8. All other structural elements in the SFRS are then provided with sufficient reserve capacity to ensure that the chosen energy-dissipating mechanisms are maintained in the selected locations throughout the deformations that can occur.

21.2.2 Seismic-force-resisting systems

Clauses 21.2 to 21.8 specify requirements covering the design and detailing of the standard seismic-force-resisting systems (SFRSs) identified in the *National Building Code of Canada (NBCC)*.

Clauses 21.2 to 21.8 were developed for the design of individual SFRSs that are continuous over the full building height and do not have significant discontinuity in strength or stiffness.

When an SFRS has a strength, stiffness, or geometrical irregularity as defined by the *NBCC*, or when combinations of SFRSs acting in the same direction are not continuous over the building height, or when elements from two or more SFRS types are combined to create a hybrid system, the design of the SFRS and gravity-load resisting frame shall account for the actual inelastic behaviour of the SFRS. An evaluation shall be carried out to

a) verify the compatibility of the system(s);
b) confirm the assumed inelastic (energy-dissipating) mechanism(s);
c) confirm that the inelastic demands are less than the inelastic capacities; and
d) account for redistribution of forces.

The requirement for such an evaluation may be waived if the performance has been previously verified by experimental evidence and analysis.

A reinforced concrete seismic-force-resisting system other than those specified in Clauses 21.2 to 21.8 may be used as an alternative solution if it can be demonstrated through testing, research, and analysis that the seismic performance of the structural system is at least equivalent to that of a standard SFRS that meets the requirements of Clauses 21.2 to 21.8.

Note: *The requirements for determining R_d and R_o of the system are given in the* National Building Code of Canada *and Commentary J, and the Compliance requirements for alternative solutions are given in Division A of NBCC.*

21.2.3 Design based on nonlinear dynamic analysis

Nonlinear dynamic analysis used for the basis of design shall include a sufficient number of appropriate ground motion time histories and shall include appropriate nonlinear material models for the structural elements that consider the reversed cyclic loading characteristics such as strength and stiffness degradation, ductility capacity and general hysteretic response. The nonlinear analysis and resulting design shall be reviewed by a qualified independent review panel.

Note: *The* National Building Code of Canada *and Commentary J provide the requirements for a special study using nonlinear dynamic analysis. The 2011 LATBSDC (Los Angeles Tall Buildings Structural Design Council)* Alternate Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Area *provides the requirements for a qualified independent review panel.*

21.2.4 Applicable clauses

21.2.4.1

The requirements of Clauses 1 to 18 and 23 shall apply to the design and detailing of structural members unless modified by the requirements of Clause 21.

21.2.4.2

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.

21.2.4.3

Regardless of the type of seismic-force-resisting system (SFRS), all members of the structure assumed not to be part of the SFRS might be required to satisfy Clause 21.11 depending on the value of $I_E F_a S_a(0.2)$ and the calculated maximum interstorey drift as given in Clause 21.11.1.1.

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

For the purpose of determining forces in and deflections of the structure, reduced section properties shall be used. The effective section 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	$I_e = 0.4\, I_g$
Column	$I_e = \alpha_c\, I_g$
Coupling beam (Clause 21.5.8.1)	$A_{ve} = 0.15 A_g$; $I_e = 0.4\, I_g$
Coupling beam (Clause 21.5.8.2)	$A_{ve} = 0.45\, A_g$; $I_e = 0.25\, I_g$
Slab frame element	$I_e = 0.2\, I_g$
Wall	$A_{xe} = \alpha_w A_g$; $I_e = \alpha_w I_g$

Where the values of α_c and α_w specified in Table 21.1 shall be determined as follows:

a)

$$\alpha_c = 0.5 + 0.6 \frac{P_s}{f'_c A_g} \leq 1.0$$

Equation 21.1

b)

$$\alpha_w = 1.0 - 0.35\left(\frac{R_d R_o}{\gamma_w} - 1.0\right) \geq 0.5 \text{ and } \leq 1.0$$ **Equation 21.2**

Where γ_w in Equation 21.2 shall be determined at the base of the wall, and in lieu of more detailed analysis, γ_w may be taken equal to R_o.

21.2.5.3

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 $R_d R_o / I_E$. The value of Q shall not exceed 1/3.

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 grade

21.2.7.1.1

Reinforcement for SFRS designed with a force modification factor, R_d, greater than 2.5 shall be weldable grade in compliance with CSA G30.18. Reinforcement for SFRS designed with a force modification factor, R_d, of 2.5 or less shall comply with CSA G30.18; but need not be weldable grade.

21.2.7.1.2

The design, detailing, and ductility requirements for structures designed using a reinforcement grade greater than 400 shall account for the increased strain demands.

Note: *The procedures specified in Clause 21, with the exception of Clause 21.2.8.2 were developed for Grade 400 reinforcement. The additional strains required for higher yield-strength steel will generally reduce ductility.*

21.2.7.2 Lap splices

21.2.7.2.1

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 resisting earthquake-induced forces.

21.2.7.2.2
Restrictions or special requirements for lap splices are given in Clauses 21.3.1.3.3 and 21.3.2.5.2 for ductile moment-resisting frame members; Clause 21.5.4.1 for moderately ductile and ductile shear walls; Clause 21.5.6.5 for ductile walls; Clause 21.6.3.7.2 for conventional walls; Clause 21.8.2.1 for precast concrete frames; and Clauses 21.9.4.3 and 21.9.4.4 for structural diaphragms.

21.2.7.3 Mechanical splices

21.2.7.3.1
Mechanical splices shall be classified as either Type 1 or Type 2, as follows:
a) Type 1 mechanical splices shall comply with Clause 12.14.3.4.
b) Type 2 mechanical splices shall comply with Clause 12.14.3.4 and shall develop the minimum tensile strength of the spliced bar.

Note: *See CSA G30.18 for determining the minimum tensile strength.*

21.2.7.3.2
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.

21.2.7.3.3
Restrictions or special requirements for mechanical splices are given in Clause 21.5.4.2 for moderately ductile and ductile walls; Clause 21.8.2.1 for ductile moment-resisting frames constructed using precast concrete; and Clause 21.9.4.4 for structural diaphragms.

21.2.7.4 Welded splices

21.2.7.4.1
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.

21.2.7.4.2
Welding of stirrups, ties, inserts, or similar elements to longitudinal reinforcement that is required by design shall not be permitted.

21.2.8 Special ties for compression members

21.2.8.1 Buckling prevention ties
Buckling prevention ties shall comply with Clause 7.6.5.5 or 7.6.5.6 and shall be detailed as hoops, seismic crossties or spirals. 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.

21.2.8.2 Confinement reinforcement

Confinement reinforcement shall satisfy the following conditions:

a) the volumetric ratio of circular hoop reinforcement, ρ_s, shall be not less than

$$P_s = C_c k_p \frac{f'_c}{f_{yh}}$$ **Equation 21.3**

where
C_c = 0.3 for systems with R_d = 2.0 or 2.5
= 0.4 for systems with R_d > 2.5
f_{yh} = shall not be taken as greater than 500 MPa
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:

$$A_{sh} = C_h k_n k_p \frac{A_g f'_c}{A_{ch} f_{yh}} s h_c$$ **Equation 21.4**

$$A_{sh} = 0.09 \frac{f'_c}{f_{yh}} s h_c$$ **Equation 21.5**

where
C_h = 0.15 for systems with R_d = 2.0 or 2.5
= 0.2 for systems with R_d > 2.5
f_{yh} = shall not be taken as greater than 500 MPa

c) transverse reinforcement may be provided by single or overlapping hoops. Seismic 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.

21.3 Ductile moment-resisting frames (R_d = 4.0)

21.3.1 Ductile moment-resisting frame members subjected to predominant flexure

21.3.1.1 General

The requirements of Clause 21.3.1 shall apply to ductile moment-resisting frame members that are subjected to an axial compressive force due to factored load effects not exceeding $A_g f'_c/10$. Where the compressive force exceeds $A_g f'_c/10$, these members shall be designed in accordance with Clause 21.3.2

21.3.1.2 Dimensional limitations

Ductile moment-resisting frame members subjected to primarily flexure shall satisfy the following:

a) the clear span of the member shall be not less than four times its effective depth;

b) the width of the member, b_w, shall not be less than the smaller of 0.3h and 250 mm; and

c) the width of the member shall not exceed the width of the supporting member, c_2, plus a distance on each side of the supporting member equal to the smaller of the width of the supporting member, c_2, and 0.75 times the depth of the supporting member, c_1.

Ductile frame members not meeting these dimensional limitations shall be designed as specified in Clause 21.11 and shall not be considered part of the SFRS.

21.3.1.3 Longitudinal reinforcement

21.3.1.3.1
At any section of a flexural member, the areas of top reinforcement and bottom reinforcement shall each be not less than $1.4b_w d/f_y$, and the reinforcement ratio, ρ, shall not exceed 0.025. At least two effectively continuous bars shall be provided at both top and bottom.

21.3.1.3.2
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.

21.3.1.3.3
Lap splices of flexural reinforcement may be used 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.

21.3.1.4 Transverse reinforcement

21.3.1.4.1
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.

21.3.1.4.2
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.

21.3.1.4.3
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.

21.3.1.4.4
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 seismic 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.

21.3.1.4.5

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.

21.3.1.5 Shear resistance requirements

21.3.1.5.1 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 resistance 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 $R_d R_o$ equal to 1.3.

21.3.1.5.2 Shear reinforcement

Shear reinforcement shall be designed to the requirements of Clause 11, with the following exceptions:
a) the values of θ = 45° and β = 0 shall be used in the regions specified in Clause 21.3.1.4.1; and
b) transverse reinforcement required to resist shear shall be hoops over the lengths of members, as specified in Clause 21.3.1.4.1.

21.3.2 Ductile moment-resisting frame members subjected to flexure and significant axial load

21.3.2.1 General

The requirements of Clause 21.3.2 shall apply to ductile moment-resisting frame members that are subjected to an axial compressive force due to factored load effects that exceeds $A_g f'_c/10$

21.3.2.2 Dimensional limitations

Ductile moment-resisting frame members subjected to flexure and significant axial load shall satisfy the following:
a) the shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall be not less than 300 mm; and
b) the ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be not less than 0.4.

21.3.2.3 Design of nonconforming members

Ductile frame members not meeting the requirements of Clause 21.3.2.2 shall be designed as specified in Clause 21.11 and shall not be considered part of the SFRS.

21.3.2.4 Minimum flexural resistance of columns

21.3.2.4.1

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 21.3.2.4.2. Columns not meeting the requirements of Clause 21.3.2.4.2 shall not be considered as contributing to the resistance of the SFRS and shall meet the requirements of Clause 21.11.

21.3.2.4.2

The flexural resistances of the columns and the beams shall satisfy

$$\Sigma M_{nc} \geq \Sigma M_{pb}$$ **Equation 21.6**

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 resistance 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 21.6 shall be satisfied for beam moments acting in either direction.

21.3.2.4.3

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 21.3.1.5.1 and using nominal rather than probable resistance. Allowance may be made for the reduction in accumulated beam shears with increasing numbers of storeys.

Note: *Cut off at $R_d R_o$ equal to 1.3 is included in Clause 21.3.1.5.1.*

21.3.2.5 Longitudinal reinforcement

21.3.2.5.1

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.

21.3.2.5.2

Lap splices may be used 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 21.3.2.6.2 and 21.3.2.6.3.

21.3.2.6 Transverse reinforcement

21.3.2.6.1

Shear reinforcement shall meet the requirements of Clause 21.3.2.7.

21.3.2.6.2

Confinement reinforcement in accordance with Clause 21.2.8.2 shall be provided unless a larger amount is required by Clause 21.3.2.6.3 or 21.3.2.7.

21.3.2.6.3
Confinement 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:

$$s_x = 100 + \left(\frac{350 - h_x}{3}\right)$$ **Equation 21.7**

21.3.2.6.4
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.

21.3.2.6.5
Confinement reinforcement 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.

21.3.2.6.6
Columns that can develop plastic hinges because of their connection to rigid members such as transfer girders, foundations or discontinued walls, or because of their position at the base of the structure shall be provided with confinement reinforcement over their clear height. This transverse reinforcement shall continue into the rigid 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 21.10.4.

21.3.2.6.7
Where confinement reinforcement 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.

21.3.2.7 Shear resistance

21.3.2.7.1
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 resistance 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 resistance 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 1.3.

21.3.2.7.2

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 21.3.2.6.5; and

b) the transverse reinforcement required to resist shear shall be hoops or spirals.

21.3.3 Joints of ductile moment-resisting frames

21.3.3.1 General

21.3.3.1.1

The requirements of Clauses 21.3.3.1.2 to 21.3.3.5.7 shall apply to joints of ductile frames serving as parts of the SFRS.

21.3.3.1.2

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 $R_d R_o$ equal to 1.3.

21.3.3.1.3

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 21.3.3.5 and in compression as specified in Clauses 12.3 and 12.5.5.

21.3.3.2 Transverse reinforcement in joints

21.3.3.2.1

Confinement reinforcement, as specified in Clauses 21.2.8.2 and 21.3.2.6 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 21.3.3.2.2.

21.3.3.2.2

Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half of the amount required by Clause 21.2.8.2 but not less than the amount required by Clause 21.3.2.6.3 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 21.3.2.6.3 may be taken as 150 mm.

21.3.3.2.3

Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of Clause 21.3.1.4.2, and the requirements of Clauses 21.3.1.4.3 and 21.3.1.4.4, if such confinement is not provided by a beam framing into the joint.

21.3.3.3 Longitudinal column reinforcement

21.3.3.3.1

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.

21.3.3.3.2

Longitudinal column reinforcement in rectangular column cores shall have the reinforcement in each face uniformly distributed along that face, with a centre-to-centre spacing corresponding to the tie spacing specified in Clause 21.3.2.6.4.

21.3.3.4 Shear resistance of joints

21.3.3.4.1

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.

21.3.3.4.2

The shear force, V_{fb}, in the joint determined using the forces specified in Clause 21.3.3.1.2, and accounting for other forces on the joint, shall not exceed the factored resistance specified in Clause 21.3.3.4.1.

21.3.3.5 Development length for tension reinforcement in joints

21.3.3.5.1

Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the column core and anchored in tension using standard 90° hooks.

21.3.3.5.2

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

$$\ell_{dh} = 0.2\frac{f_y}{\sqrt{f'_c}}d_b \qquad \textbf{Equation 21.8}$$

21.3.3.5.3
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 21.3.3.5.2.

21.3.3.5.4
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 21.3.3.5.2 or 21.3.3.5.3, 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 21.3.3.5.2 or 21.3.3.5.3, if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.

21.3.3.5.5
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.

21.3.3.5.6
The diameter of straight beam and column bars passing through the joint shall satisfy the following equation:

$$d_b \le \lambda \frac{\ell_j}{24k_2}$$ **Equation 21.9**

21.3.3.5.7
If epoxy-coated reinforcement is used, the development lengths specified in Clauses 21.3.3.5.2 to 21.3.3.5.5 shall be multiplied by the applicable factor specified in Clause 12.2.4 or 12.5.3.

21.4 Moderately ductile moment-resisting frames (R_d = 2.5)

21.4.1 General
The requirements specified in Clause 21.4 shall apply to moment-resisting frames SFRS designed using a force modification factor, R_d, of 2.5.

21.4.2 Dimensional limitations

21.4.2.1 Beams
Beams shall satisfy the following dimensional limitations:
a) the clear span of the member shall be not less than three times its effective depth;
b) the width-to-depth ratio of the cross-section shall be not less than 0.3; and
c) 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 the smaller of the width of the supporting member or three-quarters of the depth of the supporting member.

21.4.2.2 Columns
Columns shall satisfy the following dimensional limitations:
a) the shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall be not less than 250 mm; and

b) the ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be not less than 0.4.

21.4.3 Detailing of beams

21.4.3.1

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.

21.4.3.2

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 mid-span. 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.

21.4.3.3

Stirrups shall be spaced not more than $d/2$ throughout the length of the member.

21.4.4 Detailing of columns

21.4.4.1

Transverse reinforcement shall be detailed as hoops and seismic crossties or spirals.

21.4.4.2

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 resistance 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 $R_d R_o$ equal to 1.3.

21.4.4.3

Confinement reinforcement in accordance with Clause 21.2.8.2 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.

21.4.4.4
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.

21.4.4.5
Columns that can develop plastic hinges because of their connection to rigid members such as transfer girders, foundations or discontinued walls, or because of their position at the base of the structure, shall be provided with transverse confinement reinforcement over their clear height in accordance with the requirements of Clause 21.2.8.2.

21.4.5 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 resistances 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 1.3.

21.4.6 Joints in frames

21.4.6.1
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 1.3. Where beams frame into the joint from two directions, each direction may be considered independently.

21.4.6.2
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: $1.7\lambda\phi_c\sqrt{f'_c}A_j$;
b) for joints confined on three faces or on two opposite faces: $1.2\lambda\phi_c\sqrt{f'_c}A_j$; and
c) for other joints: $1.0\lambda\phi_c\sqrt{f'_c}A_j$.

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.

21.4.6.3
The amount of transverse reinforcement in each joint shall not be less than
a) the larger of the amount of transverse reinforcement provided in the column above the joint and the column below the joint; or
b) one half of the amount required in Item a) for confined joints as defined in Clause 21.4.6.2.

21.4.6.4
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.

21.4.6.5
The diameter of straight beam and column bars passing through the joint, d_b, shall satisfy the following equation:

$$d_b \le \lambda \frac{\ell_j}{20k_2}$$ **Equation 21.10**

21.5 Ductile and moderately ductile walls (R_d = 2.0, 2.5, 3.5, or 4.0)

21.5.1 General

21.5.1.1 Minimum requirements
All shear walls shall satisfy the requirements of Clause 14 except as modified by the requirements of Clause 21.5.

21.5.1.2 Shear walls designed for flexural ductility
All shear walls and coupled or partially coupled shear walls with $h_w / \ell_w > 2.0$ and designed for forces calculated using $R_d \ge 2.0$ shall meet the additional requirements specified in Clauses 21.5.2 to 21.5.9.

21.5.1.3 Walls with multi-level openings
A wall with large openings at multiple levels shall be designed in accordance with the additional requirements of Clause 21.5.8 unless the openings are limited to a small portion of the wall height and the solid wall segment(s) above and below the openings, that connect the vertical wall piers, have sufficient strength and stiffness for the wall assembly to act as a single cantilever wall.

Note: *The intent of this Clause is to ensure that any wall that may act as a coupled or partially coupled wall satisfies all the requirements for one of the four systems in Clause 21.5.8.*

21.5.1.4 Squat walls
All shear walls with $h_w / \ell_w \le 2.0$ shall meet the additional requirements specified in Clause 21.5.10.

21.5.1.5 Tilt-up and precast walls
Tilt-up concrete walls shall be designed in accordance with Clause 21.7. Precast walls shall be designed in accordance with Clause 21.8, except that solid precast wall panels may be designed in accordance with Clauses 21.7 if the wall panels also meet the requirements of Clause 23.

21.5.2 Requirements for strength and ductility over height

21.5.2.1 Plastic hinge regions in walls

21.5.2.1.1 General
Shear walls and coupled or partially coupled shear walls with $h_w / \ell_w > 2.0$ and $R_d \ge 2.0$ shall be designed for flexural ductility resulting from yielding of the vertical reinforcement in plastic hinge regions.

21.5.2.1.2 Minimum height of plastic hinge regions

A plastic hinge region shall extend a minimum distance of $0.5\ell_w + 0.1h_w$ above the critical section where the vertical reinforcement will first yield; where ℓ_w is the length of the longest shear wall or overall length of the coupled shear walls in the direction under consideration. The plastic hinge region shall extend below the critical section where the vertical reinforcement will first yield in accordance with Clauses 21.5.2.1.3 and 21.5.2.1.4.

21.5.2.1.3 Plastic hinge region at base

When first yielding of the vertical reinforcement in a wall is expected to occur at a well-defined critical section near the base of the wall and the SFRS does not contain structural irregularity types 1, 3, 4, 5, or 6 defined in the *NBCC* anywhere over the building height, the walls may be designed for a single plastic hinge region at the well-defined critical section at the base.

The plastic hinge region at the base shall extend below the critical section all the way to the footing unless all walls have adequate flexural over-strength in accordance with Clause 21.5.2.2.4 to ensure that yielding will not occur below the critical section; in which case the plastic hinge region shall extend down the distance specified in Clause 21.5.2.1.2 or to the footing, whichever is less.

21.5.2.1.4 Plastic hinge regions at irregularities

For buildings containing a Type 1 (vertical stiffness) irregularity or Type 3 (vertical geometrical) irregularity defined in the *NBCC* anywhere over the building height, the walls shall be detailed for plastic hinge regions at the base and at the location of each irregularity. The plastic hinge region shall extend from the minimum distance specified in Clause 21.5.2.1.2 above the irregularity to the minimum distance specified in Clause 21.5.2.1.2 below the irregularity.

In addition, each individual wall that contains a significant structural irregularity, such as a large opening, anywhere over the height, shall be designed such that there is no discontinuity in flexural capacity and shear capacity or shall be detailed for plastic hinging at the location of the irregularity.

21.5.2.2 Design for shear force and bending moment

21.5.2.2.1 Shear force and bending moment envelopes

The factored shear force envelope and factored bending moment envelope shall be calculated for each wall in accordance with the *NBCC* and increased as required by Clauses 21.5.2.2.2 to 21.5.2.2.9.

Note: *The* National Building Code of Canada *(*NBCC*) specifies when a dynamic analysis must be used to determine the shear force and bending moment envelopes and when the equivalent static force procedure may be used.*

21.5.2.2.2 Design for bending moment at base

Each wall shall be proportioned such that the factored bending resistance calculated in accordance with Clause 10 is greater than the factored bending moment at the critical section defined in Clause 21.5.2.1.2.

The properties of the wall cross section that affect the bending resistance of the wall shall be maintained over the height of the plastic hinge specified in Clause 21.5.2.1.2.

Note: *The cross sectional properties that affect bending resistance include concrete geometry, concrete strength and the reinforcing steel. The bending resistance of the wall will reduce over the height of the plastic hinge due to the reduction in axial compression force from gravity loads, which is not a property of the cross section.*

21.5.2.2.3 Design for bending moment above plastic hinge at base

The factored bending moment envelopes determined in accordance with Clause 21.5.2.2.1 shall be increased to ensure that flexural yielding of the wall will not first occur above the plastic hinge region as follows:

a) When dynamic analysis is used to determine the bending moment envelope, the factored bending moments at all elevations above the plastic hinge region shall be increased by the ratio of factored bending moment resistance to factored bending moment, calculated at top of plastic hinge region.
b) When the equivalent static force procedure is used, the factored bending moment in the wall shall be assumed to vary linearly from the factored bending moment resistance at the top of the plastic hinge region to zero at a point located the height given in Clause 21.5.2.1.2 above the top of the wall.

21.5.2.2.4 Design for bending moment below plastic hinge at base

To ensure adequate flexural over-strength below the plastic hinge region at the base, each wall shall satisfy all of the following:

a) the factored flexural resistance at any point below the critical section shall be greater than the factored bending moment determined by analysis in accordance with Clause 21.5.2.2.1 increased by the ratio of nominal bending moment resistance to factored bending moment, both calculated at the bottom of the plastic hinge region;
b) the factored flexural resistance at any point below the critical section shall be greater than the factored bending moment determined by analysis in accordance with Clause 21.5.2.2.9; and
c) the portion of wall immediately below the critical section at the base shall contain a minimum of 20% additional flexural tension reinforcement than the wall immediately above the critical section.

21.5.2.2.5 Design for shear force at base

Each wall shall be proportioned such that the factored shear resistance calculated in accordance with Clause 21.5.9 is greater than the factored shear force at the base. The factored shear force at the base shall be determined in accordance with Clause 21.5.2.2.1, increased to account for flexural overstrength in accordance with Clause 21.5.2.2.6, and further increased for inelastic effects of higher modes in accordance with Clause 21.5.2.2.7. The factored shear force need not be taken larger than the shear force resulting from design load combinations that include earthquake, with load effects calculated using $R_d R_o$ equal to 1.3.

The properties of the wall cross section that affect the shear resistance of the wall shall be maintained over the height of the plastic hinge region specified in Clause 21.5.2.1.2.

Note: *The cross sectional properties that affect shear resistance include concrete geometry, concrete strength, and reinforcing steel.*

21.5.2.2.6 Accounting for flexural overstrength

The factored shear force at the base determined in accordance with Clause 21.5.2.2.1 shall be increased by the ratio of the bending moment capacity given below to the factored bending moment, both calculated at the base.

	$R_d \leq 2.5$	$R_d \geq 3.5$
Bending moment capacity:	nominal	probable

21.5.2.2.7 Accounting for inelastic effects of higher modes

Except for coupled and partially coupled shear walls, the increased factored shear force determined in accordance with Clause 21.5.2.2.6 shall be further increased depending on the fundamental lateral period of vibration of the building T_a in the direction under consideration as follows:

$T_a \le T_L$	$T_a \ge T_U$
1.0	$1.0 + 0.25\left(R_dR_o/\gamma_w - 1\right) \le 1.5$ and ≥ 1.0

For T_a between T_L and T_U, linear interpolation shall be used, where T_L and T_U shall be determined as follows:

	T_L	T_U
$S(0.2)/S(2.0) < 10.0$	0.5 s	1.0 s
$S(0.2)/S(2.0) \ge 10.0$	0.2 s	0.5 s

21.5.2.2.8 Design for shear force above plastic hinge at base

The factored shear force envelope for each wall over the height of the structure determined in accordance with Clause 21.5.2.2.1 shall be increased by a constant factor over the height including the base equal to the amount that the factored shear force at the base must be increased in accordance with Clause 21.5.2.2.5.

21.5.2.2.9 Design for shear force below plastic hinge at base

When a shear wall is connected to a stiff supporting structure, such as large foundation walls, by multiple floor diaphragms below the plastic hinge, the factored shear force and corresponding factored bending moment applied to the shear wall below the plastic hinge shall be determined using an analysis that considers the lower-bound or upper-bound value of effective stiffnesses of the members as appropriate to determine a safe estimate of the factored shear force.

Note: *A description of the type of simplified analyses that can be done considering only the structure below the plastic hinge and the appropriate effective stiffnesses, accounting for the level of cracking and quantity of reinforcement, are given in the explanatory notes of the Cement Association of Canada's* Concrete Design Handbook*.*

21.5.3 Minimum wall thickness

21.5.3.1 Plastic hinge region

The wall thickness within a plastic hinge shall be not less than limit A except as permitted by Clause 21.5.3.3, but in no case shall be less than limit B:

	$R_d \le 2.5$	$R_d \ge 3.5$
Limit A	$\ell_u/14$	$\ell_u/10$
Limit B	$\ell_u/20$	$\ell_u/14$

21.5.3.2 Outside plastic hinge region

The wall thickness outside a plastic hinge shall be not less than limit B given in Clauses 21.5.3.1 except as permitted by Clause 21.5.3.3, but in no case shall be less than $\ell_u/20$.

21.5.3.3 Conditions for reduced wall thickness

Limit A in Clause 21.5.3.1 and Limit B in Clause 21.5.3.2 shall not be required to apply to

a) any part of a wall that under factored vertical and lateral loads are not more than halfway from the neutral axis to the compression face of the wall section;
b) 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 flange providing effective lateral support shall be not less than $\ell_u/5$; and
c) 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.

21.5.4 Reinforcement

21.5.4.1 General

All concentrated reinforcement and all distributed reinforcement over the full height of all walls shall meet the following requirements:

a) Reinforcing bars shall not be offset bent.
b) Lap splices shall have a minimum length of $1.5\ell_d$.
c) Reinforcing bars shall be anchored, spliced, or embedded in accordance with the requirements for reinforcement in tension specified in Clause 12 except as modified by Clause 21.5.4.2.

21.5.4.2 Mechanical splices

Where Type 2 mechanical splices are used, not more than alternate bars in each layer of distributed reinforcement and concentrated 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.

21.5.4.3 Maximum percentage

The reinforcement ratio within any region of concentrated reinforcement, including regions containing lap splices, shall be not more than 0.06 in walls designed with $R_d \geq 3.5$.

21.5.5 Distributed reinforcement

21.5.5.1 Minimum amount

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.

21.5.5.2 Maximum spacing

In plastic hinge regions, the spacing of horizontal reinforcement shall not exceed the following values:

$R_d \leq 2.5$	$R_d \geq 3.5$
400 mm	300 mm

Note: *The maximum spacing of vertical and horizontal wall reinforcement is 500 mm according to Clause 14.1.8.4.*

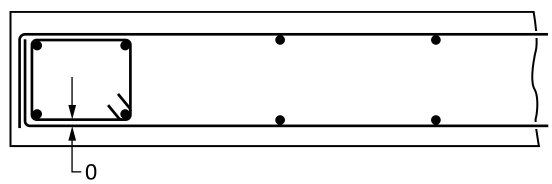
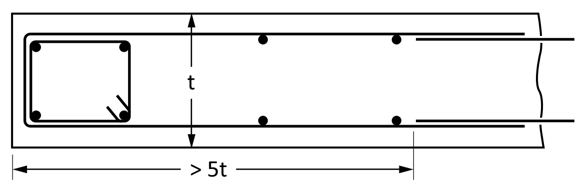
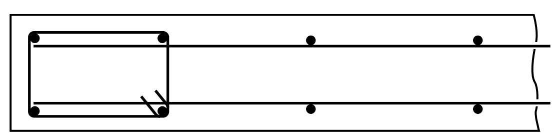
21.5.5.3 Anchorage of horizontal reinforcement

Horizontal reinforcement shall extend to the ends of walls and shall be anchored at each end as follows:

For wall systems designed using:	Permitted Anchorage
R_d = 1.5	Type 1, 2, or 3
R_d = 2.0 or 2.5	Type 2 or 3
R_d = 3.5 or 4.0	Type 3

In plastic hinge regions of walls designed with R_d = 3.5 or 4.0, the horizontal reinforcement required for shear resistance shall be anchored with straight bar embedment, hook, or mechanical anchorage to develop $1.25f_y$ within the region of tied concentrated vertical reinforcement.

Anchorage types shall be as follows:

Type	Brief description	Figure
1	Standard 90° hook around vertical end bar	
2	U-bar around vertical end bars; staggered lap splices minimum five times wall thick from end of wall	
3	Anchored within tied vertical reinforcement	

21.5.5.4 Ties for vertical distributed reinforcement

In plastic hinge regions, if the area of vertical distributed reinforcement is greater than $0.005A_g$ or the maximum bar size is greater than 20M; the vertical distributed reinforcement shall be tied with buckling prevention ties.

Note: *Clause 14.1.8.7 requires that such reinforcement be tied as a compression member in accordance with Clause 7.6.5 outside the plastic hinge region.*

21.5.6 Concentrated vertical reinforcement

21.5.6.1 Minimum amount

Over the full height of all walls, concentrated vertical reinforcement consisting of a minimum of four bars placed in at least two layers shall be provided at the ends of all walls. The minimum area of concentrated reinforcement at each end of the wall shall be as follows:

$R_d \leq 2.5$	$R_d \geq 3.5$
$0.0005\, b_w \ell_w$	$0.001\, b_w \ell_w$

21.5.6.2 Plastic hinge regions

In regions of plastic hinging, the minimum area of concentrated reinforcement at each end of the wall shall be as follows:

$R_d \leq 2.5$	$R_d \geq 3.5$
$0.00075\, b_w \ell_w$	$0.0015\, b_w \ell_w$

21.5.6.3 Flanged walls

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 area of the concentrated reinforcement specified in Clauses 21.5.6.1 and 21.5.6.2, with the remainder placed at the end of the wall web.

21.5.6.4 Ties for concentrated reinforcement

All concentrated reinforcement shall, as a minimum, be tied as a compression member in accordance with Clause 7.6.5 and all ties shall be detailed as hoops or seismic crossties. In regions of plastic hinging, all concentrated reinforcement shall have buckling prevention ties.

21.5.6.5 Limited splicing in ductile walls

When $R_d \geq 3.5$, not more than 50% of the reinforcement at each end of the walls in plastic hinge regions shall be spliced at the same location and a total of at least one-half of the height of each storey shall be completely clear of lap splices in the concentrated reinforcement.

21.5.7 Ductility of walls

21.5.7.1 Requirements above and within plastic hinge region

21.5.7.1.1 Above plastic hinge region

To ensure a wall has adequate ductility to tolerate limited yielding of vertical reinforcement due to higher mode bending moments, at any point over the height of the wall, the distance, *c*, determined in accordance with Clause 21.5.7.4 shall not be greater than

a) $0.5\ell_w$ for walls designed with $R_d \leq 2.5$; or

b) $0.4\ell_w$ for walls designed with $R_d \geq 3.5$.

21.5.7.1.2 Within plastic hinge region

To ensure a wall has adequate ductility within the plastic hinge region, the inelastic rotational capacity of the wall, θ_{ic}, determined in accordance with Clause 21.5.7.3, shall be greater than the inelastic rotational demand, θ_{id}, determined in accordance with Clause 21.5.7.2.

21.5.7.2 Inelastic rotational demand at base

The inelastic rotational demand at the base of a wall, θ_{id}, shall be taken as

$$\theta_{id} = \frac{\left(\Delta_f R_o R_d - \Delta_f \gamma_w\right)}{\left(h_w - \ell_w / 2\right)}$$ **Equation 21.11**

but shall not be taken less than 0.003 for R_d = 2.0 and 0.004 for R_d = 3.5,

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 hinge region) in the direction considered

21.5.7.3 Inelastic rotational capacity

The inelastic rotational capacity of a wall, θ_{ic}, shall be taken as

$$\theta_{ic} = \left(\frac{\varepsilon_{cu}\ell_w}{2c} - 0.002\right)$$ **Equation 21.12**

but shall not be taken greater than 0.025,

where

ℓ_w = the length of the individual wall under consideration

c = the neutral axis distance determined in accordance with Clause 21.5.7.4 and ε_{cu} shall be taken as 0.0035 unless the compression region of the wall contains confinement reinforcement in accordance with Clause 21.5.7.5

Note: *The value of 0.025 is the upper limit on inelastic rotation capacity governed by tension steel strain.*

21.5.7.4 Compression strain depth in wall

The distance from the extreme compression fibre to the neutral axis, c, when concrete reaches the maximum compression strain ε_{cu} at the extreme compression fibre shall be determined by plane sections analysis for the factored axial load acting on the wall and a bending moment causing the maximum compression strain at the extreme compression fibre, or as follows:

$$c = \frac{P_s + P_n + P_{ns} - \alpha_1 \phi_c f'_c A_f}{\alpha_1 \beta_1 \phi_c f'_c b_w}$$ **Equation 21.13**

21.5.7.5 Confinement of concrete

When ε_{cu} in Clause 21.5.7.3 is taken greater than 0.0035, the compression region of the wall shall contain confinement reinforcement. The amount of confinement reinforcement shall be determined 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. ε_{cu} shall not be taken greater than 0.014.

21.5.7.6 Simplified procedure for moderately ductile walls

For shear walls designed with $R_d \leq 2.0$, the requirements of Clause 21.5.7.1.2 shall be considered satisfied if the distance , c, determined in accordance with Clause 21.5.7.4 satisfies either one of the following two conditions:

a) $c \leq 0.33\ell_w$ when deflection of top of wall due to effects of factored loads, Δ_f, does not exceed $h_w / 350$; or

b) $c \leq 0.15\ell_w$ when Δ_f does not meet the condition given in Item a), or Δ_f is not calculated.

21.5.8 Additional requirements for coupled shear walls (R_d = 2.5 or 4.0) and partially coupled shear walls (R_d = 2.0 or 3.5)

Note: *There are four types of coupled or partially coupled wall systems as follows:*

	Moderately Ductile	*Ductile*
Coupled (DOC ≥ 66%)	$R_d = 2.5$	$R_d = 4.0$
Partially Coupled (DOC < 66%)	$R_d = 2.0$	$R_d = 3.5$

Degree of coupling (DOC) is the portion of base overturning moment resistance provided by axial forces in wall piers resulting from shear in coupling beams.

21.5.8.1 Design of coupling beams without diagonal reinforcement

21.5.8.1.1 Dimensional limitations

Coupling beams without diagonal reinforcement shall satisfy the following requirements:

a) The dimensional limitations of Clause 21.3.1.2 shall be satisfied for R_d = 4.0 or 3.5 and the dimensional limitations of Clause 21.4.2.1 shall be satisfied for R_d = 2.5 or 2.0.

b) The clear span, ℓ_u, shall be not less than $2\ell_d$.

c) The maximum shear force shall be limited to $0.1(\ell_u / d)\sqrt{f'_c}\, b_w d$.

21.5.8.1.2 Design as frame members

Coupling beams with longitudinal and transverse reinforcement shall meet the requirements of Clause 21.3.1 for R_d = 4.0 or 3.5 and Clause 21.4.3 for R_d = 2.5 or 2.0.

21.5.8.1.3 Anchorage of longitudinal reinforcement

Anchorage of the longitudinal reinforcement into the wall shall meet the requirements of Clause 21.3.3.1.3. Alternatively, the anchorage of the longitudinal reinforcement shall meet the requirements of Clause 21.5.8.2.5.

21.5.8.1.4 Wide beams

If a coupling beam is wider than the thickness of the wall pier, the following additional requirements shall be met:

a) the front interface between the coupling beam and the wall pier shall be designed for the beam forces within the wall width; and

b) the side interface(s) between the coupling beam and the wall pier shall be designed to transfer all of the forces in the beam overhang(s) to the wall.

21.5.8.1.5 Non-centred beams

If a coupling beam is not centred on the wall pier, the following additional requirements shall be met:

a) the coupling beam and adjoining wall piers shall be designed for the eccentricity; and

b) the coupling beam stiffness shall be reduced to account for the out-of-plane deformations.

21.5.8.2 Design of coupling beams with diagonal reinforcement

21.5.8.2.1 Dimensional limitations

Coupling beams with diagonal reinforcement shall satisfy the following requirements:
a) the beam depth shall be not greater than $2.0\ell_u$;
b) the beam width shall be less than or equal to the wall thickness;
c) the beam shall be centred on the wall pier; and
d) the wall piers at each end of coupling beam shall have sufficient length to contain the diagonal reinforcement embedment specified in Clause 21.5.8.2.5.

21.5.8.2.2 Quantity of diagonal reinforcement

Coupling beams with diagonal reinforcement shall be designed such that the entire factored in-plane shear force and factored bending moment is resisted by diagonal reinforcement in two directions.

Note: *When dimensions of a coupling beam are such that the factored shear force exceeds* $1.0\sqrt{f'_c}bh$*, it can be difficult to construct the beam due to reinforcement congestion, particularly at the intersection with the concentrated wall reinforcement.*

21.5.8.2.3 Concentric reinforcement

The centroid of each group of diagonal reinforcing bars shall be centred in the beam.

21.5.8.2.4 Buckling prevention ties on diagonal reinforcement

The diagonal reinforcement in each direction shall be enclosed by hoops or spirals that extend up to the concentrated reinforcement specified in Clause 21.5.8.3.1. The maximum spacing of the hoops or pitch of the spiral shall not exceed the smallest of
a) six diagonal bar diameters;
b) 24 tie diameters; or
c) 100 mm.

21.5.8.2.5 Anchorage of diagonal reinforcement

The diagonal reinforcing bars shall be anchored into the wall at each end by one of the following:
a) a minimum straight embedment of $1.5\ell_d$, where 1.5 includes the top bar factor; but not the bundled bar factor;
b) a minimum straight embedment of $1.0\ell_d$ plus a standard hook contained within confinement reinforcement; or
c) for headed and mechanically-anchored bars, the minimum straight embedment shall satisfy the minimum tension embedment for seismic applications; but shall not be less than the basic compression development length unless it can be shown that the concrete breakout resistance of the head or mechanical anchorage is able to transfer the compression force into the wall with the contribution of the straight embedment ignored.

21.5.8.3 Design of wall piers

21.5.8.3.1 Concentrated wall reinforcement at coupling beams

Concentrated vertical reinforcement, as specified in Clause 21.5.6.1, shall be provided in the wall piers at both ends of all coupling beams. The concentrated reinforcement shall be tied as specified in

Clause 21.5.6.4, except that for wall systems designed with R_d = 4.0 or 3.5, the concentrated reinforcement shall have buckling prevention ties over the full height.

21.5.8.3.2 Bending resistance of wall piers

Except as permitted by Clause 21.5.8.3.3, the wall pier at each end of a coupling beam shall be designed such that the factored bending moment resistance of the wall pier about its centroid, calculated using axial loads P_s and $\pm P_n$, exceeds the bending moment at its centroid resulting from the nominal resistance of the coupling beams framing into the wall pier and the factored bending moment applied to the wall pier.

21.5.8.3.3 Plastic hinges in wall pier

If the wall pier at one end of a coupling beam has a factored bending resistance that does not meet the condition specified in Clause 21.5.8.3.2, the following additional requirements shall be satisfied:

a) the coupling beam shall meet the requirements of Clause 21.5.8.1;

b) the wall pier shall be designed to the requirements specified in Clauses 21.3.2.6.2, 21.3.2.6.3, 21.3.2.6.6, and 21.3.2.7 for R_d = 4.0 or 3.5,
and shall be designed to the requirements specified in Clauses 21.4.4 and 21.4.5 for R_d = 2.5 or 2.0; and

c) the joint between the wall and coupling beam shall meet the requirements specified in Clause 21.3.3 for R_d = 4.0 or 3.5; and shall meet the requirements specified in Clause 21.4.6 for R_d = 2.5 or 2.0.

21.5.8.3.4 Axial forces in wall piers

All coupled and partially coupled shear walls shall be designed with the portion of factored overturning moment resisted by axial forces in the wall piers increased at each level by the ratio of sum of coupling beam nominal or factored capacity as given in the table below, to the sum of factored forces in coupling beams above the level under consideration. For assemblies of coupled and partially coupled shear walls connected together by coupling beams that function as a closed tube(s) the factored forces in the coupling beams used to calculate the ratio shall be determined without accidental torsion.

	R_d = 4.0 or 3.5	R_d = 2.5 or 2.0
Coupling beam capacity:	nominal	factored

21.5.8.4 Ductility

21.5.8.4.1 General

To ensure ductility of coupled wall systems,

a) the inelastic rotational demand on wall piers determined in accordance with Clause 21.5.8.4.2 shall not be greater than the inelastic rotational capacity of wall piers determined in accordance with Clause 21.5.8.4.3; and

b) the inelastic rotational demand on coupling beams determined in accordance with Clause 21.5.8.4.4 shall not be greater than the inelastic rotational capacity of coupling beams determined in accordance with Clause 21.5.8.4.5.

21.5.8.4.2 Inelastic rotational demand at base of wall piers

The inelastic rotational demand on coupled and partially coupled wall piers shall be taken as

$$\theta_{id} = \frac{\Delta_f R_o R_d}{h_w}$$ **Equation 21.14**

but shall not be taken less than 0.003 for R_d = 2.5 or 2.0 and 0.004 for R_d = 4.0 or 3.5, where
$\Delta_f R_o R_d$ = the design displacement

21.5.8.4.3 Inelastic rotational capacity of wall piers

The inelastic rotational capacity of wall piers shall be calculated using the methods given in Clause 21.5.7.3, except that ℓ_w shall be taken as the length of the longest individual wall pier in the direction considered for partially coupled walls and as the overall length of the interconnected wall piers for coupled walls.

21.5.8.4.4 Inelastic rotational demand on coupling beams

The inelastic rotational demand on coupling beams shall be taken as

$$\theta_{id} = \left(\frac{\Delta_f R_o R_d}{h_w}\right)\frac{\ell_{cg}}{\ell_u}$$ **Equation 21.15**

21.5.8.4.5 Inelastic rotational capacity of coupling beams

The inelastic rotational capacity of coupling beams, θ_{ic}, shall be taken as
a) 0.02 for coupling beams without diagonal reinforcement; and
b) 0.04 for coupling beams with diagonal reinforcement designed in accordance with Clause 21.5.8.2.

21.5.9 Shear resistance of flexural shear walls

21.5.9.1 General

The shear design of flexural shear walls shall meet the requirements specified in Clauses 11, 14, and 21.5.9.2 to 21.5.9.5.

21.5.9.2 Shear depth

The effective shear depth, d_v, of a wall need not be taken as less than $0.8\ell_w$.

21.5.9.3 Opening in walls

The effect of openings in walls shall be accounted for.

Note: *Strut-and-tie models in accordance with Clause 11.4 can be used to confirm that the diagonal compression due to shear force in a wall can be transmitted around an opening and can be used to determine the additional reinforcement required around an opening.*

21.5.9.4 Outside plastic hinge regions

Outside regions of plastic hinging, the shear resistance of a flexural wall can be determined using the procedures of Clause 11.3 except that the maximum possible shear resistance $V_{r,max} = 0.2\phi_c f'_c b_w d_v$.

The following simplified procedure may be used:
a) the value of β in Clause 11.3.4 shall be taken as 0.18; and
b) the value of θ in Clause 11.3.5 shall be taken as 35°.

21.5.9.5 Plastic hinge regions

21.5.9.5.1 General

In regions of plastic hinging, the shear resistance of a flexural wall shall be determined using the procedures of Clause 11.3 except that the shear resistance shall be reduced to account for the inelastic rotational demand in accordance with Clauses 21.5.9.5.2 to 21.5.9.5.3.

21.5.9.5.2 General method for ductile and moderately ductile walls

The shear resistance shall be calculated as follows:

a) The maximum possible shear resistance $V_{r,max} = 0.1\phi_c f'_c b_w d_v$ unless it is shown that the inelastic rotational demand on the wall, θ_{id} given by Equation 21.11 or 21.14 is less than 0.015. When $\theta_{id} \leq 0.005$, the maximum possible shear resistance $V_{r,max} = 0.15\phi_c f'_c b_w d_v$. 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 21.11 or 21.14 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.

21.5.9.5.3 Simplified method for moderately ductile walls

In lieu of the requirements of Clause 21.5.9.5.2, the following simplified procedure may be used for shear walls designed with $R_d \leq 2.5$:

a) the maximum possible shear resistance $V_{r,max} = 0.125\phi_c f'_c b_w d_v$;

b) the value of β in Clause 11.3.4 shall be taken as 0.1, and;

c) the value of θ in Clause 11.3.5 shall be taken as 45°.

21.5.9.6 Strut-and-tie models

When strut-and-tie models in accordance with Clause 11.4 are 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 compression member in accordance with Clause 7.6.5, 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.

21.5.10 Moderately ductile squat shear walls (R_d = 2.0)

21.5.10.1 General

The requirements specified in Clauses 21.5.10.2 to 21.5.10.8.7 shall apply to walls with h_w / ℓ_w of 2.0 or less designed using an R_d = 2.0.

21.5.10.2 Capacity design

The foundation and diaphragm components of the SFRS shall have factored resistances that are greater than the nominal wall capacity; but need not be taken larger than the forces calculated with design load combinations that include earthquake effects calculated using R_d R_o equal to 1.3. The nominal wall capacity shall be taken as the smaller of

a) the shear corresponding to the development of the nominal moment capacity of the wall; or
b) the nominal shear resistance of the wall, which shall be taken as not less than $0.25\sqrt{f'_c}b_w l_w$

Note: *Squat walls can develop either a flexural or a shear mechanism that will limit the seismic forces resisted by the wall. The procedures used to calculate nominal shear resistance assumes that the concrete is fully cracked; however, the seismic forces may be as large as the diagonal cracking shear.*

21.5.10.3 Wall thickness

The wall thickness shall be not less than ℓ_u / 20.

21.5.10.4 Reinforcement

The requirements specified in Clause 21.5.4 shall apply.

21.5.10.5 Distributed reinforcement

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 0.002 in each direction.

When the shear force assigned to the wall exceeds $0.18\phi_c\sqrt{f'_c}b_w\ell_w$,

a) the reinforcement ratio for both the vertical and horizontal distributed reinforcement shall be not less than 0.003 in each direction;
b) at least two curtains of reinforcement shall be provided;
c) the reinforcement spacing in each direction shall not exceed 400 mm; and
d) the horizontal reinforcement shall be anchored in accordance with Clause 21.5.5.3 for R_d = 2.0.

21.5.10.6 Concentrated reinforcement

When the shear force assigned to the wall exceeds $0.18\phi_c\sqrt{f'_c}b_w\ell_w$, concentrated vertical reinforcement consisting of a minimum of four 15M reinforcing bars placed in at least two layers shall be provided at the end of the wall and at junctions of intersecting walls. The concentrated vertical reinforcement shall be at least tied as a compression member in accordance with Clause 7.6.5 and the ties shall be detailed as hoops.

21.5.10.7 Overturning resistance

The vertical tension force required to resist overturning may be provided by a combination of concentrated reinforcement and distributed vertical reinforcement. Plane-sections analysis may be used for these calculations. When the height-to-length ratio of the wall is less than 1.0, the distributed vertical reinforcement required for shear in accordance with Clause 21.5.10.8.7 shall not be considered to contribute to the overturning moment resistance of the wall.

Note: *While plane sections will often not remain plane in squat walls, the method correctly accounts for compatibility of concrete and reinforcement strains at any point along the wall and satisfies equilibrium.*

21.5.10.8 Shear design of squat walls

21.5.10.8.1 General

The shear design of squat walls can be done in accordance with the strut-and-tie method in Clause 11.4 and with Clauses 21.5.10.8.2 or may be done in accordance with Clauses 21.5.10.8.3 to 21.5.10.8.7.

21.5.10.8.2 Strut-and-tie models

When the strut-and-tie models in accordance with Clause 11.4 are used for squat walls, the additional requirements of Clause 21.5.9.6 shall apply.

21.5.10.8.3 Opening in walls

The effect of openings in walls shall be accounted for.

Note: *Strut-and-tie models in accordance with Clause 11.4 may be used to confirm that the diagonal compression due to shear force in a wall can be transmitted around an opening and may be used to determine the additional reinforcement required around an opening.*

21.5.10.8.4 Shear stress

The factored shear stress, v_f, shall be computed from the following equation, from the factored shear force and dimensions of the wall:

$$v_f = V_f/(b_w \cdot 0.8\,\ell_w)$$ **Equation 21.16**

21.5.10.8.5 Maximum shear stress

The factored shear stress applied to a wall shall not exceed $0.15\phi_c f_c'$.

21.5.10.8.6 Distributed horizontal reinforcement

The required amount of distributed horizontal reinforcement for shear, ρ_h, shall be determined from Equation 21.17 using a value of θ chosen between a maximum value of 45° and a minimum value of 30°; however the same value of θ shall be used to determine the required amount of distributed vertical reinforcement for shear in accordance with Clause 21.5.10.8.7.

$$\rho_h = v \cdot \tan\theta/\phi_s f_y$$ **Equation 21.17**

21.5.10.8.7 Distributed vertical reinforcement

The required amount of distributed vertical reinforcement required to resist shear shall be determined from Equation 21.18 as a function of the required amount of distributed horizontal reinforcement, ρ_h, determined in accordance with Clause 21.5.10.8.6, and the same value of θ used to determine the required amount of distributed horizontal reinforcement.

$$\rho_v = \rho_h \cot^2\theta - \frac{P_s}{\phi_s f_y A_g}$$ **Equation 21.18**

When the height-to-length ratio of the wall is less than 1.0, the distributed vertical reinforcement required for shear given by Equation 21.18 shall be in addition to the distributed vertical reinforcement that contributes to the overturning moment resistance of the wall determined in accordance with Clause 21.5.10.7.

Note: *When h_w/ℓ_w is less than 1.0, the shear force is resisted by diagonal compression stresses that are relatively uniform across the base of the wall. The required vertical reinforcement given by Equation 21.18 is needed to balance the vertical component of this compression.*

21.6 Conventional construction (R_d = 1.3 or 1.5)

21.6.1 General

21.6.1.1

All members of the structure assumed not to be part of the SFRS might be required to satisfy Clause 21.11 depending on the value of $I_E F_a S_a(0.2)$ and the calculated maximum interstorey drift as given in Clause 21.11.1.1.

21.6.1.2

The foundation supporting the SFRS shall satisfy the requirements of Clause 21.10.

21.6.1.3

The elements of the SFRS shall satisfy the requirements of Clauses 4 to 18 and 23, and the additional requirements of Clauses 21.6.2 to 21.6.4.

21.6.2 Moment-resisting frames (R_d = 1.5)

21.6.2.1 Required resistance

All members of a moment-resisting frame, including beams, columns and beam-column joints, shall have factored shear and bending moment resistances greater than the factored forces due to lateral loads calculated in accordance with *NBCC*.

21.6.2.2 Column ties

Except when $I_E F_a S_a(0.2)$ is less than 0.2 or the factored resistances of the columns are greater than the effects of factored loads calculated using $R_d R_o$ equal to 1.3, columns shall contain buckling prevention ties satisfying Clause 21.2.8.1 over the storey height when

a) the sum of the factored resistances of columns framing into a joint is not greater than the factored resistance of the beams framing into the joint; or
b) columns can develop plastic hinges because of their connection to rigid members such as transfer girders, foundations or discontinued walls, or because of their position at the base of the structure.

21.6.2.3 Shear resistance of frame members

Except when $I_E F_a S_a(0.2)$ is less than 0.2 or the factored resistance of the frame is greater than the effects of factored loads calculated using $R_d R_o$ equal to 1.3, frame members shall have a factored shear resistance not less than the lesser of

a) the sum of the maximum shear associated with development of factored 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 shear force resulting from design load combinations that include earthquake calculated using $R_d R_o$ equal to 1.3.

21.6.2.4 Shear resistance of joints in frames

21.6.2.4.1 Design shear forces

The design shear forces acting in a beam column joint shall be those induced by the factored moments acting on the beam or beams framing into the joint. Where beams frame into the joint from two directions, each direction may be considered independently.

21.6.2.4.2 Factored shear resistance of joints

The factored shear resistance of the joint shall satisfy the requirements of Clause 21.4.6.2.

Note: *Additional requirements for beam-column joints are given in Clauses 7.7, 11.7, and 12.11.2.*

21.6.3 Shear walls (R_d = 1.5)

21.6.3.1 Minimum requirements

All shear walls shall meet the requirements of Clause 14.

21.6.3.2 Anchorage of horizontal reinforcement

Anchorage of horizontal reinforcement shall be in accordance with requirements of Clause 21.5.5.3.

21.6.3.3 Design of squat shear walls

The design of squat walls with h_w / ℓ_w of 2.0 or less shall be done in accordance with the strut-and-tie method in Clause 11.4 or shall be done in accordance with the appropriate parts of Clause 21.5.10 as follows:

a) The shear design shall be in accordance with Clauses 21.5.10.8.3 to 21.5.10.8.7, except that the maximum factored shear stress given in Clause 21.5.10.8.5 shall be increased to $0.20\phi_c f'_c$.

b) The design for overturning resistance of squat walls shall be in accordance with Clause 21.5.10.7.

The additional requirements in Clauses 21.6.3.4 to 21.6.3.7 shall not apply.

21.6.3.4 Design shear force

The factored shear force envelope determined in accordance with *NBCC* shall be increased by the ratio of the factored bending resistance to factored bending moment both calculated at the base of the wall; however, the factored shear force need not be taken larger than that resulting from design load combinations including earthquake, with load effects calculated using $R_d R_o$ equal to 1.3.

21.6.3.5 Shear resistance

The shear resistance of a flexural wall with h_w / ℓ_w greater than 2.0 shall be determined using the procedures of Clause 11.3 except that the maximum shear resistance of the wall shall not exceed $V_{r,max} = 0.2\phi_c f'_c b_w d_v$, where the effective shear depth, d_v, of a wall need not be taken less than $0.8\ell_w$.

21.6.3.6 Ductility requirements above potential hinge region

At every section above the potential plastic hinge region, the distance from the extreme compression fibre to the neutral axis, c, determined by plane sections analysis for the factored axial load acting on the wall and a bending moment causing the maximum compression strain ε_{cu} at the extreme compression fibre, shall not exceed $0.6\ell_w$. If the distance from the extreme compression fibre to the neutral axis, c, exceeds $0.3\ell_w$, concentrated vertical reinforcement consisting of a minimum of four bars placed in at least two layers shall be provided at the ends of all walls. This reinforcement shall be at least tied as a compression member in accordance with Clause 7.6.5 and the ties shall be detailed as hoops.

Note: *The intent is to ensure that if the shear wall was subjected to a sufficiently large bending moment, vertical reinforcement at the tension end of the wall will yield before concrete will crush at the compression end of the wall. The compression strain depth c depends primarily on the geometry of the cross section, the concrete strength and the axial compression force applied to the wall.*

21.6.3.7 Additional requirements at base of wall

21.6.3.7.1 Height of potential plastic hinge region

Shear walls designed with R_d = 1.5 are expected to experience yielding of vertical reinforcement when subjected to the design earthquake forces. The region of possible yielding shall be assumed to extend a minimum distance of $1.0\ell_w$ above the critical section where the vertical reinforcement will first yield.

21.6.3.7.2 Splices

Over the height defined in Clause 21.6.3.7.1, splices of the vertical reinforcement shall satisfy Clause 21.2.7.2.1.

21.6.3.7.3 Wall thickness

Over the height defined in Clause 21.6.3.7.1, the wall thickness shall be not less than $\ell_u/20$ except as permitted by Clauses 21.5.3.3, but no case shall be less than $\ell_u/25$.

Note: *The minimum wall thickness according to Clause 14.1.7 is* $\ell_u/25$.

21.6.3.7.4 Concentrated reinforcement

Over the height defined in Clause 21.6.3.7.1, concentrated vertical reinforcement consisting of a minimum of four bars placed in at least two layers shall be provided at the ends of all walls. This reinforcement shall be at least tied as a compression member in accordance with Clause 7.6.5 and the ties shall be detailed as hoops.

21.6.3.7.5 Ductility

Over the height defined in Clause 21.6.3.7.1, the distance from the extreme compression fibre to the neutral axis, c, determined by plane sections analysis for the factored axial load acting on the wall and a bending moment causing the maximum compression strain ε_{cu} at the extreme compression fibre, shall not exceed $0.5\ell_w$.

Note: *The intent is to ensure that the vertical reinforcement at the tension end of the wall will yield before concrete will crush at the compression end of the wall. The compression strain depth c depends primarily on the geometry of the cross section, the concrete strength, and the axial compression force applied to the wall.*

21.6.4 Two-way slabs without beams (R_d = 1.3)

21.6.4.1

The factored slab moment at support including 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 4.1).

21.6.4.2

The fraction of the moment, M_s, determined using Equation 13.5 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.

21.6.4.3

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.

21.6.4.4
Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.

21.6.4.5
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.

21.6.4.6
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.

21.6.4.7
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.

21.6.4.8
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 slab design satisfies the requirements of Clause 21.11.4.2.

21.7 Tilt-up construction (R_d = 1.5 or 2.0)

21.7.1 General

21.7.1.1 Application
Clause 21.7 provides seismic design requirements for reinforced concrete wall panels ranging from solid wall panels to wall panels with large openings (frames). All tilt-up wall panels shall meet the requirements of Clause 23.

21.7.1.2 Types of seismic-force-resisting systems
The *NBCC* specifies three types of seismic-force-resisting systems for concrete tilt-up construction. Clauses 21.7.1 to 21.7.4 provide design requirements for concrete tilt-up walls and frames designed with $R_d \geq 1.5$. Clause 21.7.5 provides additional requirements for concrete tilt-up walls and frames designed with $R_d = 2.0$.

Note: *Limited Ductility (R_d = 1.5) walls and frames may be designed using a force-based approach, while moderately ductile (R_d = 2.0) walls and frames are designed using a displacement-based approach that explicitly accounts for the inelastic displacement demands on wall panels.*

21.7.1.3 Governing ultimate limit state

21.7.1.3.1 Ductile limit states
All tilt-up construction with $R_d \geq 1.5$ shall be designed such that the governing ultimate limit state of the tilt-up walls is ductile. Acceptable limit states include
a) rocking of individual wall panels or wall panel groups;
b) sliding along the base of the tilt-up wall; or

c) for panels with large openings (frames), yielding of the beams and columns.

Note: *Rocking of individual wall panels or wall panel groups might require yielding of panel-to-panel shear connectors and/or tie-down connectors. Sliding along the base of walls can be difficult to achieve unless walls have a simple configuration, and post-earthquake damage due to sliding can be difficult to repair.*

21.7.1.3.2 Non-ductile limit states

The following non-ductile ultimate limit states shall be prevented:

a) failure of any connection between the wall panels and roof diaphragm; and
b) shear failure of a wall panel or portion of a wall panel.

21.7.1.4 Building lateral period

When established methods of mechanics considering roof diaphragm flexibility are used to determine the fundamental lateral period of a tilt-up building in accordance with the *NBCC*, the wall panels in the building shall meet the requirements for moderately ductile (R_d = 2.0) walls and frames.

Note: *When the influence of the flexible diaphragm is accounted for in the calculation of the building period, a displacement-based approach is used to account for the increased inelastic displacement demands on wall panels due to the flexible roof diaphragm.*

21.7.2 Seismic force demands

21.7.2.1 In-plane shear

21.7.2.1.1 Factored in-plane shear force

Wall panels shall be designed for the factored in-plane shear forces transferred to the panel by the diaphragm and the additional in-plane shear force due to the panel self-weight. A rational analysis shall be used to distribute the in-plane shear force to individual panels in one wall.

Note: *A rational analysis needs to account for the deformation compatibility of the panels and structural members used to transfer the in-plane shear such as drag struts and collector elements, as well as the deformation capacity of all connectors.*

21.7.2.1.2 Minimum shear resistance

Wall panels shall have a factored shear resistance greater than the shear force due to the effects of factored loads determined in accordance with Clause 21.7.2.1.1, but not less than the smaller of

a) the shear force corresponding to the development of the factored overturning moment capacity for R_d = 1.5, or nominal overturning moment capacity for R_d = 2.0, of the individual wall panel or wall panel group; or
b) the shear force resulting from design load combinations that include earthquake effect, with loads calculated using $R_d R_o$ equal to 1.3.

21.7.2.2 Out-of-plane shear and bending moment

Wall panels shall be designed for the out-of-plane bending moments and shear forces resulting from the wall panel acting as a flexural member spanning between lateral supports. Out-of-plane forces shall be calculated in accordance with the *NBCC.*

21.7.2.3 Transfer of forces to foundation

A reliable load path shall be provided to transfer all in-plane and out-of-plane forces to the supporting soil.

Note: *A direct or indirect connection to the wall footing may be used to provide a reliable load path. Slab-sliding resistance is difficult to predict and unreinforced or jointed floor slabs might not be reliable.*

21.7.2.4 Connection forces

21.7.2.4.1 Out-of-plane forces

The connections to wall panels shall be designed for the factored out-of-plane forces calculated in accordance with the requirements for the connection of elements to the structure in the *NBCC* assuming a flexible element (wall panel) and non-ductile connections; but not less than the forces specified in Clause 23.2.9.2.

Note: *The Cement Association of Canada's* Concrete Design Handbook *summarizes the appropriate values of C_p, A_r, and R_p to be used.*

21.7.2.4.2 In-plane forces

Wall panel connections that transfer in-plane shear forces shall be designed for the factored forces resulting from design load combinations that include earthquake effect, with loads calculated using $R_d R_o$ that depends on the type of connector as follows:

a) Ductile wall connections shall be designed using the same $R_d R_o$ as the seismic-force-resisting-system.
b) Non-ductile wall connections shall be designed using $R_d R_o$ equal to 1.3.
c) When a ductile structural member is used to transmit the in-plane shear force to a non-ductile connection, the non-ductile connection need not be designed for a force greater than the probable capacity of the ductile member.

21.7.3 Design requirements

21.7.3.1 Strength and ductility of connectors

All wall panel connectors shall be designed based on experimentally-established strengths and ductility or shall be designed in accordance with established design procedures such as given in Annex D.

Note: *The Cement Association of Canada's* Concrete Design Handbook *has information about standard tilt-up connectors that have experimentally established strengths and ductility.*

21.7.3.2 Overturning capacity

The required factored overturning moment capacity of wall panels shall be provided using any combination of the panel self weight, vertical load acting directly on the panel, panel-to-panel shear connectors and tie-down anchors. Panel-to-panel shear connectors shall be ductile. For other connections, if non-ductile connectors or anchors are used, the displacement compatibility of the connectors shall be accounted for. The effective length of the wall panel shall account for concrete spalling at the edge of the panel.

Note: *When a panel rocks up on a corner, the concrete outside of tied vertical reinforcement will likely spall off thereby reducing the effective length of the panel.*

21.7.3.3 Sliding shear resistance at base of wall

The required factored sliding shear resistance shall be provided by a combination of shear friction and shear connectors. All wall panels shall have shear connectors at the base and cannot rely only on shear friction. The sliding shear friction resistance between the base of wall panels and foundation shall be determined in accordance with Clause 11.5.1 with $c = 0$, $\mu = 0.75$, and $\phi_c = 0.65$

21.7.3.4 Shear design of solid panels

If the applied in-plane shear stress exceeds $0.1\phi_c\sqrt{f'_c}$, 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 0.002 in each direction, and the shear design of the wall panel shall be in accordance with Clause 21.6.3.3.

21.7.3.5 Design of compression members

Portions of a wall panel, such as a column or edge of a solid panel, subjected to axial compression stresses greater than $0.09\phi_c f'_c$ due to load combinations including earthquake effects shall be designed for slenderness effects in accordance with Clause 10.15 and shall contain tied vertical reinforcement in accordance with Clause 21.7.3.6.

Note: *The compression edge of unconnected panels are particularly prone to slenderness effects.*

21.7.3.6 Ties for tilt-up compression members

Where required according to Clause 21.7.3.5, tilt-up compression members shall contain vertical reinforcement with ties that satisfy the following:

a) The ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support.
b) Crossties or legs of overlapping hoops shall have centre-to-centre spacings not exceeding 350 mm.
c) The tie shall consist of a 10M bar or larger.
d) The maximum spacing of the ties shall not exceed the smallest of 16 times the diameter of the smallest enclosed longitudinal bar, 300 mm or the thickness of the panel.
e) In panels designed using R_d = 2.0, all ties shall be detailed as hoops or seismic crossties and the spacing of the ties shall not exceed 12 times the diameter of the smallest enclosed longitudinal bar.

21.7.4 Design of tilt-up frames

21.7.4.1 General

Wall panels with openings shall either be designed as solid panels with the forces transmitted around the openings or shall be designed as a frame in accordance with Clauses 21.7.4.2 to 21.7.4.5.

Note: *Strut-and-tie models in accordance with Clause 11.4 may be used to confirm that the forces applied to a solid wall panel can be transmitted around an opening by concrete compression stresses and additional reinforcement provided around the opening.*

21.7.4.2 Influence of panel connectors on ductility

When determining that the governing ultimate limit state of a tilt-up frame is ductile as required by Clause 21.7.1.3, the influence of forces due to panel connections shall be accounted for.

Note: *Panel-to-panel shear connections between panel legs (lower columns) can influence the load distribution or failure mode of the panel legs.*

21.7.4.3 Design for plastic hinging

21.7.4.3.1 Columns

When a plastic hinge is expected to form in a column, the following requirements shall be satisfied:

a) The column dimension perpendicular to the axis of bending shall not be less than 600 mm.
b) The plastic hinge region shall not be taken less than 1.5 times the largest column dimension from the face of the joint.

c) Over the plastic hinge region defined in Item b) and over the beam-column joint for a height not less than the largest column dimension, the column shall contain ties in accordance with Clause 21.7.3.6, except that
   i) all ties shall be detailed as hoops or seismic crossties for panels designed using R_d = 2.0; and
   ii) the spacing of the ties shall not exceed 12 times the diameter of the smallest enclosed longitudinal bar for panels designed using R_d = 1.5 and 8 times the diameter of the smallest enclosed longitudinal for panels designed using R_d = 2.0.

21.7.4.3.2 Beams

When a plastic hinge is expected to form in a beam, the plastic hinge region shall not be taken less than twice the beam depth from the face of the joint. Over the plastic hinge region, the beam shall contain 10M or larger stirrups spaced at the smallest of

a) $d/4$;
b) 12 times the diameter of the smallest longitudinal bar in the top or bottom layer of reinforcement; and
c) 300 mm.

In panels designed using R_d = 2.0, the stirrups shall be detailed as hoops.

21.7.4.3.3 Cover spalling

When calculating the factored moment resistance of a column or beam, the influence of cover spalling on the compression face of the member shall be accounted for.

21.7.4.4 Minimum shear resistance of frame members

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 force associated with development of the factored moment resistances for R_d = 1.5, or nominal moment resistances for R_d = 2.0, 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 shear force resulting from design load combinations that include earthquake effect, with loads calculated using $R_d R_o$ equal to 1.3.

The moment resistances in Item a) shall not be reduced due to cover spalling.

21.7.4.5 Design of joints

The joints of tilt-up frames shall satisfy the requirements of Clauses 21.4.6.1 to 21.4.6.3. The diameter of straight beam and column reinforcing bars passing through the joint shall not be larger than one-eighth of the panel thickness for R_d = 1.5, or one-tenth of the panel thickness for R_d = 2.0.

Note: *The maximum bar diameter does not apply to a bar that is anchored beyond the joint by a standard hook.*

21.7.5 Additional requirements for Moderately Ductile wall panels (R_d = 2.0)

21.7.5.1 Ductile roof diaphragm

The flexible roof diaphragm of low-rise tilt-up buildings designed for earthquake loads calculated using R_d = 2.0 shall be designed to exhibit ductile behaviour in accordance with the applicable CSA standard.

21.7.5.2 Inelastic displacement demand

The inelastic displacement demand at the top of the wall shall be calculated as ($\Delta_f R_d R_o - \Delta_f \gamma_w$).

Note: *The inelastic displacement demand is assumed to equal the design displacement at the top of the wall panel minus an estimated elastic portion of the displacement.*

21.7.5.3 Displacement design of solid wall panels

The inelastic displacement capacity of the wall and connections shall be greater than the inelastic displacement demand.

Note: *The inelastic displacement demands in solid wall panels is usually concentrated in the panel-to-panel shear connections and/or the panel-to-base connections. The Cement Association of Canada's* Concrete Design Handbook *contains information about the displacement capacity of standard tilt-up connections.*

21.7.5.4 Displacement design of tilt-up frames

Panels with openings designed as frames shall not have rotational demands greater than 0.04 on any of the members. When the rotational demand on any member exceeds 0.02 radians, the entire frame shall satisfy all the requirements of Clause 21.4.

21.8 Precast concrete

21.8.1 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 21.8. Solid precast wall panels may be designed in accordance with Clause 21.7 if the wall panels also meet the requirements of Clause 23.

21.8.2 Ductile moment-resisting frames constructed using precast concrete (R_d = 4.0)

21.8.2.1

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 21.3.1.5.1 or 21.3.2.7.1; 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 21.2.7.3.

21.8.2.2

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 bending resistance 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 21.3.2.7.1.

21.8.2.3

Ductile moment-resisting frames constructed using precast concrete and not meeting the requirements of Clause 21.8.2.1 or 21.8.2.2 shall comply with the acceptance criteria for moment frames based on structural testing (ACI 374.1) 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.

21.8.3 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 21.5 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.

21.8.4 Moderately ductile shear walls constructed using precast concrete (R_d = 2.0)

21.8.4.1

Moderately ductile shear walls constructed using precast concrete shall meet all of the requirements of Clause 21.5 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 shall apply.

21.8.4.2

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.

21.8.4.3

Elements of the connection that are not designed to yield shall develop at least 150% of the specified yield strength of the yielding element.

21.9 Structural diaphragms (R_d = 2.0, 2.5, 3.5, or 4.0)

21.9.1 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 21.9.2 to 21.9.8.

21.9.2 Design forces

Design forces for diaphragms and their connections shall comply with the requirements of the *NBCC.*

21.9.3 Diaphragm systems

21.9.3.1
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.

21.9.3.2
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.

21.9.4 Reinforcement

21.9.4.1
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.

21.9.4.2
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.

21.9.4.3
All continuous reinforcement in struts, ties, chords, and collector elements shall be anchored or spliced as specified in Clauses 12 and 21.2.7.3. Anchorage and splice lengths for reinforcement not contained within confinement reinforcement 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.

21.9.4.4
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 21.2.7.3 and 21.2.7.4, respectively. Type 2 splices shall be required where mechanical splices are used to transfer forces between collectors and the vertical components of the SFRS.

21.9.4.5
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 may be used to resist diaphragm design forces if a complete load path is provided.

21.9.5 Monolithic concrete systems

21.9.5.1

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.

21.9.5.2

The factored shear resistance of a shear panel shall be taken as

$$V_r = A_{cv}(0.2\phi_c\sqrt{f'_c} + \phi_s\rho_n f_y) \le 0.8\phi_c A_{cv}\sqrt{f'_c}$$ **Equation 21.19**

21.9.5.3

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.

21.9.6 Precast systems

21.9.6.1

Cast-in-place composite and non-composite toppings may be used 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.

21.9.6.2

The factored shear resistance of a shear panel shall be taken as

$$V_r = \phi_s A_{cv}\rho_n f_y \le 0.6\phi_c A_{cv}\sqrt{f'_c}$$ **Equation 21.20**

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.

21.9.6.3

Chords, collectors, struts, and ties shall comply with Clause 21.9.5.3.

21.9.7 Composite systems

21.9.7.1

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.

21.9.7.2

The factored shear resistance 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.

21.9.7.3
For decks bounded by steel beams and girders designed as full composite members with headed stud shear connectors, the shear resistance of a reinforced topping slab shall be taken as

$$V_r = \phi_s A_{cv} \rho_n f_y \leq 0.6 \phi_c A_{cv} \sqrt{f'_c}$$ **Equation 21.21**

where A_{cv} is calculated based on the topping thickness above the flutes.

21.9.7.4
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 21.9.5.3.

21.9.8 Construction joints
All construction joints in diaphragms shall comply with Clause 6.3. Contact surfaces shall be treated as specified in Clause 11.5.

21.10 Foundations (R_d = 1.3, 1.5, 2.0, 2.5, 3.5, or 4.0)

21.10.1 General

21.10.1.1
Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and the ground may be designed in accordance with Clause 21.10.2 if the foundation meets the limitations given in Clause 21.10.2.1; otherwise, they shall be designed using the general method specified in Clause 21.10.3.

21.10.1.2
The requirements of Clause 21.10 for piles, drilled piers, caissons, and slabs-on-grade shall be in addition to the requirements specified in Clause 15.

21.10.2 Design of foundations restrained against rotation

21.10.2.1 General
Foundations, including foundation walls and footings, that are restrained from rotating by structural elements may be designed in accordance with Clauses 21.10.2.2 to 21.10.2.3 if the structure restraining foundation movement is shown to have sufficient stiffness and strength to prevent significant increase in displacement of the SFRS; otherwise, the foundation shall be designed in accordance with Clause 21.10.3.

Note: *Examples of foundations that are restrained against rotation include: foundations that are tied down by soil anchors, foundations supported on piles and foundations supporting shear walls that are connected by multiple diaphragms to foundation walls supported on separate foundations. Foundation movements, which may be determined using the procedures in Clause 21.10.3.3.1, can influence the distribution of forces between the foundation and the restraining structure.*

21.10.2.2 Factored resistance

Foundations shall have a factored overturning resistance and a factored shear resistance, including factored shear resistance of walls and factored sliding shear resistance of footings, not less than required to resist the smaller of

a) the factored gravity loads and the nominal overturning capacity of the SFRS and the corresponding shear force; or
b) the forces from design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 1.3.

21.10.2.3 Design of foundation walls restrained by diaphragms

When an SFRS is designed using $R_d \geq 2.0$, all foundation walls that provide the restraint to foundation movement required by Clause 21.10.2.1 due to being interconnected by multiple diaphragms shall be designed for the forces determined in accordance with Clause 21.5.2.2.9. In addition, all foundation walls that are part of the SFRS shall be designed in accordance with Clause 21.5.2.2.4.

Note: *When an SFRS is designed using $R_d < 2.0$, consideration should be given to the large shear forces that can develop due to bending moment reversals in the wall from diaphragm forces.*

21.10.3 Design of foundations — General method

21.10.3.1 General

Foundations, including foundation walls, shall be designed to meet the requirements of Clauses 21.10.3.2 to 21.10.3.4.

21.10.3.2 Factored resistance

21.10.3.2.1 General

All foundations shall satisfy both of the following requirements:

a) The factored overturning resistance of foundations, including foundation walls and footings, shall satisfy one of Clauses 21.10.3.2.2 to 21.10.3.2.4.
b) The factored shear resistance of foundations, including factored shear resistance of walls and factored sliding shear resistance of footings, shall not be less than that required to resist the factored gravity loads and the shear force corresponding to the required factored overturning resistance in Item a); but need not exceed the forces from design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 1.3.

21.10.3.2.2 Maximum required overturning resistance

The overturning capacity of a foundation calculated using a bearing stress in the soil or rock equal to 1.5 times the factored bearing resistance and all other resistances equal to 1.3 times the factored resistance need not exceed the overturning moment resulting from design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 1.0. The factor of 1.3 shall not apply to the portion of resistance to uplift or overturning resulting from gravity loads.

Note: *Overstrength in the bearing resistance of soil or rock can result in a very small increase in overturning capacity of the foundation. Thus, the minimum overstrength is applied to the resistance and the forces are calculated using $R_d R_o$ equal to 1.0 rather than 1.3.*

21.10.3.2.3 Capacity-protected foundations

Except as given in Clauses 21.10.3.2.2 and 21.10.3.2.4, foundations shall have a factored overturning resistance greater than or equal to what is required to resist the factored gravity loads and the overturning capacity of the SFRS given below:

	$R_d R_o/\gamma_w \leq 2.5$	$R_d R_o/\gamma_w > 2.5$
Overturning capacity of SFRS:	Nominal	Probable

When the SFRS is not a concrete wall, the equivalent overstrength factor for the SFRS shall be substituted for γ_w. In lieu of a more detailed analysis, γ_w may be taken equal to R_o.

21.10.3.2.4 Not capacity-protected (NCP) foundations

Foundations that are not restrained against rotation may be designed as NCP foundations if it can be shown that the SFRS and the members not considered part of the SFRS can tolerate the increased displacements. Foundations that are restrained by flexible structures may also be designed as NCP foundations if it can be shown that the restraining structure can also tolerate the increased displacements.

Note: *Examples of foundations that cannot be designed as NCP foundations include foundations tied down by soil anchors, foundations supported on piles, raft foundations, and foundations supporting walls that are restrained by a stiff structure consisting of multiple diaphragms connected to large foundation walls. The procedures in Clause 21.11 can be used to demonstrate that the members not considered part of the SFRS and a flexible restraining structure have adequate displacement capacity.*

NCP foundations shall satisfy the requirements of Clauses 21.10.3.3.3 and 21.10.3.4, and shall have a factored overturning resistance not less than what is required to resist the factored gravity loads and the larger of

a) 75% of the nominal overturning capacity of the SFRS; or
b) the overturning moment resulting from design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 2.0.

21.10.3.3 Foundation movements

21.10.3.3.1 General

The increased displacements due to movements of foundations shall be accounted for in the design of the SFRS and in the design of the members not considered part of the SFRS.

Note: *A description of how to estimate foundation movements are given in the explanatory notes of the Cement Association of Canada's* Concrete Design Handbook*.*

21.10.3.3.2 Movements of capacity-protected foundations

When the factored overturning resistance of a foundation satisfies Clause 21.10.3.2.1 or 21.10.3.2.2, the foundation movements may be calculated using a static analysis. The footing rotation is calculated as the difference in vertical displacements at the "toe" and "heel" of the footing, divided by the length of footing, ℓ_f. The analysis shall account for the assumed bearing stress distribution in the soil or rock to resist the applied loads and the corresponding stiffness of the soil or rock.

In lieu of a more detailed analysis, the interstorey drift ratio of the building determined from a fixed-base model shall be increased at every level, including immediately above the footing, by an interstorey drift ratio equal to the footing rotation, in radians.

In lieu of a more detailed analysis, the footing rotation may be estimated from Equation 21.22 when the applied overturning moment is sufficiently large to cause the "heel" of the footing to up-lift, which occurs when $M_f > P_f\,\ell_f/6$.

Equation 21.22

$$\theta = 0.3\left(\frac{q_s}{G_0}\right)\left(\frac{\ell_f}{a_s}\right)\left\{1 + 2\left(\frac{a_s}{b_f}\right)^{1.5}\right\}$$

where

θ = footing rotation in radians

a_s = length of uniform bearing stress in soil or rock required to resist the applied loads

b_f = width of footing (parallel to axis of rotation)

q_s = magnitude of uniform bearing stress in soil or rock required to resist the applied loads

G_o = initial shear modulus of soil or rock, which may be estimated in kPa units from $\gamma_s / V_s^2 / 1000$ when γs, the density of soil or rock, is in kg/m³, and V_s, the shear wave velocity measured in the soil or rock immediately below the foundation, is in units of m/s

ℓ_f = length of footing (perpendicular to axis of rotation)

When the overturning moment is less than the moment required to cause up-lift, the footing rotation may be assumed to vary linearly from zero at $M_f = 0$ up to the rotation given by Equation 21.22 when $M_f = P_f\,\ell_f/6$.

21.10.3.3.3 Movements of not capacity-protected (NCP) foundations

Except as given below, the rotations of NCP foundations designed in accordance with Clause 21.10.3.2.4 shall be determined using a dynamic analysis that accounts for the reduced rotational stiffness of the footing due to footing uplift and soil deformation.

In lieu of using dynamic analysis to determine the increased drifts due to rotations of NCP foundations, the interstorey drift ratio of the building determined from a fixed-base model shall be increased at every level, including immediately above the footing, by a value equal to the largest of

a) 50% of the displacement at the top of the SFRS determined from a fixed-base model, divided by the height above the footing;

b) the rotation of the foundation calculated using Equation 21.22 when subjected to an overturning moment equal to the nominal overturning capacity of the SFRS; but need not exceed 3.0 times the rotation of the foundation when subjected to an overturning moment equal to the factored overturning resistance of the foundation; or

c) an interstorey drift ratio equal to 0.005.

Note: *The drift ratio increases are upper-bound values that can be used to avoid calculating the actual increased drift ratios in those cases where the SFRS and the members not considered part of the SFRS can easily tolerate the increases. When an NCP foundation is subjected to an overturning moment greater than its factored overturning resistance for Item b), the magnitude of uniform bearing stress in soil or rock required to resist the applied loads will exceed the factored bearing resistance when calculating rotations using Equation 21.22.*

21.10.3.4 Design of footings in NCP foundations

When NCP foundations are designed in accordance with Clause 21.10.3.2.4, the factored flexural and shear resistances of the footing shall be sufficient to develop the smallest of

a) the forces determined using a bearing stress in the soil or rock equal to 2.5 times the factored bearing resistance;

b) the forces resulting from an applied moment on the foundation equal to the nominal overturning resistance of the SFRS; or

c) the forces from design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 1.3.

Note: *The plan dimensions of the footing, which control the foundation overturning resistance, are determined using the factored bearing resistance of the soil or rock. The required amount of flexural reinforcement in the footing and the shear design of the footing, which may be influenced by the shear span-to-depth ratio, must account for possible overstrength in the bearing resistance of soil or rock. The increase in bearing stress in soil or rock results in the same bearing stress resultant; but the vertical force is located closer to the toe of the foundation.*

21.10.4 Footings, foundation mats, and pile caps

21.10.4.1

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. In addition, the reinforcement shall satisfy the requirements of Clauses 21.10.4.2 to 21.10.4.4

21.10.4.2

Columns designed assuming fixed-end conditions at the foundation shall comply with Clause 21.10.2.1 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.

21.10.4.3

Concentrated wall reinforcement shall extend to the bottom of the footing, mat, or pile cap and terminate with a 90° hook or mechanical anchorage.

21.10.4.4

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.

21.10.4.5

Footings or pile caps that are a part of the foundation system resisting tension due to uplift forces shall have top reinforcement in each direction to satisfy the flexural requirements but not less than 0.001 times the gross sectional area in each direction.

21.10.5 Grade beams and slabs on grade

21.10.5.1

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.

21.10.5.2

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.

21.10.5.3

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 21.3.3.

21.10.5.4

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 21.9. The design drawings shall clearly state that the slab on grade is a structural diaphragm and part of the SFRS.

21.10.6 Piles

21.10.6.1

The requirements of Clauses 21.10.6.2 to 21.10.6.6 shall apply to concrete piles supporting structures designed for earthquake resistance.

21.10.6.2

Piles 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.

21.10.6.3

At sites where $I_E F_a S_a(0.2)$ is greater than 0.75 and the factored moment in piles, drilled piers, or caissons is greater than 75% of the factored moment resistance, these members shall have confinement reinforcement at the following locations:

a) at the top of these members 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 Item a) for members in air, in water, or in soil incapable of providing lateral support; and
c) within five member diameters of the interface between soils of different strength or stiffness.

21.10.6.4

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.

21.10.6.5

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.

21.10.6.6

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.*

21.11 Members not considered part of the seismic-force-resisting system (R_d = 1.5, 2.0, 2.5, 3.5, or 4.0)

Note: *One of the most common causes of building collapse during an earthquake is failure of one or more components of the gravity-load resisting frame. The intent of this clause is to ensure an adequate level of strength and/or ductility for all structural members not considered part of the seismic-force-resisting system (SFRS) but subjected to seismically induced deformations. This is accomplished by ensuring elements either remain elastic or yield in bending and contain appropriate detailing to ensure adequate shear resistances and flexural ductility.*

21.11.1 General

21.11.1.1 Application

Independent of the R_d used to design the SFRS, the requirements of Clause 21.11 shall apply to all members of the structure not considered part of the SFRS unless the building is located where $I_E F_a S_a(0.2)$ is less than or equal to 0.35 or the maximum interstorey drift ratio at any level is less than 0.005.

The interstorey drift ratio shall be determined from an analysis in accordance with the National Building Code of Canada, incorporating the effects of torsion, including accidental torsion, and accounting for foundation movements in accordance with Clauses 21.10.3.3.

21.11.1.2

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 21.11.2 to 21.11.4 provided that

a) the effects of non-structural elements are accounted for in the design of structural elements if the non-structural elements increase the seismically induced deformation of or the forces applied to a structural element; and

b) the non-structural elements are attached to the building in accordance with the *NBCC*.

21.11.1.3

Structural members not considered part of the seismic-force-resisting system (SFRS) shall meet the requirements of Clauses 21.11.2 to 21.11.4.

21.11.2 Seismic demands

21.11.2.1 General analysis requirements

An analysis shall be done to determine the forces and deformations induced in structural members not considered to be part of the SFRS due to seismic demands on the SFRS. Such an analysis shall satisfy the following:

a) the complete structure shall be displaced laterally to the design displacements $\Delta_f R_d R_o$, determined from an analysis in accordance with the *NBCC*, incorporating the effects of torsion; including accidental torsion, and accounting for foundation movements in accordance with Clauses 21.10.3.3;

b) the inelastic displacement profile of the SFRS shall be accounted for;

Note: *Yielding of the SFRS causes concentration of deformations at plastic hinge locations. In lieu of using a nonlinear model of the SFRS, a linear model with appropriately reduced section properties at plastic hinge locations may be used to estimate the inelastic displacement profile.*

c) cracking of concrete may be accounted for in determining the section properties used for structural members not considered part of the SFRS; however, an upper-bound estimate of effective stiffness shall be used in order to determine a safe estimate of the induced forces.

Note: *Low estimates of average section properties such as those given in Clause 21.2.5.2 are used for the SFRS to make a safe estimate of the design displacements of the overall building. Higher estimates of section properties must be used for each structural member not considered part of the SFRS to make a safe estimate of the forces induced in these members by the design displacements of the SFRS.*

d) The increased displacements due to foundation movement determined in accordance with Clauses 21.10.3.3 shall be accounted for.

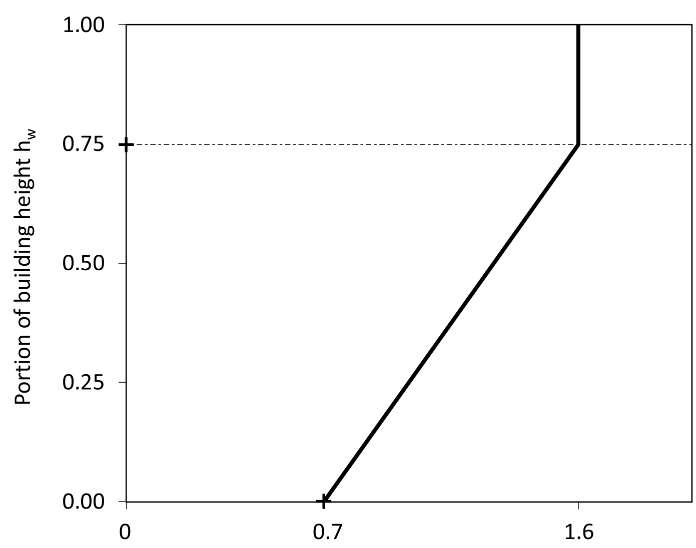
21.11.2.2 Simplified analysis of shear wall buildings

When the SFRS consists of shear walls or coupled walls, the requirements of Clauses 21.11.2.1 a), b), and d) may be satisfied by the following simplified analysis:

a) The shear force and bending moments induced in members of a gravity-load resisting frame shall be determined at each level by subjecting the frame to the interstorey drift ratio given in Figure 21.1 for that level. The deflection Δ used to calculate the global drift ratio Δ / h_w in Figure 21.1 shall be the design lateral deflection at the top of the gravity-load resisting frame determined from an analysis in accordance with the *NBCC*, incorporating the effects of torsion, including accidental torsional moments, and accounting for foundation movements in accordance with Clause 21.10.3.3. The height of the building h_w shall be measured from the base of the plastic hinge zone in the SFRS. All gravity-load resisting frames shall be investigated in each direction of loading.

Note: *A gravity-load resisting frame may consist of a single column or wall and the attached beams or slabs or may consist of a combination of columns, walls, beams, and slabs.*

Figure 21.1
Envelope of minimum interstorey drift ratios over building height.



b) The additional vertical load that is induced in vertical-load resisting members due to lateral deformation of the structure shall be determined by summing the shear forces from all horizontal members supported by the vertical-load resisting member using the procedure in Clause 21.11.2.2 a) for each level and summing the contribution for all levels above the level of interest.

c) Over the height of the plastic hinge region of the SFRS, the minimum curvature demand on all columns and walls shall not be taken less than the curvature demand associated with the inelastic rotational demands on the SRFS given in Clauses 21.5.7.2 and 21.5.8.4.2.

21.11.3 Design of members in gravity-load resisting frames

21.11.3.1 Shear resistance

The factored shear resistance of each structural member not considered part of the SFRS shall be sufficient to carry all shear forces due to factored gravity loads in addition to the shear forces induced in the member when the structure is subjected to the seismic demands given in Clause 21.11.2. The factored shear resistance need not exceed the maximum shear force that can develop due to the probable bending moment resistance of the member or adjacent members as given in Clauses 21.3.1.5.1 for beams and 21.3.2.7.1 b) for columns.

21.11.3.2 Resistance of members transferring gravity loads

The factored resistance of all members not considered part of the SFRS that transfer gravity loads from upper floors, including columns, walls, transfer girders and transfer slabs shall have sufficient capacity to resist all vertical forces due to factored gravity loads in addition to the vertical forces induced in the member when the structure is subjected to the seismic deformation demands given in Clause 21.11.2. The additional vertical force due to seismic deformations need not exceed the maximum axial force that can develop due to the nominal flexural resistance of the attached horizontal members.

21.11.3.3 Design of gravity-load resisting columns and bearing walls

21.11.3.3.1 Limitations on thin bearing walls

If the interstorey drift ratio determined from an analysis in accordance with the *NBCC* and incorporating the effects of torsion, including accidental torsional moments, exceeds 0.005 at any point in the structure, all bearing walls in the entire structure that are assumed to support gravity loads shall contain two layers of uniformly distributed reinforcement and the two layers shall have a minimum clear spacing of 50 mm.

Note: *The maximum interstorey drift at any point in the structure is used as an indicator of seismic demands and the flexibility of the structure. Walls with a single layer of reinforcement might not be able to tolerate cycles of combined in-plane and out-of-plane displacement.*

21.11.3.3.2 Plastic hinge regions of shear wall buildings

When the SFRS consists of shear walls or coupled walls, all columns and walls that support gravity loads shall meet the following requirements over the storeys that the SFRS is required to be detailed for plastic hinging to occur as specified in Clause 21.5.2.1:

a) All columns and walls shall have a curvature capacity greater than the curvature demand given in Clause 21.11.2.2 c). This requirement may be met by limiting the distance to the neutral axis, c, in these members determined from a plane sections analysis for the factored resistance or from Clause 21.5.7.4 to:

$$c \le \frac{\varepsilon_{cu}}{(2\theta_{id} + 0.004)} \cdot \ell_w$$ **Equation 21.23**

where

ℓ_w = the length of the shear walls or coupled walls of the SFRS parallel to the distance c as defined in Clause 21.5.7.2 or 21.5.8.4.3

θ_{id} = the inelastic rotational demand on the SFRS determined from Clause 21.5.7.2 or Clause 21.5.8.4.2

ε_{cu} shall be taken as 0.0035 unless the compression region of the member is confined as a column. When ε_{cu} is taken greater than 0.0035 but less than 0.010, the amount of confinement reinforcement shall be determined with k_p taken as $(0.1 + 30\varepsilon_{cu})$. This reinforcement shall be provided over a distance of not less than c from the compression face of the member.
For shear walls designed with $R_d \leq 2.5$, in lieu of determining the limiting distance c in columns and walls from Equation 21.23, the simplified procedure given in Clause 21.5.7.6 may be used.

b) Unless the design displacement $\Delta_f R_o R_d$ is less than $h_w/200$: all columns shall contain at least buckling prevention ties in accordance with Clause 21.2.8.1 over the full height that the SFRS is required to be detailed for plastic hinging to occur, and; all walls shall have concentrated vertical reinforcement consisting of a minimum of four bars at each end of the wall and at the ends and intersections of all wall flanges. The concentrated reinforcement in walls shall be at least tied as a compression member in accordance with Clause 7.6.5, and the ties shall be detailed as hoops.

21.11.3.3.3 Design of columns and walls for plastic hinging

The seismic design requirements for columns and walls that are part of the gravity-load resisting frame depend on the inelastic flexural deformation demands on the member. When seismic demands on the gravity-load resisting frame are determined using a linear model, the design requirements shall be determined from how much the calculated induced bending moment due to the seismic deformation demands given in Clause 21.11.2 exceeds the factored bending resistance of the member.

Note: *Factored resistances are used to account for the uncertainty in displacement demands – the resistances are reduced rather than the displacement demands increased. Multiples of factored resistance are used as indicators of inelastic displacement demands.*

The calculated induced bending moment determined from a linear analysis shall be limited depending on the type of member, axis of bending in walls, and level of applied axial compression as follows:

Maximum calculated induced bending moment

Type of column or wall *	Axial compression †	
	$P_s \leq 0.2 f'_c A_g$	$P_s \geq 0.4 f'_c A_g$
Ductile columns satisfying: Clauses 21.3.2.2, 21.3.2.5, 21.3.2.6, 21.3.2.7	$5.0M_r$	$3.0M_r$
Moderately ductile columns satisfying: Clauses 21.4.2.2, 21.4.4 except 21.4.4.2, 21.4.5	$3.0M_r$	$2.0M_r$
Tied columns satisfying Clause 7.6.5 and the dimensional limitations of Clause 21.4.2.2.	$2.0M_r$	$1.5M_r$
Other columns or walls tied as compression members in accordance with Clause 7.6.5 over full length	$1.5M_r$	$1.0M_r$
Strong-axis bending of walls with two layers of reinf. and concentrated reinf. satisfying Clause 21.6.3.7.4	$1.2M_r$‡	$0.8M_r$‡

(Continued)

(Concluded)

Type of column or wall *	Axial compression †	
	$P_s \leq 0.2\, f'_c A_g$	$P_s \geq 0.4\, f'_c A_g$
Strong-axis and weak-axis bending of walls with two layers of reinforcement.	$1.0M_r$‡	$0.7M_r$‡
Strong-axis and weak-axis bending of walls with single layer of reinforcement.	$0.7M_r$‡	$0.5M_r$‡

* *All members shall satisfy the shear design requirements of Clause 21.11.3.1.*
† *Linear interpolation shall be used for intermediate levels of axial compression.*
‡ *The induced bending moment in these members shall be determined using $E_c I_e = 1.0 E_c I_g$.*

21.11.3.4 Design of gravity-load resisting beams

21.11.3.4.1 General

The design requirements for beams that are part of the gravity-load resisting frame depend on the inelastic flexural deformation demands on the member. When seismic demands on the gravity-load resisting frame are determined using a linear model, the design requirements shall be determined from how much the induced bending moment due to the seismic deformation demands given in Clause 21.11.2 exceeds the factored bending resistance of the flexural member as follows:

Note: *Factored resistances are used to account for the uncertainty in displacement demands – the resistances are reduced rather than the displacement demands increased.*

a) When the induced bending moment determined from a linear analysis is greater than the given limit, the member shall meet the corresponding detailing requirements as follows:

Induced bending moment	Beam detailing requirements
$< 1.0M_r$	No additional requirements
$\geq 1.0M_r$; but $< 2.0M_r$	Limited ductility — Clause 21.11.3.4.2.
$\geq 2.0M_r$; but $< 3.0M_r$	Moderately ductile — Clause 21.11.3.4.3.
$\geq 3.0M_r$; but $< 5.0M_r$	Ductile — Clause 21.11.3.4.4.

b) When the induced bending moment due to seismic deformation demands determined from a linear analysis is greater than 5.0 times the factored bending resistance, the design of the structure shall be modified to reduce the induced bending moment or increase the bending resistance of the member. The effect of these changes on the seismic demands in the rest of the gravity-load resisting frame shall be accounted for.

21.11.3.4.2 Detailing beams for limited ductility

When a beam in a gravity-load resisting frame requires limited ductility as given in Clause 21.11.3.4.1 a), the member shall meet all of the following requirements:

a) Throughout the length of the beam, at least two effectively continuous longitudinal bars shall be provided at both top and bottom, and stirrups meeting Clauses 21.11.3.1 and 11.2.8.2 shall be spaced at not more than $d/2$.

b) The positive moment reinforcement required by Clause 12.11.1 shall be anchored to develop the specified yield strength, f_y, in tension at the face of the support.

c) At the locations of flexural yielding, the flexural tension reinforcement shall not be less than $1.4b_w d/f_y$, and the reinforcement ratio, ρ, shall not exceed 0.025.

21.11.3.4.3 Detailing beams for moderately ductile behaviour

When a beam in a gravity-load resisting frame requires moderate ductility as given in Clause 21.11.3.4.1 a), the member shall meet all of the following requirements:

a) All requirements given in Clause 21.11.3.4.2 except as modified below.

b) For a distance d on either side of a section where flexural yielding may occur, stirrups meeting Clause 21.11.3.1 shall be spaced the smaller of $d/4$ and 12 times the diameter of the smallest enclosed longitudinal bar.

c) All top longitudinal reinforcement terminated in a column shall extend to the far face of the column core and be anchored by a standard 90° hook located within the column core so as to develop the yield strength of the reinforcement.

21.11.3.4.4 Detailing beams for ductile behaviour

When a beam in a gravity-load resisting frame requires a high level of ductility as given in Clause 21.11.3.4.1 a), the member shall meet all of the following requirements:

a) All requirements given in Clause 21.11.3.4.3 except as modified below.

b) The clear span of member shall be not less than three times its effective depth.

c) The shear design of the member shall be in accordance with Clause 21.3.1.5.

d) For a distance d on either side of a section where flexural yielding may occur, the stirrups shall be detailed as hoops and seismic crossties and shall be spaced at the smaller of $d/4$ and 8 times the diameter of the smallest enclosed longitudinal bar.

e) Lap splices shall not be located within a distance d on either side of a section where flexural yielding may occur.

f) Transverse hoop reinforcement shall be provided over the depth of the beam-column joints and shall be spaced at a maximum of 8 times the diameter of the smallest enclosed longitudinal column bar.

21.11.4 Design of slab-column connections for seismic drift demands

21.11.4.1 Reduction of punching shear resistance due to drift demands

Where the maximum gravity load two-way shear stresses determined using seismic load combinations and excluding shear stresses from unbalanced bending moment, 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 21.11.4.2, with R_E calculated as follows:

$$R_E = \left(\frac{0.005}{\delta_i}\right)^{0.85} \leq 1.0$$

Equation 21.24

where the interstorey drift ratio, δ_i, shall be determined in accordance with Clause 21.11.2. For shear wall or coupled wall buildings, the minimum interstorey drift ratios are given in Clause 21.11.2.2.

21.11.4.2 Design of shear reinforcement in slabs

When shear reinforcement is required by Clause 21.11.4.1, or the slab design is to qualify for the exemption in Clause 21.6.4.8, 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 v_f, excluding shear stresses from unbalanced moment, and determined using seismic load combinations, does not exceed $v_r = R_E(0.5v_c + v_s)$ with v_c calculated in accordance with Clause 13.3.8.3, 13.3.9.3, or 18.12.3.3, and with v_s calculated in accordance with Clause 13.3.8.5 or 13.3.9.4.

b) The factored shear stress resistance of the shear reinforcement, v_s, 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, except shear reinforcement shall extend a minimum of $4d$ beyond the face of the column and stud spacing shall be $\leq d/2$.

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

a) the influence of climatic conditions;
b) selection and proportioning of materials;
c) mixing, placing, and curing of concrete;
d) the degree of restraint to movement; and
e) 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:

$$P_f = 0.45\phi_c \alpha_1 f'_c A_g \left[1 - \left(\frac{k\ell_c}{32t}\right)^2\right]$$ **Equation 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 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

$$V_r = \frac{2}{3}\left(0.18\lambda\phi_c\sqrt{f_c'}\,bh\right)$$ **Equation 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

$$V_r = \frac{2}{3}\left(\left(1+\frac{2}{\beta_c}\right)0.18\,\lambda\,\phi_c\sqrt{f_c'}\,b_o h\right)$$ **Equation 22.3**

but

$$V_r \le \frac{2}{3}(0.37\,\lambda\,\phi_c\sqrt{f_c'}\,b_o h)$$ **Equation 22.4**

22.7 Slabs on grade
Plain concrete slabs shall be designed with due regard to loading and foundation conditions.

Notes:

1) *See CSA A23.1 for additional information on slabs on grade including tolerances for slab thickness, surface flatness, and concrete mix designs.*

2) *See ACI 360R and 302R for information on the design and the construction of slabs on grade and for guidance on curing, curling, and crack control.*

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:

$$V_r = \frac{3}{4}(0.18\lambda\phi_c\sqrt{f'_c})$$ **Equation 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 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. The seismic design of tilt-up wall panels shall also meet the requirements of Clause 21, where applicable.

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, or 21.7, 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 to tilt-up panels shall take into account in-plane and out-of-plane forces; the additional effects of shrinkage, creep, temperature, movement; and the seismic design requirements where applicable.

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 and to provide a load path to the lateral load resisting system. Panels supported at the top by steel roof deck, wood decking, or plywood only shall have additional tie struts or continuity ties perpendicular to the wall and connecting the panel back to the primary roof structural members, such as beams or joists. The ties shall be designed for the forces specified in Clause 23.2.9.2. Steel ties shall have a minimum thickness of 3 mm.

23.2.9.2
Connection of tilt-up panels to floor and roof diaphragms, including at the base, shall be designed for the required out-of-plane forces specified in Clause 23.2.8.1. The connection horizontal force shall not be less than 2% of the total factored vertical load that the wall is designed to carry at the level of support or 5 kN per metre length of wall, whichever is greater.

23.2.9.3 Effective reinforcement
Where vertical reinforcement is placed in two layers, the effect of compression reinforcement shall be ignored when calculating flexural resistance for out-of-plane forces.

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:

$$\frac{P_{wf} + P_{tf}}{A_g} < 0.09\phi_c f_c' \qquad \textbf{Equation 23.1}$$

23.3.1.3

The factored moment, M_f, shall be determined at the mid-height of the panel and shall be equal to

$$M_r = M_b\delta_b \qquad \textbf{Equation 23.2}$$

where

$$M_b = \frac{W_t \ell^2}{8} + P_{tf}\frac{e}{2} + \left(P_{wf} + P_{tf}\right)\Delta_o$$

$$\delta_b = \frac{1}{1 - \frac{P_f}{\phi_m K_{bf}}} \geq 1.0$$

where

$$P_f = P_{wf} + P_{tf}$$

$$K_{bf} = \frac{48 E_c I_{cr}}{5\,\ell^2}$$

where

$$I_{cr} = \frac{bc^3}{3} + \frac{E_s}{E_c} A_{s,\text{eff}} (d - c)^2$$

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

$$M_r \geq M_f \qquad \textbf{Equation 23.3}$$

The resisting moment may be calculated using an effective area of reinforcement, $A_{s,\text{eff}}$, as follows:

$$A_{s,\text{eff}} = A_s + \frac{P_f}{\phi_s f_y}\left(\frac{h}{2d}\right) \qquad \textbf{Equation 23.4}$$

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:

$$\Delta_s = \frac{5M_s\ell^2}{48E_cI_e} = \frac{M_s}{K_{bs}} \qquad \textbf{Equation 23.5}$$

where
M_s = $M_{bs}\,\delta_{bs}$

where

$$M_{bs} = \frac{W_s\ell^2}{8} + P_{ts}\frac{e}{2} + (P_{ws} + P_{ts})\Delta_o$$

$$\delta_{bs} = \frac{1}{1 - \frac{P_s}{K_{bs}}} \geq 1.0$$

where
P_s = $P_{ws} + P_{ts}$

$$K_{bs} = \frac{48E_cI_e}{5\,\ell^2}$$

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.

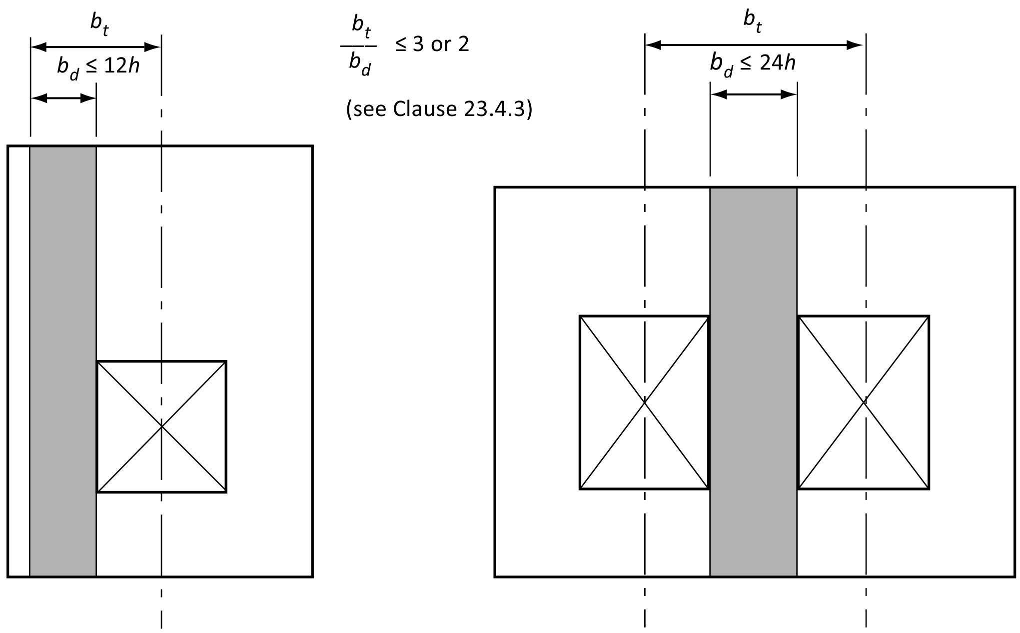
23.4.1.2

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

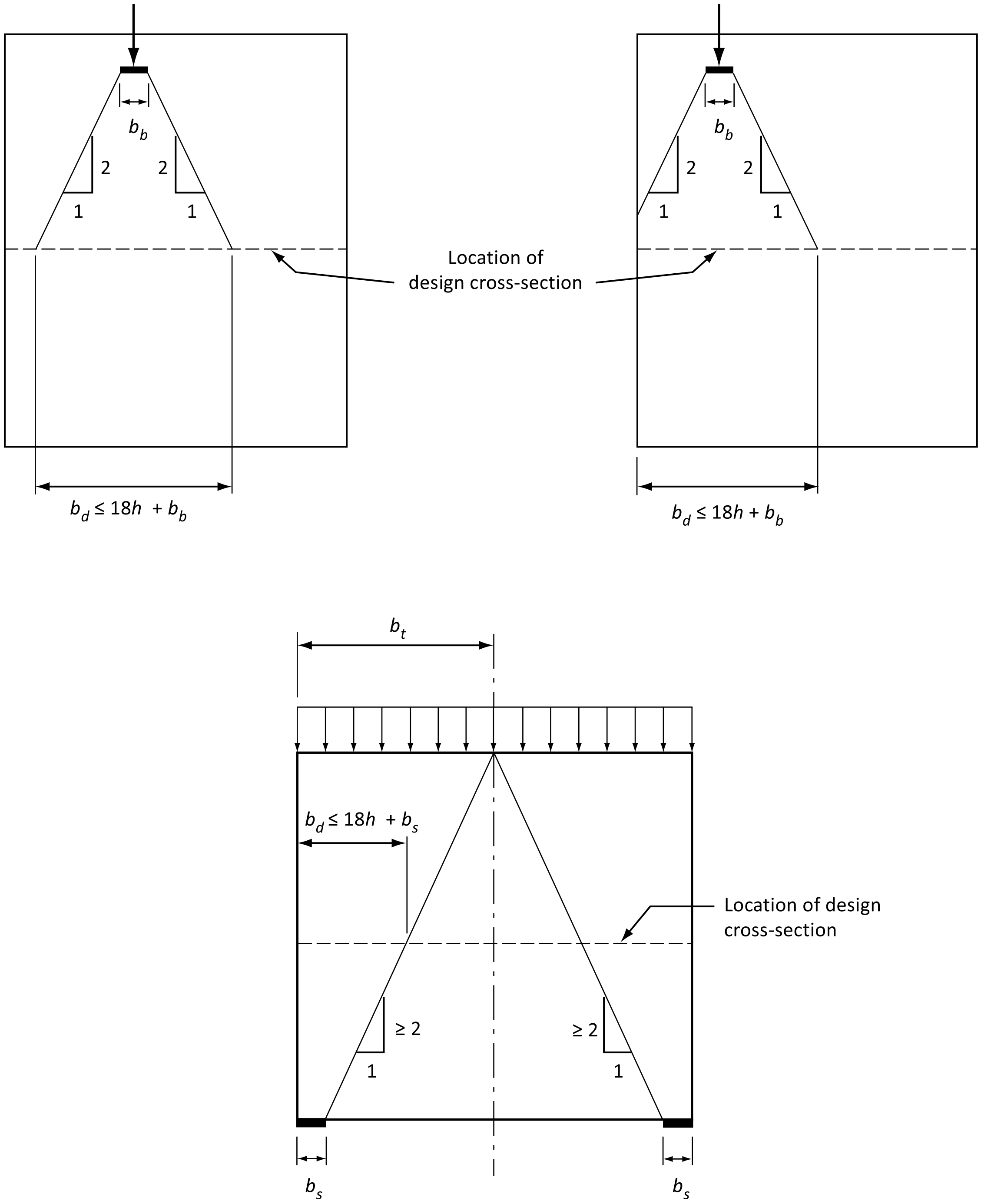
Unless a more detailed analysis, accounting for the internal force effects, indicates otherwise, the ratio of tributary width to design width shall not exceed 3. For panels with a single layer of reinforcement and $\ell/h > 40$, the ratio shall not exceed 2.

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 4.1.2, 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 so that the expected failure mode is ductile.

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 A23.1-14, **Concrete materials and methods of concrete construction**

Notes:
1) *This Annex is not a mandatory part of this Standard.*
2) *This Annex provides portions from an unpublished draft of CSA A23.1-14.*
3) *A number of clauses from CSA A23.1-14 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.*
4) *This Annex reprints only those portions of Clause 2 of CSA A23.1 applicable to the other clauses reprinted in this Annex.*

A.1

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 Group

A23.3-14
Design of concrete structures

CAN/CSA-G30.18-M92 (R2007)
Billet-steel bars for concrete reinforcement

CAN/CSA-S6-06
Canadian Highway Bridge Design Code

CAN/CSA-S413-07
Parking structures

CAN/CSA-S474-04 (R2009)
Concrete structures

S478-95 (R2007)
Guideline on durability in buildings

CAN/CSA-S806-02 (R2007)
Design and construction of building components with fibre-reinforced polymers

W59-03 (R2008)
Welded steel construction (metal-arc welding)

W186-M1990 (R2007)
Welding of reinforcing bars in reinforced concrete construction

ACI (American Concrete Institute)
201.2R-08
Guide to Durable Concrete

ANSI/AWS (American National Standards Institute/American Welding Society)
D1.1:2008
Structural Welding Code — Steel

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 4.1.1.10 and 7.4 and Tables 1 to 4 and 20, as appropriate.

Notes:

1) *Although minimum requirements for concrete durability are specified, it should be stressed that a durable concrete also depends upon the use of high-quality materials, an effective quality control program, and good quality of work in manufacturing, placing, finishing, and curing the concrete.*

2) *For exposure conditions not covered by this Standard and for general information on concrete durability, see ACI MCP, ACI 201.2R, and PCA IS001.08T.*

3) *For parking structures, highway bridges, and offshore structures, see CSA S413, CAN/CSA-S6, and CAN/CSA-S474, respectively.*

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).
d) Hooks for crossties shall have a bend of a 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.

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 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 greater.

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 greater than 16 °C, unless bending tests that are otherwise in accordance with CSA G30.18 confirm that bars bent at temperatures below 16 °C are acceptable.

Note: *See Stecich et al., 1984.*

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.

Notes:

1) *Black (1973) states "Construction conditions might require straightening of bars embedded in concrete. Field bending should not be done without authorization of the engineer. The engineer must determine if the bars should be bent cold or if heating should be employed. Bends should be gradual and must be straightened as required.*

"Tests have shown that Grade 400 reinforcing bars can be cold bent and straightened up to 90° at or near the minimum bend diameter. If cracking or breaking occurs, heating to a maximum temperature of 820 °C should be beneficial for avoiding overstressing and damage for subsequent bars. Bars that fracture during bending or straightening must be spliced outside the bend region.

"Heating must be done in a manner that will avoid damage to the concrete. If the bend area is within approximately 150 mm of the concrete, some protective insulation might be required. Heating of the bar

should be controlled by temperature-indicating crayons or other suitable means. The heated bars should not be artificially cooled until they have naturally cooled to at least 300 °C".

2) *See Stecich et al., 1984.*

6.6.2.5.3

The bending tolerances shall be sufficiently accurate to comply with the placing and protection tolerances specified in Clause 6.6.8.

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.

Note: *The intention of this Clause is to provide adequate spacing for concrete to be placed in the presence of closely spaced mats of steel.*

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 CAN/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 largest of the limits for each relevant exposure condition in Table 17.

Note: *See Clause 6.6.8 for tolerances of concrete cover and Clause 6.8.2.4 for additional cover requirements.*

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 Appendix D of the NBCC.*

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 (however, 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 provide a more rigid reinforcement mat or cage, i.e. in prefabricated reinforcing cage, 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. Weldable grade bars shall be used unless a fusion weld is employed.

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 and CSA W47.1.

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 or to the requirements of CSA W47.1 and CSA W59 at the option of the Contractor.

Note: *Refer to Clause 6.6.10.1 for requirements for welding reinforcing bars.*

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 AWS D1.1/D1.1M.*

6.8 Post-tensioning

6.8.2 Unbonded tendons

6.8.2.4.1
The concrete cover to the anchorage, measured in a direction perpendicular to the tendon, shall be not less than 40 mm.

6.8.2.4.2

The stressing pocket shall be sufficiently deep so that the cover to the end cap, measured parallel to the tendon, will be at least 40 mm and the cover to the anchorage will be at least 60 mm.

Table 1
Definitions of C, F, N, A, and S classes of exposure
(See Clauses 3, 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 8.13.3, Tables 2 and 17, and Annex L.)

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 or A-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 structures exposed to seawater located within the tidal and splash zones, concrete exposed to seawater spray, and salt water pools. For seawater or seawater-spray exposures the requirements for S-3 exposure also have to be met.
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 structures exposed to seawater. For seawater or seawaterspray exposures the requirements for S-3 exposure also have to be met.
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 that when in service is neither exposed to chlorides nor to freezing and thawing nor to sulphates, either in a wet or dry environment. Examples: footings and interior slabs, walls, and columns.
N-CF	Interior concrete floors with a steel-trowel finish that are not exposed to chlorides, nor to sulphates either in a wet or dry environment. Examples: interior floors, surface covered applications (carpet, vinyl tile) and surface exposed applications (with or without floor hardener), ice-hockey rinks, freezer warehouse floors.
A-XL	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 might be generated, with higher durability performance expectations than A-1 class
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 might 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.

(Continued)

Table 1 (Concluded)

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 and to seawater or seawater spray (Tables 2 and 3).
R-1	Residential concrete for footings for walls, columns, fireplaces and chimneys.
R-2	Residential concrete for foundation walls, grade beams, piers, etc.
R-3	Residential concrete for interior slabs on ground not exposed to freezing and thawing or deicing salts.

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 exposed to sulphates shall comply with the minimum requirements of S class noted in Tables 2 and 3. In particular, Classes A-1 to A-4 in municipal sewage elements could be subjected to sulphate exposure.*

5) *No hydraulic cement concrete will be entirely resistant in severe acid exposures. The resistance of hydraulic cement concrete in such exposures is largely dependent on its resistance to penetration of fluids.*

Table 16
Bend diameter for standard hooks
(See Clause 6.6.2.3.)

Bar size, mm	Minimum bend diameter,* mm Steel grade 300R‡	 400R or 500R	 400W or 500W§
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.*

‡ *R refers to "Regular" grade.*

§ *W refers to "Weldable" grade*

Table 17
Concrete cover
(See Clauses 4.3.2.2.1 and 6.6.6.2.3.)

Exposure condition	Exposure class (see Tables 1 and 2) N	 F-1, F-2, S-1, S-2, S-3	 C-XL, A-XL, C- 1, C-3, A-1, A-2, A-3
Cast against and permanently exposed to earth, including footings and piles	75 mm	75 mm	75 mm
Beams, girders, and columns	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.*

(Continued)

Table 17 (Concluded)

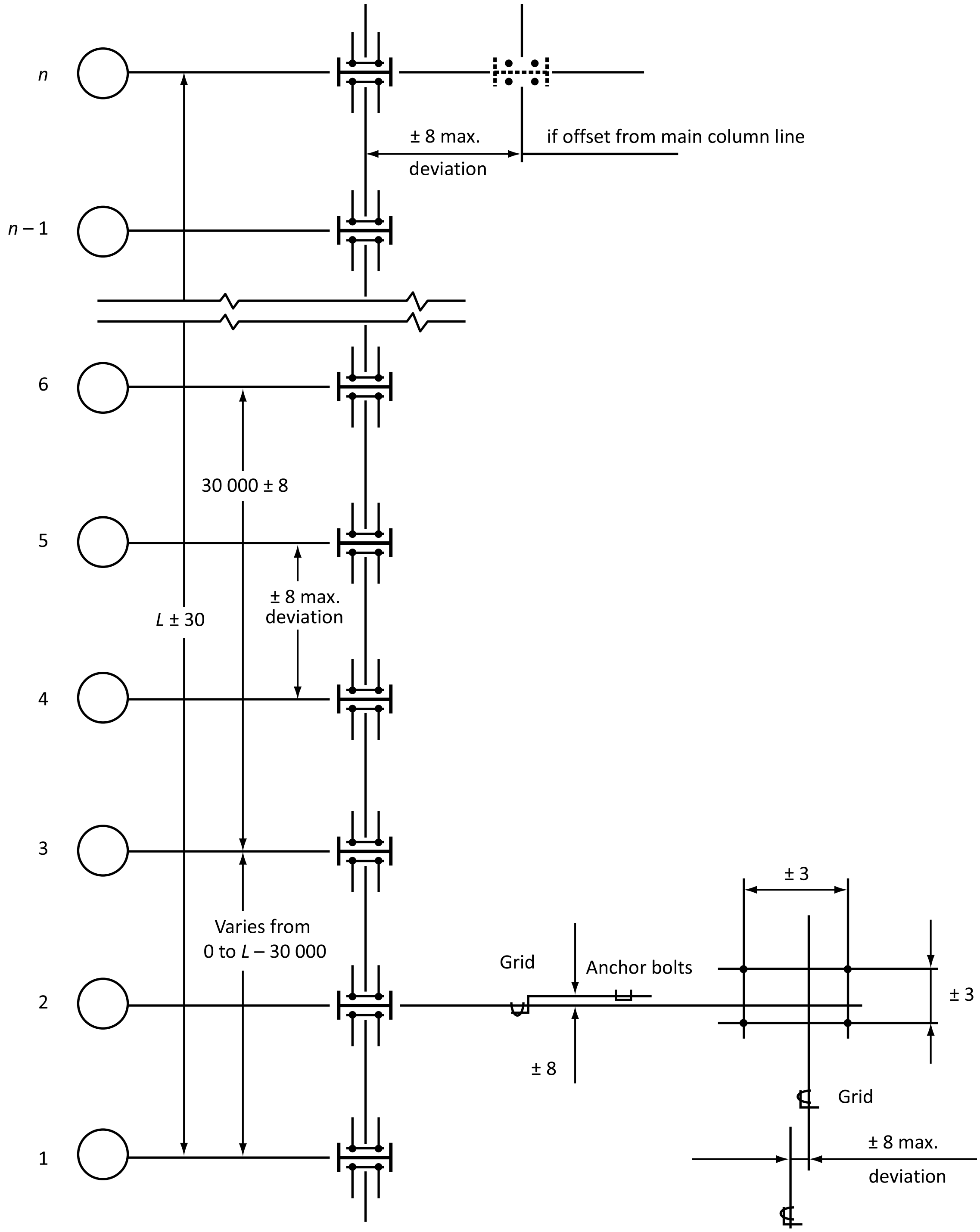
Notes:

1) *Greater cover or protective coatings might 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 CSA S413.*

3) *For information on the additional protective measures and requirements for bridges, see CAN/CSAS6.*

Figure 3
Tolerances on anchor bolt placement
(See Clause 6.7.3.1.)



Note: *All measurements are in millimetres*

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 $b_w h_b^3 / \ell_n h_n^3$ 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)

$$M_a neg = C_a neg w_f \ell_a^2$$ **Equation B.1**

b)

$$M_b neg = C_b neg w_f \ell_b^2$$ **Equation 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)

$$M_{al} pos = C_{al} w_{lf} \ell_a^2$$ **Equation B.3**

b)

$$M_{ad} pos = C_{ad} w_{df} \ell_a^2$$ **Equation B.4**

c)

$$M_{bl} pos = C_{bl} w_{lf} \ell_b^2$$ **Equation B.5**

d)

$$M_{bd} pos = C_{bd} w_{df} \ell_b^2$$ **Equation 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:

$$\frac{W_f \ell_a}{3}$$ **Equation B.7**

a) for the long span:

$$\frac{W_f \ell_a}{3} \times \frac{(3 - m^2)}{2}$$ **Equation 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

(Continued)

Table B.2 (Concluded)

$m = \ell_a / \ell_b$	Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
	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	0.015	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*, 2015

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*, 2015 (NBCC). 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 NBCC* User's Guide *and should be used in conjunction with the resistance factors specified in Clause 8.4.*

4) *The NBCC 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 NBCC 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:*

C = *live load due to cranes including self weight*

C_7 = *crane bumper impact load*

C_d = *self weight of all cranes positioned for maximum effects*

D = *a permanent load due to the weight of building components, as specified in Subsection 4.1.4 of the* NBCC

E = *earthquake load and effects — a rare load due to an earthquake, as specified in Subsection 4.1.8 of the* NBCC

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*

L_{xc} = *live load exclusive of crane loads*

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 load due to snow, including ice and associated rain, as specified in Article 4.1.6.2 of the* NBCC*, or due to rain, as specified in Article 4.1.6.4 of the* NBCC

T = *effects due to contraction, expansion, or deflection caused by temperature changes, shrinkage, moisture changes, creep, ground settlement, or a combination thereof (see Appendix A of the* NBCC)

W = *wind load — a variable load due to wind, as specified in Subsection 4.1.7 of the* NBCC

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 NBCC.*

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 *NBCC* 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, which shall be determined in accordance with Clause C.1.2.2.

C.1.2.2

Except as provided in Clause C.1.2.3, the effect of factored loads for a building or structural component shall be determined in accordance with the requirements of this Clause and the following load combination cases, the applicable combination being that which results in the most critical effect:

a) for load cases without crane loads, the load combinations listed in Table C.1 a); and
b) for load cases with crane loads, the load combinations listed in Table C.1 b).

C.1.2.3

Other load combinations that must also be considered are the principal loads acting with the companion loads taken as zero.

C.1.2.4

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, with load factors of 1.5, 1.0, and 1.25 assigned to H, P, and T respectively.

Table C.1 a)
Load combinations without crane loads 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	1.4*D*†	—
2	(1.25*D*‡ or 0.9*D*§) + 1.5*L***	1.0*S*†† or 0.4*W*
3	(1.25*D*‡ or 0.9*D*§) + 1.5*S*	1.0*L*††, ‡‡ or 0.4*W*
4	(1.25*D*‡ or 0.9*D*§) + 1.4*W*	0.5*L*‡‡ or 0.5*S*
5	1.0*D*§ + 1.0*E*§§	0.5*L*††, ‡‡ + 0.25*S*††

* *See Clauses C.1.2.2, C.1.2.3, and C.1.2.4.*
† *See Clause C.1.2.9.*
‡ *See Clause C.1.2.8.*
§ *See Clause C.1.2.5.*
** *See Clause C.1.2.6.*
†† *See Article 4.1.5.5 of the NBCC.*
‡‡ *See Clause C.1.2.7.*
§§ *See Clause C.1.2.10.*

Notes:
1) *This Table corresponds to Table 4.1.3.2.A of the* NBCC.
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.*

Table C.1 b)
Load Combinations With Crane Loads 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	(1.25D† or 0.9D‡) + (1.5C + 1.0L_{XC})	1.0S§ or 0.4W
2	(1.25D† or 0.9D‡) + (1.5L_{XC}** + 1.0C)	1.0S§ or 0.4W
3	(1.25D† or 0.9D‡) + 1.5S	(1.0C + 1.0L_{XC}§, ††)
4	(1.25D† or 0.9D‡) + 1.4W	(1.0C‡‡ + 0.5L_{XC}§, ††
5	(1.25D† or 0.9D‡) + C_7	—
6	1.0D‡ + 1.0E§§	1.0C_d + 0.5L_{XC}§, ††+ 0.25S§

* *See Clauses C.1.2.2, C.1.2.3, and C.1.2.4.*
† *See Clause C.1.2.8.*
‡ *See Clause C.1.2.5.*
§ *See Article 4.1.5.5 of the* NBCC.
** *See Clause C.1.2.6.*
†† *See Clause C.1.2.7.*
‡‡ *Side thrust due to cranes need not be combined with full wind load.*
§§ *See Clause C.1.2.10.*

C.1.2.5

Except as provided in Sentence 4.1.8.16.(1) of the *NBCC*, the counteracting factored dead load, 0.9D in the load combinations specified in Cases 2, 3, and 4 and 1.0D in load Case 5 of Table C.1 a) and 0.9D in the load combination specified in Cases 1 to 5 and 1.0D in load combination Case 6 in Table C.1 b), 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 *NBCC*.

C.1.2.6

The principal-load factor 1.5 for live load, L, in Table C.1 a) and L_{XC} in Table C.1 b) may be reduced to 1.25 for liquids in tanks.

C.1.2.7

The companion-load factor for live load, L, in Table C.1 a) and L_{XC} in C.1 b) shall be increased 0.5 for storage occupancies and for equipment areas and service rooms in Table 4.1.5.3 of the *NBCC*.

C.1.2.8

Except as provided in Clause C.1.2.9, the load factor 1.25 for dead load, D, for soil, superimposed earth, plants, and trees in Table C.1 a) and C.1 b) 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.1.2.9

A principal-load factor of 1.5 shall be applied to the weight of saturated soil used in load combination Case 1 of Table C.1 a).

C.1.2.10
Earthquake load, *E*, in load combination Case 5 of Table C.1 a) and Case 6 of Table C.1 b) includes horizontal earth pressure due to earthquake determined in accordance with Sentence 4.1.8.16.(4) of the *NBCC*.

C.1.2.11
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.1.2.12
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 *NBCC* 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 *NBCC* and in the Standards listed in Section 4.3 of the *NBCC* (see Appendix A of the *NBCC*).

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 between
a) connected structural elements; or
b) safety-related attachments and structural elements.

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 cast-in anchors and to post-installed expansion (torque-controlled and displacement-controlled), undercut and adhesive anchors (see Figure D.1). Adhesive anchors shall be installed in concrete having a minimum age of 21 days at time of anchor installation. Specialty inserts, through-bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, grouted anchors, and direct anchors such as powder or pneumatic-actuated nails or bolts are not included in the provisions of Annex D. Reinforcement used as part of the embedment shall be designed in accordance with the applicable clauses of this Standard.

D.1.3

Design provisions are included for the following types of anchors:
a) Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal to or exceeding 1.4 N_{pr}, where N_{pr} is given in Equation D.16;
b) Hooked bolts having a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal to or exceeding 1.4 N_{pr}, where N_{pr} is given in Equation D.17;
c) Post-installed expansion and undercut anchors that meet the assessment criteria of ACI 355.2/355.2R; and
d) Adhesive anchors that meet the assessment criteria of ACI 355.4M.

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 — a statistical term meaning 90% confidence that there is a 95% probability of the actual strength exceeding the nominal strength.

Adhesive — chemical components formulated from organic polymers, or a combination of organic polymers and inorganic materials that cure when blended together.

Adhesive anchor — a post-installed anchor, inserted into hardened concrete with an anchor hole diameter not greater than 1.5 times the anchor diameter, that transfers loads to the concrete by bond between the anchor and the adhesive, and bond between the adhesive and the concrete.

Anchor — a steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads to the concrete. Cast-in anchors include headed bolts, hooked bolts (J- or L-bolts) and headed studs. Post-installed anchors include expansion anchors, undercut anchors, and adhesive anchors. Steel elements for adhesive anchors include threaded rods, deformed reinforcing bars, or internally threaded steel sleeves with external deformations.

Anchor group — a number of similar anchors having approximately equal effective embedment depths with spacing s between adjacent anchors such that the projected areas overlap. See Clause D.4.1.2..

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.4A).

Anchor reinforcement — reinforcement used to transfer the full design load from the anchors into the structural member. See Clauses D.6.2.9 and D.7.2.9 and Figures D.10, D.17A, and D.17B.

Attachment — the structural assembly, external to the surface of the concrete, that transmits loads to or receives loads from the anchor.

Brittle steel element — an element with a tensile test elongation of less than 14% or reduction in area less than 30%, or both.

Cast-in anchor — a headed bolt, headed stud, 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.4A, D.4B and D.5).

Concrete pryout strength — the strength corresponding to formation of a concrete spall behind a short, stiff anchor, displaced in the direction opposite to the applied shear force (see Figure D.4B).

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% and a reduction in area of at least 30%. A steel element meeting the requirements of CSA G40.21 or ASTM A307 shall be considered as a ductile steel element. Deformed reinforcing bars meeting the requirements of CSA G30.18, shall be considered as ductile steel elements.

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 or from the surrounding concrete. The effective embedment depth will normally be the depth of the concrete 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 to and from the concrete by direct bearing, friction, or both. Expansion anchors may 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 either 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/AWS D1.1 and is affixed to a plate or similar steel attachment by stud arc welding before casting. The underside of the plate or steel attachment is assumed to be cast flush with the concrete surface.

Hooked bolt — a cast-in anchor anchored mainly by bearing of the 90° bend (L-bolt) or 180° bend (J-bolt) against the concrete at its embedded end and having a minimum e_h of $3d_a$.

Horizontal or upwardly inclined anchor — an anchor installed in a hole drilled horizontally or in a hole drilled at any orientation above horizontal. See Figure D.2.

Manufacturer's Printed Installation Instructions (MPII) — published instructions for the correct installation of the anchor under all covered installation conditions as supplied in the product packaging.

Post-installed anchor — an anchor installed in hardened concrete. Expansion, undercut and adhesive anchors are examples of post-installed anchors.

Projected area — the area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface. See Clauses D.6.2.1 and D.7.2.1.

Projected influence area — the rectilinear area on the free surface of the concrete member that is used to calculate the bond strength of adhesive anchors. See Clause D.6.5.1.

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 Figure D.4A.

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.

Stretch length — length of anchor, extending beyond concrete in which it is anchored, subject to full tensile load applied to anchor, and for which cross-sectional area is minimum and constant. See Figure D.3.

Supplementary reinforcement — reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load from the anchors into the 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_{brg} = bearing area of the head of stud anchor bolt or headed deformed bar, mm^2

A_{Na} = projected influence area of an adhesive anchor or group of adhesive anchors, for calculation of bond strength in tension, mm^2 (see Clause D.6.5.1 and Figure D.11)

A_{Nao} = projected influence area of a single adhesive anchor, for calculation of bond strength in tension if not limited by edge distance or spacing, mm^2. See Clause D.6.5.1 (see Figure D.11)

A_{Nc} = projected concrete failure area of a single anchor, or a group of anchors, for calculation of resistance in tension, mm^2, as specified in Clause D.6.2.1 (see Figure D.7)

A_{Nco} = projected concrete failure area of a single anchor, for calculation of resistance in tension, when not limited by edge distance or spacing, mm^2, as specified in Clause D.6.2.1 (see Figure D.6)

$A_{se,N}$ = effective cross-sectional area of anchor in tension, mm^2

$A_{se,V}$ = effective cross-sectional area of anchor in shear, mm^2

A_{Vc} = projected concrete failure area of a single anchor or group of anchors, for calculation of resistance in shear, mm^2, as defined in Clause D.7.2.1 (see Figure D.13)

A_{Vco} = projected concrete failure area of one anchor, for calculation of resistance in shear, when not limited by corner influences, spacing, or member thickness, mm^2, as specified in Clause D.7.2.1 (see Figure D.12)

c_{ac} = critical edge distance required to develop the basic resistance as controlled by concrete breakout or bond of a post-installed anchor in tension in uncracked concrete without supplementary reinforcement to control splitting, mm, as specified in Clause D.9.7

$c_{a,max}$ = maximum distance from centre of an anchor shaft to the edge of concrete, mm

$c_{a,min}$ = minimum distance from centre of an anchor shaft to the edge of concrete, mm

c_{a1} = distance from the centre of an anchor shaft to the edge of concrete in one direction, mm. If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance. Where anchors subjected to shear are located in narrow sections of limited thickness, see Clause D.7.2.4 (see Figures D.7 and D.12)

c'_{a1} = limiting value of c_{a1} when anchors are located less than $1.5h_{ef}$ from three or more edges (see Figure D.15)

c_{a2} = distance from centre of an anchor shaft to the edge of concrete in the direction orthogonal to c_{a1} (see Figure D.7)

c_{Na} = projected distance from centre of an anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor, mm (see Clause D.6.5.1)

d_a = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, mm (see Clause D.9.4)

d'_a = value substituted for d_a when an oversized anchor is used, mm (see Clause D.9.5)

e_h = distance from the inner surface of the shaft of a J-bolt or L-bolt to the outer tip of the J-bolt or L-bolt

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. e'_N is always positive (see Figure D.9)

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.16)

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_{uta} = specified tensile strength of anchor steel
f_{ya} = specified yield strength of anchor steel
F_a = acceleration-based site coefficient, as specified in the *National Building Code of Canada*
h_a = thickness of member in which an anchor is anchored, measured parallel to anchor axis
h_{ef} = effective anchor embedment depth, mm (see Figure D.1). Where anchors subject to tension are close to three or more edges, see Clause D.6.2.3.
h'_{ef} = limiting value of h_{ef} when anchors are located less than 1.5 h_{ef} from three or more edges (see Figure D.8)
I_E = earthquake importance factor of the structure, as specified in the *National Building Code of Canada*
k_c = coefficient for factored concrete breakout resistance in tension
k_{cp} = coefficient for pryout resistance
k_{05} = coefficient associated with the 5 percent fractile.
ℓ_e = load-bearing length of anchor for shear, not to exceed $8d_a$
n = number of anchors in a group
N_{agr} = factored bond resistance in tension of a group of adhesive anchors, see Clause D.6.5.1
N_{ar} = factored bond resistance in tension of a single adhesive anchor, see Clause D.6.5.1
N_{bar} = factored bond resistance of a single adhesive anchor in tension in cracked concrete, see Clause D.6.5.2
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, see Clause D.6.3.1
N_f = factored tensile load
N_{fa} = factored tensile load on an anchor or individual anchor in a group of anchors
$N_{fa,g}$ = total factored tensile load applied to an anchor group
$N_{fa,i}$ = factored tensile load applied to the most highly stressed anchor in the group of anchors
$N_{fa,s}$ = factored sustained tension 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_{sar} = factored resistance of a single anchor or individual anchor in a group of anchors in tension as governed by the steel resistance, as specified in Clauses D.6.1.1 and D.6.1.2
N_{sbgr} = factored side-face blowout resistance of a group of anchors, as specified in Clause D.6.4.2
N_{sbr} = factored side-face blowout resistance of a single anchor, as specified in Clause D.6.4.1
R = resistance modification factor, as specified in Clause D.5.3
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_y = factor applied to f_{ya} to estimate the probable yield stress, as specified in Clause D.4.3.5.3

s = anchor centre-to-centre spacing

s_s = sample standard deviation, MPa

$S_a(0.2)$ = 5% 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_{fa} = factored shear force applied to a single anchor or group of anchors

$V_{fa,g}$ = factored shear force applied to anchor group

$V_{fa,i}$ = factored shear force applied to the most highly stressed anchor in a group of anchors

V_r = factored shear resistance

V_{sar} = factored resistance in shear of a single anchor or individual anchor in a group of anchors as governed by the steel resistance, as specified in Clauses D.7.1.1 and D.7.1.2

λ_a = factor to account for low-density concrete in certain concrete anchorage applications

ϕ_c = material resistance factor for concrete

ϕ_s = steel embedment material resistance factor for reinforcement

τ_{cr} = characteristic bond stress of adhesive anchor in cracked concrete, MPa (see Clause D.6.5.2)

τ_{uncr} = characteristic bond stress of adhesive anchor in uncracked concrete, MPa (see Clause D.6.5.2)

$\psi_{c,N}$ = modification factor for anchor resistance in tension based on presence or absence of cracks, as specified in Clause D.6.2.6

$\psi_{c,P}$ = modification factor for pullout resistance of anchors based on presence or absence of cracks as specified in Clause D.6.3.6

$\psi_{c,V}$ = modification factor for resistance in shear of anchors based on presence or absence of cracks as specified in Clause D.7.2.7

$\psi_{cp,N}$ = factor used to modify the tensile strength of post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for splitting tensile stresses due to installation (see Clause D.6.2.7)

$\psi_{cp,Na}$ = factor used to modify tensile strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation (see Clause D.6.2.7)

$\psi_{ec,N}$ = modification factor for resistance in tension to account for anchor groups loaded eccentrically, as specified in Clause D.6.2.4

$\psi_{ec,Na}$ = factor used to modify tensile strength of adhesive anchors based on eccentricity of applied loads (see Clause D.6.5.5)

$\psi_{ec,V}$ = modification factor for resistance in shear to account for anchor groups loaded eccentrically, as specified in Clause D.7.2.5

$\psi_{ed,N}$ = factor used to modify tensile strength of anchors based on proximity to edges of concrete member (see Clause D.6.2.5)

$\psi_{ed,Na}$ = factor used to modify tensile strength of adhesive anchors based on proximity to edges of concrete members (see Clause D.6.5.4)

$\psi_{ed,V}$ = modification factor for resistance in shear based on proximity to edges of concrete members, as specified in Clause D.7.2.6

$\psi_{h,V}$ = factor used to modify shear strength of anchors located in concrete members with $h_a < 1.5c_{a1}$, as specified in Clause D.7.2.8.

D.4 General requirements

D.4.1 Analysis

D.4.1.1

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.4.1.2

Anchor group effects shall be considered wherever two or more anchors have spacing less than the critical spacing as follows:

Failure mode under investigation	Critical spacing
Concrete breakout strength in tension	$3h_{ef}$
Bond strength in tension	$2c_{Na}$
Concrete breakout in shear	$3c_{a1}$

Only those anchors susceptible to the particular failure mode under investigation shall be included in the group.

D.4.2 Load combinations

Anchors shall be designed for 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.4.3.8 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.4.3.8 shall apply.

D.4.3.4

Post-installed anchors shall be qualified for earthquake loading in accordance with ACI 355.2 or ACI 355.4. The factored pullout resistance, N_{pr}, and factored steel resistance of the anchor in shear, V_{sar}, of expansion and undercut anchors shall be based on the results of the ACI 355.2 simulated seismic tests. For adhesive anchors, the steel factored resistance in shear V_{sar} and the characteristic bond stresses τ_{uncr} and τ_{cr} shall be based on results of the ACI 355.4 simulated seismic tests.

D.4.3.5 Requirements for tensile loading

D.4.3.5.1

Where the tension component of the seismic force applied to a single anchor or group of anchors is equal to or less than 20% of the total factored anchor tensile force associated with the same load combination, a single anchor or group of anchors may be designed to satisfy Clauses D.6 and D.5.1.1 and Table D.1.

D.4.3.5.2

Where the tensile component of the seismic force applied to anchors exceeds 20% of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with Clause D.4.3.5.3. The anchor design tensile resistance shall be determined using Clause D.4.3.5.4.

D.4.3.5.3

Anchors and their attachments shall be designed using one of the following options:

a) For single anchors, the nominal concrete-governed resistance shall be greater than the probable steel resistance of the anchor. For anchor groups the ratio of the tensile load on the most highly stressed anchor to the probable steel strength of the anchor shall be equal to or greater than the ratio of the tensile load on the tension-loaded anchors to the nominal concrete-governed strength of those anchors:
   i) In each case,
      1) The probable steel resistance shall be the gross section strength of the steel anchor determined using a probable yield stress $R_y f_{ya}$ where R_y is taken as 1.1 and ϕ_s is taken as 1.0.
      2) The nominal concrete governed strength shall be taken as strength considering pullout, side-face blowout, concrete breakout, and bond strength as applicable with ϕ_c taken as 1.0. For consideration of pullout in groups, the ratio shall be calculated for the most highly stressed anchor.
   ii) In addition, the following shall be satisfied:
      1) Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least eight anchor diameters unless otherwise determined by analysis.
      2) Where anchors are subject to load reversals, the anchor shall be protected against buckling.
      3) Where connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall not be less than 1.3 unless the threaded portions are upset. The upset portions shall not be included in the stretch length. See Figure D.3.
      4) Deformed reinforcing bars used as ductile steel elements to resist earthquake effects shall be limited to CSA G30.18 weldable grade, satisfying the requirements of Clause 21.

b) The anchor or group of anchors shall be designed for the maximum probable tension force that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects for the attachment. The anchor design tensile strength resistance shall be calculated from Clause D.4.3.5.4.
c) The anchor or group of anchors shall be designed for the maximum tension force that can be transmitted to the anchors by a non-yielding attachment. The anchor design tensile strength shall be calculated from D.4.3.5.4.
d) The anchor or group of anchors shall be designed for the maximum tension force obtained from design load combinations that include earthquake effects, with loads calculated using R_dR_o equal to 1.3 or as specified in Clause 4.1.8.18 of the *National Building Code of Canada* (*NBCC*). The anchor design tensile strength shall satisfy the tensile strength requirements of Clause D.4.3.5.4.

D.4.3.5.4

The anchor design tensile resistance for resisting earthquake seismic forces shall be determined from consideration of the following for the failure modes given in Table D.1 assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked:
a) N_{sra} for a single anchor or for the most highly stressed individual anchor in a group of anchors;
b) $0.75N_{cbr}$ or $0.75N_{cbgr}$, except that N_{cbr} or N_{cbgr} need not be calculated where anchor reinforcement satisfying D.6.2.9 is provided;
c) $0.75N_{cpr}$ for a single anchor or for the most highly stressed individual anchor in a group of anchors;
d) $0.75N_{sbr}$ or $0.75N_{sbgr}$; and
e) $0.75N_{ar}$ or $0.75N_{agr}$.

D.4.3.5.5

Where anchor reinforcement is provided in accordance with Clause D.6.2.9, no reduction in design tension strength beyond that specified in Clause D.6.2.9 shall be required.

D.4.3.6 Requirements for shear loading

D.4.3.6.1

Where the shear component of the strength-level earthquake seismic force (factored earthquake load) applied to the anchor or group of anchors is equal to or less than 20% of the total factored anchor shear force associated with the same load combination, the anchor or group of anchors may be designed to satisfy Clause D.7 and the shear strength requirements of Clause D.5.1.2.

D.4.3.6.2

Where the shear component of the seismic force applied to anchors exceeds 20% of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with Clause D.4.3.6.3. The anchor design shear resistance for resisting earthquake forces shall be determined in accordance with Clause D.7.

D.4.3.6.3

Anchors and their attachments shall be designed using one of the following options:
a) The anchor or group of anchors shall be designed for the maximum probable shear force that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects in the attachment.

b) The anchor or group of anchors shall be designed for the maximum probable shear force that can be transmitted to the anchors by a non-yielding attachment.
c) The anchor or group of anchors shall be designed for the maximum shear force obtained from design load combinations that include earthquake effects, with loads calculated using R_dR_o equal to 1.3 or as specified in Clause 4.1.8.18 of the *NBCC*. The anchor design shear resistance shall satisfy the shear resistance requirements of Clause D.5.1.2.

D.4.3.6.4
Where anchor reinforcement is provided in accordance with Clause D.7.2.9, no reduction in design shear strength beyond that specified in Clause D.7.2.9 shall be required.

D.4.3.7
Single anchors or groups of anchors that are subjected to both tension and shear forces shall be designed to satisfy the requirements of Clause D.8, with the anchor design tensile strength calculated from Clause D.4.3.5.4.

D.4.3.8
Anchor reinforcement used in structures where $R_d > 2.5$ shall be deformed reinforcement satisfying the requirements of Clause 21.2.7.1.

D.4.4
Adhesive anchors installed horizontally or upwardly inclined shall be qualified in accordance with ACI 355.4 requirements for sensitivity to installation direction.

D.4.5
For adhesive anchors subjected to sustained tension loading, Clause D.5.1.3 shall be satisfied. For groups of adhesive anchors, Equation D.1 shall be satisfied for the anchor that resists the highest sustained tension load. Installer certification and inspection requirements for horizontal and upwardly-inclined adhesive anchors subjected to sustained tension loading shall be in accordance with Clauses D.10.2.2 through D.10.2.4.

D.4.6
Modification factor λ_a for lightweight concrete in this annex shall be taken as

Cast-in and undercut anchor concrete failure	$1.0\,\lambda$
Expansion and adhesive anchor concrete failure	$0.8\,\lambda$
Adhesive anchor bond failure per Equation D.24	$0.6\,\lambda$

where λ is determined in accordance with Clause 8.6.5. An alternate value of λ_a may be used where tests have been performed and evaluated in accordance with ACI 355.2 or ACI 355.4.

D.4.7 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 (see Figures D.4A and D.4B):

a) steel strength of anchor in tension (Clause D.6.1);
b) concrete breakout resistance of anchor in tension (Clause D.6.2);
c) pullout resistance of cast-in, post-installed expansion or undercut anchor in tension (Clause D.6.3);
d) concrete side-face blowout strength of headed anchor in tension (Clause D.6.4);
e) bond strength of adhesive anchor in tension (Clause D.6.5);
f) steel strength of anchor in shear (Clause D.7.1);
g) concrete breakout strength of anchor in shear (Clause D.7.2); and
h) concrete pryout resistance of anchor in shear (Clause D.7.3).

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to precluding splitting failure as required by Clause D.9.

D.5.1.2

The design of anchors shall be in accordance with Table D.1. In addition, the design of anchors shall satisfy Clause D.4.3 for earthquake loading and Clause D.5.1.3 for adhesive anchors subject to sustained tensile loading.

Table D.1
Required resistance of anchors, except as noted in Clause D.4.3
(See Clause D.5.1.2.)

Failure mode	Single anchor	Anchor group*	
		Individual anchor in a group	**Anchors as a group**
Steel strength in tension (D.6.1)	$N_{sar} \geq N_{fa}$	$N_{sar} \geq N_{fa,i}$	
Concrete breakout strength in tension (D.6.2)	$N_{cbr} \geq N_{fa}$		$N_{cbgr} \geq N_{fa,g}$
Pullout strength in tension (D.6.3)	$N_{pr} \geq N_{fa}$	$N_{pr} \geq N_{fa,i}$	
Concrete side-face blowout strength in tension (D.6.4)	$N_{sbr} \geq N_{fa}$		$N_{sbgr} \geq N_{fa,g}$
Bond strength of adhesive anchor in tension (D.6.5)	$N_{ar} \geq N_{fa}$		$N_{agr} \geq N_{fa,g}$
Steel strength in shear (D.7.1)	$V_{sar} \geq V_{fa}$	$V_{sar} \geq V_{fa,i}$	

(Continued)

Table D.1 (Concluded)

Failure mode	Single anchor	Anchor group*	
		Individual anchor in a group	Anchors as a group
Concrete breakout strength in shear (D.7.2)	$V_{cbr} \geq V_{fa}$		$V_{cbgr} \geq V_{fa,g}$
Concrete pryout strength in shear (D.7.3)	$V_{cpr} \geq V_{fa}$		$V_{cpr} \geq V_{fa,g}$

** Required resistance of anchors in groups shall account for all applicable failure modes for individual anchors and for the group.*

D.5.1.3
For the design of adhesive anchors to resist sustained tension loads, in addition to Clause D.5.1.2,

$$0.55\, N_{bar} \geq N_{fa,s}$$ **Equation D.1**

where N_{bar} is determined in accordance with Clause D.6.5.2.

D.5.1.4
When both N_{fa} and V_{fa} are present, interaction effects shall be considered using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by Clause D.8.

D.5.2 Calculating anchor resistance

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. See Figure D.5.

D.5.2.2
The effect of reinforcement provided to restrain the concrete breakout may be included in the design models specified in Clause D.5.2.1. Where anchor reinforcement is provided in accordance with Clauses D.6.2.9 and D.7.2.9, calculation of the concrete breakout strength in accordance with Clauses D.6.2 and D.7.2 is not required.

D.5.2.3
For anchors with diameters not exceeding 100 mm, the concrete breakout resistance shall be considered satisfied by the design procedure specified in Clauses D.6.2 and D.7.2.

D.5.2.4

For adhesive anchors with embedment depths $4d_a \le h_{ef} \le 20d_a$, the bond strength requirements shall be considered satisfied by the design procedure of Clause D.6.5.

D.5.3

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, side face 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 or ACI 355.4		
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	0.75

** 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 except where pullout or pryout resistance governs. 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 in tension, N_{sar}, shall not exceed

$$N_{sar} = A_{se,N}\phi_s f_{uta}R \quad \textbf{Equation D.2}$$

where $A_{se,N}$ is the effective cross-sectional area of an anchor in tension, mm², f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ or 860 MPa and R is as specified in Clause D.5.3.

D.6.2 Concrete breakout resistance of anchor in tension

D.6.2.1

The factored concrete breakout resistance, N_{cbr} of a single anchor or N_{cbgr} for a group of anchors, shall not exceed

a) for a single anchor:

$$N_{cbr} = \frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_{br} \quad \textbf{Equation D.3}$$

b) for an anchor group:

$$N_{cbgr} = \frac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_{br} \quad \textbf{Equation D.4}$$

In these equations factors, $\Psi_{ec,N}$, $\Psi_{ed,N}$, $\Psi_{c,N}$, and$\Psi_{cp,N}$ are defined in Clauses D.6.2.4, D.6.2.5, D.6.2.6, and D.6.2.7 respectively. N_{br} is the factored concrete breakout resistance value for a single anchor in tension in cracked concrete. A_{Nc} is the projected concrete failure area of a single 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.7). A_{Nc} shall not exceed nA_{Nco}, where n is the number of tensioned anchors in the group. A_{Nco} is the projected concrete failure area of a single anchor with an edge distance equal to or greater than $1.5h_{ef}$ (see Figure D.6).

$$A_{Nco} = 9{h_{ef}}^2 \quad \textbf{Equation D.5}$$

D.6.2.2

The factored concrete breakout resistance of a single anchor in tension in cracked concrete, N_{br}, shall not exceed

$$N_{br} = k_c\phi_c\lambda_a\sqrt{f'_c}\,h_{ef}^{1.5}R \quad \textbf{Equation D.6}$$

where

k_c = 10 for cast-in headed studs, headed bolts, and hooked bolts

= 7.0 for post-installed anchors

R = as specified in Clause D.5.3

The k_c factor for post-installed anchors may be increased above 7.0 in accordance with ACI 355.2 or ACI 355.4 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, N_{br}, shall not exceed

$$N_{br} = 3.9\phi_c\lambda_a\sqrt{f'_c}\,h_{ef}^{5/3}R \quad \textbf{Equation D.7}$$

where R is as specified in Clause D.5.3.

D.6.2.3

Where anchors are located less than 1.5 h_{ef} from three or more edges the value of h_{ef} used for the calculation of A_{Nc} in accordance with Clause D.6.2.1, as well as in Equations D.3 to , D.10, and D.11 shall be the larger of $c_{a,max}$/1.5 and s/3 where s is the maximum spacing between anchors within the group (see Figure D.7).

D.6.2.4

The modification factor for anchor groups loaded eccentrically in tension, $\Psi_{ec,N}$, shall be computed as

$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)}$$ **Equation D.8**

but $\Psi_{ec,N}$ shall not be taken greater than 1.0 (see Figure D.9).

This equation shall be valid for

$$e'_N \le \frac{s}{2}$$ **Equation D.9**

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.8 and for the calculation of N_{cbgr} in accordance with Equation D.4.

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.4.

D.6.2.5

The modification factor for edge effects for single anchors or anchor groups loaded in tension, $\Psi_{ed,N}$, shall be computed as

If $c_{a,min} \ge 1.5\ h_{ef}$, then

$$\psi_{ed,N} = 1.0$$ **Equation D.10**

If $c_{a,min} < 1.5\ h_{ef}$, then

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,\min}}{1.5h_{ef}}$$ **Equation D.11**

D.6.2.6

For anchors 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 anchors; and

$\psi_{c,N}$ = 1.4 for post-installed anchors when the value of k_c = 7.0 is used in Equation D.6.

When the value of k_c used in Equation D.6 is taken from an ACI 355.2 or ACI 355.4 product evaluation report for post-installed anchors qualified for use in both cracked and uncracked concrete, the value of both k_c and $\psi_{c,N}$ shall be based on ACI 355.2 or ACI 355.4 product evaluation report.

Where k_c used in Equation D.6 is taken from an ACI 355.2 or ACI 355.4 product evaluation report for post-installed anchors approved for use in uncracked concrete, $\psi_{c,N}$ shall be taken as 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. Post-installed anchors shall be qualified for use in cracked concrete in accordance with ACI 355.2 or ACI 355.4. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with Clause 10.6.1, or equivalent crack control shall be provided by confining reinforcement.

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, $\Psi_{cp,N}$, shall be computed as follows using the critical distance c_{ac} as defined in Clause D.9.7.

If $c_{a,min} \geq c_{ac}$, then

$$\psi_{cp,N} = 1.0 \qquad \textbf{Equation D.12}$$

If $c_{a,min} < c_{ac}$, then

$$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}} \qquad \textbf{Equation D.13}$$

but $\Psi_{cp,N}$ determined from Equation D.13 shall not be taken less than $1.5h_{ef}/c_{ac}$, where the critical distance, c_{ac}, is as specified in Clause D.9.7.

For all other cases, including cast-in anchors, $\Psi_{cp,N}$, shall be taken as 1.0.

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.6.2.9

Where anchor reinforcement is developed in accordance with Clause 12 on both sides of the breakout surface, the design strength of the anchor reinforcement may be used instead of the concrete breakout strength in determining N_r. See Figure D.10. The anchor reinforcement capacity shall be taken as:

$$N_r = \phi_s A_s f_y R \qquad \textbf{Equation D.14}$$

where R = 0.85.

D.6.3 Pullout resistance of cast-in, post-installed expansion, and undercut anchors in tension

D.6.3.1

The factored pullout resistance of a single cast-in, post-installed expansion, and post-installed undercut anchor in tension, N_{cpr}, shall not exceed

$$N_{cpr} = \psi_{c,P} N_{pr}$$ **Equation D.15**

where $\Psi_{c,P}$ is defined in Clause D.6.3.6.

D.6.3.2

For post-installed expansion and undercut anchors, the values of N_{pr} shall be based on the 5% fractile of results of tests performed and evaluated in accordance with ACI 355.2. The factored pullout resistance, N_{pr}, for use in Equation D.15 shall be obtained by multiplying the resulting 5% fractile test value by ϕ_c before being applied to Equation D.15. It is not permissible to calculate the pullout strength, N_{pr}, in tension for such anchors.

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 procedures but without the benefit of friction may be used. When this procedure is used, the factored pullout resistance, N_{pr}, for use in Equation D.15 shall be obtained by multiplying the resulting 5% fractile test value by ϕ_c before being applied to Equation D.15.

D.6.3.4

The factored pullout resistance in tension of a single headed stud or headed bolt, N_{pr}, for use in Equation D.15 shall not exceed

$$N_{pr} = 8 A_{brg} \phi_c f'_c R$$ **Equation D.16**

D.6.3.5

The factored pullout resistance in tension of a single J-bolt or L-bolt, N_{pr}, for use in Equation D.15 shall not exceed

$$N_{pr} = 0.9 \phi_c f'_c e_h d_a R$$ **Equation D.17**

where $3d_a \le e_h \le 4.5d_a$

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 ($h_{ef} > 2.5c_{a1}$), the factored side-face blowout resistance, N_{sbr}, shall not exceed

$$N_{sbr} = 13.3 c_{a1} \sqrt{A_{brg}} \phi_c \lambda_a \sqrt{f'_c} R$$ **Equation D.18**

If c_{a2} for the single headed anchor is less than $3c_{a1}$, the value of N_{sbr} shall be modified by multiplying it by the factor $(1 + c_{a2}/c_{a1})/4$, where $1.0 \le c_{a2}/c_{a1} \le 3.0$.

D.6.4.2

For multiple-headed anchors with deep embedment close to an edge ($h_{ef} > 2.5c_{a1}$) and spacing between anchors less than $6c_{a1}$, the factored resistance of the anchor group for a side-face blowout failure, N_{sbgr}, shall not exceed

$$N_{sbgr} = \left(1 + \frac{s}{6c_{a1}}\right)N_{sbr}$$ **Equation D.19**

where s = 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.6.5 Bond strength of adhesive anchor in tension

D.6.5.1

The factored bond resistance in tension, N_{ar}, of a single adhesive anchor or N_{agr} of a group of adhesive anchors, shall not exceed

a) For a single adhesive anchor:

$$N_{ar} = \left(\frac{A_{Na}}{A_{Nao}}\right)\psi_{ed,Na}\psi_{cp,Na}N_{bar}$$ **Equation D.20**

b) For a group of adhesive anchors:

$$N_{agr} = \left(\frac{A_{Na}}{A_{Nao}}\right)\psi_{ec,Na}\psi_{ed,Na}\psi_{cp,Na}N_{bar}$$ **Equation D.21**

Factors $\Psi_{ec,Na}$, $\Psi_{ed,Na}$, and $\Psi_{cp,Na}$ are defined in Clauses D.6.5.3, D.6.5.4, and D.6.5.5, respectively. A_{Na} is the projected influence area of a single adhesive anchor or group of adhesive anchors that shall be approximated as a rectilinear area that projects outward a distance, c_{Na}, from the centerline of the adhesive anchor or, in the case of a group of adhesive anchors, from a line through a row of adjacent adhesive anchors. A_{Na} shall not exceed nA_{Nao}, where n is the number of adhesive anchors in the group that resist tension loads. A_{Nao} is the projected influence area of a single adhesive anchor with an edge distance equal to or greater than c_{Na} (see Figure D.11):

$$A_{Nao} = (2c_{Na})^2$$ **Equation D.22**

where

$$c_{Na} = 10d_a\sqrt{\frac{\tau_{uncr}}{7.60}}$$ **Equation D.23**

and constant 7.60 carries the unit of N/mm^2.

D.6.5.2

The factored bond resistance of a single adhesive anchor in tension in cracked concrete, N_{bar}, shall not exceed

$$N_{bar} = \lambda_a\phi_c\tau_{cr}\pi d_a h_{ef} R$$ **Equation D.24**

The characteristic bond stress, τ_{cr}, shall be taken as the 5% fractile of results of tests performed and evaluated according to ACI 355.4.

Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ACI 355.4.

For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} may be used in place of τ_{cr} in Equation D.24 and shall be taken as the 5% fractile of results of tests performed and evaluated in accordance with ACI 355.4.

In lieu of using test results, the minimum characteristic bond stress values in Table D.2 may be used, provided Items a) to e) are satisfied:

a) anchors shall meet the requirements of ACI 355.4;
b) anchors shall be installed in holes drilled with a rotary impact drill or rock drill;
c) concrete at time of anchor installation shall have a minimum compressive strength of 17 MPa;
d) concrete at time of anchor installation shall have a minimum age of 21 days; and
e) concrete temperature at time of anchor installation shall be at least 10 °C.

Table D.2
Minimum characteristic bond stresses*†
(See Clause D.6.5.2.)

Installation and service environment	Moisture content of concrete at time of anchor installation	Peak in-service temperature of concrete, °C	τ_{cr}, MPa	τ_{uncr}, MPa
Outdoor	Dry to fully saturated	80	1.40	4.50
Indoor	Dry	43	2.10	6.90

* *Where anchor design includes sustained tension loading, multiply values of τ_{cr} and τ_{uncr} by 0.4.*
† *Where anchor design includes earthquake loads for structures where, ($I_E F_a S_a(0.2) \geq 0.35$) multiply values of τ_{cr} by 0.8 and τ_{uncr} by 0.4.*

D.6.5.3

The modification factor for adhesive anchor groups loaded eccentrically in tension, $\Psi_{ec,Na}$, shall be computed as

$$\psi_{ec,Na} = \frac{1}{\left(1 + \frac{e'_N}{c_{Na}}\right)} \qquad \textbf{Equation D.25}$$

but $\Psi_{ec,Na}$ shall not be taken greater than 1.0.

If the loading on an adhesive anchor group is such that only some adhesive anchors are in tension, only those adhesive anchors that are in tension shall be considered when determining the eccentricity e'_N for use in Equation D.25 and for the calculation of N_{agr} in accordance with Equation D.21.

In the case where eccentric loading exists about two orthogonal axes, the modification factor, $\Psi_{ec,Na}$, shall be computed for each axis individually and the product of these factors used as $\Psi_{ec,Na}$ in Equation D.21.

D.6.5.4

The modification factor for edge effects for single adhesive anchors or adhesive anchor groups loaded in tension, $\Psi_{ed,Na}$, shall be computed as

If $c_{a,min} \geq c_{Na}$

then

$$\psi_{ed,Na} = 1.0$$ **Equation D.26**

If $c_{a,min} < c_{Na}$

then

$$\psi_{ed,Na} = 0.7 + 0.3\left(\frac{c_{a,\min}}{c_{Na}}\right)$$ **Equation D.27**

D.6.5.5

The modification factor for adhesive anchors designed for uncracked concrete in accordance with Clause D.6.5.2 without supplementary reinforcement to control splitting, $\Psi_{cp,Na}$, shall be computed as

If $c_{a,min} \geq c_{ac}$

then

$$\psi_{cp,Na} = 1.0$$ **Equation D.28**

If $c_{a,min} < c_{ac}$

then

$$\psi_{cp,Na} = C_{a,\min} / c_{ac}$$ **Equation D.29**

but $\psi_{cp,Na}$ determined from Equation D.29 shall not be taken less than c_{Na}/c_{ac} where the critical edge distance, c_{ac}, is defined in Clause D.9.7. For all other cases $\psi_{cp,Na}$ shall be taken as 1.0.

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_{sar}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor. Where concrete breakout is a potential failure mode, the required steel shear strength shall be consistent with the assumed breakout surface.

D.7.1.2

The factored resistance of an anchor in shear shall not exceed the following Items a) to c):

a) for cast-in headed stud anchors:

$$V_{sar} = A_{se,V}\, \phi_s\, f_{uta} R$$ **Equation D.30**

where $A_{se,V}$ is the effective cross-sectional area of an anchor in shear, mm² and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ or 860 MPa and R is as specified in Clause D.5.3;

b) for cast-in headed bolts, hooked bolt anchors, and post-installed anchors where sleeves do not extend through the shear plane:

$$V_{sar} = A_{se,V}\ \phi_s\, 0.6\, f_{uta} R \qquad \textbf{Equation D.31}$$

where $A_{se,V}$ is the effective cross-sectional area of a single anchor in shear, mm² and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ or 860 MPa; and R is as specified in Clause D.5.3; and

c) for post-installed anchors with sleeves extending through the shear plane, V_{sar} shall be based on the 5% fractile of results of tests performed and evaluated in accordance with ACI 355.2. When this procedure is used, V_{sar} shall be obtained by multiplying the resulting 5% fractile test value by $\phi_s R$. Alternatively, Equation D.31 may be used.

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 multiplied by a 0.80 factor.

D.7.2 Concrete breakout resistance of anchor in shear

D.7.2.1

The factored concrete breakout resistance in shear V_{cbr} of a single anchor or V_{cbrg} of a group of anchors shall not exceed the following (see Figure D.4B):

a) For shear force perpendicular to the edge on a single anchor:

$$V_{cbr} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,v} V_{br} \qquad \textbf{Equation D.32}$$

b) For shear force perpendicular to the edge on an anchor group:

$$V_{cbgr} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{br} \qquad \textbf{Equation D.33}$$

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.32 or D.33, respectively, with the shear force assumed to act perpendicular to the free edge and with $\psi_{ed,V}$ taken to be equal to 1.0 (see Figures D.13 and D.14).

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.14).

Factors $\psi_{ec,V}$, $\psi_{ed,V}$, $\psi_{c,V}$, and $\psi_{h,V}$ are defined in Clauses D.7.2.5, D.7.2.6, D.7.2.7, and D.7.2.8, respectively. V_{br} is the factored concrete breakout resistance value for a single anchor. A_{Vc} 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_{a1} shall be taken as the distance from the edge to this axis. A_{Vc} shall not exceed nA_{Vco}, where n is the number of anchors in the group. See Figure D.13.

A_{Vco} is the projected area for a single anchor in a deep member and with a distance from edges equal or greater than $1.5c_{a1}$ in the direction perpendicular to the shear force. This area, A_{Vco}, may be evaluated as the base of a half-pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$, as follows (see Figure D.12):

$$A_{Vco} = 4.5(c_{a1})^2 \qquad \textbf{Equation D.34}$$

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_{a1} 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. See Figure D.13.

D.7.2.2

The factored concrete breakout resistance in shear of a single anchor in cracked concrete, V_{br}, shall not exceed the smaller of Item a) or b):

a)

$$V_{br} = 0.58\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\,\phi_c\lambda_a\sqrt{f'_c}\,c_{a1}^{\,1.5}R \qquad \textbf{Equation D.35}$$

where

ℓ_e = the load-bearing length of the anchor for shear

ℓ_e = h_{ef} for anchors with a constant stiffness over the full length of embedded section, such as headed studs and post-installed anchors with one tubular shell over full length of the embedment depth,

ℓ_e = $2d_a$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve, and

$\ell_e \le 8d_a$ in all cases.

b)

$$V_{br} = 3.75\lambda_a\phi_c\sqrt{f'_c}\,(c_{a1})^{1.5}R \qquad \textbf{Equation D.36}$$

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, V_{br}, shall be the smaller of Equations D.36 and D.37:

$$V_{br} = 0.66\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\,\phi_c\lambda_a\sqrt{f'_c}\,c_{a1}^{\,1.5}R \qquad \textbf{Equation D.37}$$

where ℓ_e is defined in D.7.2.2 and R is specified in Clause D.5.3,

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) anchor spacing, s, is not less than 65 mm; and

c) supplementary reinforcement is provided at the corners if $c_{a2} \le 1.5h_{ef}$.

D.7.2.4

Where anchors are located in narrow sections of limited thickness such that both edge distances c_{a2} and thickness h_a are less than $1.5c_{a1}$, the value of c_{a1} used for the calculation of A_{Vc} in accordance with Clause D.7.2.1 as well as in Equation D.34 to D.38 and D.40 to D.42 shall not exceed the largest of

a) $c_{a2}/1.5$, where c_{a2} is the largest edge distance;

b) $h_a/1.5$; and

c) $s/3$, where s is the maximum spacing perpendicular to direction of shear between anchors within a group. See Figure D.15.

D.7.2.5

The modification factor for eccentrically-loaded anchor groups loaded eccentrically in shear, $\Psi_{ec,V}$, shall be computed as

$$\psi_{ec,V} = \frac{1}{1 + \frac{2e'_V}{3c_{a1}}}$$ **Equation D.38**

This equation shall be valid for

$$e'_V \leq \frac{s}{2}$$ **Equation D.39**

but $\Psi_{ec,V}$, shall not be taken greater than 1.0.

If the loading on an anchor group is such that only some anchors are loaded in shear in the same direction, only those anchors that are loaded in shear in the same direction shall be considered when determining the eccentricity of e'_V for use in Equation D.38 and for the calculation of V_{cbgr} in accordance with Equation D.33. See Figure D.16.

D.7.2.6

The modification factor for edge effect for a single anchor or group of anchors loaded in shear, $\Psi_{ed,V}$, shall be computed as follows using the smaller value of c_{a2}:

If $c_{a2} \geq 1.5\, c_{a1}$ then $\Psi_{ed,V} = 1.0$ **Equation D.40**

If $c_{a2} < 1.5\, c_{a1}$ then $\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right)$ **Equation D.41**

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 without supplementary reinforcement or with edge reinforcement smaller than a 15M bar	$\psi_{c,V} = 1.0$
For anchors in cracked concrete with reinforcement of a 15M bar or greater between the anchor and the edge	$\psi_{c,V} = 1.2$
For anchors in cracked concrete with reinforcement of a 15M bar or greater between the anchor and the edge and with the reinforcement enclosed within stirrups spaced not more than 100 mm apart	$\psi_{c,V} = 1.4$

D.7.2.8

The modification factor for anchors located in a concrete member where $h_a < 1.5c_{a1}$, $\psi_{h,V}$ shall be computed as

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}}$$ **Equation D.42**

but $\psi_{h,V}$ shall not be taken less than 1.0.

D.7.2.9

Where anchor reinforcement is developed in accordance with Clause 12 on both sides of the breakout surface or encloses the anchor and is developed beyond the breakout surface, the design strength of the anchor reinforcement may be used instead of the concrete breakout strength in determining V_r. See Figures D.17A and D.17B. The anchor reinforcement capacity shall be taken as

$$V_r = \phi_s A_s f_y R$$ **Equation D.43**

where R = 0.85.

D.7.3 Concrete pryout resistance of an anchor in shear

The factored pryout resistance, V_{cpr} for a single anchor or V_{cpgr}, for a group of anchors, shall not exceed

a) for a single anchor:

$$V_{cpr} = k_{cp} N_{cpr}$$ **Equation D.44**

For cast-in, expansion, and undercut anchors, N_{cpr} shall be taken as N_{cbr} determined from Equation D.3, and for adhesive anchors, N_{cpr} shall be the lesser of N_{ar} determined from Equation D.20 and N_{cbr} determined from Equation D.3.

b) for a group of anchors:

$$V_{cpgr} = k_{cp} N_{cpgr}$$ **Equation D.45**

For cast-in, expansion, and undercut anchors, N_{cpgr} shall be taken as N_{cbgr} determined from Equation D.4, and for adhesive anchors N_{cpgr} shall be the lesser of N_{agr} determined from Equation D.21 and N_{cbgr} determined from Equation D.4.

In Equations D.44 and D.45,

k_{cp} = 1.0 for $h_{ef} < 65$ mm

= 2.0 for $h_{ef} \geq 65$ mm

D.8 Interaction of tensile and shear forces

D.8.1

Unless determined in accordance with Clause D.5.1.4, 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 to D.8.4. The value of N_r and V_r shall be the required strengths as determined from Clause D.5.1.2 or D.4.3.

D.8.2

If $\frac{V_f}{V_r} \leq 0.2$ for the governing resistance in shear, the full resistance in tension may be used, as follows:

$N_r \geq N_f$ (see Figure D.18).

D.8.3

If $\frac{N_f}{N_r} \leq 0.2$, for the governing resistance in tension, the full resistance in shear may be used, as follows (see Figure D.18): $V_r \geq V_f$.

D.8.4

If $\frac{V_f}{V_r} > 0.2$ for the governing resistance in shear, and $\frac{N_f}{N_r} > 0.2$ for the governing resistance in tension, the following shall apply (see Figure D.18):

$$\frac{N_f}{N_r} + \frac{V_f}{V_r} \leq 1.2$$

Equation D.46

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 supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI 355.2 OR ACI 355.4 may be used.

D.9.2

Unless determined in accordance with Clause D.9.5, the minimum centre-to-centre spacing of anchors shall be $4d_a$ for cast-in anchors that will not be torqued and $6d_a$ 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 anchors that will not be torqued shall be based on the minimum cover requirements for reinforcement specified in Clause 7.9. For cast-in anchors that will be torqued, the minimum edge distances shall be $6d_a$.

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 OR ACI 355.4, and shall be not less than twice the nominal maximum aggregate size. In the absence of such product-specific ACI 355.2 or ACI 355.4 test information, the minimum edge distance shall not be less than the following:

Adhesive anchors	$6d_a$
Undercut anchors	$6d_a$
Torque-controlled anchors	$8d_a$
Displacement-controlled anchors	$10d_a$

D.9.5

For anchors where installation does not produce a splitting force and that will not be torqued, 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_a a smaller value d'_a 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 a diameter, d'_a.

D.9.6

Unless determined from tests in accordance with ACI 355.2, 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, h_a, and the member thickness less 100 mm.

D.9.7

Unless determined from tension tests in accordance with ACI 355.2 OR ACI 355.4, the critical edge distance, c_{ac}, shall not be taken less than the following:

Adhesive anchors	$2h_{ef}$
Undercut anchors	$2.5h_{ef}$
Torque-controlled expansion anchors	$4h_{ef}$
Displacement-controlled expansion anchors	$4h_{ef}$

D.9.8

Contract documents shall specify use of anchors with the minimum edge distance assumed in the design.

D.10 Installation and inspection of anchors

D.10.1

Anchors shall be installed by qualified personnel in accordance with the contract documents. The contract documents shall require installation of post-installed anchors in accordance with the manufacturer's printed installation instructions (MPII). Installation of adhesive anchors shall be performed by personnel trained to install adhesive anchors.

D.10.2

D.10.2.1

The level of inspection required varies by anchor category type for both mechanical and adhesive anchors. Adhesive anchors shall be subject to Clauses D.10.2.2 to D.10.2.4.

D.10.2.2

For adhesive anchors, the contract documents shall specify proof loading where required in accordance with ACI 355.4. For adhesive anchors, the contract documents shall specify either of the following as determined by ACI 355.4:

a) periodic special inspection; or
b) continuous special inspection with proof loading where required in accordance with ACI 355.4. The proof loading program shall comply with ACI 355.4.

The contract documents shall also specify all parameters associated with the characteristic bond stress used for the design in accordance with D.6.5 including minimum age of concrete, concrete temperature range, moisture condition of concrete at time of installation, type of lightweight concrete if applicable, and requirements for hole drilling and preparation.

D.10.2.3

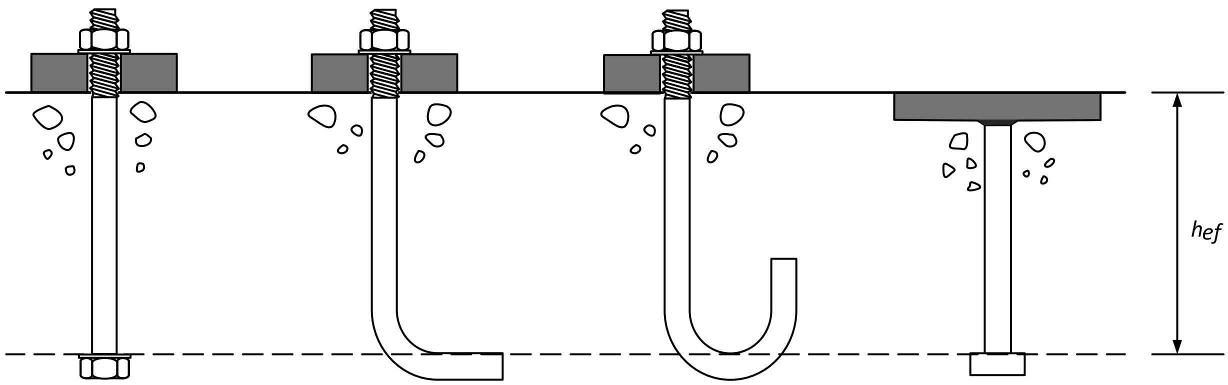
Installation of adhesive anchors horizontally or upwardly inclined to support sustained tension loads shall be performed by personnel certified by an applicable certification program.

Certification shall include written and performance tests in accordance with the ACI/CRSI Adhesive Anchor Installer Certification program, or equivalent.

D.10.2.4

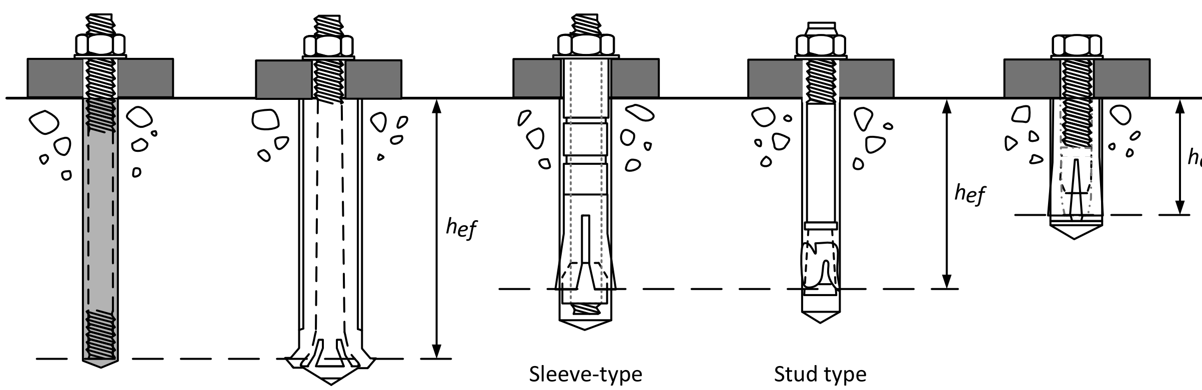
Adhesive anchors installed in horizontal and upwardly inclined orientations to resist sustained tension loads shall be continuously inspected during installation by an inspector specially approved for that purpose. The special inspector shall furnish a report to the licensed design professional and building official that the work covered by the report has been performed and that the materials used and the installation procedures used conform with the approved contract documents and the MPII.

Figure D.1
Types of anchors
(See Clauses D.1.2, D.2, and D.3.)



a) Hex head bolt with washer
b) L-bolt
c) J-bolt
d) Welded headed stud

a) Cast-in anchors



a) Adhesive anchor
b) Undercut anchor
c) Torque-controlled expansion anchors
d) Drop-in type displacement-controlled expansion anchor

b) Post-installed anchors

Figure D.2
Possible orientations of horizontal or upwardly inclined anchors
(See Clauses D.2 and D.4.3.5.3.)

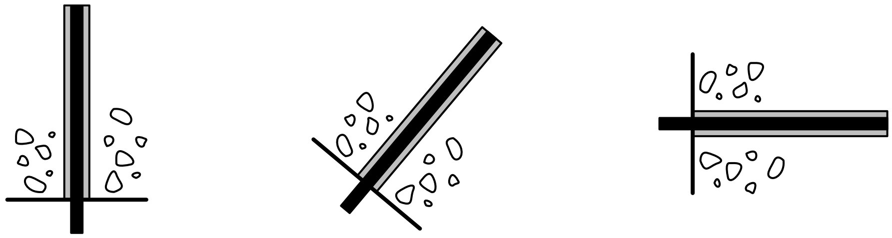
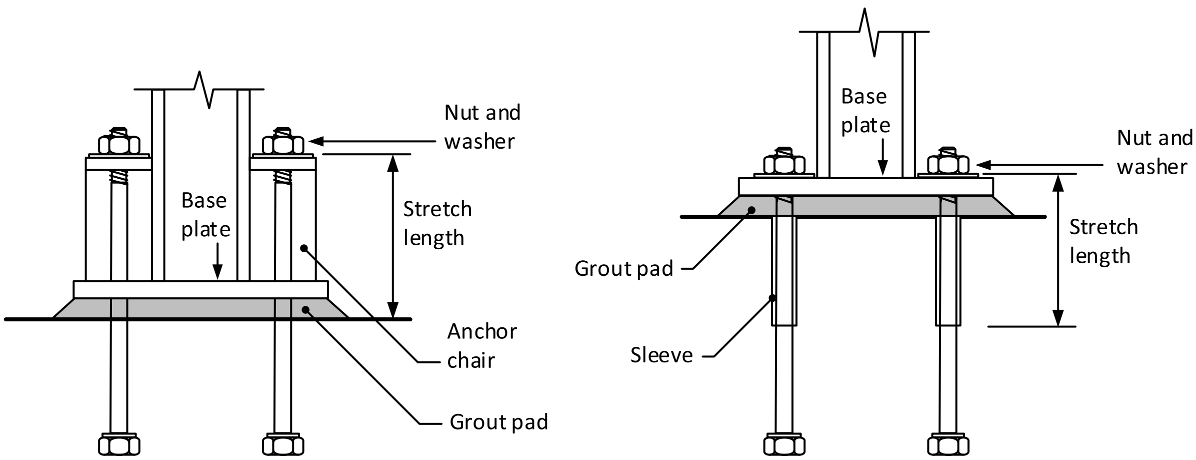


Figure D.3
Illustrations of stretch length
(See Clause D.2.)



a) Anchor chair **b) Sleeve**

Figure D.4A
Failure modes for anchors under tensile loading
(See Clauses D.5.1.1 and D.6.2.1.)

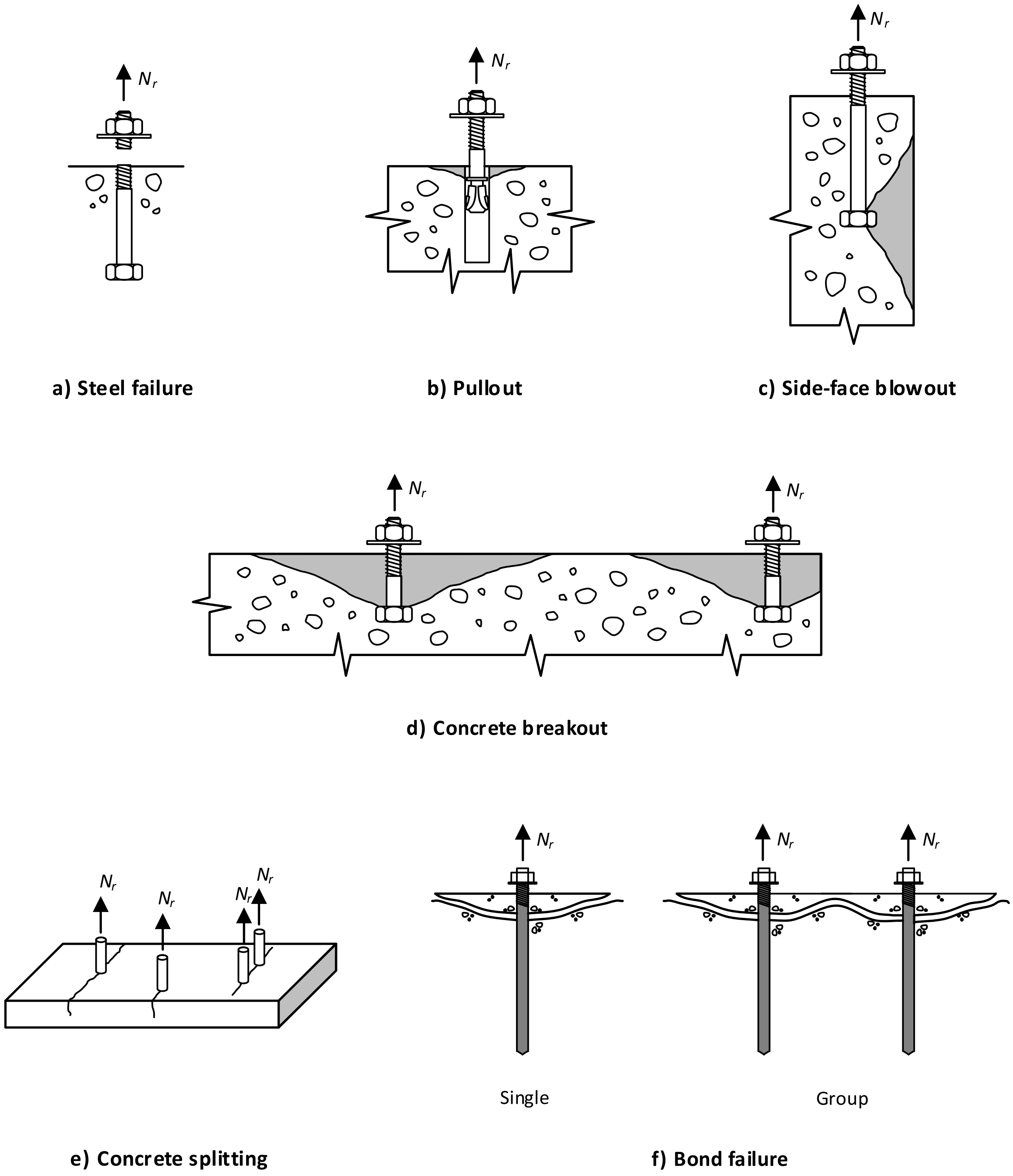
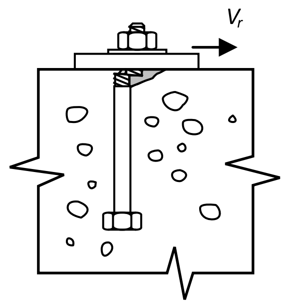
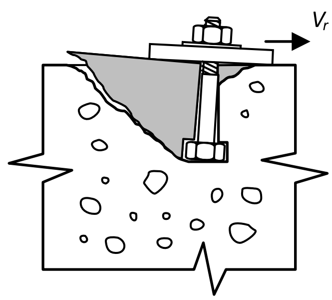


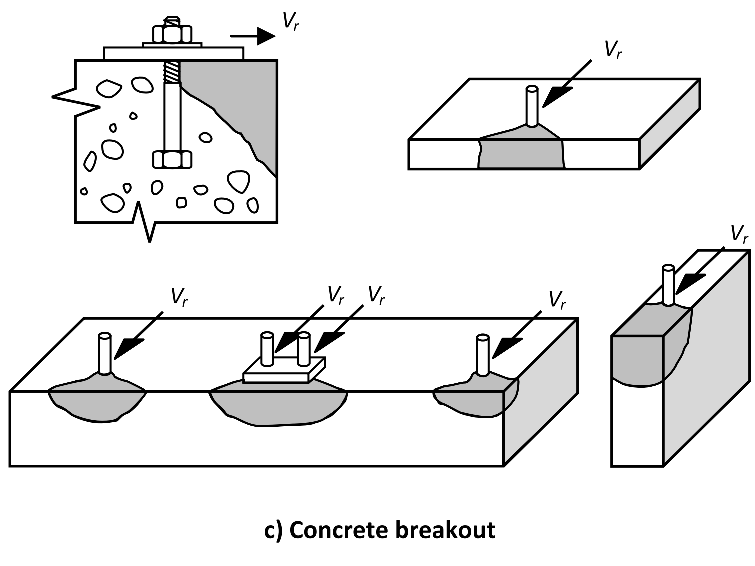
Figure D.4B
Failure modes for anchors under tensile loading
(See Clauses D.5.1.1 and D.7.2.1.)



a) Steel failure preceded by concrete spall

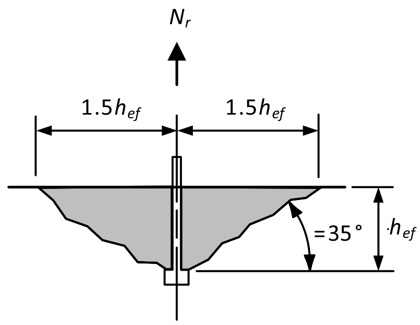


b) Concrete pryout for anchors far from a free edge

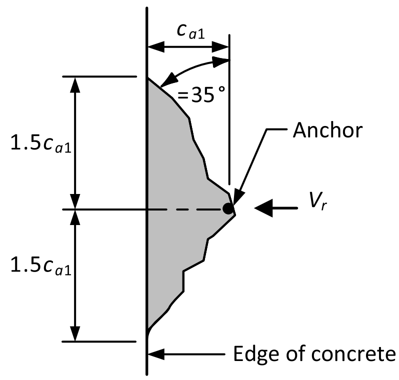


c) Concrete breakout

Figure D.5
Breakout cones
(See Clause D.5.2.1.)



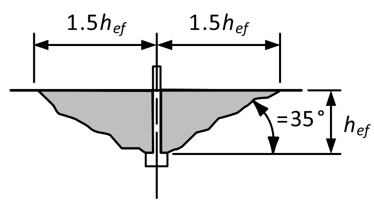
a) Breakout cone for tension



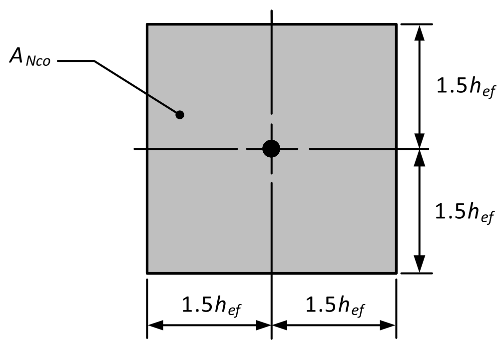
Plan view

b) Breakout cone for shear

Figure D.6
Calculation of A_{Nco}
(See Clause D.6.2.1.)



a) Section through failure cone



b) Plan view

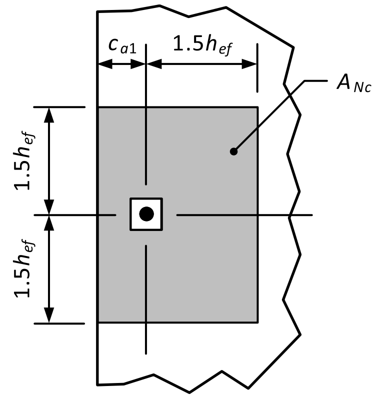
Notes:

1) *The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is 1.5 h_{ef}.*

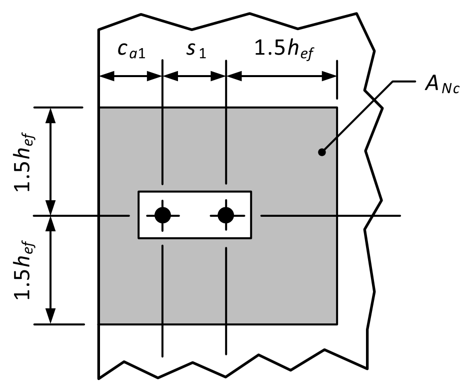
2)

$$A_{Nco} = (2 \times 1.5h_{ef}) \times (2 \times 1.5h_{ef})$$
$$= 3h_{ef} \times 3h_{ef}$$
$$= 9h_{ef}^2$$

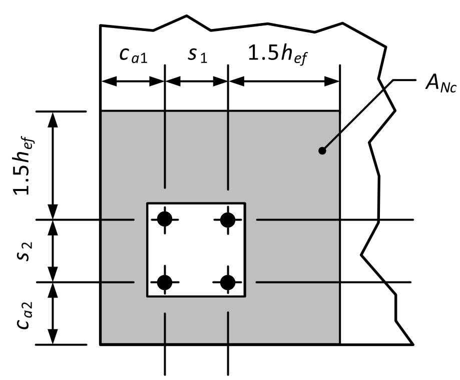
Figure D.7
Projected areas for single anchors and groups of anchors
(See Clauses D.6.2.1 and D.6.2.3.)



$A_{NC} = (c_{a1} + 1.5h_{ef})(2 \times 1.5h_{ef})$
if $c_{a1} < 1.5h_{ef}$

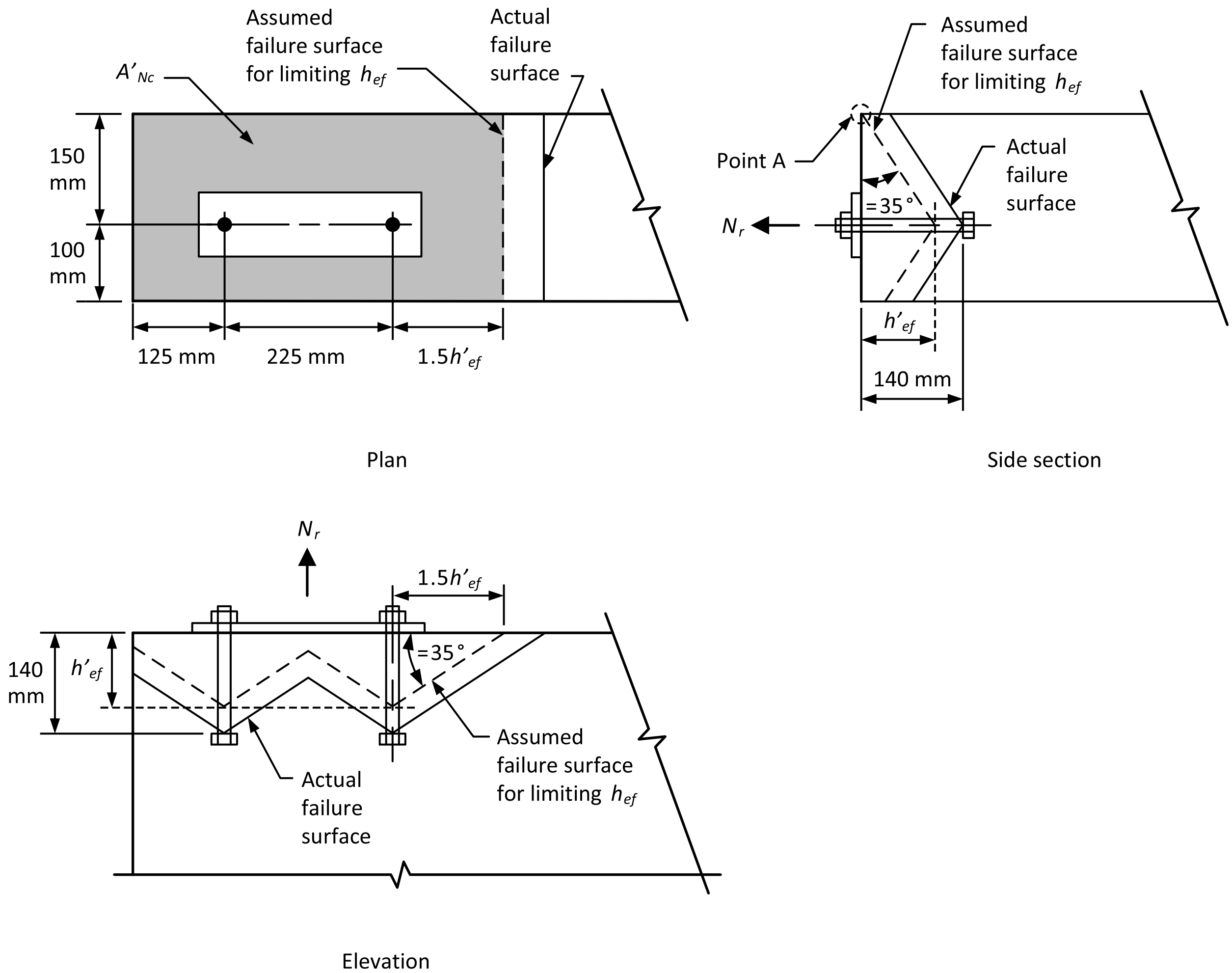


$A_{NC} = (c_{a1} + s_1 + 1.5h_{ef})(2 \times 1.5h_{ef})$
if $c_{a1} < 1.5h_{ef}$ and $s_1 < 3h_{ef}$



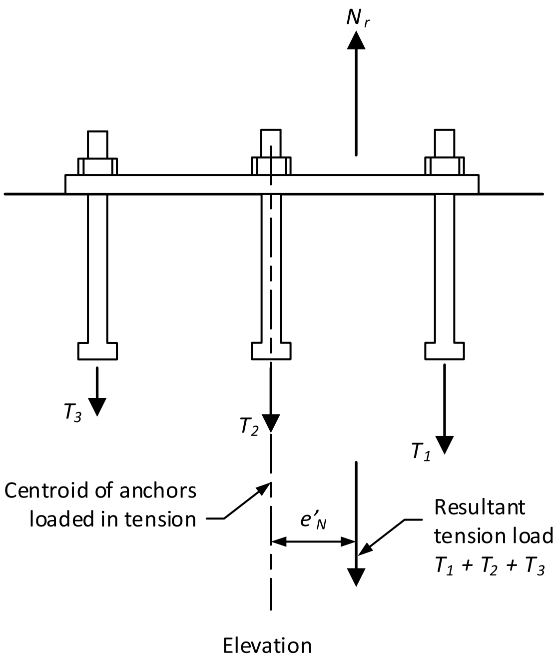
$A_{NC} = (c_{a1} + s_1 + 1.5h_{ef})(c_{a2} + s_2 + 1.5h_{ef})$
if c_{a1} and $c_{a2} < 1.5h_{ef}$
and s_1 and $s_2 < 3h_{ef}$

Figure D.8
Example of tension where anchors are located in narrow members
(See Clause 3.)

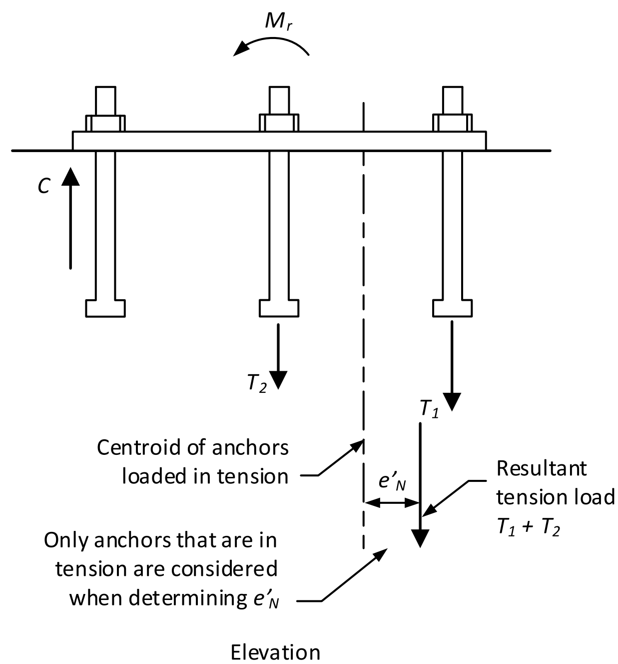


Note: *The actual h_{ef} = 140 mm, but three edges are ≤ 1.5 h_{ef} (shown as h'_{ef} in the Figure) is the larger of $c_{a,max}$/1.5 and one-third of the maximum spacing for an anchor group: h'_{ef} = max (150/1.5, 225/3) = 100 mm. Therefore, use h_{ef} = 100 mm for the value of h_{ef} in Equations D.3 to D.10 including the calculation of A_{Nc}. A'_{Nc} = (150 + 100) (125 + 225 + (1.5 x 100)) = 129 032 mm². Point A shows the intersection of the assumed failure surface for limiting h_{ef} with the concrete surface.*

Figure D.9
Definition of dimension e'_N for a group of anchors
(See Clause D.6.2.4.)

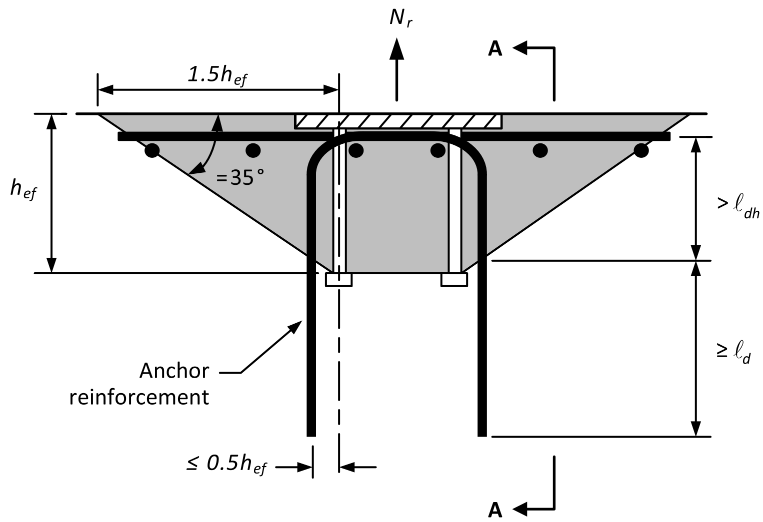


a) Where all anchors in a group are in tension

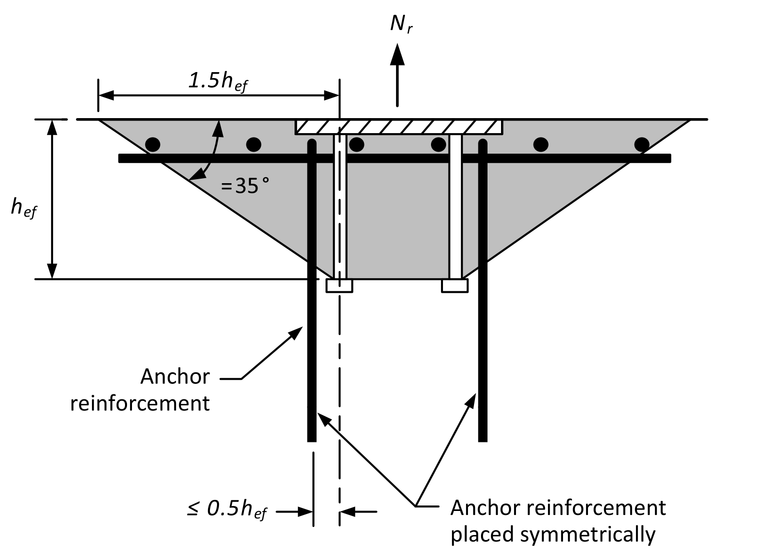


b) Where only some anchors in a group are in tension

Figure D.10
Anchor reinforcement for tension
(See Clause D.6.2.9.)



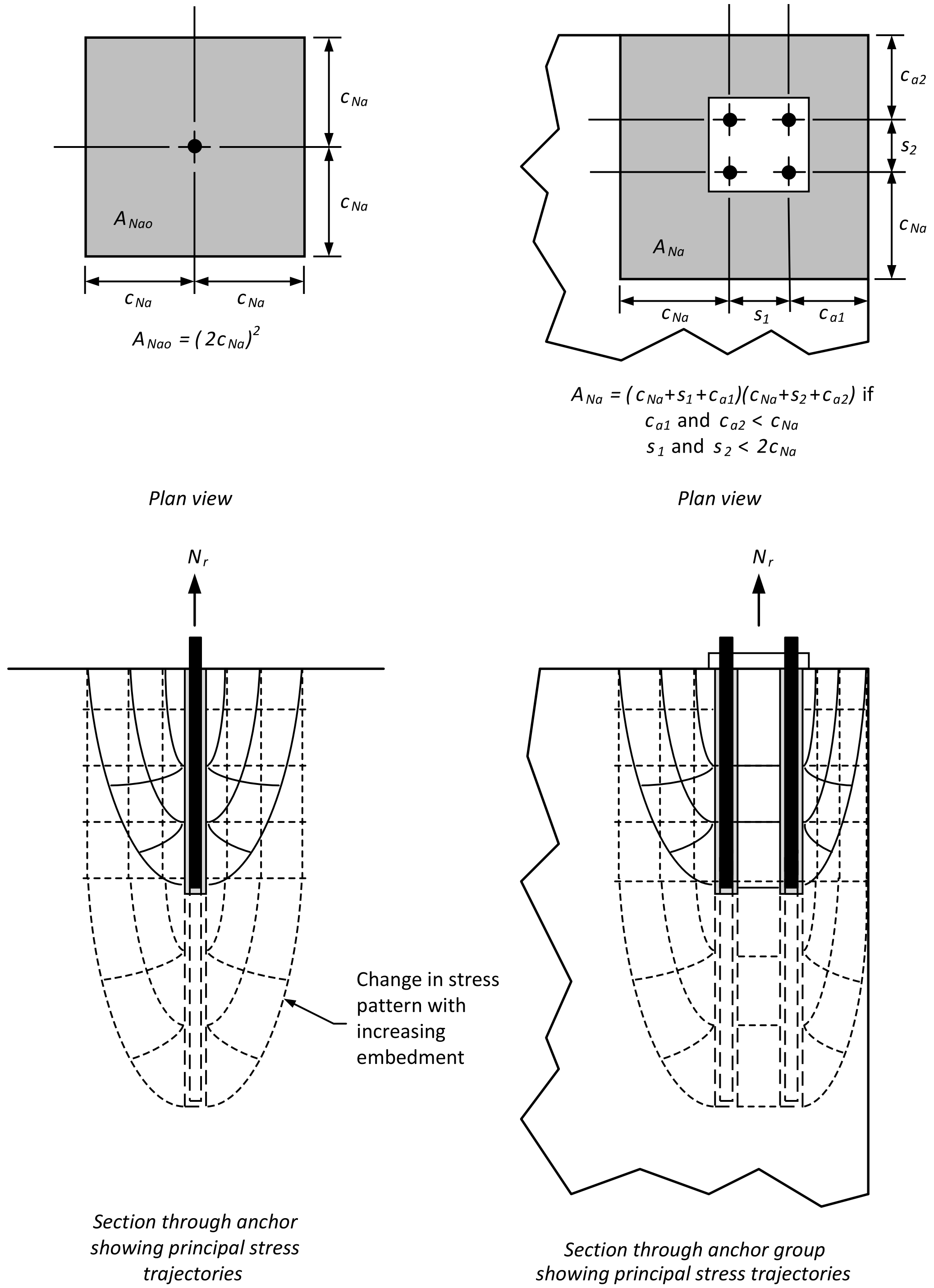
Elevation



Section A-A

Figure D.11
Calculation of influence areas A_{Nao} and A_{Na}

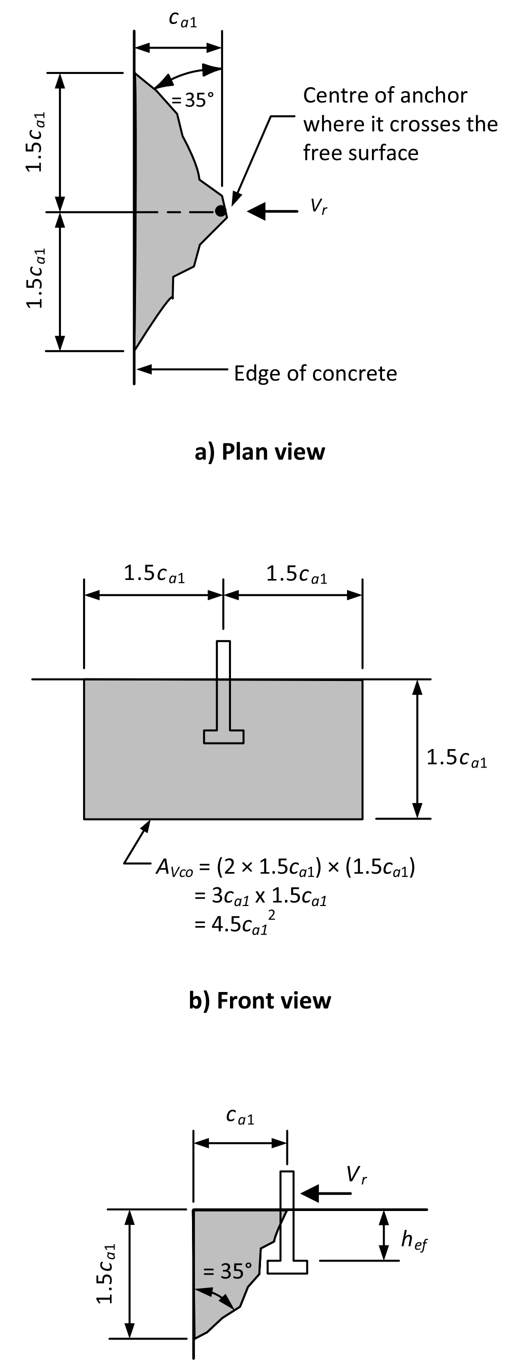
(See Clause D.6.5.1.)



a) Single adhesive anchor away from edges and other anchors

b) Group of four adhesive anchors located near a corner

Figure D.12
Calculation of A_{Vco}
(See Clause D.7.2.1.)



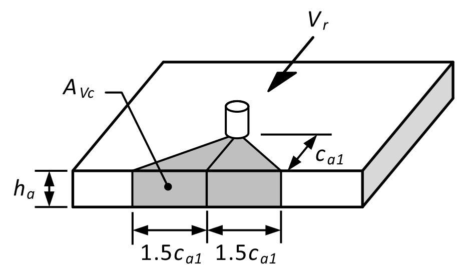
a) Plan view

b) Front view

c) Side section

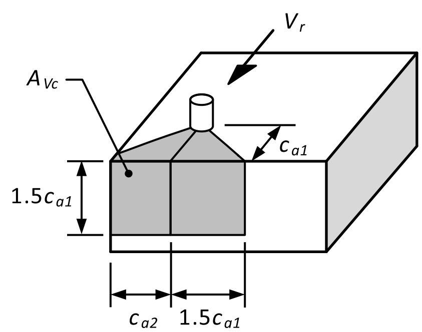
Note: *The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is $1.5c_{a1}$.*

Figure D.13
Projected areas for single anchors and anchor groups
(See Clause D.7.2.1.)



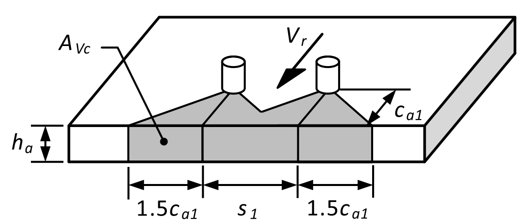
$$A_{Vc} = 2 \times 1.5c_{a1} \times h_a$$

a) A single anchor, if $h_a < 1.5c_{a1}$



$$A_{Vc} = 1.5c_{a1} \times (1.5c_{a1} + c_{a2})$$

b) A single anchor, if $c_{a2} < 1.5c_{a1}$

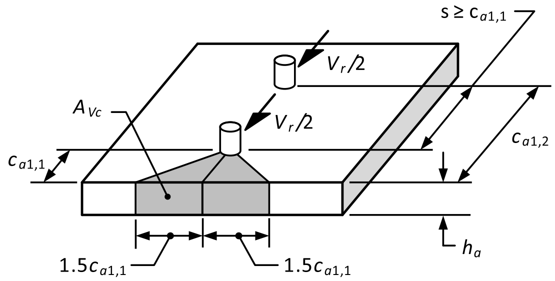


$$A_{Vc} = (2 \times 1.5c_{a1} + s_1) \times h_a$$

c) Two loaded anchors aligned parallel to edge, if $h_a < 1.5c_{a1}$ and $s_1 < 3c_{a1}$

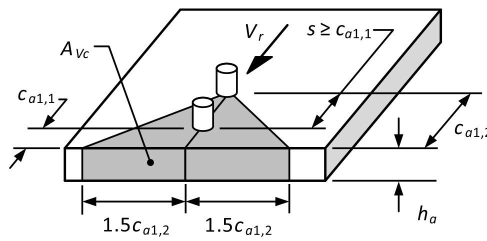
(Continued)

Figure D.13 (Concluded)



$A_{Vc} = 2 \times 1.5c_{a1,1} \times h_a$

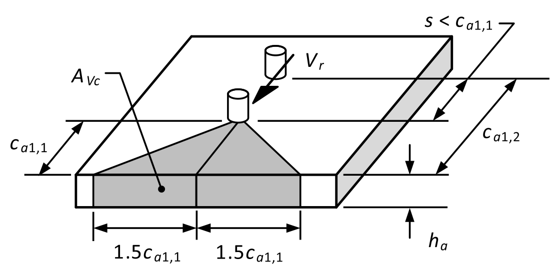
Case 1: One assumption of the distribution of forces indicates that half of the shear force would be critical on the front anchor and the projected area. For the calculation of concrete breakout, c_{a1} is taken as $c_{a1,1}$



$A_{Vc} = 2 \times 1.5c_{a1,2} \times h_a$

Case 2: Another assumption of the distribution of forces indicates that the total shear force would be critical on the rear anchor and its projected area. Only this assumption needs to be considered when anchors are welded to a common plate independent of s. For the calculation of concrete breakout, c_{a1} is taken as $c_{a1,2}$

Note: *For s ≥ $c_{a1,1}$, both Case 1 and Case 2 should be evaluated to determine which controls for design except as noted for anchors welded to a common plate.*



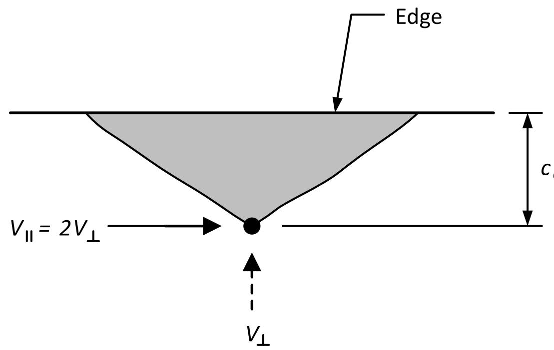
$A_{Vc} = 2 \times 1.5c_{a1,1} \times h_a$

Case 3: Where $s < c_{a1,1}$, apply the entire shear load V to the front anchor. This case does not apply for anchors welded to a common plate. For the calculation of concrete breakout, c_{a1} is taken as $c_{a1,1}$.

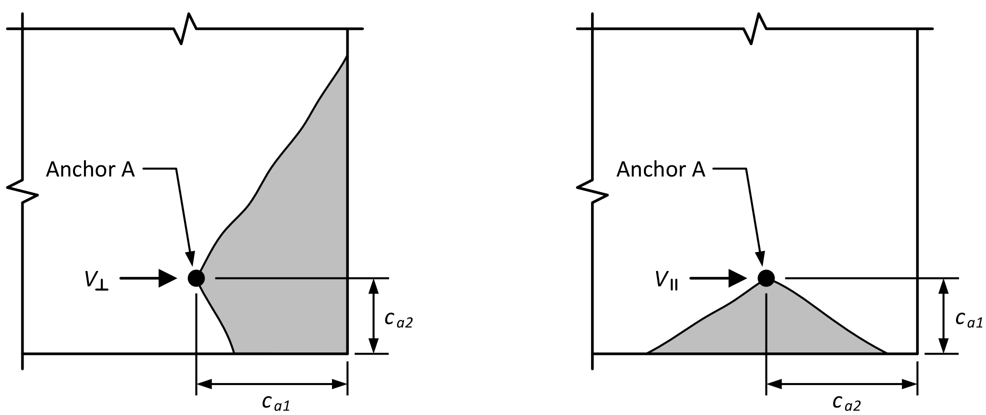
d) Two loaded anchors aligned perpendicular to edge if $h_a < 1.5\ C_{a1}$

Figure D.14
Shear loading parallel or perpendicular to edge, or near a corner

(See Clause D.7.2.1.)

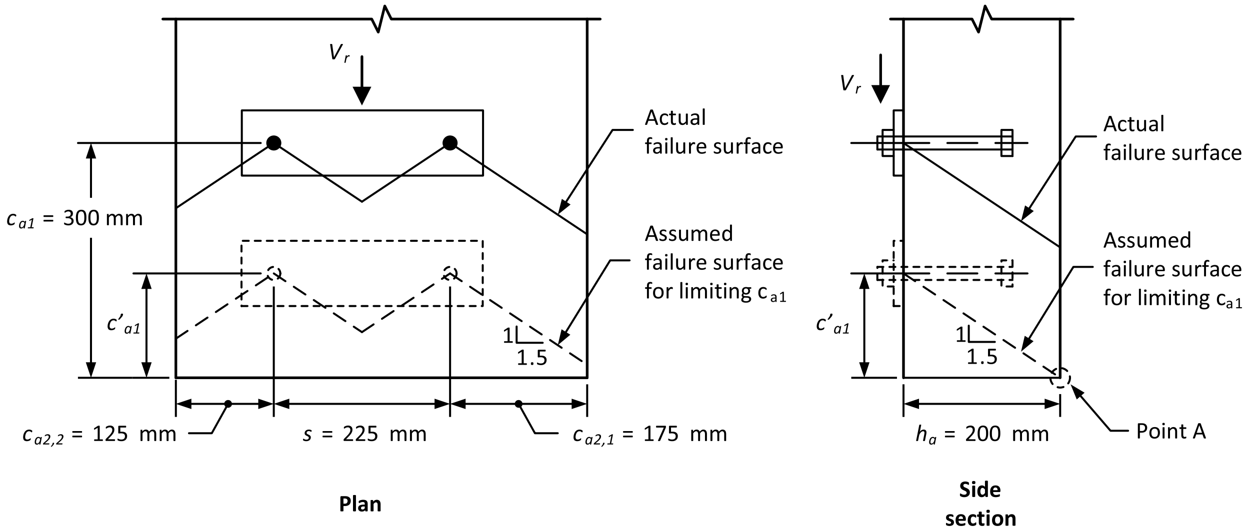


a) Shear force parallel to edge



b) Shear force near a corner

Figure D.15
Example of shear where anchors are located in narrow members of limited thickness
(See Clause D.7.2.4.)



a) The actual c_{a1} = 300 mm.
b) The two edge distances c_{a2} as well as h_a are all less than $1.5c_{a1}$.
c) The limiting value of c_{a1} (shown as c'_{a1} in the figure) to be used for the calculation of A_{Vc} and in Equations D.32 to D.38 and D.40 to D.42 is determined as the largest of the following:
$(c_{a2,max})/1.5 = (175)/1.5 = 116.7$ mm.
$(h_a)/1.5 = (200)/1.5 = 133.3$ mm.
$s/3 = 225/3 = 75$ mm.
d) For this case, A_{Vc}, A_{Vco}, $\psi_{ed,V}$, and $\psi_{h,V}$ are determined as follows:
$A_{Vc} = (125 + 225 + 175)(1.5 \times 133.3) = 105\,000$ mm^2.
$A_{Vco} = 4.5 \times 133.3^2 = 80\,000$ mm^2.
$\psi_{ed,V} = 0.7 + (0.3 \times 125)/133.3 = 0.98$
$\psi_{h,V} = 1.0$ because $c_{a1} = (h_a)/1.5$. Point A shows the intersection of the assumed failure surface with the concrete surface that establishes the limiting value of c_{a1}.

Figure D.16
Definition of dimension e'_V for a group of anchors
(See Clauses D.3 and D.7.2.5.)

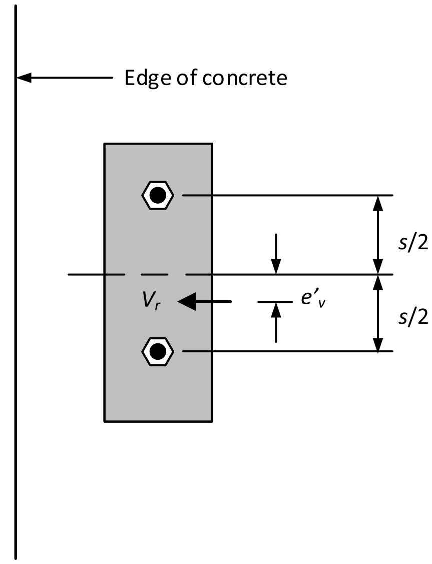
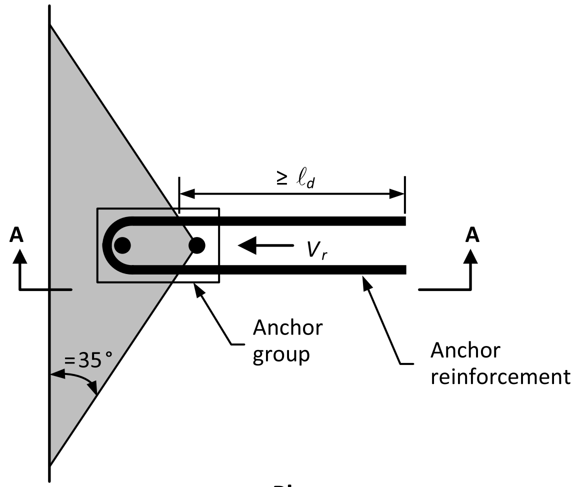
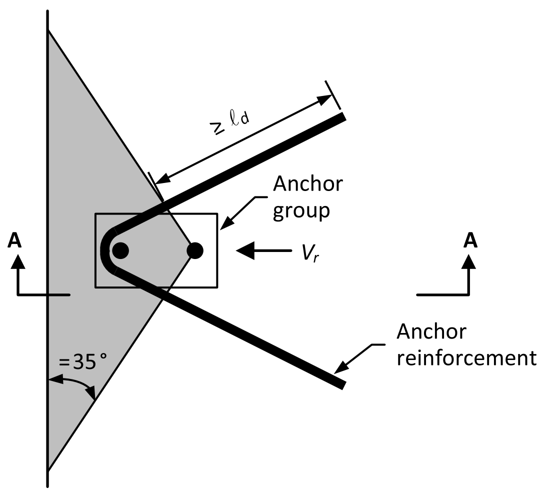


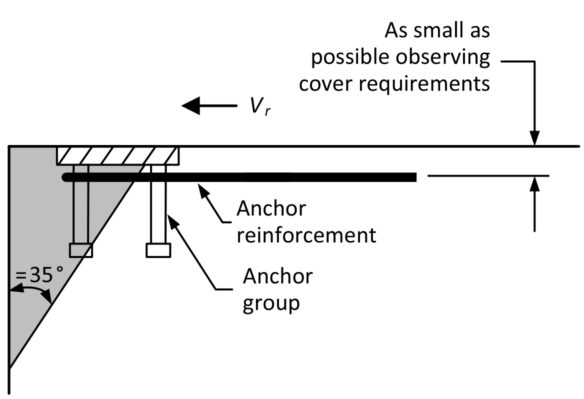
Figure D.17A
Hairpin anchor reinforcement for shear
(See Clause D.7.2.9.)



Plan

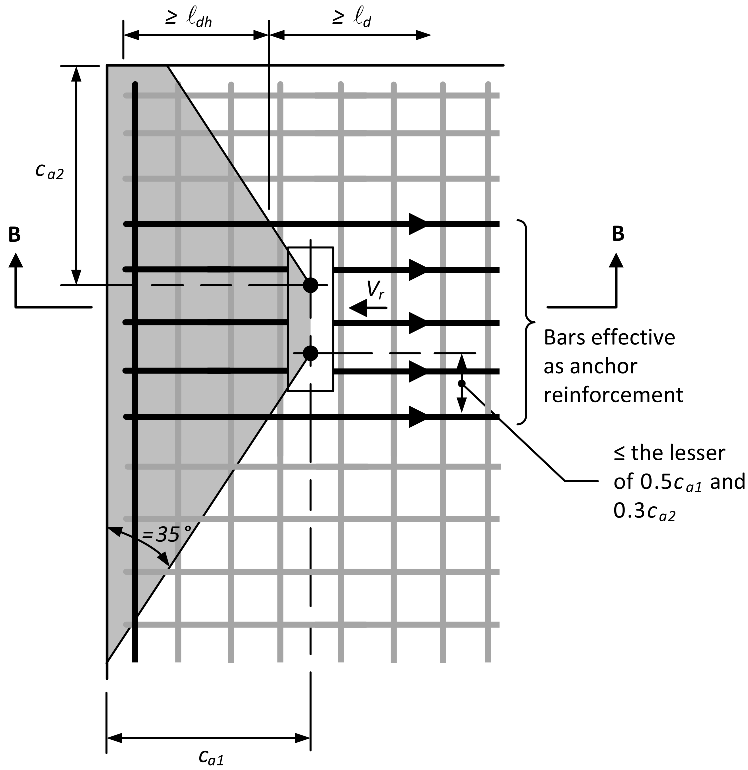


Plan

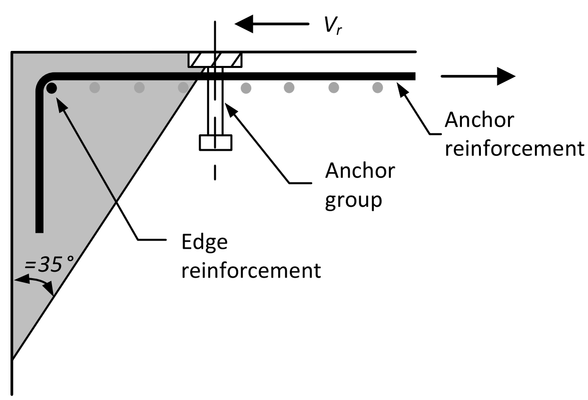


Section

Figure D.17B
Edge reinforcement and anchor reinforcement for shear
(See Clause D.7.2.9.)

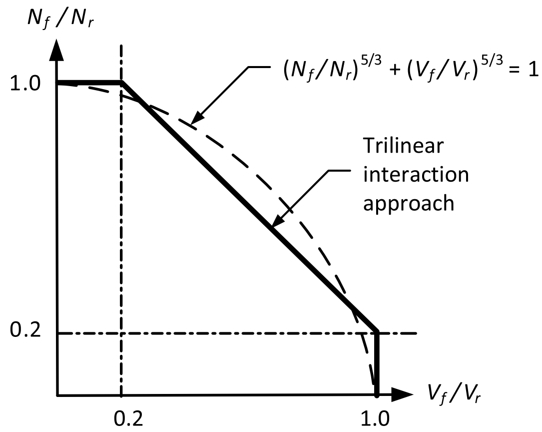


Plan



Section B–B

Figure D.18
Shear and tensile load interaction equation
(See Clauses D.8.2 to D.8.4.)



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