18.3.1.3

Where computed tensile stresses exceed the values specified in Items c) and d) 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 A416/ A416M, ASTM A421/A421M, and ASTM A722/A722M, 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:

$$M_r \geq 1.2M_{cr}$$ **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 but not less than the requirements of Clause 13.10.8.4.

18.10 Frames and continuous construction

Moments for computing the required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads specified in Clause 8.3. Where a minimum area of bonded reinforcement is provided as specified in Clause 18.8, negative moments may be redistributed as specified in Clause 9.2.4.

18.11 Compression members — Combined flexure and axial loads

18.11.1 General

The design of prestressed concrete members subject to combined flexure and axial loads shall be based on Clauses 10.9 to 10.16. The effects of prestress, creep, shrinkage, and temperature change shall be included.

18.11.2 Limits for reinforcement of prestressed compression members

18.11.2.1
Members with average prestress, f_{cp}, less than 1.5 MPa shall have the minimum reinforcement specified in Clauses 7.6 and 10.9 for columns or Clause 14.1.8 for walls.

18.11.2.2
Except for walls, members with average prestress, f_{cp}, equal to or greater than 1.5 MPa shall have all of their prestressing tendons enclosed by spirals or lateral ties as follows:
a) spirals shall comply with Clause 7.6.4; and
b) ties shall comply with Clause 7.6.5, excluding Clauses 7.6.5.2 a) and 7.6.5.5.

18.12 Two-way slab systems

18.12.1 General
Factored moments and shears in prestressed slab systems reinforced for flexure in two directions shall be determined as specified in Clause 13.8 or by more detailed design procedures.

18.12.2 Stresses under specified loads

18.12.2.1
When Clause 13.8 is used, flexural stresses due to unfactored gravity loads in column strips shall be determined by taking 75% of interior negative moments, 100% of exterior negative moments, and 60% of positive moments unless a more detailed analysis is performed.

18.12.2.2
Concrete stresses due to prestressing may be assumed to be uniformly distributed across the slab unless a more detailed analysis is performed.

18.12.2.3
The minimum average compressive stress, f_{cp}, shall be 0.8 MPa.

18.12.3 Shear resistance

18.12.3.1
In the vicinity of concentrated loads or reactions, the maximum factored shear stress, v_f, calculated as specified in Clauses 13.3.5 and 13.3.6, shall not exceed v_r.

18.12.3.2
The factored shear stress resistance, v_r, in two-way slabs shall be not greater than the factored shear stress resistance provided by the concrete, v_c, computed as specified in Clause 13.3.4 or 18.12.3.3, unless shear reinforcement is provided as specified in Clause 13.3.7, 13.3.8, or 13.3.9.

18.12.3.3
At columns supporting two-way slabs of uniform thickness, the factored shear stress resistance provided by the concrete shall be determined by

$$v_c = \beta_p \lambda \phi_c \sqrt{f'_c} \sqrt{1 + \frac{\phi_p f_{cp}}{0.33 \lambda \phi_c \sqrt{f'_c}}} + \frac{V_p}{b_o d}$$ **Equation 18.5**

where

β_p = the smaller of 0.33 or $(\alpha_s d / b_o + 0.15)$

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

b_o = the perimeter of the critical section specified in Clause 13.3.3

f_{cp} = the average value of f_{cp} for the two directions and shall not be taken greater than 3.5 MPa

V_p = the factored vertical component of all prestress forces crossing the critical section

f'_c shall not be taken greater than 35 MPa and the slab shall extend at least $4h_s$ from all faces of the column. Equation 13.5, 13.6, or 13.7 shall apply to edge and corner columns when the slab extends less than $4h_s$ from a column face.

18.12.4 Shear and moment transfer
The fraction of the unbalanced moment transferred by eccentricity of shear shall comply with Clause 13.3.5.3.

18.12.5 Minimum bonded non-prestressed reinforcement

18.12.5.1
The minimum requirements for bonded reinforcement in two-way slabs shall be as specified in Clauses 18.8, 18.9, and 18.12.5.2.

18.12.5.2
In negative moment areas at column supports, the bonded reinforcement, A_s, shall be distributed within a zone equal to the column width plus 1.5 times the slab thickness beyond each side of the column. At least four bars or wires shall be provided in each direction. The spacing of the bonded reinforcement shall not exceed 300 mm.

18.12.6 Spacing of tendons

18.12.6.1
The spacing of tendons or groups of tendons in one direction shall not exceed eight times the slab thickness or 1500 mm unless adequate additional bonded reinforcement is provided so that the slab has the strength to span between tendons.

18.12.6.2
Tendon spacing shall be given special consideration in slabs supporting concentrated loads.

18.12.6.3
In slabs without beams, a minimum of two tendons or bars shall be provided in each direction over each column. These tendons or bars shall satisfy the requirements specified in Clause 13.10.6.

18.13 Tendon anchorage zones

18.13.1
Post-tensioning anchorage zones shall be designed to resist the specified tensile strength of the tendons.

18.13.2
One of the following methods shall be used for the design of anchorage zones:
a) equilibrium based on strut-and-tie models (see Clause 11.4);
b) elastic stress analysis (finite element methods or equivalent);
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 Structural diaphragms

19.1 General

19.1.1
Clause 19 shall apply to the design of non-prestresed and prestressed concrete structural diaphragms and their connections.

19.1.2
Diaphragms that are part of a seismic-force-resisting system with *Rd* > 1.5 shall also meet the requirements of Clause 21.

19.2 Design forces

19.2.1
Design shall consider the following forces:
a) diaphragm in-plane forces due to lateral loads acting on the building;
b) diaphragm transfer forces between LFRS elements;
c) connection forces between the diaphragm and other structural or non-structural elements;
d) force components resulting from bracing of vertical or sloping building elements; and
e) diaphragm out-of-plane forces due to gravity and other loads applied to the diaphragm surface.

19.3 Analysis and design of structural diaphragms

19.3.1
The interaction of all structural and non-structural elements affecting the response of the structure to lateral and stability loads shall be considered in the analysis.

19.3.2
A set of reasonable and consistent assumptions for diaphragm stiffness shall be used. Where element force distribution varies with stiffness assumptions, an upper and lower bound force envelope shall be considered.

19.3.3
Diaphragms shall be designed for concurrent in-plane and out-of-plane loads and forces.

Note: *See NBCC Part 4 for load factors, principal loads, and companion loads.*

19.3.4
Diaphragms shall have thickness as required for stability, strength, and stiffness under factored load combinations.

19.4 Diaphragm systems

19.4.1
For in-plane forces, 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 tension and compression forces;

b) collectors and distributors arranged to transfer the forces to, from, and between the LFRS elements; and

c) shear panels or a system of struts and ties to transfer forces to, from, and between the chords, collectors, and distributors.

19.4.2
Diaphragm elements shall be made effectively continuous as follows:

a) There shall be a load path within and between diaphragm elements and other structural and non-structural elements for force transfer.

b) The force transferred between all edges of shear panels, as well as edges and ends of diaphragm elements, shall be considered.

c) Collectors and distributors shall extend over sufficient lengths of the diaphragm and the LFRS to transfer the required forces.

19.5 Reinforcement

19.5.1
The minimum reinforcement for cast-in-place structural diaphragms shall comply with Clause 7.8.

19.5.2
The diameter of the bars used in diaphragm struts, ties, chords, collector, and distributor elements shall not exceed one-sixth of the minimum element dimension at the bar location.

19.5.3
All continuous reinforcement in struts, ties, chords, collector, and distributor elements shall be anchored and/or spliced as specified in Clause 12.

19.5.4
Reinforcement provided for shear strength shall be continuous and shall may be distributed uniformly across the shear plane.

19.5.5
Reinforcement designed to resist diaphragm in-plane forces shall be in addition to reinforcement required to resist other load effects (see Clause 19.3.3).

19.5.6
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 lateral 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.

19.6 Monolithic concrete systems

19.6.1
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.

19.6.2
The factored shear resistance of a shear panel shall be taken as

$$V_r = \left(0.2\phi_c\lambda\sqrt{f_c'} + \phi_s\rho_n f_y\right)A_{cv} \le 0.14\phi_c f_c' A_{cv}$$ **Equation 19.1**

19.6.3
Chords, collectors, distributors, and struts shall be proportioned to have concrete compressive stresses less than $0.2\phi_c f_c'$ unless an analysis is done accounting for slenderness of the diaphragm and out-of-plane loads. Horizontal reinforcement required as compression reinforcement shall be tied in accordance with Clause 7.6.5 unless the area of horizontal reinforcement is less than $0.005A_g$ and the bar size is 20M or smaller.

19.6.4 Reinforcement splices
No more than 50% of the reinforcement in chords, collectors, distributors, and struts shall be spliced at the same location and laps shall be staggered with at least one lap length from the end of one lap to the start of the next.

19.7 Precast systems

19.7.1
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 a composite topping is placed shall be clean, free of laitance, and intentionally roughened.

19.7.2

The factored shear resistance of a shear panel shall be calculated in accordance with Clause 19.6.2 where A_{cv} is calculated based on the thickness of the cast-in-place topping. 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.

19.7.3

Chords, collectors, struts, and ties shall comply with Clauses 19.6.3 and 19.6.4.

19.8 Composite systems

19.8.1

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. For composite systems, A_{cv} is calculated based on the topping thickness above the flutes.

19.8.2

The factored shear resistance of composite toppings on steel decks may be taken from manufacturer's data but shall be not more than an upper limit of $0.14\phi_c f'_c A_{CV}$.

19.8.3

Where steel deck toppings are reinforced for shear, the minimum reinforcement shall be $0.001A_{cv}$. The maximum bar diameter shall not exceed one-quarter of the topping thickness at the bar location.

19.8.4

For decks bounded by steel beams and girders and where topping reinforcement is continuous over the members, the shear resistance of the reinforced topping shall be calculated in accordance with Clause 19.6.2.

19.8.5

Chords, collectors, struts, and ties may be structural steel and/or reinforced concrete sections. Structural steel members used for this purpose shall have mechanical shear connectors designed to transfer the shear forces from the topping. Reinforced concrete sections shall comply with Clauses 19.6.3 and 19.6.4.

19.9 Construction joints

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

