5.7.1.7.6 Transverse wood plank deck-on-girders
Transverse bending moments due to live load in wood plank decks shall be determined using the equivalent strip method and distributing a wheel load over the width of a plank or 0.25 m, whichever is larger.

5.7.1.8 Transverse vertical shear

5.7.1.8.1 Transverse vertical shear in shear-connected beam bridges
The maximum intensity of transverse vertical shear, V_y, in kN/m, shall be assumed to occur when there is only one design vehicle on the bridge.

The following simplified method may be used for shear-connected beam bridges:

(a) The value of β is calculated as follows:

$$\beta = \pi\left[\frac{B}{L}\right]\left[\frac{D_x}{D_{xy}}\right]^{0.5}$$

(b) In accordance with Figure 5.4, the value of transverse vertical shear intensity, V_y, in kN/m is calculated as follows:

$V_y = kW$

where

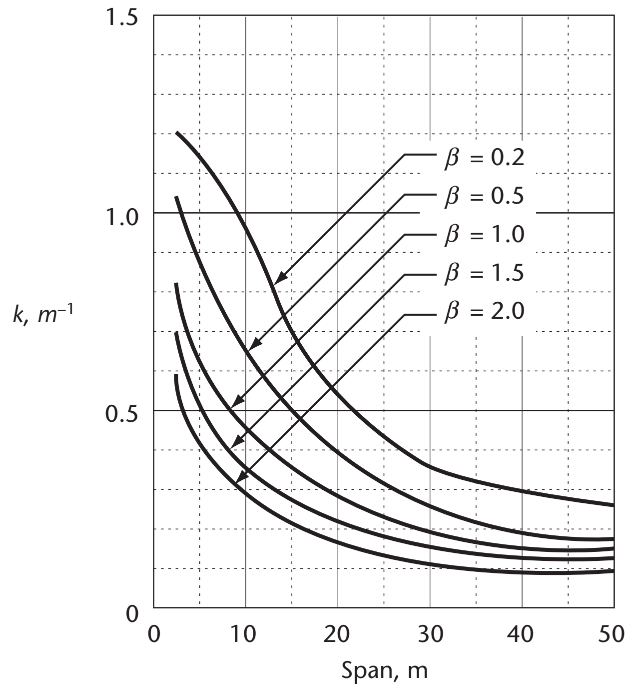
k = applicable value obtained from Figure 5.4, m^{-1}

W = heaviest axle load of the design vehicle, kN

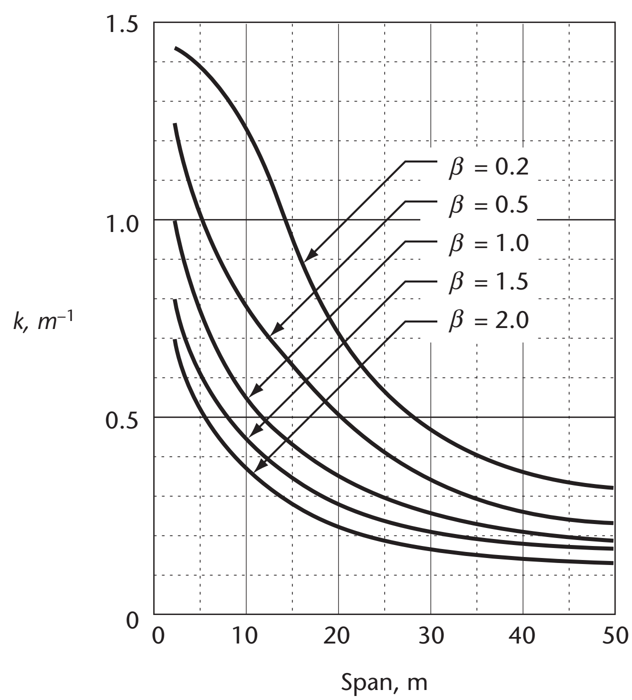
Linear interpolation for this intensity is to be used for widths falling between the widths specified in Figure 5.4.

(c) The intensity of transverse vertical shear obtained in accordance with Item (b) shall be multiplied by (1 + *DLA*) to obtain the design intensity of transverse vertical shear, where *DLA* is the applicable dynamic load allowance for a single vehicle, as specified in Clause 3.8.4.5.

5.7.1.8.2 Transverse vertical shear in transverse wood plank deck-on-girders
The transverse vertical shear due to live load on wood plank decks shall be determined using the equivalent strip method and distributing a wheel load over the width of a plank or 0.25 m, whichever is larger.



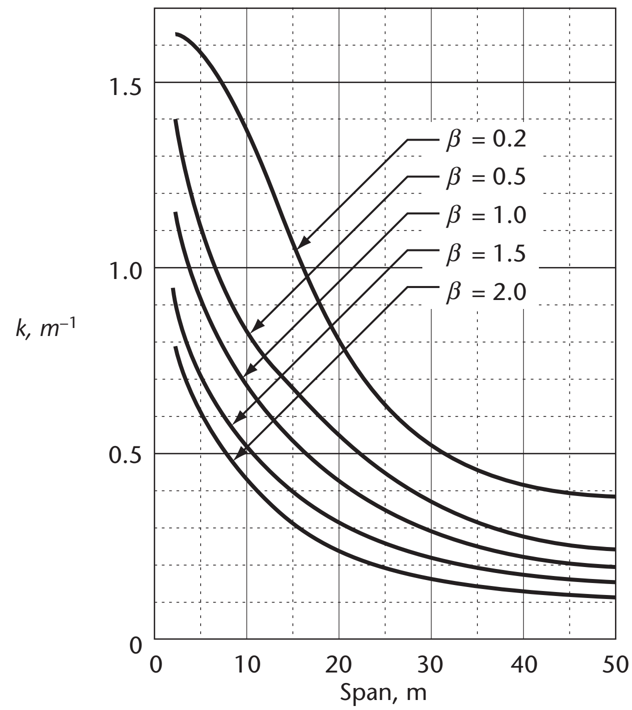
(a) *B* = 7.5 m



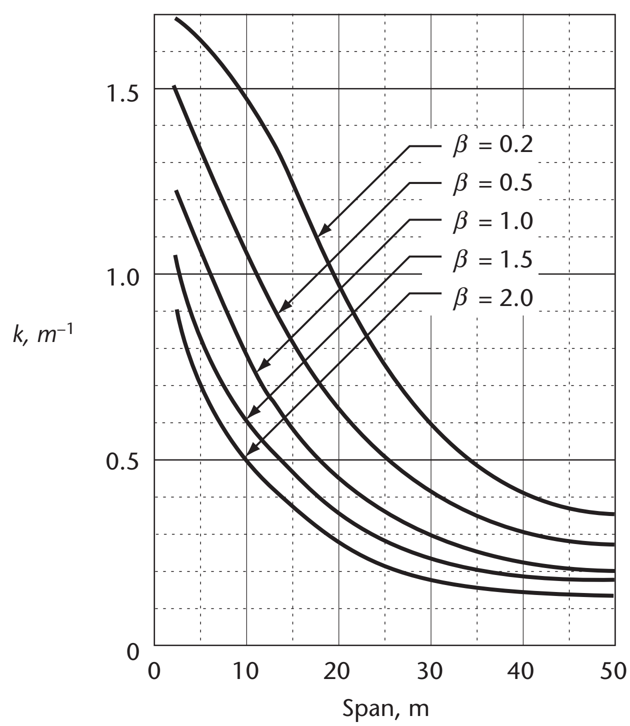
(b) *B* = 10.0 m

Figure 5.4
Values of *k* for calculating transverse vertical shear in shear-connected beam bridges
(See Clause 5.7.1.8.1.)

(Continued)



(c) *B* = 12.5 m



(d) *B* = 15.0 m

Figure 5.4 (Concluded)

5.7.1.9 Analysis of stringers in truss and arch bridges
In analyzing the stringers in truss and arch bridges, the portion of the bridge between adjacent floor beams may be analyzed using the methods specified in Clause 5.6.1.2 for dead load and Clauses 5.7.1.2 and 5.7.1.4 for live load; in both cases, the distance between the adjacent floor beams shall be taken as L, which, if less than 3.0 m, shall be taken as 3.0 m.

When stringers are designed with continuity at the floor beam supports, the flexibility of the floor beams shall be considered.

5.7.1.10 Analysis of floor beams in truss and arch bridges
A live load situated between two floor beams shall be divided between the two beams by simple static division, using the lever principle, without any dispersion of the load along the beams. The line of the lever shall be perpendicular to the floor beams.

5.7.1.11 Analysis of orthotropic steel decks

5.7.1.11.1 General
Force effects in orthotropic decks may be determined by elastic methods of analysis, e.g., equivalent grillage, or by finite strip or finite element methods as specified in Clause 5.9.

In lieu of a more precise analysis, the use of the approximate methods of analysis, as specified in Clauses 5.7.1.11.3 to 5.7.1.11.5, shall be permitted.

5.7.1.11.2 Wheel load distribution
A 45° distribution in all directions of the tire pressure calculated in accordance with Clauses 3.8.3.2, 3.8.4.3, and 3.8.4.4 from the surface contact area to the middle of the steel deck plate (including dynamic load allowance for a single axle in accordance with Clause 3.8.4.5) may be assumed.

5.7.1.11.3 Effective width of deck
The effective width of deck shall be as specified in Clause 5.8.2.2.

5.7.1.11.4 Approximate analysis of decks with open ribs
The rib may be analyzed as a continuous beam supported by the floor beams.

For rib spans not exceeding 4.6 m, the load on one rib due to wheel loads may be determined as the reaction of transversely continuous deck plate on rigid ribs. For rib spans greater than 4.6 m, the effect of rib flexibility on the lateral distribution of wheel loads shall be considered, and for this purpose elastic analysis shall be employed.

For rib spans not greater than 3 m, the flexibility of the floor beams shall be considered when force effects are calculated.

5.7.1.11.5 Approximate analysis of decks with closed ribs
For the analysis of decks with closed ribs, semi-empirical methods may be used. The load effects on a closed rib with the span not greater than 6.1 m may be calculated from wheel loads placed over one rib only, with the effects of the adjacent transversely located wheel loads disregarded.

5.7.2 Refined methods of analysis
For short- and medium-span bridges where the simplified methods specified in Clause 5.7.1 are not applicable, a refined method of analysis in accordance with Clause 5.9 shall be used. In cases where the requirements of Clause 5.7.1 are met, a refined method of analysis may nevertheless be used.

For long-span bridges, a refined method in accordance with Clause 5.10 shall be used.

5.8 Idealization of structure and interpretation of results

5.8.1 General

In the analysis, the structure, boundary conditions, and loading shall be idealized in such a way that the total idealization represents realistically the properties of the actual structure, the actual boundary conditions, and the actual dead and applied loads.

The method of idealizing the structure specified in Clause A5.2.1 may be used. In applying the results of the analysis to the actual structure, the structural responses carried by any component of the mathematical model shall be deemed to be carried by the portion or portions of the actual structure for which the given component is the analogue.

5.8.2 Effective flange widths for bending

5.8.2.1 Concrete slab-on-girders

In the calculation of bending resistances and bending stresses in slab-on-girder bridges and box girder bridges with a concrete slab, a reduced cross-section shall be used. The reduced cross-section shall comprise a left-hand overhang, a central portion, and a right-hand overhang. The overhang, b_e, shall be determined as follows:

$$b_e / b = 1 - \left[1 - \frac{L}{15b}\right]^3 \text{ for } L/b \le 15$$

$$= 1 \text{ for } L/b > 15$$

where

b_e = dimension shown in Figure 5.5 for the applicable type of bridge cross-section

b = the dimension shown in Figure 5.5 for the applicable type of bridge cross-section

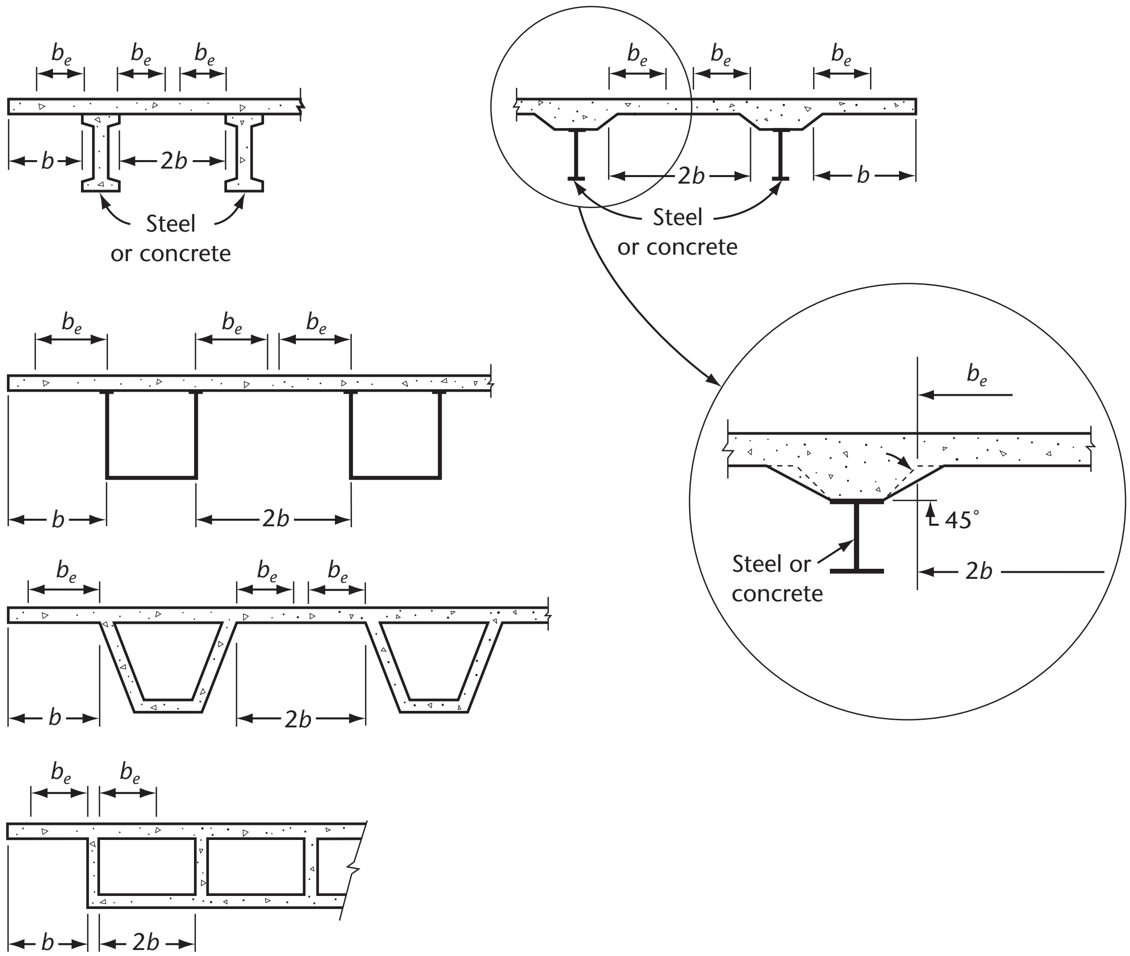


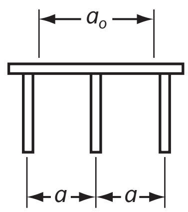
Figure 5.5
b_e and b for various cross-sections
(See Clause 5.8.2.1.)

5.8.2.2 Orthotropic steel decks

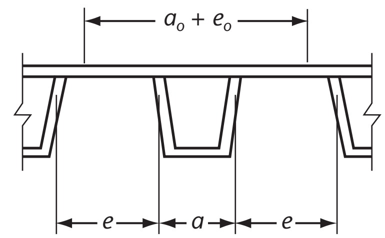
5.8.2.2.1 Longitudinal ribs

The effective width of the deck acting as the top flange of one longitudinal stiffener or one rib shall be determined from Table 5.11.

Table 5.11
Effective deck plate width for a longitudinal rib
(See Clause 5.8.2.2.1.)



Effective width = a_o



Effective width = $a_o + e_o$

Rib section properties for calculation of deck rigidity and flexural effects due to dead load	$a_o = a$	$a_o + e_o = a + e$
Rib section properties for calculation of flexural effects due to wheel loads	$a_o = 1.1a$	$a_o + e_o = 1.3(a + e)$

5.8.2.2.2 Longitudinal girders and transverse beams

The effective width of the deck acting as the top flange of a longitudinal superstructure component or transverse beam may be determined using an accepted method of analysis or may be taken as shown in Figure 5.6.

The effective span, L, shown as ℓ_1 and ℓ_2 in Figure 5.6 shall be taken as the actual span for simple spans and as the distance between points of dead load contraflexure for continuous spans.

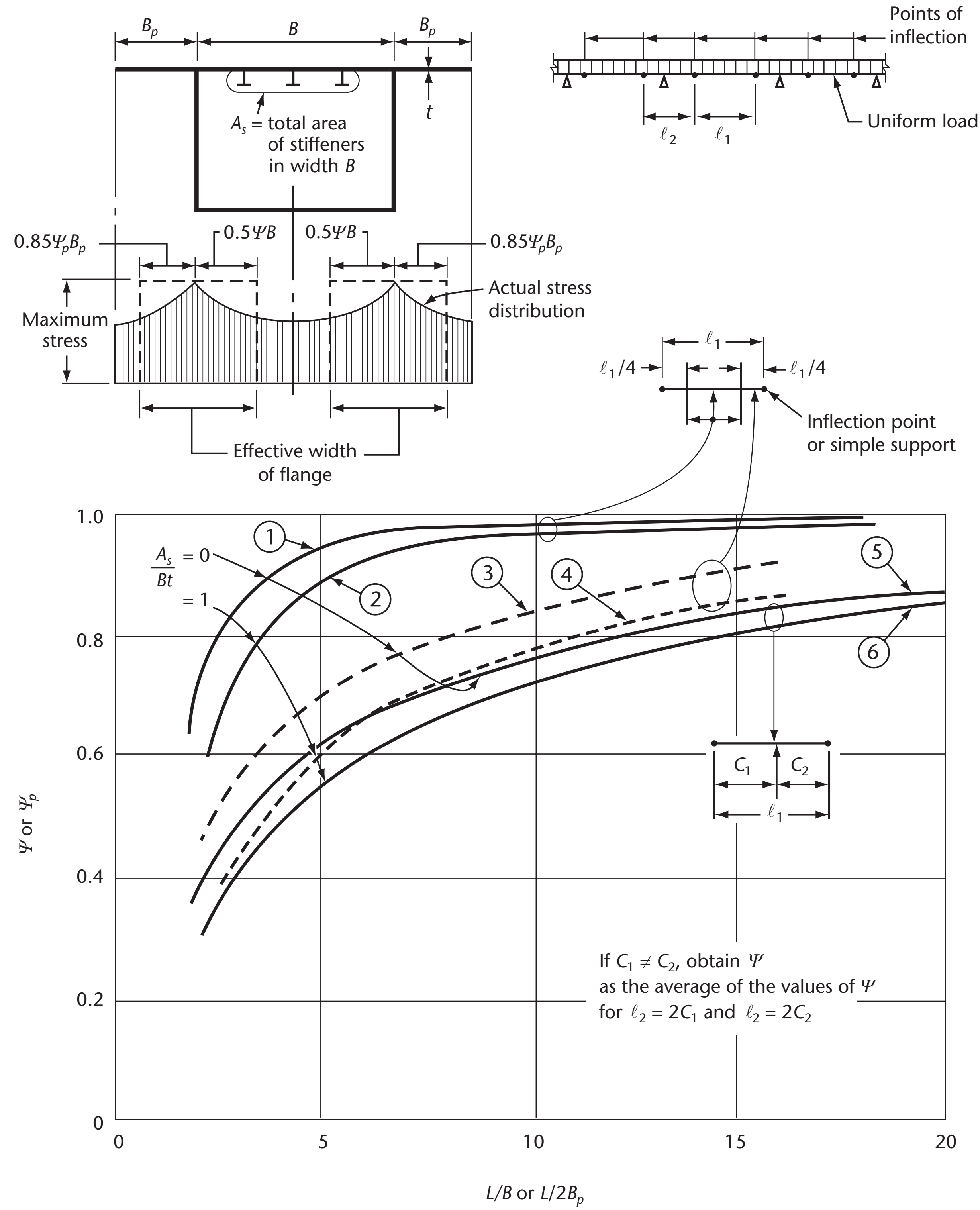


Figure 5.6
Effective width of orthotropic deck
(See Clause 5.8.2.2.2.)

(Continued)

Notes:

(1) *Curves 1 and 2 apply to the middle half of the positive moment region of beams.*

(2) *Curves 3 and 4 apply to areas in positive moment regions located between the inflection point or simple support and one-quarter of the length of the positive moment region.*

(3) *Curves 5 and 6 apply to negative moment regions.*

Legend:

B = spacing of longitudinal or transverse beams, as applicable, mm

B_p = length of the cantilever portion of the transverse beam, mm

ℓ_1, ℓ_2 = distances between the points of inflection of the longitudinal or transverse beams, as applicable, mm

L = ℓ_1 for positive moment regions of the longitudinal or transverse beams, as applicable

= ℓ_2 for negative moment regions of the longitudinal beams or transverse beams

Note: *For cantilever portions of transverse beams, L shall be taken as twice the length of the cantilever.*

ψ = effective plate width factor for interior portions of deck between beams

ψ_p = effective width factor for exterior or cantilever portions of deck

Figure 5.6 (Concluded)

5.8.3 Idealization for analysis

For the purposes of analysis, the stiffness properties for concrete and composite members shall be based on uncracked sections or on cracked and/or uncracked sections consistent with the anticipated behaviour.

5.9 Refined methods of analysis for short- and medium-span bridges

5.9.1 Selection of methods of analysis

The refined methods of analysis for short- and medium-span bridges are as follows:

(a) grillage analogy;
(b) orthotropic plate theory;
(c) finite element;
(d) finite strip;
(e) folded plate; and
(f) semi-continuum.

Unless specified elsewhere in this Code or Approved, the method or methods of analysis may be selected from Table 5.12. Other methods may be used if they are capable of providing a level of accuracy comparable to that of the methods specified in Items (a) to (f).

5.9.2 Specific applications

Influence surfaces may be used to evaluate relevant responses in bridge superstructures if they are developed from the refined methods specified in Clause 5.9.1 or from model analysis in accordance with Clause 5.9.3. The use of influence surfaces developed using other methods shall require Approval.

5.9.3 Model analysis

The use of model analysis (which involves testing a physical model of the whole or part of a bridge) shall be acceptable as an alternative or addition to other methods of analysis permitted in this Section. The model analysis and the interpretation of the results for the purpose of design shall require Approval.

Table 5.12
Selection of methods of analysis
(See Clause 5.9.4.)

Method of analysis	Bridge type(s) (see Clause 5.1) for which the method is appropriate	Limitations on applicability
Simplified method specified in Clause 5.6.1.2 for dead load	Slab Voided slab Slab-on-girder Shear-connected beam Floor systems of truss, arch, or rigid frame and integral abutment Bridges incorporating longitudinal wood beams Box girder — Single cell Box girder — Multi-cell Box girder — Multi-spine	Elements of the structure shall meet the requirements of Clause 5.6.1.1
Simplified method specified in Clauses 5.7.1.2, 5.7.1.4, 5.7.1.6, and 5.7.1.7 for live load	Slab Voided slab Slab-on-girder Shear-connected beam Floor systems of truss, arch, or rigid frame and integral abutment Bridges incorporating longitudinal wood beams	Structure shall meet the requirements of Clause 5.7.1.1
Simplified method specified in Clauses 5.7.1.3 and 5.7.1.5 for live load	Box girder — Multi-spine	Structure shall meet the requirements of Clause 5.7.1.1
Grillage analogy	Slab Voided slab Slab-on-girder Shear-connected beam Floor systems of truss, arch, or rigid frame and integral abutment Bridges incorporating longitudinal wood beams Box girder — Multi-cell Box girder — Multi-spine Orthotropic decks	—
Orthotropic plate theory	Slab Voided slab Slab-on-girder Shear-connected beam Floor systems of truss, arch, or rigid frame and integral abutment Bridges incorporating longitudinal wood beams Box girder — Multi-spine Orthotropic decks	Structure shall meet the requirements of Clause 5.7.1.1

(Continued)

Table 5.12 (Concluded)

Method of analysis	Bridge type(s) (see Clause 5.1) for which the method is appropriate	Limitations on applicability
Finite element	All bridge types	For shear-connected beam bridges, it is possible that special elements with zero transverse rigidity will be necessary
Finite strip	Slab Voided slab Slab-on-girder Shear-connected beam Floor systems of truss, arch, or rigid frame and integral abutment Bridges incorporating longitudinal wood beams Box girder — Single cell Box girder — Multi-cell Box girder — Multi-spine Orthotropic decks Cable stayed Suspension	The support conditions are closely equivalent to line support at the ends of the bridge. In the case of multi-span bridges, isolated column supports shall be permitted. For shear-connected beam bridges, it is possible that special elements with zero transverse rigidity will be necessary.
Folded plate	Slab Voided slab Slab-on-girder Shear-connected beam Floor systems of truss, arch, or rigid frame and integral abutment Bridges incorporating longitudinal wood beams Box girder — Single cell Box girder — Multi-cell Box girder — Multi-spine Orthotropic decks	Not applicable to bridges with (a) a skew parameter greater than that permitted by Clause 5.6.1.1; or (b) support conditions other than those permitted by Clause 5.6.1.1. For shear-connected beam bridges, it is possible that special elements with zero transverse rigidity will be necessary.
Semi-continuum	Slab Voided slab Slab-on-girder Shear-connected beam Floor systems of truss, arch, or rigid frame and integral abutment Bridges incorporating longitudinal wood beams Box girder — Multi-spine Orthotropic decks	Structures shall meet the requirements of Clause 5.7.1.1
Conventional methods of analysis for truss, arch, or rigid frame and integral abutment	Trusses, arches, and rigid frames and integral abutment, as applicable	—
Influence surface	All bridge types	In accordance with Clause 5.9.2
Model analysis	All bridge types	In accordance with Clause 5.9.3
Other methods	All bridge types	Require Approval

5.10 Long-span bridges

5.10.1 General

In the analysis of cable-stayed bridges, suspension bridges, and long-span arches, the deflected shape of the structure shall be used in the formulation of equilibrium. For other types of long-span bridges, the analysis may be based on typical assumptions associated with small-deflection, linear-elastic structures. The elastic method used shall be capable of determining all essential structural responses.

5.10.2 Cable-stayed bridges

Spatial or planar structural analysis may be used to determine the distribution of force effects in the components of a cable-stayed bridge if the tower geometry, the number of planes of stays, and the torsional stiffness of the deck superstructure are considered.

Cable-stayed bridges shall be investigated for

(a) non-linear effects that could result from the change in cable sag at all limit states;
(b) deformation of the deck superstructure at all limit states; and
(c) material non-linearity at the ultimate limit states.

The change in force effects due to deflection may be investigated using any method that satisfies large-deflection theory and accounts for the change in orientation at the ends of the cable stays.

Cable-stayed bridges shall be investigated for the effects of the loss of any cable stay in order to ensure the integrity of the structure in the event of such a loss.

Cable stays shall be designed to be easily replaceable.

5.10.3 Suspension bridges

For suspension bridges, force effects shall be analyzed using the large-deflection theory for vertical, torsional, and lateral loads. Linear and elastic material properties may be assumed; however, the non-linear geometrical relationship between force and deformation shall be accounted for.

The effects of wind loads shall be analyzed, taking into consideration the tension stiffening of the cables.

5.11 Dynamic analysis

5.11.1 General requirements of structural analysis

5.11.1.1 General

For analysis of the dynamic behaviour of bridges, the stiffness, mass, and damping characteristics of the structural components shall be modelled.

The minimum number of degrees of freedom included in the analysis shall be based on the number of natural frequencies to be obtained and the reliability of the assumed mode shapes. The model shall be compatible with the accuracy of the solution method. Dynamic models shall include relevant aspects of the structure and the excitation. The relevant aspects of the structure may include distribution of mass, distribution of stiffness, and damping characteristics. The relevant aspects of excitation may include frequency of the forcing function, duration of application, and direction of application.

5.11.1.2 Distribution of masses

The modelling of mass shall be consistent with the number of mode shapes used in the analysis.

5.11.1.3 Stiffness

The stiffnesses of the elements of the model shall be consistent with the corresponding portions of the bridge being modelled.

5.11.1.4 Damping
Equivalent viscous damping may be used to represent energy dissipation.

5.11.1.5 Natural frequencies
For the purpose of Clause 5.11.2, and unless otherwise specified by the Regulatory Authority, elastic undamped natural modes and frequencies of vibration shall be used. For the purpose of Clause 5.11.4 and Section 4, all relevant damped modes and frequencies shall be considered.

5.11.2 Elastic dynamic responses

5.11.2.1 Vehicle-induced vibrations
Vehicle-induced vibrations shall be accounted for by applying a dynamic load allowance, i.e., an equivalent static load equal to a fraction of the applied live load. The dynamic load allowance shall be as specified in Clause 3.8.4.5.

5.11.2.2 Wind-induced vibrations
In accordance with Clauses 3.10.4.1 and 3.10.4.2, wind-sensitive structures shall be analyzed for dynamic effects such as buffeting by turbulent or gusting winds and unstable wind–structure interaction such as vortex shedding, galloping, and flutter.

Slender or torsionally flexible structures shall be analyzed for lateral buckling, excessive thrust, and divergence.

Oscillatory deformations under wind that could lead to excessive stress levels, structural fatigue, and user inconvenience shall be avoided. Bridge decks, cable stays, and hanger cables shall be protected against excessive vortex and against oscillations induced by wind and rain. Where practical, the employment of dampers shall be considered to control excessive dynamic responses. Where dampers or shape modification are not practical, the structural system shall be changed to achieve such control.

5.11.3 Inelastic-dynamic responses

5.11.3.1 General
Energy dissipation by one or more of the following mechanisms during a major earthquake or ship collision may be taken into account:
(a) elastic or inelastic deformation of the object that could collide with the structure;
(b) inelastic deformation of the structure and its attachments;
(c) permanent displacements of the masses of the structure and its attachments; and
(d) inelastic deformation of special-purpose mechanical energy dissipaters.

5.11.3.2 Plastic hinges and yield lines
For the purpose of analysis, energy absorbed by inelastic deformation in a structural component may be assumed to be concentrated in plastic hinges and yield lines. The location of these sections may be established by successive approximation to obtain a lower bound solution for the energy absorbed. For these sections, moment-rotation hysteresis curves may be determined using verified analytic material models.

5.11.4 Analysis for collision loads
Where permitted by Section 3, dynamic analysis of ship collision may be replaced by an equivalent static elastic analysis. Where an inelastic analysis is specified, the effect of other loads that could be present shall be considered.

5.11.5 Seismic analysis
The minimum analysis requirements for seismic effects shall be as specified in Clauses 4.4.5, 4.5, and 4.11.5.

5.12 Stability and magnification of force effects

5.12.1 General
Stability effects are divided into two categories: member stability and structural stability. The stability of individual members, of the components of structural assemblies, and of structural systems shall be considered in the analysis. Stability analyses of structural assemblies and individual members shall be performed as specified in the clauses of Sections 8 to 10 and 16 that apply to the material(s) used for the members.

5.12.2 Member stability analysis for magnification of member bending moments
Member stability analysis shall be performed in order to account for
(a) the interaction between axial compression forces and bending moments or out-of-straightness of a member; and
(b) the possible increase of the bending moment magnitude between the two ends of a member.
Each member shall be considered individually.

5.12.3 Structural stability analysis for lateral sway
Structural stability analysis shall be performed to account for gravity loads undergoing lateral sway arising from horizontal loads or out-of-plumbness of the structure. This structural analysis shall encompass all members or structural components resisting the sway.

5.12.4 Structural stability analysis for assemblies of individual members
The structural stability of an assembly of individual members shall be considered for the condition of the buckling of such an assembly acting as a whole.

Annex A5.1 (normative) ***Factors affecting structural response***

Note: *This Annex is a mandatory part of this Code.*

A5.1.1 General

The factors affecting structural response are specified in this Annex. The method of analysis chosen shall be capable of evaluating behaviour affected by these factors.

A5.1.2 Continuity of spans

When the simplified methods specified in Clauses 5.7.1.2 to 5.7.1.5 are to be used for a multi-span bridge, the effect of continuity may be accounted for, provided that the ends of the bridge are free of externally applied restraint against rotation, by taking the value of L for obtaining F as follows:

(a) for positive moments in an exterior span: 80% of the distance between the external support and the internal support;

(b) for positive moments in an interior span: 60% of the distance between the internal supports; and

(c) for negative moments in the region of an internal support: 25% of the sum of the spans on either side of the support.

The positive moment and negative moment regions (only for the purpose of obtaining the value of F) will then be as shown in Figure A5.1.1. Unless specified elsewhere in this Section, points of inflexion shall not be assumed to occur at the positions shown in Figure A5.1.1 for any purpose other than calculating F.
A value of 3.0 m shall be assumed for L if it is found to be less than 3.0 m.

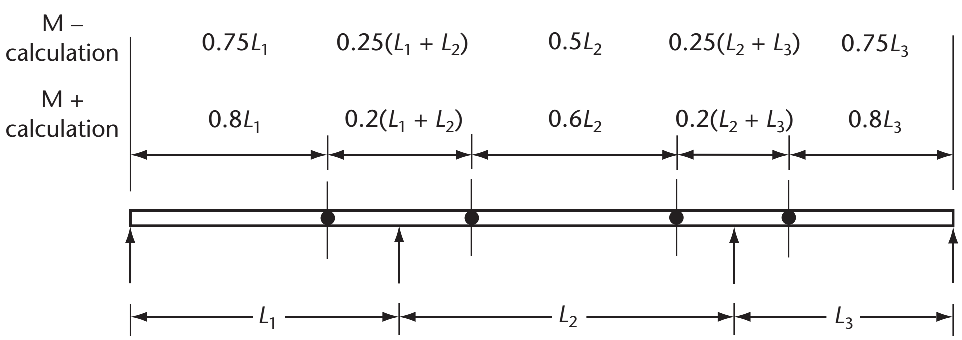


Figure A5.1.1
Assumed points of inflexion under dead loads
(See Clause A5.1.2.)

A5.1.3 Plan geometry

A5.1.3.1 Shallow superstructures on skew spans

A5.1.3.1.1

If, for solid and voided slab bridges, the skew parameter $\varepsilon = \beta \tan \psi / L$ does not exceed 1/6, and, for slab-on-girder bridges, the skew parameter $\varepsilon = S \tan \psi / L$ does not exceed 1/18, the angle of skew may be ignored for the calculation of longitudinal moment and shears if the analysis is carried out for a right span that is equal to the skew span of the actual structure.

Note: *S is the girder spacing and B is the bridge width.*

A5.1.3.1.2

A5.1.3.1.2.1

For values of the skew parameter greater than those specified in Clause A5.1.3.1.1, the effects of longitudinal and transverse torsion under live and dead loads and prestressing forces shall be considered, except that slab-on-girder bridges satisfying all of the applicable conditions listed in Clause 5.7.1.1 may be analyzed using the simplified methods of Clause 5.7.1, supplemented as specified in Clause A5.1.3.1.2.2.

A5.1.3.1.2.2

For slab-on-girder bridges with skew, a simplified method of analysis for longitudinal bending moment may be used in the absence of a more refined method, i.e., the corresponding bridge without skew, using the skew span, may be analyzed for longitudinal bending moment in accordance with Clauses 5.7.1.2.1 and 5.7.1.2.2. The bending moments thus obtained may be used for design without modification.

A5.1.3.1.3

For the calculation of longitudinal vertical shear in slab-on-girder bridges with skew, the increase of shear forces near an obtuse corner as compared to skewless bridges shall be taken into account in accordance with a suitable method.

A5.1.3.2 Bridges curved in plan

For both live and dead loads, longitudinal twisting moments and the associated effects of torsional and distortional warping shall be considered.

If L^2/BR for a bridge as shown in Figure A5.1.2 is not greater than 0.5, the bridge may be treated as straight for the calculation of values in simplified methods of analysis if there are at least two intermediate diaphragms per span.

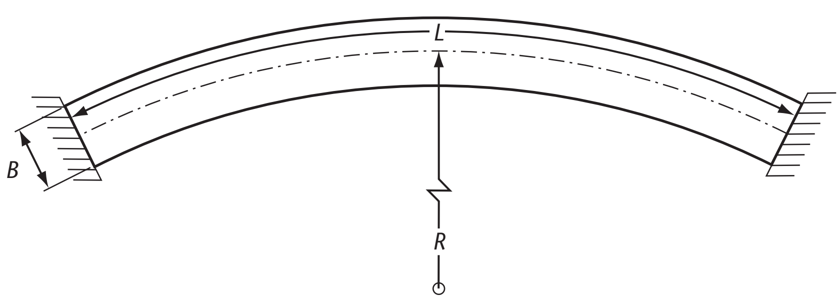


Figure A5.1.2
Bridges curved in plan
(See Clause A5.1.3.2.)

A5.1.3.3 Other plan geometries

When a bridge superstructure with a plan geometry that is not rectangular, skewed, or curved is analyzed, the method of analysis shall be capable of deriving all relevant structural responses and shall be compatible with the requirements specified in Clauses A5.1.3.1 and A5.1.3.2.

A5.1.4 Transverse variation of longitudinal section

A slab that is tapered in the vicinity of its free edges for a distance of up to 2.5 m may be regarded as being of constant thickness if the total width of the slab is conceptually reduced so as to have the cross-sectional area shown in Figure A5.1.3.

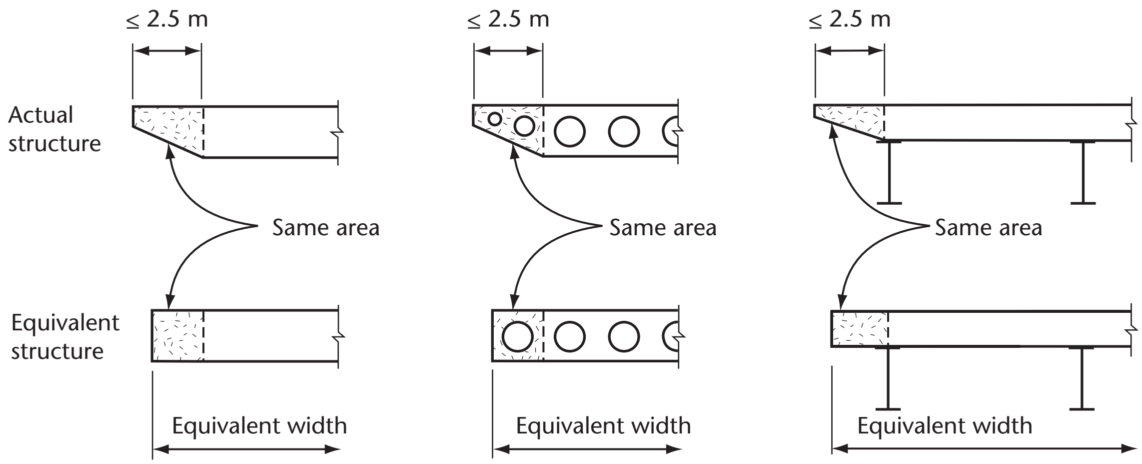


Figure A5.1.3
Idealization of cross-section with a varying thickness
(See Clause A5.1.4.)

A5.1.5 Diaphragms and cross-frames

For bridges of the shallow superstructure type, the effect on the structural responses of diaphragms and cross-frames between supports may be ignored. In the case of box girders, all diaphragms and cross-frames shall be taken into consideration if the number of diaphragms and cross-frames is less than the minimum number required in Table 5.1 when the analysis is based on a method other than the simplified method specified in Clauses 5.6 and 5.7.

A5.1.6 Wind bracing

Engineering judgment shall be used to decide whether the forces in the wind bracing arising from its acting integrally with the rest of the structure need be considered; if so, the analysis shall be able to predict these forces.

A5.1.7 Interaction of floor system and its support system

In truss bridges and arch bridges where the floor system is connected to the trusses or arches in such a way that at least a part of the floor system acts integrally with the trusses or arches, the effective contribution from the floor system may be included.

A5.1.8 Barrier and parapet walls

In cases where the bridge incorporates barrier or parapet walls that are structurally integral with the bridge,

(a) the effect of the barrier or parapet walls shall be ignored in calculating the distribution of loads for ultimate limit states and serviceability limit states;

(b) the barrier or parapet walls may be included in the bridge cross-section in calculating the distribution of loads for the FLS and superstructure vibration; and
(c) the beneficial effect of barrier walls may be included in calculating the distribution of loads for the ULS of deck slabs only.

A5.1.9 Support conditions other than line support

In cases where the support condition includes isolated supports or generates an irregular pattern of support forces, the analysis shall be capable of assessing the local behaviour arising from the support condition.

A5.1.10 Movement of supports and supports for continuous and skew spans

The methods of analysis shall take into account the anticipated support conditions in new bridges and the actual support conditions in existing bridges, either directly or through subsequent adjustments to the results of the analysis. In continuous or skew structures, the analysis shall be capable of taking into account differential settlement.

A5.1.11 Temperature effects

Stresses due to changes in the mean temperature of the bridge or to temperature gradients shall be assessed in accordance with Section 3.

A5.1.12 Creep and shrinkage

The structural response of a bridge superstructure due to the creep and shrinkage effects of concrete shall be provided for in accordance with Sections 3 and 8.

A5.1.13 Secondary force effects and elastic shortening

The influence of secondary force effects and elastic shortening shall be considered. Elastic methods of analysis shall be used for this purpose.

A5.1.14 Construction sequence

Due account shall be taken of the change in nature of the structural system and of changes in material properties that occur during the construction sequence. The behaviour at any stage of the construction sequence shall be analyzed using elastic methods of analysis.

Annex A5.2 (informative)
Two-dimensional analysis

Note: *This Annex is not a mandatory part of this Code.*

A5.2.1 Two-dimensional analysis of steel or concrete superstructures

A5.2.1.1
For two-dimensional analysis, shallow steel or concrete superstructures may be idealized as grillages or orthotropic plates.

A5.2.1.2
For considering the flexural behaviour of the bridge types described in Clause A5.2.1.1 using two-dimensional mathematical models, the following parameters are necessary:

(a) Parameters that depend on the material:

E = modulus of elasticity

ν = Poisson's ratio (taken as 0.15 for concrete and 0.30 for steel)

G = shear modulus

= $E/2(1 + \nu)$

n = modular ratio, E_s/E_c

(b) Parameters that depend on the cross-section:

i_L = longitudinal moment of inertia per unit width

j_L = longitudinal torsional inertia per unit width

i_T = transverse moment of inertia per unit length

j_T = transverse torsional inertia per unit length

s_v = shear area per unit length (needed only for slabs with rectangular voids)

A5.2.1.3
The properties required for analysis as grillage or orthotropic plate may be calculated as specified in Table A5.2.1 and the values of the parameters may be calculated as specified in Table A5.2.2. For both reinforced concrete and prestressed concrete, the uncracked section should be used in calculating these parameters. The factors F_1 and F_2 indicated in Table A5.2.2 for voided slabs may be obtained from Figure A5.2.1. In other cases, the torsional inertia of a section may be calculated by dividing the section into a number of rectangles and adding the torsional inertias of all rectangles. The torsional inertia of a single rectangle with sides a and b may be taken to be $J = Kab^3$, where a is the longer and b the shorter of the two sides of the rectangle and K is a constant depending on the ratio of a/b, which can be interpolated from Table A5.2.3.

A5.2.1.4
In the absence of a more detailed analysis, the equivalent areas for in-plane analysis of slabs with circular voids may be taken as follows:

(a) A_x = the equivalent area of the transverse section, per unit width

$$= t - \frac{\pi(t_v)^2}{4S}$$

(b) A_y = the equivalent area of the longitudinal section, per unit length

$$= t\left[1 - 0.5\left[\frac{t_v}{t}\right]\left[1 + \frac{t_v}{t}\right] + 0.1\left[1.7 - \frac{t_v}{t}\right]\left[\frac{S - t_v}{t}\right]\right]$$

Table A5.2.1
Properties of idealized orthotropic plate or grillage
(See Clause A5.2.1.3.)

	Properties to define a two-dimensional orthotropic plate	Grillage beam properties to define a two-dimensional orthogonal grillage
Longitudinal direction	$D_x = Ei_L$	Moment of inertia, $I_x = i_L \times$ (longitudinal grillage beam spacing)
	$D_{xy} = Gj_L$*	Torsional inertia, $J_x = j_L \times$ (longitudinal grillage beam spacing)
	$D_1 = \nu \times$ (lesser of D_x or D_y)	No equivalent to D_1
Transverse direction	$D_y = Ei_T$	Moment of inertia, $I_y = i_T \times$ (transverse grillage beam spacing)
	$D_{yx} = Gj_T$*	Torsional inertia, $J_y = j_T \times$ (transverse grillage beam spacing)
	$D_2 = D_1$	No equivalent to D_2
	$S_y = Gs_v$†	Transverse shear area = $s_v \times$ (transverse grillage beam spacing)

**There is a lack of consistency in the application of this value in various analyses. When the analysis uses the following expression to calculate of M_{xy}, the values of D_{xy} and D_{yx} are calculated as specified in this Table:*

$$M_{xy} = D_{xy}\frac{\partial^2 w}{\partial xy}$$

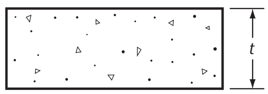
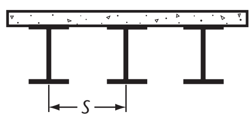
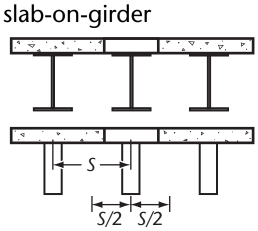
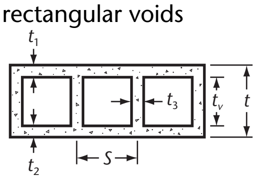
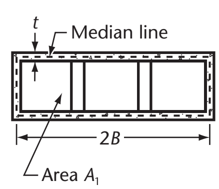
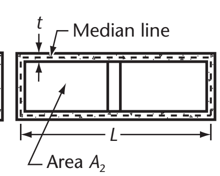
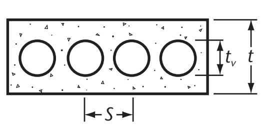
However, when the following expression for calculating M_{xy} is used, the values of D_{xy} and D_{yx} are taken as half those specified in this Table:

$$M_{xy} = 2D_{xy}\frac{\partial^2 w}{\partial xy}$$

For grillage properties, the expressions specified in this Table for torsional inertia are always correct as stated.
†Required only for voided slabs with rectangular voids.

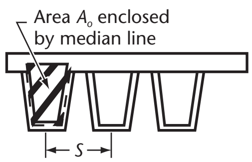
Table A5.2.2
Expressions for structural parameters

(See Clause A5.2.1.3.)

Bridge type and transverse section	Structural parameters				
	i_L	i_T	j_L	j_T	s_v
Slab 	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{6}$	$\frac{t^3}{6}$	May be ignored
Non-composite slab-on-girder 	$\frac{t^3}{12}+\frac{I_{xt}}{S}$ where I_{xt} = transformed moment of inertia of the girder about its own x-axis	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{6}$	$\frac{t^3}{6}$	May be ignored
Composite slab-on-girder 	$\frac{I_x}{S}$ where I_x = the combined transformed moment of inertia of slab portion located in width S	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{6}+\frac{J}{S}$ where J = transformed torsional inertia of the girder multiplied by $n_g = 0.88n$ for steel portions	$\frac{t^3}{6}$	May be ignored
Voided slab with rectangular voids 	$\frac{t^3-t_v^3}{12}$	$\frac{t^3-t_v^3}{12}$	$\frac{2A_1^2}{B\sum\frac{ds}{t}}$ Intermediate webs ignored	$\frac{2A_2^2}{L\sum\frac{ds}{t}}$ Intermediate diaphragm, if any, ignored	*
Voided slab with circular voids 	$\frac{t^3}{12}-\frac{\pi t_v^4}{64S}$	$\frac{F_1 t^3}{12}$ (F_1 from Figure A5.2.1)	$\frac{F_2 t^3}{6}$ (F_2 from Figure A5.2.1)	$\frac{F_2 t^3}{6}$ (F_2 from Figure A5.2.1)	May be ignored

(Continued)

Table A5.2.2 (Concluded)

Bridge type and transverse section	Structural parameters				
	i_L	i_T	j_L	j_T	s_v
Multi-spine girder 	$\frac{I_x}{S}$ where I_x = the combined transformed moment of inertia of slab portion located in width S	$\frac{t^3}{12\left(1-v^2\right)}$ for the portion of the deck between the spines. The value of i for the portion of the deck included in the spine is calculated by considering the total transverse stiffness of the spine, including that of the bracing and diaphragms within the box.	$\frac{4A_0^2}{S\sum\frac{ds}{n_g t}}$ where n_g = 1.0 for concrete portions = 0.88n for steel portions	$\frac{t^3}{6}$	May be ignored for slab between spines. See note (*) for portion within spine. The stiffness of internal braces in spines may be included.

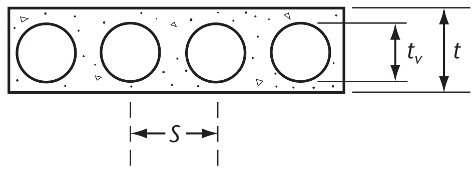
$$^{*}\ s_v = \left[\frac{t_1^3 + t_2^3}{S^2}\right]\frac{E}{G}\left[\frac{t_3^3 S}{S t_3^3 + \left(t_1^3 + t_2^3\right)\left[\frac{t + t_v}{2}\right]}\right]$$ *for voided slabs with rectangular voids.*

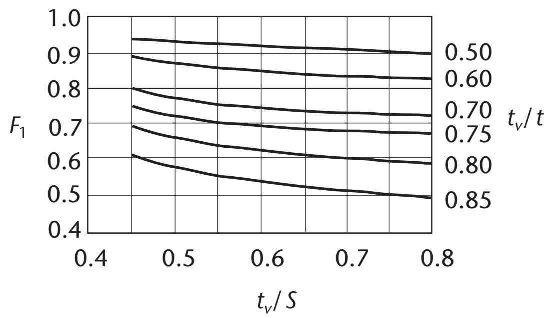
Note: *All parameters are in terms of deck slab concrete units.*

Table A5.2.3
Torsional constant, *K*, for rectangular sections where $a \geq b$

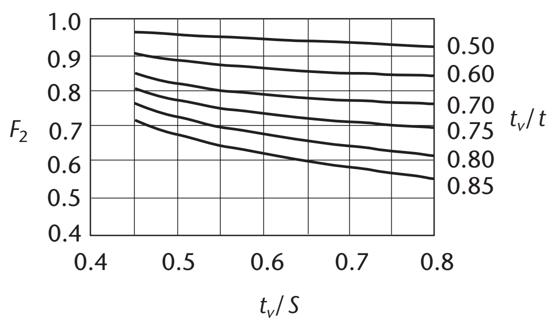
(See Clause A5.2.1.3 and Table A5.2.4.)

a/b	1	1.2	1.5	2.0	2.5	3.0	4.0	5.0	10.0	$\geq$ 420
K	0.141	0.166	0.196	0.229	0.249	0.263	0.281	0.291	0.312	0.333





Coefficients for D_y



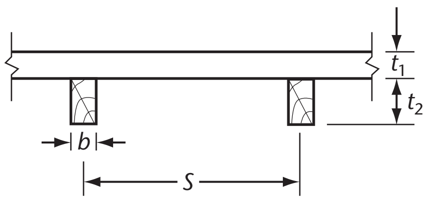
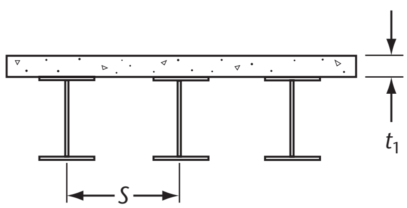
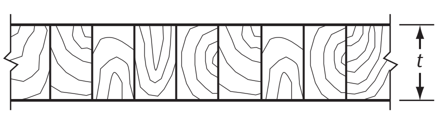
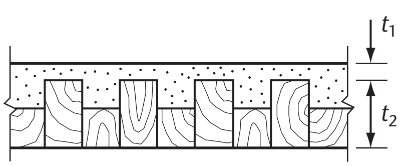
Coefficients for D_{xy}

Figure A5.2.1
Modification factors for voided slabs
(See Clause A5.2.1.3 and Table A5.2.2.)

A5.2.2 Two-dimensional analysis of wood floor systems

The six structural parameters needed to idealize a wood floor system for two-dimensional analysis may be calculated as specified in Table A5.2.4. The footnote to Table A5.2.4 applies to the calculation of D_{xy}. In the absence of actual material properties, a value of 9600 MPa for E_L needs to be employed for load distribution analysis of wood floor systems. The effective values of E_T and G_{LT} need to be $0.015E_L$ and $0.030E_L$, respectively. E_T is the modulus of elasticity in the principal direction L shown in Figure A5.2.2 and G_{LT} is the shear modulus in the LT plane. For analysis of elastic shortening due to load perpendicular to the grain, E_T needs to be taken as $0.05E_L$ in the absence of actual properties.

Table A5.2.4
Calculation of structural parameters for bridges incorporating wood beams
(See Clause A5.2.2.)

Bridge type and transverse section	Structural parameters				
	D_x	D_y	D_{xy}	D_{yx}	$D_1 = D_2$
Transverse laminated wood decks on longitudinal wood beams 	$\frac{E_L b t_2^3}{12S}$	$\frac{E_L t_1^3}{12}$	$\frac{G_{LT} K t_2 b^3}{S}$ *	0.0	0.0
Transverse laminated decks on longitudinal steel beams 	$\frac{E_s I}{S}$ where I = moment of inertia of a girder in steel units	$\frac{E_L t_1^3}{12}$	0.0	0.0	0.0
Glue-laminated and transversely laminated prestressed decks 	$\frac{E_L t^3}{12}$	$\frac{E_T t^3}{12}$	$\frac{G_{LT} t^3}{6}$	$\frac{G_{LT} t^3}{6}$	0.0
Composite concrete slabs on longitudinally laminated wood decks 	$E_c I$ where I = combined moment of inertia of a unit width of concrete and wood using a modular ratio of E_c/E_L	$\frac{E_c t_1^3}{12}$	$\frac{G_c t_1^3}{6}$	$\frac{G_c t_1^3}{6}$	νD_y

**K* is obtained from Table A5.2.3.*

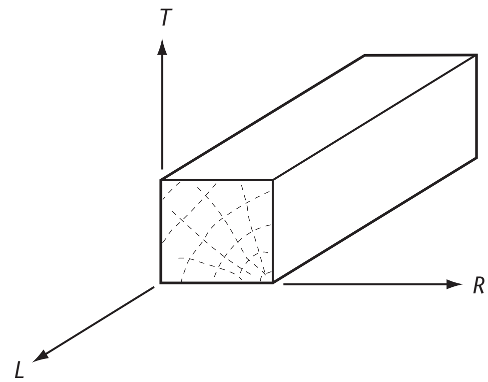


Figure A5.2.2
Principal directions in wood specimen
(See Clause A5.2.2.)

Section 6 — Foundations

Section 6
Foundations

6.1 Scope

This Section specifies minimum requirements for the design of foundations and the estimation of earth pressures on retaining structures, including requirements pertaining to geotechnical investigations and design reports. This Section does not apply to buried structures that fall within the scope of Section 7 or to the design of structures in permafrost, and is not mandatory for the design of temporary structures.

Note: *See Figure A3.1.5 for a permafrost region map of Canada.*

6.2 Definitions

The following definitions apply in this Section:

Active pressure — the lateral earth pressure exerted on a retaining structure when it is able to move away from the backfill by an amount sufficient to mobilize the soil strength fully.

Assessed value — a value determined through assessment.

Assessment — the estimation of resistance and deformation values for a site by reference to values established for other sites known to have similar stratigraphy.

At-rest pressure — the lateral earth pressure within soil before it is displaced or excavated.

Backfill — the fill retained by a structure, including fill Approved for use as engineered fill, e.g., earth backfill, rock fill, slag, and polystyrene. Backfill also includes retained materials such as in-situ soil or rock.

Bearing surface — the contact surface between a foundation or component and the soil or rock on which it bears.

Bond length — the portion of a ground anchor that transmits the tendon force to the surrounding soil or rock.

Deep foundation — a foundation that transfers load to soil or rock through a combination of toe bearing and shaft resistance at a depth exceeding three times the effective pile width below the surface of backfill or original ground level. The minimum depth for a deep foundation is 3 m below the base of the pile cap.

Deformation — the total or differential movement of a foundation, consisting of one or more of settlement, heave, horizontal displacement, and rotation.

Double corrosion protection (in relation to ground anchors) — a system of double covering of the tendon to protect against corrosion, normally consisting of encapsulation within a sealed tube that is encased in an outer tube filled with grout.

Downdrag load — the load transferred to a deep foundation unit when the surrounding soil settles in relation to it.

Dynamic analysis — calculation of the impact force, driving resistance, and energy of a pile by wave propagation theory without the use of field measurements.

Dynamic test — determination of the resistance, impact force, and developed driving energy of a driven pile by analysis of the measured strain induced by the driving of the pile.

Effective height (in relation to a retaining structure) — the overall vertical dimension of the surface over which horizontal earth pressure is assumed to act.

Factored geotechnical resistance at ULS — the product of the resistance factor and the ultimate soil or rock resistance.

Free-stressing length — the portion of the ground anchor tendon that is free to elongate during stressing.

Geotechnical Engineer — an Engineer or foundation engineering specialist responsible for the work related to soil and rock, including site investigation, foundation recommendations, inspection, and quality control.

Geotechnical reaction at SLS — the reaction of the soil or rock at the deformation associated with an SLS condition.

Geotechnical report — a report prepared by the geotechnical Engineer to satisfy the requirements of Clause 6.5.

Geotechnical resistance at ULS — the resistance of soil or rock corresponding to a failure mechanism predicted from theoretical analysis using unfactored geotechnical parameters obtained from testing or estimated from assessed values.

Ground anchor — a structural component installed in soil or rock to resist loads transferred to it in tension.

Ground anchor tendon — an assembly consisting of prestressing steel, a corrosion protection system, and an end anchorage.

Groundwater — a free body of water in the ground.

> **Artesian groundwater** — groundwater, in a confined aquifer, under pressure that results in its hydrostatic elevation being higher than the elevation of the top of the confined aquifer at the location of measurement.
>
> **Groundwater level (groundwater table)** — the top surface of a free body of water in the ground.

Lockoff load — the load in a ground anchor immediately after the load has been transferred from the jack to the stressing anchorage.

Long-term deformation — the time-dependent deformation in soil or rock occurring as a result of consolidation, creep, or both.

Passive resistance — the resistance occurring as a result of the movement of a retaining structure, footing, or pile toward backfill, soil, or rock.

Pile — a deep foundation unit wholly or partially embedded in the ground and installed by casting-in-place, driving, augering, jetting, or other means.

Post-grouting — the pressure grouting of the bond length of a ground anchor after the initial bond grout has set.

Relaxation — a reduction in the resistance of a pile over time due to the dissipation of pore water pressure.

Restrained structure — a wall, abutment, or other retaining structure that cannot move sufficiently to mobilize active earth pressure.

Shallow foundation — a foundation in which a footing transfers load directly to the soil- or rock-bearing surface, normally at a depth less than the effective footing width.

Short-term deformation — the deformation in soil or rock that occurs on the application of load.

Temporary structure — a structure with a service life of less than two years.

Transfer load — see **Lockoff load**.

Unrestrained structure — a wall, abutment, or other earth-retaining structure that can move by an amount sufficient to mobilize active pressure in the retained soil.

6.3 Abbreviations and symbols

6.3.1 Abbreviations

The following abbreviations apply in this Section:

MSE — mechanically stabilized earth

SLS — serviceability limit state

ULS — ultimate limit state

6.3.2 Symbols

The following symbols apply in this Section:

A' = effective contact area, m^2

A'_s = effective peripheral area of the pile shaft within the supporting stratum, m^2

A'_t = effective cross-sectional area of the pile tip, m^2

B = width of a shallow foundation, m

B' = effective width of a shallow foundation, m

b = equivalent diameter of a deep foundation unit, taken as the diameter of a round pile or as the face-to-face dimension of an octagonal, hexagonal, or square pile, m

c = undrained shear strength, kPa

c' = effective cohesion, kPa

D = embedment depth of a shallow foundation, m

e_B = eccentricity of load from the centroid of the footing in the short direction, m

e_L = eccentricity of load from the centroid of the footing in the long direction, m

H = unfactored horizontal force, kN

H_f = factored horizontal load, kN

H_{ri} = factored horizontal shear resistance of the interface between the foundation and the soil, kN

H_{rs} = factored horizontal shear resistance of the soil, kN

i_c = inclination factor associated with N_c

i_q = inclination factor associated with N_q

$i\gamma$ = inclination factor associated with N_γ

L = length of footing or pile, m

L' = effective length of footing or pile, m

N_c = bearing coefficient for cohesion

N_q = bearing coefficient for overburden pressure
N_γ = bearing coefficient for soil weight
Q = applied load, kN
q = applied pressure, kPa
q' = effective overburden pressure at the foundation level, kPa
q_u = ultimate geotechnical pressure resistance, kPa
R_u = ultimate resistance of a deep foundation unit, kN
r_s = ultimate unit shaft resistance within supporting stratum, kPa
r_t = ultimate unit toe resistance, kPa
s_c = shape factor associated with N_c
s_q = shape factor associated with N_q
s_γ = shape factor associated with N_γ
V = unfactored vertical force, kN
z = depth below ground surface, m
γ = unit weight, kN/m^3
γ' = effective unit weight, kN/m^3
δ_i = angle of inclination of force from the vertical, degrees
ϕ = angle of internal friction, degrees
ϕ' = effective angle of internal friction, degrees

6.4 Design requirements

6.4.1 Limit states

6.4.1.1 General

Foundations and retaining structures shall be proportioned to satisfy the requirements of this Section at the SLS and ULS. The design process shall include a consideration of the manner in which a structure and the supporting soil or rock will approach a limit state.

6.4.1.2 Ultimate limit state

The ULS conditions to be considered shall include those in which a failure mechanism forms in the soil or rock and those in which loss of static equilibrium or rupture of a portion of the structure occurs because of deformation of the soil or rock.

The following shall be considered both singly and in combination:
(a) overall stability of a foundation and of any adjacent slope;
(b) bearing resistance;
(c) pullout or uplift resistance; and
(d) sliding, horizontal shear resistance, and passive resistance.

6.4.1.3 Serviceability limit state

The SLS conditions to be considered shall be those causing the structure to become unserviceable and shall include the following:
(a) foundation deformations that cause SLS limitations for the structure to be exceeded;
(b) deformations that cause the riding surface or transitions between the approaches and the bridge superstructure to become unacceptable; and
(c) deformations that cause unacceptable structure misalignment, distortion, or tilting.

6.4.2 Effects on surroundings

Changes that could occur at or near the site during and after construction shall be investigated during design. Effects on existing structures and on the adjacent ground to be considered shall include
(a) changes in bearing and sliding resistance due to excavation or ground disturbance;
(b) changes in groundwater level;
(c) effects of blasting and pile driving;
(d) effects of soil compaction;
(e) effects of load changes on pressures within subsurface layers; and
(f) effects of temperature changes, e.g., by heating or freezing.

6.4.3 Effects on structure

For the appropriate limit state, consideration shall be given to all loads, imposed deformations, and foundation and component deformations. The variability and interdependence of these at various times during the design life of the structure shall be taken into account.

The effects to be considered shall include
(a) groundwater effects, including seepage, piping, and subsurface erosion;
(b) forces due to lateral and vertical soil movements;
(c) dynamic effects, including earthquakes and blasting;
(d) frost penetration;
(e) the variability of soil and rock strata;
(f) scour and excavation; and
(g) backfill compaction.

6.4.4 Components

All footings, foundation components, and retaining structure components, including deep foundation units, ground anchors and ties, pole footings, and all components of crib and bin walls and MSE systems, shall be considered structural components and shall comply with the applicable requirements of this Code for the material in question.

6.4.5 Consultation

Consultation between the structural Engineer and the geotechnical Engineer shall take place during planning, design, and construction. The geotechnical Engineer shall review the geotechnical aspects of the Plans before construction.

6.4.6 Inspection and quality control

During construction, deep and shallow foundations, MSE, and ground anchors shall be inspected by the geotechnical Engineer to confirm that the site conditions are consistent with the design assumptions and to ensure that the geotechnical aspects of the work are carried out as intended. The results of the inspection and of observations at the site shall be documented.

6.5 Geotechnical investigation

6.5.1 General

A geotechnical investigation shall be conducted to assess the suitability of the site for the proposed structure. Preliminary design information shall be established before and during the geotechnical investigation. To the extent practicable, this information shall include the following:
(a) the type of structure;
(b) the probable substructure locations;
(c) the minimum footing depths;
(d) the approximate magnitude and direction of foundation loads;
(e) the approximate acceptable short-term and long-term foundation deformations; and

(f) the changes to the site and the surrounding area that could be caused by the structure during and after construction.

The investigation shall be of sufficient scope to provide information on the subsurface conditions and to verify the assumptions made for the design and construction of the structure and associated approaches.

The investigation shall include a subsurface exploration of sufficient depth to identify any stratum that could affect the performance of the proposed structure and approaches, and shall provide necessary information for design and construction, including appropriate geotechnical parameters to be used in design.

The site investigation shall establish the geology, geomorphology, and hydrogeology of the site, determine whether the chemical nature of the soil, bedrock, and groundwater could affect the durability of the structure or its foundation units, provide details of matters requiring inspection during construction, and, where appropriate, provide requirements for post-construction observations.

6.5.2 Investigation procedures

Site investigations, field tests, and laboratory testing shall be carried out in accordance with recognized or standardized procedures. The procedures used shall be documented.

6.5.3 Geotechnical parameters

Geotechnical parameters shall be appropriate to the nature of the soil, rock, or anticipated backfill, the mode of failure or deformation being considered, and the variability of the soils and rock occurring at the site. Test procedures that will ensure appropriate accuracy shall be used.

6.5.4 Shallow foundations

For shallow foundations, values of the factored geotechnical resistance at the ULS for the probable depths of embedment and footing sizes shall be determined. Geotechnical reactions at the SLS for associated deformation values shall be estimated in accordance with Clause 6.6.3.

6.5.5 Deep foundations

For deep foundations, values of the factored geotechnical resistance at the ULS for the appropriate types and lengths of deep foundation units shall be determined. Installation procedures and group effects shall also be investigated. Foundation loads at SLS for associated deformation values shall be estimated in accordance with Clause 6.6.3.

6.5.6 Report

The geotechnical report shall discuss and provide recommendations related to the following, as applicable:

(a) the procedures used in the investigation;
(b) the geology, geomorphology, and hydrogeology of the site;
(c) the surface and subsurface conditions at the site (to be described in detail);
(d) the groundwater elevations, including anticipated fluctuations;
(e) representative values of the geotechnical parameters;
(f) the types, depths, and widths of foundations;
(g) the factored geotechnical resistance at the ULS for shallow foundations or deep foundation units or groups;
(h) the deformations for anticipated SLS loads and the relevant range of footing sizes and typical pile configurations. When estimates of SLS loads are not available, SLS reactions for a range of corresponding estimated deformations shall be provided. This range shall include foundation settlements of 25 and 50 mm;
(i) the effect on the geotechnical design of the construction of any associated works;
(j) the values of earth pressures and corresponding values of soil or rock parameters for the design of retaining structures;
(k) the construction and inspection measures required during construction and any special monitoring requirements related to the performance of the structure;

(l) the geotechnical implications for adjacent property of the proposed structure and its construction;
(m) the impact of events such as landslides and earthquakes on the structure;
(n) the stability and settlement of approaches to the structure; and
(o) erosion and frost protection requirements.

Unless otherwise Approved, the report shall be signed and sealed by two Engineers, one of whom shall be a principal Engineer.

6.6 Resistance and deformation

6.6.1 General

Geotechnical resistances or reactions used in the design shall ensure acceptable performance of the structure at both the ULS and the SLS. The methods of analysis shall include consideration of the duration of the loading and construction sequence.

When site-specific geotechnical parameters are chosen for calculation of geotechnical resistance at the ULS and geotechnical reactions at the SLS, the variability of the conditions at the site, and the type of foundation and construction sequence, shall be considered.

6.6.2 Ultimate limit state

6.6.2.1 Procedures

The geotechnical resistance at the ULS of shallow or deep foundations and anchors shall be determined from calculations, field tests, or assessments for a given soil or rock at a specific site. Unfactored geotechnical parameters shall be used to determine the geotechnical resistance at the ULS. The factored geotechnical resistance at the ULS of a deep or shallow foundation shall be the ultimate geotechnical resistance multiplied by the relevant resistance factor specified in Table 6.1, unless a higher value is Approved.

6.6.2.2 Geotechnical formulas

The geotechnical formulas used for calculating ultimate resistance shall be appropriate to the soil and rock conditions at the site.

6.6.2.3 In-situ tests

The parameters for calculation of geotechnical resistance at the ULS of a shallow foundation, anchor, pile, or group of piles may be determined from in-situ tests at the site. The factored resistance at the ULS shall be the ultimate geotechnical resistance obtained from the in-situ tests using an Approved method of interpretation and multiplied by the resistance factor specified in Table 6.1.

Table 6.1
Geotechnical resistance factors
(See Clauses 6.6.2.1, 6.6.2.3, 6.10.2.2, and 6.13.2.3.)

Application	Resistance factor
Shallow foundations	
Bearing resistance	0.5
Passive resistance	0.5
Horizontal resistance (sliding)	0.8
Ground anchors (soil or rock)	
Static analysis — Tension	0.4
Static test — Tension	0.6
Deep foundations — Piles	
Static analysis	
Compression	0.4
Tension	0.3
Static test	
Compression	0.6
Tension	0.4
Dynamic analysis — Compression	0.4
Dynamic test — Compression (field measurement and analysis)	0.5
Horizontal passive resistance	0.5

6.6.2.4 Assessed value
Provided that suitable geotechnical data, including the detailed stratigraphy, have been obtained from the site, ultimate geotechnical resistance may be estimated based on extrapolation of foundation performance under similar site conditions.

6.6.3 Serviceability limit state

6.6.3.1 General
The SLS to be considered shall be those of the short-term and long-term total and differential deformations. The simultaneous occurrence of several types of deformation shall be considered.

6.6.3.2 Calculations
The methods used for calculating SLS deformations and reactions shall employ unfactored geotechnical parameters appropriate to the site conditions.

6.6.3.3 Tests
The time dependency of deformations shall be considered in planning the in-situ tests and interpreting test results to determine geotechnical reaction at the SLS.

6.6.3.4 Assessed values
When applicable geotechnical data, including the detailed stratigraphy at the site, are available, measurements of actual deformation at sites with similar stratigraphy may be used to determine the geotechnical reactions and deformations at the SLS.

6.6.3.5 Loads for SLS analysis

Unfactored permanent and transitory loads shall be used for calculating total deformation in non-cohesive soils. Permanent loads and appropriate proportions of transitory loads shall be considered for the initial and time-dependent final deformations of cohesive soils.

6.6.3.6 Calculation considerations

In calculating or predicting short-term and long-term deformations for the geotechnical reaction at the SLS, the following shall be taken into account:

(a) the sequence of construction and changes in soil parameters as a consequence of construction;
(b) observations of deformation of substructures in similar subsurface conditions;
(c) the influence of soil variability, including layering;
(d) induced stress and strain levels in relation to the geotechnical resistance at the ULS and preconsolidation pressure;
(e) permeability, drainage, water content, and pore pressure;
(f) the magnitude of the strains in the soils associated with the deformations; and
(g) consolidation, creep, swelling, or collapse characteristics.

A range of possible values shall be considered when values of deformation are to be used in structural analysis.

6.7 Shallow foundations

6.7.1 General

The requirements of Clauses 6.7.2 to 6.7.5 shall apply to shallow foundations, including combined footings and mats, isolated footings, and wall footings.

6.7.2 Calculated geotechnical resistance at ULS

The geotechnical resistance at ULS for a concentrically loaded footing founded in a uniform soil stratum, as shown in Figure 6.1, shall be calculated from the following or an alternative Approved method:

$$q_u = cN_c s_c i_c + q'N_q s_q i_q + 0.5\gamma' BN_\gamma s_\gamma i_\gamma$$

The parameters used for analysis shall be stated.

When the load is eccentric, the footing shall be considered to have an effective concentrically loaded area of width B' and length L' in accordance with Figure 6.2, where for a load, Q, the stress, q, is given by

$$q = Q/B'L'$$

where

B' = $B - 2e_B$, but is less than L'

L' = $L - 2e_L$

The dimensionless bearing coefficients, N_c, N_q, and N_γ, depend only on the value of the effective internal friction angle, ϕ', and are as shown in Figure 6.3.

In the bearing resistance equation, shape factors that account for the width-to-length ratio of footings shall be calculated from

s_c = $1 + (B'/L')(N_q/N_c)$

s_q = $1 + (B'/L')(N_q/N_c)$

s_γ = $1 - 0.4(B'/L')$

The effects of a load inclination shall be accounted for by applying inclination factors as follows:

i_c = $(1 - \delta_i/90°)^2$

i_q = $(1 - \delta_i/90°)^2$

i_γ = $(1 - \delta_i/\phi')^2$

where

δ_i = angle of the resultant force with respect to the vertical

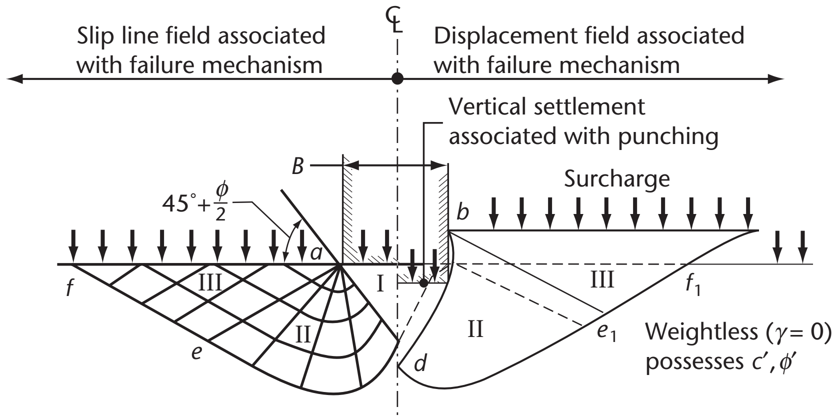
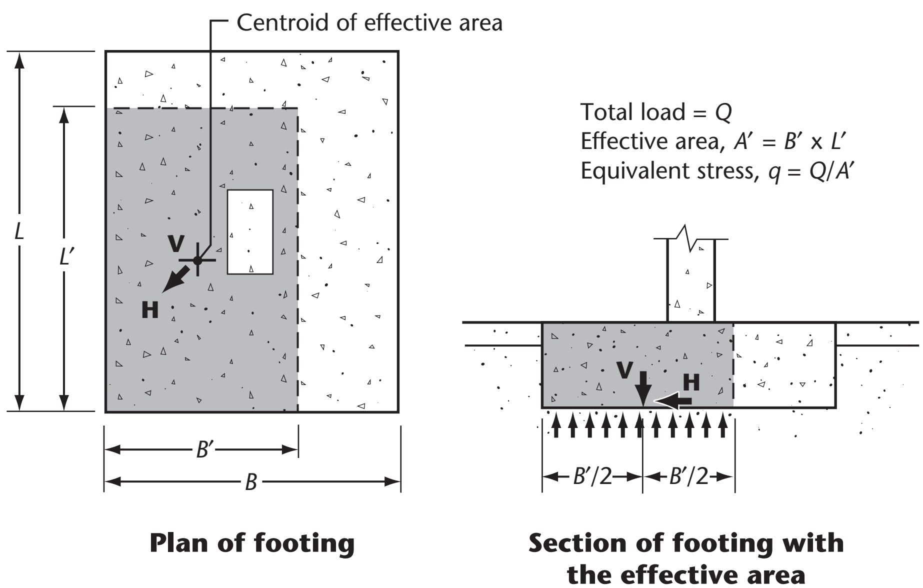


Figure 6.1
Failure mechanism for footing
(See Clause 6.7.2.)



Note: *Values are factored and apply to the ULS.*

Figure 6.2
Footing under eccentric load
(See Clause 6.7.2.)

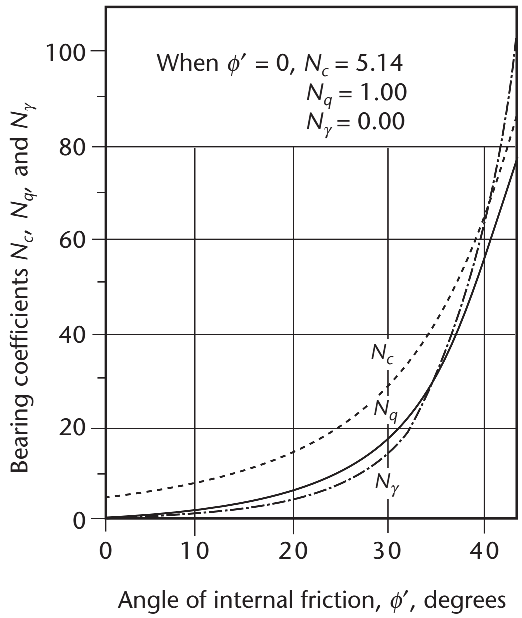


Figure 6.3
Bearing coefficients
(See Clause 6.7.2.)

6.7.3 Pressure distribution

6.7.3.1 Effective area

For proportioning of concentrically loaded footings, a contact pressure of uniform intensity at the ULS shall be assumed.

For eccentrically loaded footings, an equivalent effective area with a contact pressure of uniform intensity shall be assumed such that the centroid of the area coincides with the vertical component of the factored load.

6.7.3.2 Pressure distribution at the ULS for structural design

For the structural design of footings, the more critical of the following shall be considered:

(a) a uniform pressure distribution whose magnitude shall not be more than the factored geotechnical resistance; or

(b) a linear pressure distribution where the maximum bearing pressure could be greater than the factored geotechnical resistance.

6.7.3.3 Pressure distribution at the SLS

A linear distribution of contact pressure at the SLS shall be assumed. Tension at the interface between the footing and the soil or rock shall not be assumed.

6.7.3.4 Eccentricity limit

In the absence of detailed analysis at the ULS for soil or rock, the eccentricity of the resultant of the factored loads at the ULS acting on a foundation, as shown in Figure 6.4, shall not exceed 0.30 times the dimension of the footing in the direction of the eccentricity being considered.

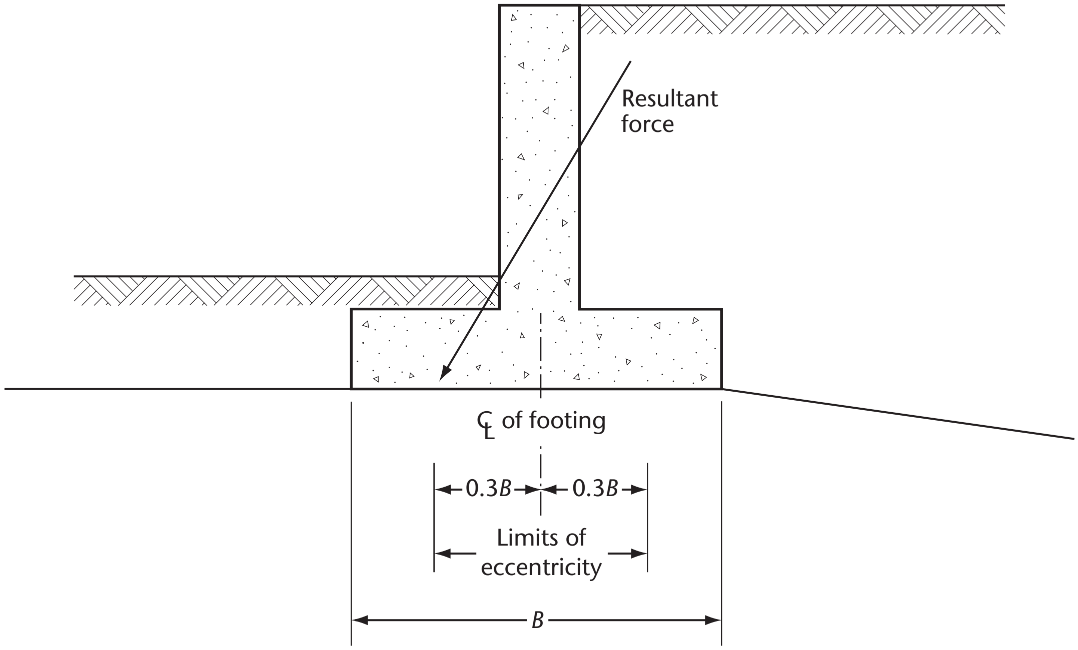


Figure 6.4
Eccentricity limit
(See Clause 6.7.3.4.)

6.7.4 Effect of load inclination

The inclination of the factored load shall be considered when the bearing resistance of shallow foundations is being determined. When the geotechnical resistance values are given for vertical forces, either the inclination reduction factors specified in Clause 6.7.2 or the reduction factors specified in Figure 6.5 shall apply in calculating the effect of load inclination. The factors specified in Figure 6.5 shall apply to the vertical factored geotechnical resistance only for embedment-to-width ratios *(D/B′)* greater than 0.125 and for ratios of horizontal force to vertical force less than 0.55. The effects of load inclination for shallow foundations on rock shall be analyzed, taking into account any weaknesses in the rock.

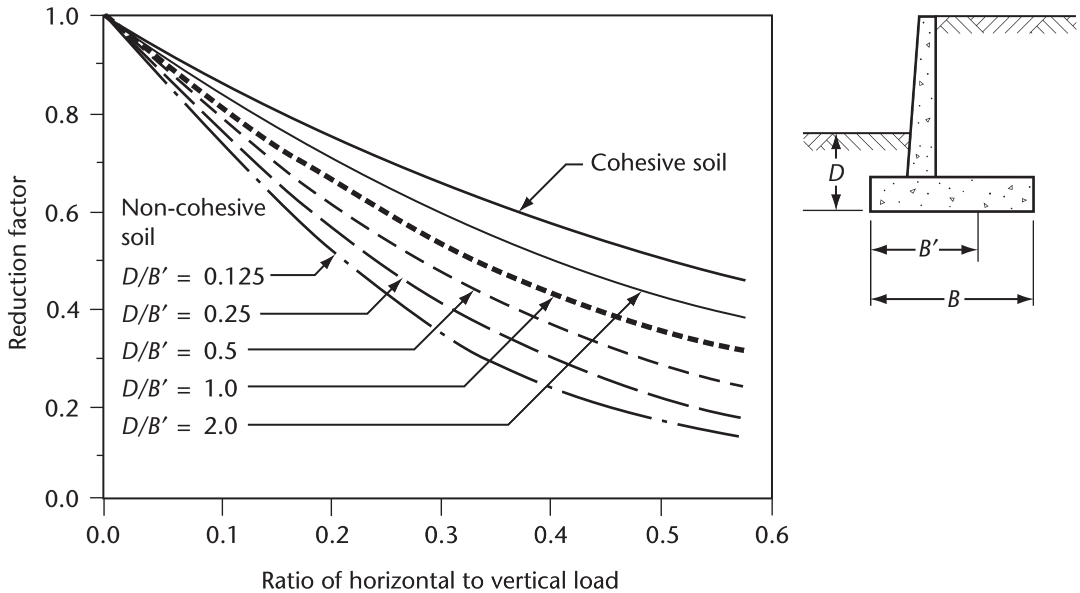


Figure 6.5
Load inclination reduction factors for bearing resistance, $\phi' = 30°$
(See Clause 6.7.4.)

6.7.5 Factored geotechnical horizontal resistance

The factored geotechnical horizontal resistance shall not be less than the horizontal component of the factored load. The factored geotechnical horizontal resistance shall be taken as the lesser of the factored horizontal shear resistance of the soil or rock below the footing and the factored horizontal shear resistance of the interface between the footing and the soil or rock. Where appropriate, the factored horizontal passive resistance of the soil or rock shall be included.

In the absence of a detailed analysis, the following shall be used to calculate the factored geotechnical horizontal resistance within the soil close to the soil-structure interface:

$$H_{rs} = 0.8A'c' + 0.8V \tan \phi' > H_f$$

In the absence of a detailed analysis, the following shall be used to calculate the factored geotechnical horizontal shear resistance at the interface between the footing and the soil or rock:

$$H_{ri} = 0.8A'c' + 0.8V \tan \delta_i > H_f$$

The effective cohesion, c', shall be zero in the absence of detailed test data. The effective contact area, A', shall be the smallest area required to carry the minimum vertical loads.

When the subgrade soil is clay, the short-term case shall also be checked using $\tan \phi' = 0$ and c' equal to either the undrained shear strength or adhesion at the interface.

When the passive pressure resistance of the soil or rock in front of the wall or some portion thereof is considered as contributing to resistance, the soil or rock properties and the acceptability of the movement required to develop the passive condition shall be considered.

The presence of planes of weaknesses and discontinuities in the soil or rock beneath a foundation and the effects of buoyancy and seepage shall be considered in determining horizontal resistance.

Sliding resistance for footings placed on smooth or inclined bedrock surfaces shall be supplemented by keys, dowels, or sockets unless horizontal resistance and stability can be ensured by other means.

6.8 Deep foundations

6.8.1 General

The requirements of this Clause apply to vertical and inclined piles acting as single units or in a group.

6.8.2 Selection of deep foundation units

For deep foundations the following shall be considered:

(a) the suitability of the type of pile;
(b) the reliability of the soil or rock in providing the required resistance;
(c) the durability of the pile material;
(d) the movement of the soil surrounding the piles;
(e) scour, future dredging, or excavation;
(f) the effect of groundwater on the installation of piles;
(g) the existence of sloping bedrock, boulders, or construction debris;
(h) the ductility of the pile in seismic areas; and
(i) the effects of frost.

6.8.3 Vertical load transfer

All loads on a deep foundation shall be assumed to be transferred to the underlying strata by the deep foundation units and any contribution arising from direct bearing of the footing on the soil shall be neglected.

6.8.4 Downdrag

Downdrag on piles caused by settlement of the surrounding soil shall be considered a load and shall be specified by the geotechnical Engineer.

For the purposes of calculation, downdrag effects on piles shall be considered to be those of settlement and structural resistance. Downdrag loads shall be evaluated by the geotechnical Engineer by taking into account the specific site conditions. If neutral plane concepts are used, the location of the plane of zero relative movement between the soil and the pile for a pile or group of piles shall be determined by using unfactored loads and unfactored geotechnical parameters.

The downdrag load, along with other loads, shall be applied to the pile or pile group using load factors specified in Section 3.

6.8.5 Factored geotechnical axial resistance

6.8.5.1 General

The methods used to establish and verify the geotechnical axial resistance shall be appropriate to the site, to the soil or rock conditions, to the type of deep foundation unit, and to the proposed method of installation. At least one of the following methods shall be used:

(a) static analysis;
(b) static pile load test;
(c) dynamic analysis (for compression only);
(d) dynamic pile test (for compression only); and
(e) assessed value.

For driven piles, an assessment shall be made to determine that the piles can be installed to design depth and provide the design resistance without inducing damaging stresses. The factored geotechnical axial resistance shall not exceed the factored structural resistance of the pile.

6.8.5.2 Static analysis

The geotechnical resistance at ULS of piles shall be calculated as follows:

$$\begin{aligned} R_u &= \text{total shaft resistance} + \text{toe resistance} \\ &= \sum A_s' r_s + A_t' r_t \end{aligned}$$

where

$A_s' r_s$ = ultimate shaft resistance, kN

$A_t' r_t$ = ultimate pile toe resistance, kN

$\sum A_s' r_s$ indicates the summation over the length of the pile in those strata considered as contributing to the shaft resistance. For re-entrant surfaces of a pile shaft, e.g., H-piles, the shaft resistance shall be taken as the lesser of the soil-pile interface resistance and the shear resistance on a plane through the soil joining the re-entrant corners.

6.8.5.3 Static pile load tests
Static test loads applied to a pile shall normally be measured to an accuracy of ± 2% and vertical displacement to a precision of ± 0.025 mm.

6.8.5.4 Dynamic analysis and tests
The dynamic analysis and tests shall consider the hammer-pile-soil systems proposed for or used at the site.

6.8.5.5 Limitation for tension piles
In determining the geotechnical axial resistance at the ULS of a pile in tension, only the shaft resistance and the weight of the pile shall be considered. For tapered piles, tensile resistance shall not be considered unless demonstrated by testing at the site.

6.8.5.6 Relaxation of driven piles
For stratigraphies where relaxation of the resistance of the driven pile can occur, the design shall take into consideration a possible loss of resistance with time.

6.8.6 Group effects — Vertical loads

6.8.6.1 Load distribution
At the SLS, piles and pile footings shall be assumed to respond linearly to applied loads.

Either linear or non-linear responses to applied loads shall be assumed in determining the loads acting on individual piles within a pile group at the ULS.

For other structural components, a linear response to applied loads shall be assumed when the forces at the ULS are calculated. For this case, the forces acting on the piles shall not be limited by the factored geotechnical resistance of the piles.

6.8.6.2 Group resistance
The factored vertical resistance of a pile group shall be determined as follows:

(a) the factored geotechnical resistance of a group of piles bearing on rock, dense sand, or gravel with no weak strata beneath the bearing layer shall be taken as the sum of the factored axial geotechnical resistances of the individual piles in the group; or

(b) the factored geotechnical resistance of a group of piles that derive their resistance primarily from shaft friction shall be taken as the lesser of the following:
   (i) the sum of the factored geotechnical resistances of the individual piles in the group; or
   (ii) the factored geotechnical resistance of an equivalent block enclosing the pile group.

6.8.7 Factored geotechnical lateral resistance

6.8.7.1 General
The factored geotechnical lateral resistance of a pile shall be determined from at least one of the following:

(a) static analysis;

(b) static tests; and

(c) assessment.

6.8.7.2 Static analysis
The factored geotechnical horizontal resistance of a pile shall be taken as the sum of the horizontal component of the factored passive resistance of the soil against the pile and the horizontal component of the factored axial load present in an inclined pile. The factored horizontal resistance of a group of piles shall take group effects into consideration.

6.8.7.3 Lateral deflection
The resistance provided by the soil to a pile as the pile deflects laterally shall be considered using unfactored geotechnical parameters. The pile shall be modelled as a beam-column supported by springs equivalent to the passive reaction distributed along the shaft. A linear or non-linear resistance-displacement relationship may be assumed. The relationship shall reflect the type of pile and the resistance and deformation characteristics of the soil. Both short-term static and cyclic responses shall be considered. Soil properties at various elevations shall be based on test data appropriate to similar soil types.

6.8.8 Structural resistance

6.8.8.1 Supported length
That portion of a deep foundation unit that is permanently in contact with soil shall be considered a laterally supported compression member.

6.8.8.2 Unsupported length
The length of pile between points of contraflexure in contact with air or water, including any buried length that could become exposed, shall be considered laterally unsupported. Engineering judgment shall be used to determine whether a soft soil provides adequate lateral support for a pile.

6.8.8.3 Structural instability
For piles not permanently in contact with soil, the possibility of structural instability of individual piles and of a pile group shall be considered.

6.8.8.4 Transporting, handling, and driving
Prefabricated deep foundation units shall be of sufficient strength to withstand force effects resulting from transporting, handling, and driving.

6.8.8.5 Factored structural resistance
The factored structural resistance of piles at the ULS shall be determined in accordance with the applicable Sections of this Code. In the case of embedded laterally supported piles, a reduction factor of 0.75 shall be applied. The reduction factor may be modified to increase the resistance if warranted by an assessment of the design and construction conditions for the site.

6.8.9 Embedment and spacing

6.8.9.1 Embedment in footing
Where the heads of piles are encased in a concrete footing or pile cap, the heads shall project at least 300 mm into the footing after all material that has been damaged by driving has been removed. For concrete and concrete-filled steel pipe piles connected to pile caps using reinforcing steel, the minimum embedment may be reduced to 100 mm.

The reinforcement shall be developed by embedment length, end anchorage, or both in accordance with Section 8.

6.8.9.2 Pile spacing
Where the centre-to-centre spacing of piles at the underside of the footing is less than 2.5b or less than 750 mm, the effects of interaction between piles shall be considered.

6.8.10 Pile shoes and splices

6.8.10.1 Pile shoes or points
Where soil or rock conditions warrant, pile shoes or points shall be used to ensure the integrity of piles during driving or to ensure effective contact with an end-bearing stratum.

Where shaft friction is required to contribute significantly to pile resistance, the pile shoes shall be shaped to avoid reducing shaft friction or pile shoes shall not be used.

6.8.10.2 Splices
The structural resistance of a pile splice shall be at least equal to that of the pile.

Where shaft friction is required to contribute significantly to pile resistance, the pile splices shall be shaped to avoid reducing shaft friction. Splicing of wood piles shall require Approval.

6.9 Lateral and vertical pressures

6.9.1 General
In calculating the magnitude and direction of the lateral pressures due to backfill, the following shall be considered:

(a) the nature and density of the backfill;
(b) the mobilized parameters of the backfill;
(c) the movement of the structure relative to the backfill;
(d) wall friction;
(e) the slope of the surface of the backfill;
(f) force effects due to compaction of the backfill;
(g) surcharge and superimposed loads;
(h) groundwater and seepage;
(i) the temperature regime;
(j) dynamic effects, including earthquakes;
(k) the adequacy of surface and subsurface drainage; and
(l) protection against pressures due to freezing of free water within the backfill.

6.9.2 Lateral pressures

6.9.2.1 General
Lateral pressures for use in the design of a structure shall include the effects of any superimposed dead and live load and the following shall apply:

(a) For an unrestrained structure, an active pressure due to backfill shall be considered in proportioning the width of the footing and the arrangement of piles. Lateral pressure due to compaction shall not be considered.
(b) For an unrestrained structure, an active pressure due to backfill and pressure due to compaction shall be considered in the proportioning of the structural sections.
(c) For a restrained structure, an at-rest pressure and pressure due to compaction shall be considered in the proportioning of the width of the footing, the arrangements of piles, and the structural sections.
(d) For structures where the interaction between structure and soil is considered to be neither restrained nor unrestrained, the earth pressure shall be determined by recognized methods of analysis from consideration of the movement of the structure relative to the retained soil and of the limit state condition in the supporting foundation.

Where it is possible for the base of a soil-retaining structure to move laterally at the ULS and thereby to mobilize passive pressure in foundation soils, the passive pressure shall be considered a resistance.

6.9.2.2 Calculated pressures

Pressures shall be calculated using representative unfactored soil parameters and recognized methods of analysis. The compaction pressures specified in Clause 6.9.3 shall be used as minimum values. Where the sloping surcharge exceeds 5°, an evaluation by a geotechnical Engineer shall be performed.

6.9.2.3 Equivalent fluid pressures

Equivalent fluid pressures shall be considered acceptable for calculating active and at-rest earth pressures for retaining structures with a stem height not exceeding 6.0 m, provided that well-drained granular material fills the space between the back face of the wall and a limiting line drawn at 45° to the horizontal from where the back face of the footing meets the bottom of the footing.

In the absence of calculated values developed from established soil properties for retaining walls having a stem height not exceeding 6.0 m, the equivalent fluid pressures, not including compaction pressures, shall be in accordance with Table 6.2. The effect of shear resistance or wall friction between the back face of the wall and the soil may be neglected.

The equivalent fluid pressure values specified in Table 6.2 shall not apply in the following circumstances:

(a) if the soil beyond the granular material limiting line specified in this Clause consists of soft cohesive soil or organic soils;

(b) if the ground surface within the granular material limiting line slopes upward from the wall at more than 5° to the horizontal; or

(c) if the groundwater table is above the base of the footing.

Table 6.2
Equivalent fluid pressure per metre width, kPa/m
(See Clause 6.9.2.3.)

	Angle of internal friction	Pressure
Active condition	30–35°	$7z$
	Greater than 35°	$6z$
At-rest condition	30–35°	$11z$
	Greater than 35°	$10z$

6.9.3 Compaction surcharge

For retained backfill that is placed and compacted in layers, the lateral force caused by compaction shall be considered.

For the calculated pressures required by Clause 6.9.2.2 and the granular backfill required by Clause 6.9.2.3, a lateral pressure varying linearly from 12 kPa at the fill surface to 0 kPa at a depth of 1.7 m below the surface for angles of internal friction from 30 to 35°, or 2.0 m below the surface for angles of internal friction greater than 35°, shall be added to the lateral earth pressure, as shown in Figure 6.6, in lieu of calculation by recognized methods of analysis.

In the absence of detailed analysis, the additional lateral pressure due to the effects of compaction for restrained structures shall be taken as the at-rest pressure intensity at a given elevation multiplied by 0.15. The total lateral pressure for restrained structures shall be added to the lateral pressure corresponding to light compaction, as shown in Figure 6.6.

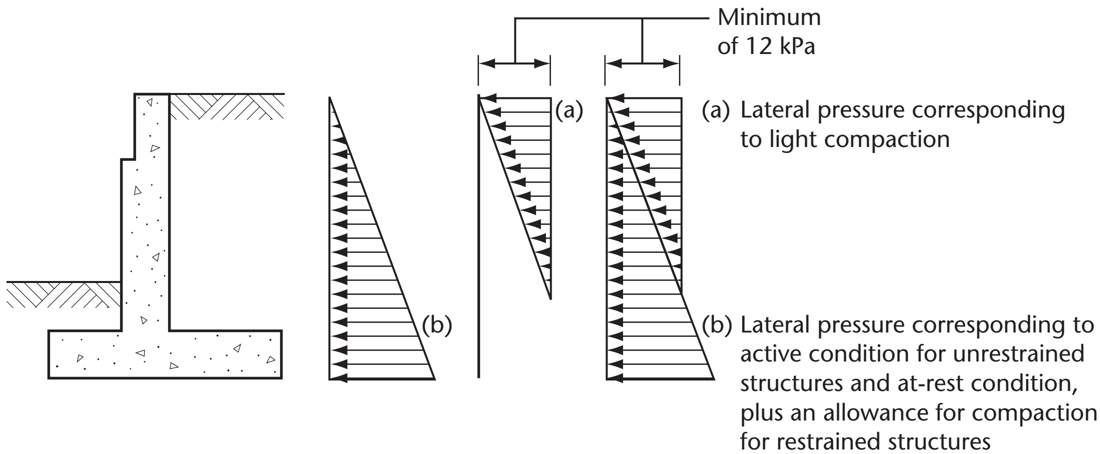


Figure 6.6
Compaction effects
(See Clause 6.9.3.)

6.9.4 Effects of loads

The effects of loads of various inclinations shall be considered in the assessment of the ultimate resistance of foundations supporting earth-retaining structures.

Where a mass of soil is retained behind a wall, combinations of load factor for both horizontal and vertical load shall be considered. For earth pressures acting on a buried structure such as a culvert or partial frame, where different soil masses are possible on either side of the structure, the maximum or minimum load factors specified in Section 3 shall apply.

6.9.5 Surcharge

The horizontal and vertical force effects due to footings and other loads placed in or on the backfill shall be taken into account.

A live load surcharge shall apply where the backfill supports highway live loads within a distance from the back face of the wall equal to the effective height. Where an approach slab extends over this distance and is supported by or at the wall, the surcharge shall not apply.

The live load surcharge shall be equal to an equivalent additional fill height of 0.80 m. The surcharge shall be assumed to act above the finished grade and over the length of the retaining structure.

6.9.6 Wheel load distribution through fill

When the depth of fill over a structure is 0.60 m or more, wheel loads shall be uniformly distributed at the surface of the structure over a rectangle, the sides of which are equal to the footprint of the wheel of the CL-W Truck as specified in Clause 3.8.3.1 plus 1.75 times the depth of fill. When distributed wheel loads from two or more wheels overlap, the total load of those wheels shall be uniformly distributed over the smallest rectangle that includes their individual areas, but the total width of distribution shall not exceed the total width of the structure supporting the fill.

When the depth of fill over a structure is less than 0.60 m, no distribution beyond the footprints of the wheels shall be considered.

6.10 Ground anchors

6.10.1 Application

Clause 6.10 applies to the design, installation, stressing, inspection, and testing of temporary and permanent ground anchor systems. It applies to soil and rock anchors, but not to soil nailing and MSE systems. Sections 8, 9, and 10 shall apply to ground anchors used with concrete structures, wood structures, and steel structures, respectively.

6.10.2 Design

6.10.2.1 General

The designer shall assess the performance requirements for the ground anchor system and the available site information.

The pullout performance shall be sufficient for the service life of the installation.

The following shall be considered:

(a) the site conditions;
(b) the soil and rock properties;
(c) ground discontinuities;
(d) ground creep susceptibility;
(e) the grout-ground interface;
(f) the diameter of the hole;
(g) the construction methods and equipment;
(h) the grouting procedure;
(i) the strength of the grout;
(j) the tendon-grout bond;
(k) the structural components;
(l) the aggressiveness of the environment; and
(m) corrosion protection.

The durability of the anchorage system shall be designed for the service life of the installation. Installations with a design life exceeding two years shall be considered permanent.

6.10.2.2 Factored geotechnical resistance at the ULS and geotechnical reaction at the SLS

Potential failure mechanisms at ultimate limit states shall be identified and evaluated. The factored geotechnical resistance at the ULS shall be determined using the resistance factors specified in Table 6.1.

The geotechnical resistance at the ULS and the geotechnical reaction at the SLS shall be determined by one or more of the following:

(a) a static analysis based on geotechnical information obtained at the site;
(b) pullout tests; or
(c) an assessment by extrapolation of anchor behaviour under similar site conditions.

6.10.2.3 Spacing, bond length, and free-stressing length

The spacing, bond length, and free-stressing length shall be sufficient to ensure performance of the ground anchor.

The centre-to-centre spacing between bond lengths shall not be less than four times the diameter of the bored hole. The bond length shall be sufficient to develop the required pullout resistance and shall not be less than 3 m. The length of the free-stressing zone shall locate the bond length outside the failure wedge.

6.10.3 Materials and installation

6.10.3.1 Prestressing steel and attachments

The prestressing steel shall comply with CSA G279.

Tendons shall be designed so that
(a) the factored resistance at the ULS does not exceed 60% of the specified tensile strength of the prestressing steel;
(b) the transfer or lockoff load does not exceed 70% of the specified strength of the prestressing steel;
(c) the maximum test load does not exceed 80% of the specified strength of the prestressing steel; and
(d) the anchorage components and couplers for tendons develop at least 100% of the specified strength of the tendon.

6.10.3.2 Grout or concrete for bond length

The strength of grout or concrete for the bond length shall be adequate for the anchor performance specified in Clause 6.10.3.1. The use of post-grouting techniques shall be considered as a means of enhancing anchor resistance. Post-grout pressures shall be controlled so that the overlying soil is not disturbed.

6.10.3.3 Backfill for free-stressing length

The backfill for the free-stressing length shall completely fill the annular space between the ground anchor tendon and the borehole wall and shall prevent any transfer of the anchor load to the free-stressing length.

6.10.3.4 Corrosion protection

For temporary anchors, the prestressing steel shall retain adequate structural strength during the required service life of the anchor and the provisions for corrosion protection shall be adequate for this purpose.

For permanent anchors, double corrosion protection, providing two separate corrosion barriers to the prestressing steel, shall be incorporated.

6.10.4 Anchor testing

6.10.4.1 General

Anchor testing shall comply with accepted and Approved standards and with the following requirements:
(a) A sufficient number of pre-production tests shall be carried out where in-situ tests are required as a basis for grout-to-ground bond design.
(b) A sufficient number of performance tests shall be carried out to determine whether the anchor design has sufficient load-carrying capacity and creep performance.
(c) Proof tests shall be carried out on all production anchors to confirm that anchor pullout performance is sufficient and that the free-length requirements have been satisfied.
(d) Liftoff tests shall be carried out on selected production anchors to confirm creep performance.

6.10.4.2 Acceptance criteria

The acceptance criteria for anchor performance shall be sufficient for meeting Approved standards and project-specific requirements.

6.11 Sheet pile structures

6.11.1 Application

Clauses 6.11.2 to 6.11.4 apply to retaining structures that consist of driven sheet piles with or without ties.

6.11.2 Design

In the design of all components and the determination of deformations, the method of analysis used shall assume a linear or non-linear resistance-displacement relationship in accordance with the resistance and deformation characteristics of the soil and of the sheet piles. Long-term and short-term static and cyclic responses shall be considered.

The determination of soil properties at various elevations shall be based on field tests or assessed values.

6.11.3 Ties and anchors

6.11.3.1 Deadman anchors

In determining the required length of tie rods connected to deadman anchors, the possible reduction in anchor resistance caused by interference between the active failure wedge behind the sheeting and the passive failure wedge in front of the anchor shall be considered.

6.11.3.2 Pile anchors

Tie rod anchorages consisting of vertical or inclined piles shall be designed in accordance with Clause 6.8.

6.11.3.3 Tie load

The factored load to be resisted by anchorage ties shall not exceed 80% of the calculated factored tensile resistance at the ULS calculated in accordance with Section 10.

6.11.3.4 Sagging of tie rods

When tie rods are underlain by compressible soils that can undergo significant vertical deformation under surcharge loading, the possibility of additional tensile stresses being induced in the tie rods because of sagging shall be considered.

6.11.4 Cellular sheet pile structures

In the proportioning of cellular sheet pile structures, the following shall be considered:

(a) the type of pile;
(b) ease of installation;
(c) the type of backfill and its geotechnical properties;
(d) the groundwater and free water levels;
(e) acceptable deformations;
(f) durability;
(g) future dredging or excavation; and
(h) scour.

6.12 MSE structures

6.12.1 Application

Clause 6.12 applies to the design of reinforced-soil-type retaining wall systems consisting of prefabricated facing elements, reinforced soil mass, and soil-reinforcing elements, normally comprising metal strips, geogrids, or metal mesh. Components made of materials that are not covered by this Code shall be used only when short- and long-term testing have established their suitability for the intended purpose. The testing shall establish all relevant properties, including those pertaining to durability, dimensional stability, and creep. The design of MSE structures shall take into consideration both overall stability and internal stability.

6.12.2 Design

6.12.2.1 General

The design shall be based on accepted methods of analysis and shall take into consideration the magnitude of strains expected in the soil-reinforcing elements and the soil; internal stability within the zone of the soil-reinforcing elements; and the site-specific external stability of the MSE geometry.

6.12.2.2 Calibration
When established empirical methods of analyzing the internal resistance of reinforced-soil-type structures are available and are based on working stress design methods, such methods shall be used to check the results of the analytical limit states design method.

6.12.2.3 Factors for consideration
In determining proportions, including those of the founding level and the overall width of MSE structures, the following shall be considered:
(a) the type and spacing of soil-reinforcing elements;
(b) the backfill type and compaction;
(c) the durability of components;
(d) scour, future dredging, or excavation; and
(e) groundwater and seepage.

6.12.3 Backfill
Backfill within and behind the reinforced soil mass shall consist of Approved earth material compacted using methods and equipment appropriate to the type of structure.

6.13 Pole foundations

6.13.1 Application
Clause 6.13 applies to the design of pole foundations, including foundations for high-mast lighting poles, in which a vertical cylindrical footing is formed by augering or excavating into the ground, securing a reinforcement cage and pole base anchorage system in the hole, and casting the footing concrete around them against undisturbed soil.

6.13.2 Design

6.13.2.1 General
The embedded length and diameter of the foundation shaft that is considered to be effective in resisting horizontal pressure shall be determined by accepted analytical methods. The foundation shaft and any concrete extension thereof that is not considered part of the foundation shall comply with all reinforced concrete substructure requirements of this Code. Anchor bolts and anchorages shall comply with the applicable requirements of Sections 8, 10, and 12.

6.13.2.2 Assumptions
Transverse loads, including those caused by bending effects, shall be assumed to be resisted by the horizontal reaction of the ground surrounding the foundation shaft. In the determination of horizontal soil reactions, the foundation shaft shall be considered to be infinitely stiff. In both cohesive and non-cohesive soils, allowance shall be made at the ULS for the end effect on transverse distribution of pressures within the soil mass that resist the horizontal movement of the foundation shaft.

When significant proportions of the horizontal soil/rock reactions are due to permanent loads, allowance shall be made for the effects of long-term loading on foundation deformations and resistance.

6.13.2.3 Ultimate limit state
At the ULS, the full passive resistance of the soil shall be assumed to have developed. The passive resistance factor applied shall be as specified in Table 6.1.

6.13.2.4 Serviceability limit state
The SLS consideration shall be rotation of the footing. Rotation shall not exceed the tilting limit for the serviceable function of the pole. The rotation shall be calculated using an Approved method of analysis.

Section 7 — Buried structures

Section 7
Buried structures

7.1 Scope

This Section specifies requirements for the analysis and design of buried structures of the following types:
(a) soil-metal structures;
(b) metal box structures; and
(c) reinforced concrete structures.

This Section also specifies construction procedures, properties and dimensions of engineered soil components, and requirements for construction supervision.

7.2 Definitions

The following definitions apply in this Section:

Arch — a soil-metal or reinforced concrete structure in which the conduit wall is not continuous around the perimeter of the bridged opening and the conduit wall is supported on footings.

Arching — the transfer of pressure or load between the soil masses adjacent to and above a conduit that move relative to one another. Positive arching results in the transfer of loads away from the conduit; negative arching produces the opposite effect.

Bedding — the prepared portion of engineered soil on which the base of a closed conduit wall is placed.

Bevelled end — the termination of the wall of a conduit, cut at a plane inclined to the horizontal.

Buried structure — a structure that has one or more conduits and is designed by taking account of the interaction between the conduit wall and engineered soil.

Camber — an adjustment required in the longitudinal profile of bedding to compensate for post-construction settlement.

Compaction — the process of soil densification, at a specified moisture content, by the application of pressure through rolling, kneading, tamping, rodding, or vibratory actions of mechanical or manual equipment.

Conduit — the bridged opening of a buried structure.

Conduit wall — the corrugated metal plate shell or reinforced concrete wall lining a conduit.

Connection — an overlapped bolted joint between two structural metal plates or a joint between two reinforced concrete elements.

Controlled low-strength material — a mixture of soil, a small amount of cement, and a large amount of water and other admixtures that flows easily in its initial stages and hardens to a 28-day compressive strength of 1 to 5 MPa.

Crown — the highest point of the transverse section of a conduit wall.

Deep corrugations — structural plate corrugations with a pitch between 380 and 400 mm and a rise between 140 and 150 mm.

Depth of cover — the vertical distance between the roadway surface and the conduit wall, as shown in Figures 7.3, 7.8, and 7.11.

Engineered soil — soil selected and placed to achieve desired geotechnical properties.

Foundation — the soil or rock underlying a conduit and the engineered soil.

Frost-susceptible soil — soil that tends to heave excessively under frost action, resulting in a severe degradation in strength and stiffness.

Haunch — the following:
(a) in a soil-metal structure or circular concrete structure, the portion of the conduit wall between the spring line and the top of the bedding or footing;
(b) in a metal-box structure, the curved portion of the conduit wall between the sidewall and top, sometimes referred to as the shoulder; and
(c) in a concrete box section, the stiffened corner portions.

Horizontally elliptical pipe — an elliptical pipe whose the major diameter is horizontal and greater than 1.10 times the minor diameter.

Invert — the lowest point of a conduit at a transverse section or the bottom segment of a conduit wall.

Longitudinal direction — the direction of a conduit axis that is parallel to the locus of the crown.

Longitudinal stiffeners — stiffeners that comprise continuous structural elements, are usually of reinforced concrete construction, and are attached along the length of the metallic shell at the junction of the top and side arcs.

Metal box structure — a structure that is fabricated from corrugated metal plates, has the details shown in Figure 7.7, and in which the design of the conduit wall is mainly governed by flexure.

Modified proctor density — the maximum dry density of a soil determined in accordance with ASTM D 1557.

Modulus of soil stiffness — the ratio of the radial contact pressure to the radial strain in a soil.

Obvert — the highest point of a conduit at a transverse section or the top segment of a metal conduit wall.

Overfill — the soil placed above and beyond required structural backfill.

Pipe-arch — a conduit that consists of arched upper and side portions and is structurally continuous with an invert whose radius of curvature is greater than that of the other portions.

Re-entrant arch — an arch whose spring lines lie above the footings.

Rise — the maximum vertical clearance inside a conduit at a transverse section, measured at the mid-depth of the corrugations of metal structures.

Round pipe — a circular or elliptical pipe whose major diameter does not exceed 1.10 times the minor diameter.

Shallow corrugations — structural plate corrugations with a pitch between 150 and 230 mm and a rise between 50 and 65 mm.

Shoulder — the portion of a conduit wall between the crown and the spring line.

Sidefill — the portion of structural backfill illustrated in Figures 7.8 and 7.9 for circular concrete pipes and in Figures 7.10 and 7.11 for concrete box sections.

Sidewall — the vertical or nearly vertical portion of a conduit wall in a metal or concrete box structure.

Soil-metal structure — a structure fabricated from corrugated metal sheets or plates.

Soil modification — improvement of soil strength, compressibility, or permeability by geotechnical means, including the use of geosynthetics.

Span — the maximum horizontal clearance inside a conduit at a transverse section, measured for soil-metal structures at the mid-depth of the corrugations.

Spring line — the locus of the outermost points of the sides of a conduit.

Standard installation — the installation of buried concrete structures as specified in Clauses 7.8.3.5 and 7.8.3.6.

Standard Proctor density — the maximum dry density of a soil, determined in accordance with ASTM D 698.

Stiffener — a structural member connected to a conduit wall to improve its strength and stiffness.

Structural backfill — the engineered soil placed around a conduit in a controlled manner, as specified in Clauses 7.6.5.6.1, 7.7.5.1.1, and 7.8.15.5.

Thrust — the circumferential compressive force in a conduit wall, per unit length of the wall.

Transverse direction — the direction in the horizontal plane perpendicular to the longitudinal direction.

Transverse section — a section in the vertical plane normal to the longitudinal direction.

Vertically elliptical pipe — an elliptical pipe whose major diameter is vertical and greater than 1.10 times the minor diameter.

7.3 Abbreviation and symbols

7.3.1 Abbreviation

The following abbreviation applies in this Section:

CLSM — Controlled low-strength material

7.3.2 Symbols

The following symbols apply in this Section:

A = cross-sectional area of a corrugated metal conduit wall per unit length in the longitudinal direction, mm^2/mm

A_H = horizontal acceleration ratio due to earthquake loading (dimensionless), equal to the zonal acceleration ratio in Clause 4.4.3

A_L = axle load, kN (see Clause 7.7.3.1.3)

A_V = vertical acceleration ratio due to earthquake loading (dimensionless), equal to two-thirds of the horizontal acceleration ratio, A_H

A_c = axle load during construction, kN

A_f = factor used to calculate the thrust due to dead load in a conduit wall

A_s = area of tensile reinforcement per width b, mm^2/mm

A_{si} = total area of the inner cage reinforcement area per width b, mm^2/mm

A_{so} = total area of the outer cage reinforcement area per width b, mm^2/mm

A_{vs} = area of stirrup reinforcement to resist shear and radial tension in each line of stirrups at circumferential spacing, s_v, mm^2 per width b

B_1 = crack-control coefficient for the effect of spacing and number of layers of reinforcement for all sizes of welded wire fabric and hot-rolled bars 10M or smaller with a longitudinal spacing of less than or equal to 100 mm

b = width of the concrete section that resists structural force effects, mm

C_s = axial stiffness parameter for soil-metal structures (see Clause 7.6.3.1.2)

C_1 = crack-control coefficient for the type of reinforcement; tandem axle coefficient for metal box structures (see Clause 7.7.3.1.3)

D_h, D_v = dimensions relating to the conduit, m (see Figures 7.1 and 7.2)

D_i = inside diameter of concrete pipe, mm

D_o = outside diameter of concrete pipe, mm

DLA = dynamic load allowance expressed as a fraction of live load

d = depth of corrugation, mm; distance from compression face to centroid of tension reinforcement, mm

d_c = corrugation depth, mm

E = modulus of elasticity, MPa

E_m = modified modulus of soil stiffness, MPa

E_s = secant modulus of soil stiffness, MPa

F_N = coefficient for effect of thrust on shear strength

F_c = factor for effect of curvature on diagonal tension (shear) strength in curved components

F_{cr} = crack-width control factor for adjusting crack control

F_d = factor for crack-depth effect resulting in increase in diagonal tension (shear) strength with decreasing d

F_m = reduction factor for modifying buckling stress in multi-conduit structures

F_{rt} = factor for pipe size effect on radial tension strength of pipe

F_y = cold-formed yield stress of a metal conduit wall, MPa

f_b = factored failure stress in compression in a metal conduit wall, MPa

f_c' = design compressive strength of concrete, MPa

f_s = maximum service load strength of reinforcing steel for crack control, MPa

f_v = maximum developable strength of stirrup material, MPa

f_y = design yield strength of reinforcement, MPa

H = depth of cover or height of overfill, m

H' = half the vertical distance between crown and spring line, m

H_c = depth of cover at intermediate stages of construction, m

H_e = effective value of depth of cover above a conduit (used for calculating bending moment due to dead load in a complete soil-metal structure), m

H_{min} = minimum allowable depth of cover above a conduit, m

h = overall thickness of member (wall thickness), mm

I = second moment of cross-sectional area, A, about the neutral axis of a corrugated section in the longitudinal direction of the conduit, mm^4/mm

K = factor representing the relative stiffness of a conduit wall with respect to the adjacent soil; the effective length factor used in a P-Δ analysis

k_{M1}, k_{M2}, k_{M3} = factors used in calculating moments in soil-metal structures (see Clauses 7.6.3.3.1 and 7.6.3.3.2)

k_R = haunch moment reduction factor for metal box structures

k_1, k_2, k_3, k_4 = factors used in calculating dead load and live load moments in soil-metal and metal box structures

L_L = line load equivalent to the live load acting on a metal structure, kN/m

L_c = line load equivalent to the construction load acting on a metal structure, kN/m

L_{hi} = horizontal dimension of concrete box section to inside of walls, mm

L_{ho} = horizontal dimension of concrete box section to outside of walls, mm

L_{vi} = vertical dimension of concrete box section to inside of top and bottom slabs, mm

L_{vo} = vertical dimension of concrete box section to outside of top and bottom slabs, mm

ℓ_t = length of dispersed live load at crown level measured transversely, m (see Clause 7.6.3.1.3)

ℓ_θ = total additional arc length beyond calculated arc lengths requiring stirrups, mm

M = unfactored moment in a soil-metal structure, kN•m/m

M_B = additional moment in the wall of a soil-metal structure due to a height of fill, H_c, above the crown, kN•m/m

M_C = additional moment in a soil-metal structure due to construction live loads, kN•m/m

M_D = sum of the intensities of bending moments at the crown and haunch in a metal box structure due to dead load, kN•m/m; moment in the wall, kN•m/m

M_E = additional moment in a metal box structure due to earthquake loading, kN•m/m

M_L = sum of the crown and haunch bending moments in a metal box structure due to live load; moment in the wall, kN•m/m

M_P = unfactored plastic moment capacity of a corrugated metal section, kN•m/m

M_{Pf} = factored plastic moment capacity of a corrugated metal section, kN•m/m

M_{cD} = crown bending moment in a metal box structure due to dead load, kN•m/m

M_{cL} = crown bending moment in a metal box structure due to live load, kN•m/m

M_{cf} = total factored crown bending moment in a metal box structure, kN•m/m

M_{hD} = haunch bending moment in a metal box structure due to dead load, kN•m/m

M_{hf} = total factored haunch bending moment in a metal box structure, kN•m/m

M_{hL} = haunch bending moment in a metal box structure due to live load, kN•m/m

M_{nu} = factored moment in concrete structures, as modified for effects of compressive or tensile thrust, N•mm per width b

M_u = factored moment acting on a cross-section of a concrete structure, N•mm per width b

M_1 = moment in a soil-metal structure resulting from fill to the crown level, kN•m/m

m_f = modification factor for multi-lane loading

N_F = flexibility number used in calculating moments in a soil-metal structure during construction

N_u = factored axial thrust acting on a cross-section of a concrete structure (positive when compressive, negative when tensile), N per width b

n = number of layers of reinforcement in a cage (n = 1 or 2)

P = unfactored thrust in the wall of a soil-metal structure, kN/m

P_{Pf} = factored compressive strength of a corrugated metal section without buckling, kN/m

P_h = total horizontal earth load acting on a buried concrete pipe, kN per width b (see Clause 7.8.4.2.2)

R = radius of curvature of a conduit wall at the mid-depth of corrugations at a transverse section, mm; rise of a metal box structure as shown in Figure 7.6, m

R_B, R_L, R_U = parameters used in calculating moments in the wall of a soil-metal structure during construction

R_b = radius of curvature of the invert of the cross-section of a pipe-arch

R_c = R at crown, mm
R_e = equivalent radius specified in Clause 7.6.3.2, mm
R_s = radius of curvature of the haunch of the cross-section of a pipe-arch
r = radius of gyration of corrugation profile, mm; radius to centreline of concrete pipe wall, mm
r_s = radius of the inside reinforcement, mm
S = least transverse clear spacing between adjacent conduits, m
S_M = fifth percentile flexural strength of a longitudinal connection per unit length, kN•m/m
S_s = fifth percentile axial strength of a longitudinal connection per unit length, kN/m
s_v = circumferential spacing of stirrups, mm
s_1 = longitudinal spacing of circumferential reinforcement, mm
T_C = additional thrust in the wall of a soil-metal structure due to construction live loads, kN/m
T_D, T_L = maximum thrust in a conduit wall per unit length due to unfactored dead and live loads, respectively, kN/m
T_E = additional thrust in the wall of a soil-metal structure due to earthquake loading, kN/m
T_f = maximum thrust in a conduit wall due to factored loads per unit length, kN/m
t_b = clear cover over reinforcement, mm
V_b = basic shear strength of a critical section of a concrete structure, where $M_{nu}/V_u d$ = 3.0, N per width b
V_c = nominal shear strength provided by a concrete cross-section, N per width b
V_u = factored shear force acting on a concrete cross-section, N per width b
W = dead weight of the column of material above a conduit per unit length of conduit (see Figure 7.2), kN/m for soil-metal structures and kN per width b for concrete pipe and concrete box sections
W_e = total vertical earth load acting on a buried concrete pipe, kN per width b (see Clause 7.8.4.2.2)
w_c = weight of a column of unit area of fill above a reference point at the top or on the sides of a buried concrete box section, kN/m^2
α_D = load factor for dead loads
α_L = load factor for live loads
γ = unit weight of soil, kN/m^3
θ = skew angle of a conduit, degrees (see Table 7.1); orientation angle in a circular concrete pipe, degrees
θ_0 = angle of radial line from vertical demarking the upper and lower portions of a conduit wall in a soil-metal structure, radians
κ = crown moment coefficient used to calculate the crown and haunch bending moments in a metal box structure
λ = factor used in calculating K
λ_h = factor used in the analysis of concrete buried structures in standard installations to account for the effect of soil-structure interaction on the horizontal soil pressures
λ_v = factor used in the analysis of concrete buried structures in standard installations to account for the effect of soil-structure interaction on the vertical soil pressures
ρ = reduction factor for buckling stress in metal conduit walls; ratio of reinforcement area to concrete area
σ = stress due to thrust in a conduit wall due to factored loads, MPa
σ_L = equivalent uniformly distributed pressure at the crown due to unfactored dispersed live load, kPa

σ_h	=	horizontal earth pressure acting on the sides of a buried concrete box section, kPa
σ_v	=	vertical earth pressure acting at the top of a buried concrete box section, kPa
ϕ_c	=	resistance factor for concrete in compression, radial tension, and in shear
ϕ_h	=	resistance factor for plastic hinge for the completed structure
ϕ_{hc}	=	resistance factor for plastic hinge during construction
ϕ_j	=	resistance factor for failure of connections
ϕ_s	=	resistance factor for flexural steel reinforcement
ϕ_t	=	resistance factor for compressive strength of soil-metal and metal box structures

7.4 Hydraulic design

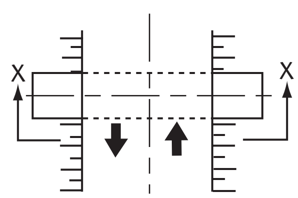
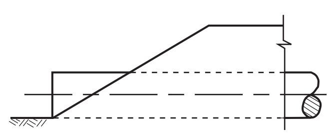
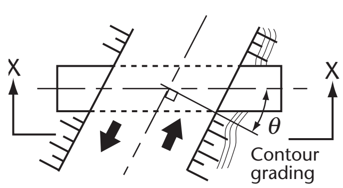
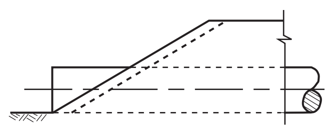
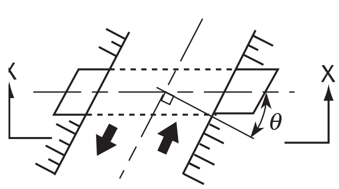
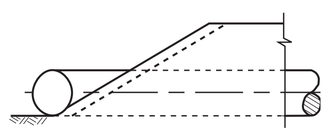
The following requirements shall apply to buried structures that are intended to convey water:

(a) the hydraulic design of the conduit shall be in accordance with Clause 1.9;

(b) the cut ends shall be as indicated in Table 7.1; and

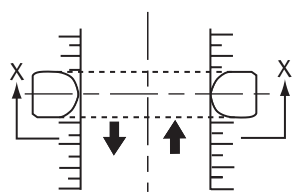
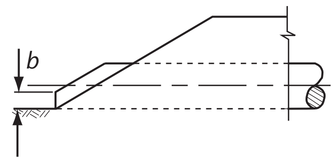
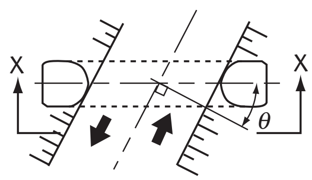
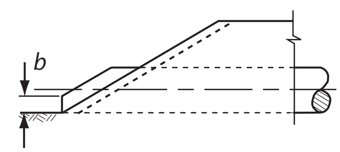
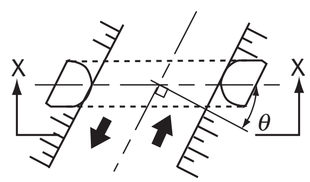
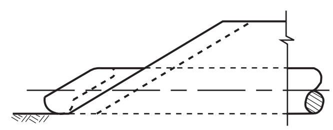
(c) for soil-metal structures and metal box structures, end treatments shall be provided in accordance with Clause 1.9.

Table 7.1
Requirements for cut ends
(See Clause 7.4.)

Description of cut end	Typical view of installation — Plan	Section X-X	Requirements
1. Square end with roadway parallel to transverse direction			None
2. Square end with roadway skew to transverse direction			θ shall be less than 40°. For θ greater than 20°, the earth pressure imbalance shall be accommodated by structural reinforcement of the conduit wall or by contour grading of the embankment slope.
3. Skew end			As for No. 2

(Continued)

Table 7.1 (Concluded)

Description of cut end	Typical view of installation		Requirements
	Plan	Section X-X	
4. Square bevel with roadway parallel to transverse direction			*b* shall not be less than $D_v/8$. The ends shall be treated as a retaining structure and shall be designed in accordance with Section 6.
5. Square bevel with roadway skew to transverse direction			As for Nos. 3 and 4
6. Skew bevel			This cut end shall be avoided

7.5 Structural design

7.5.1 Limit states

For different types of buried structures, the specific limit states corresponding to general limit states shall be as specified in Table 7.2.

Table 7.2
Specific limit states
(See Clause 7.5.1.)

General limit state	Type of structure	Specific limit state	Applicable clause
Ultimate limit state	Soil-metal with shallow corrugations	Compression failure Plastic hinge during construction Connection failure	7.6.3.2 7.6.3.3.1 7.6.3.4
	Soil-metal with deep corrugations	Compression failure Plastic hinge Plastic hinge during construction Connection failure	7.6.3.2 7.6.3.3.2 7.6.3.3.1 7.6.3.4
Serviceability limit state	Soil-metal	Deformation during construction	7.6.3.3.1
	Metal box	Deformation during construction	7.7.5.2
	Concrete	Maximum crack widths due to flexure	7.8.9.1
Fatigue limit state	Soil-metal	—	—
	Metal box	Stress range in conduit wall	7.7.3.1.5
	Concrete	Stress range of the reinforcement	7.8.10

7.5.2 Load factors

The load factors shall be as specified in Clause 3.5.1, except that the load factor for dead load for earth fill over concrete structures with curved bottom surfaces shall be increased by the installation factor specified in Clause 7.8.7.1(a). For earth pressure due to live loads, the factor for live loads shall be applied.

7.5.3 Material resistance factors

The material resistance factors specified in Table 7.3 shall be used to calculate factored resistances for conduit walls.

Table 7.3
Material resistance factors
(See Clause 7.5.3.)

Type of structure	Component of resistance	Material resistance factor
Soil-metal with shallow corrugations	Compressive strength Plastic hinge during construction Connections	$\phi_t = 0.80$ $\phi_{hc} = 0.90$ $\phi_j = 0.70$
Soil-metal with deep corrugations	Compressive strength Plastic hinge Plastic hinge during construction Connections	$\phi_t = 0.80$ $\phi_h = 0.85$ $\phi_{hc} = 0.90$ $\phi_j = 0.70$
Metal box	Compressive Strength Plastic hinge Connections	$\phi_t = 0.90$ $\phi_h = 0.90$ $\phi_j = 0.70$
Precast concrete	Cold-drawn wire and welded wire fabric Flexural reinforcement — Hot-rolled bars Concrete (normal density)	$\phi_s = 0.90$ $\phi_s = 0.90$ $\phi_c = 0.80$
Cast-in-place concrete	Flexural steel reinforcement Concrete (normal density)	$\phi_s = 0.90$ $\phi_c = 0.75$

7.5.4 Geotechnical considerations

7.5.4.1 Geotechnical investigation

The feasibility of constructing buried structures and their approaches shall be established by a geotechnical investigation of the site unless knowledge of local subsurface conditions indicates that approach fills and cuts will remain stable during and after construction. Geotechnical investigation of the foundation shall be carried out to provide the information required for the design of the footings or the base of the structure.

7.5.4.2 Soil properties

The soil properties used in the design of buried structures shall be as specified in Clause 7.6.2.3 for soil-metal structures, Clause 7.7.2.2 for metal box structures, and Clause 7.8.3.1 for concrete structures.

7.5.4.3 Camber

Whether camber is needed shall be established by considering the flow-line gradient and estimating the maximum deformation of the foundation at the invert. If the maximum foundation deformation is to be compensated, the invert grade shall be cambered by an amount sufficient to prevent the development of a sag or back slope in the flow line.

7.5.4.4 Footings

Footings shall be designed in accordance with Clause 6.7. Scour protection shall be provided in accordance with Clause 1.9.5. In the design of footings, consideration shall be given to resisting the horizontal reactions that develop in footings because of soil pressures on the conduit wall.

7.5.4.5 Control of soil migration

Where groundwater and soil characteristics can cause migration of soil fines into or out of foundation, bedding, sidefill, and backfill soils, methods to prevent migration shall be specified for the installation. These methods shall comprise the use of placed soils with filter gradation, the use of filter geotextiles, or other suitable means.

7.5.5 Seismic requirements

7.5.5.1 General

Buried structures shall be designed to resist inertial forces associated with a seismic event having a 10% chance of being exceeded in 50 years. The vertical component of the earthquake acceleration ratio, A_V, shall be two-thirds of the horizontal ground acceleration ratio, A_H. A_H shall be set equal to the zonal acceleration ratio specified in Clause 4.4.3. Amplification of these accelerations shall be considered where a significant thickness of less competent soil overlies rock or firm ground. Damage to the structure caused by excessive deformation of the soil, including the foundation soil, during a seismic event shall also be considered.

7.5.5.2 Seismic design of soil-metal structures

For soil-metal structures, the additional thrust, T_E, due to earthquake loading shall be calculated as follows:

$$T_E = T_D A_V$$

In accordance with Clause 3.5.1, the total factored thrust, T_f, including the earthquake effects, shall be calculated as follows:

$$T_f = \alpha_D T_D + T_E$$

7.5.5.3 Seismic design of metal box structures

For metal box structures, the additional moment due to the effect of earthquake, M_E, shall be calculated as follows:

$$M_E = M_D A_V$$

The total factored moments, M_{cf} and M_{hf}, including the earthquake effects, shall be calculated as follows:

$$M_{cf} = \kappa(\alpha_D M_D + M_E)$$

$$M_{hf} = (1 - \kappa)(\alpha_D M_D + M_E)$$

where α_D is obtained from Clause 3.5.1.

7.5.5.4 Seismic design of concrete structures

For concrete structures, the effects of earthquake loading shall be calculated in accordance with Clause 7.8.4.4.

7.5.6 Minimum clear spacing between conduits

For multi-conduit structures, including soil-metal structures with shallow corrugations, the minimum clear spacing between adjacent conduits shall be not less than 1000 mm or one-tenth of the largest span; this requirement may be waived for concrete boxes with cement grout between the boxes. For multi-conduit soil-metal structures with deep corrugations, the minimum clear spacing between adjacent conduits shall be 1000 mm; if CLSM is used between the conduits, the minimum clear spacing may be reduced to 800 mm if the CLSM is poured to a height where its width is at least 800 mm.

7.6 Soil-metal structures

7.6.1 General

Clauses 7.6.2 to 7.6.7 apply to the structural design of steel or aluminum structures of circular, elliptic, pipe-arch, or pear-shaped pipes with a closed or arch configuration shown in Figure 7.1.

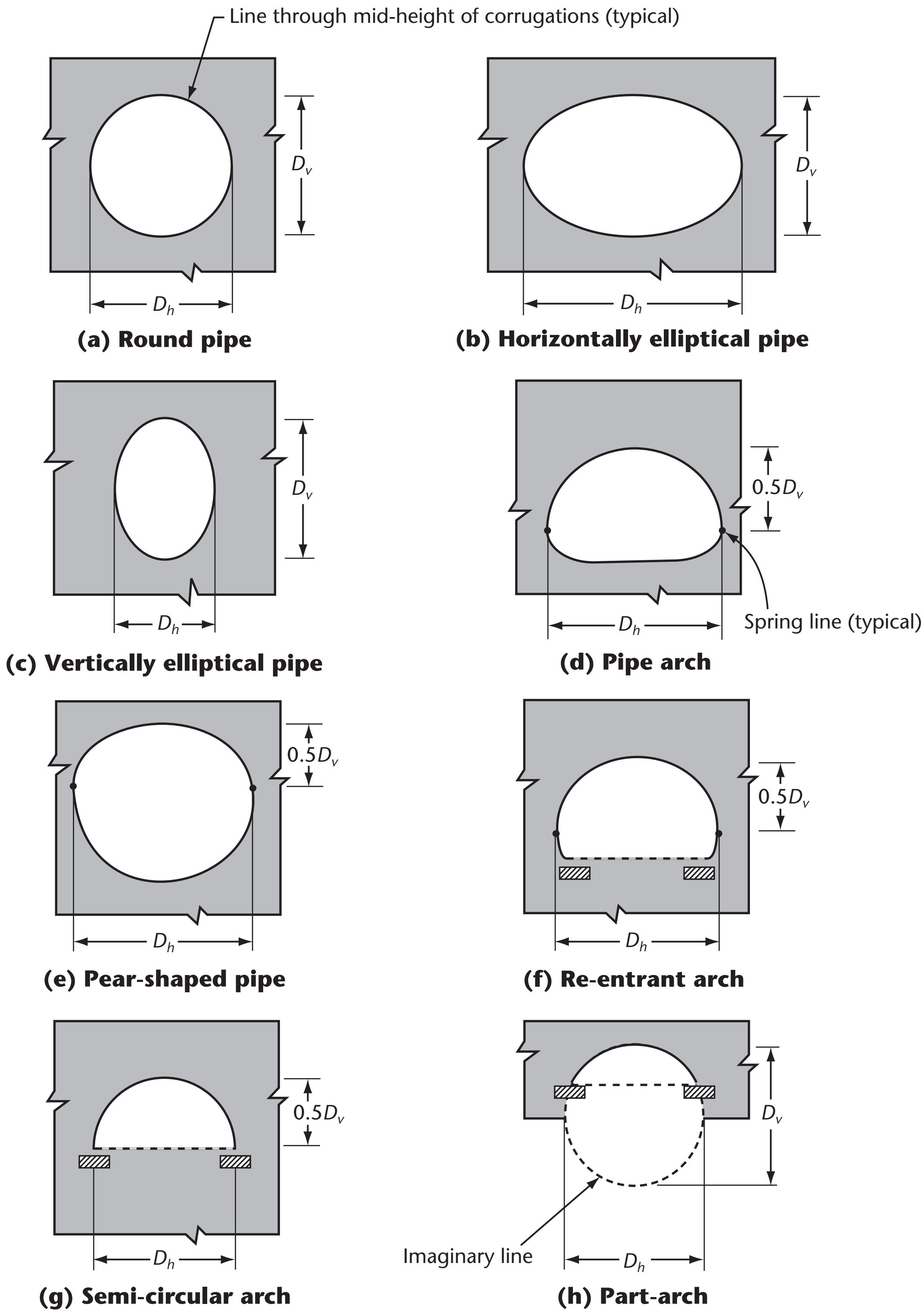


Figure 7.1
D_h and D_v for various shapes of pipe
(See Clauses 7.2, 7.6.1, 7.6.3.1.2, 7.6.3.2, and 7.6.4.1.)

7.6.2 Structural materials

7.6.2.1 Structural metal plate
Steel plates with both shallow and deep corrugations, and related components, shall satisfy the material and fabrication requirements of CSA G401. Aluminum plates and components shall satisfy the material and fabrication requirements of ASTM B 746/B 746M.

7.6.2.2 Corrugated steel pipe
Corrugated steel pipe shall satisfy the material and fabrication requirements of CSA G401.

7.6.2.3 Soil materials
Unless supported by in-situ or laboratory testing using recognized geotechnical engineering investigation and evaluation methods, the design shall be based on the soil properties specified in Table 7.4 for the various soils classified in Table 7.5. When the Standard Proctor densities are other than those specified in Table 7.5, linear interpolation shall be used to obtain the value of E_s. In the absence of laboratory data, the value of E_s for CLSM shall be assumed to be 30 MPa.

Table 7.4
Soil classifications
(See Clauses 7.6.2.3 and 7.6.5.6.2 and Table 7.5.)

Soil group	Grain size	Soil types	Unified Soil Classification symbol*
I	Coarse	Well-graded gravel or sandy gravel	GW
		Poorly graded gravel or sandy gravel	GP
		Well-graded sand or gravelly sand	SW
		Poorly graded sand or gravelly sand	SP
II	Medium	Clayey gravel or clayey-sandy gravel	GC
		Clayey sand or clayey gravelly sand	SC
		Silty sand or silty gravelly sand	SM

**According to ASTM D 2487.*

Table 7.5
Secant modulus of soil, E_s, for various soils
(See Clauses 7.2, 7.6.2.3, and 7.6.3.3.1.)

Soil group*	Standard Proctor density, %†	E_s, MPa
I	85	6
	90	12
	95	24
	100	30
II	85	3
	90	6
	95	12
	100	15

**See Table 7.4.*
†*According to ASTM D 698.*

7.6.3 Design criteria

7.6.3.1 Thrust

7.6.3.1.1 General
The thrust, T_f, in the conduit wall due to factored live loads and dead loads shall be calculated for ULS Combination 1 of Table 3.1 as follows:

$$T_f = \alpha_D T_D + \alpha_L T_L (1 + DLA)$$

where T_D and T_L are calculated in accordance with Clauses 7.6.3.1.2 and 7.6.3.1.3, respectively, and the dynamic load allowance, DLA, is obtained from Clause 3.8.4.5.2.

7.6.3.1.2 Dead loads
The thrust, T_D, in the conduit walls due to the overfill shall be calculated as follows:

$$T_D = 0.5(1.0 - 0.1C_s)A_f W$$

where
(a) C_s, the axial stiffness parameter, is calculated as follows:

$$C_s = \frac{1000E_s D_v}{EA}$$

(b) A_f is obtained from Figure 7.2 for the relevant values of D_h/D_v and H/D_h, where D_h and D_v are as shown in Figure 7.1. For H/D_h smaller than 0.2, A_f shall be obtained by graphical or numerical extrapolation, provided that the value of H is not smaller than the minimum depth of cover permitted by Clause 7.6.4.1; and
(c) W is the weight of the column of earth and the pavement above the conduit, as shown in Figure 7.2.

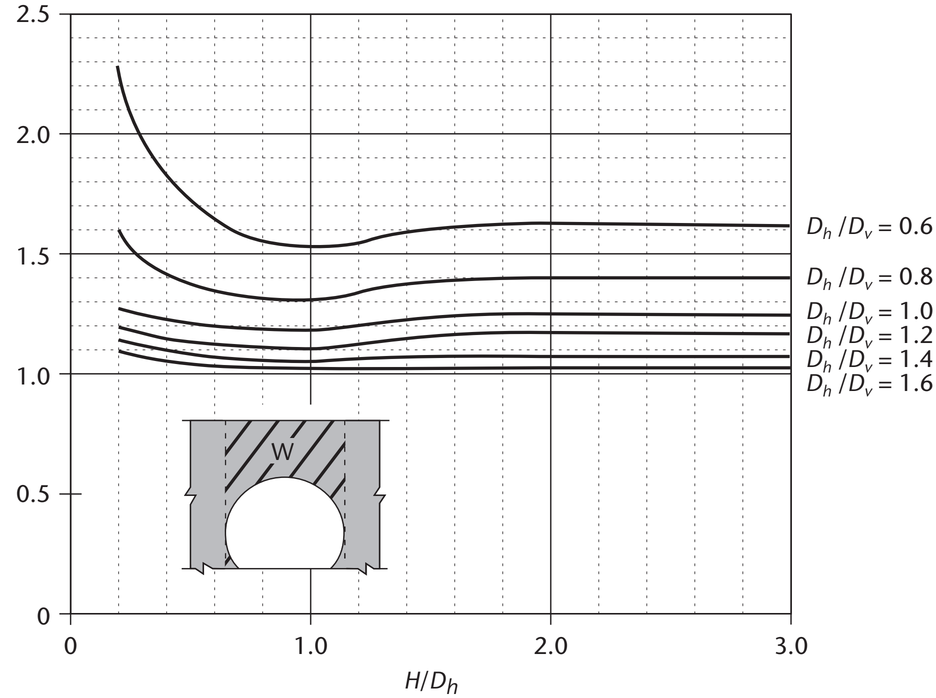


Figure 7.2
Values of A_f
(See Clause 7.6.3.1.2.)

7.6.3.1.3 Live loads

The thrust, T_L, shall be assumed to be constant around the conduit wall, and its value shall be the lesser of

$$T_L = 0.5 D_h \sigma_L m_f$$

$$= 0.5 \ell_t \sigma_L m_f$$

where

(a) ℓ_t is the distance between the outermost axles, including the tire footprints, placed in accordance with Item (c)(i) plus $2H$;

(b) m_f is the modification factor for multi-lane loading obtained from Clause 3.8.4.2, in accordance with the number of vehicles considered; and

(c) the load case yielding the maximum value of $\sigma_L m_f$ governs. σ_L is obtained as follows:

  (i) within the span length, position as many axles of the CL-W Truck or Trucks at the road surface above the conduit as would give the maximum total load;

  (ii) distribute the rectangular wheel loads through the fill down to the crown level at a slope of one vertically to one horizontally in the transverse direction of the conduit and two vertically to one horizontally in the longitudinal direction; and

  (iii) obtain the equivalent uniformly distributed pressure σ_L by assuming that the total wheel loads considered in Item (i) are uniformly distributed over the rectangular area that encloses the individual rectangular areas obtained in Item (ii).

7.6.3.2 Wall strength in compression

For the purposes of this Clause, the conduit wall shall be divided into lower and upper zones separated from each other by two symmetrical radial lines with their centre at the centre of curvature of the arc at the crown, and with an angle θ_0, in radians, from the vertical calculated as follows:

$$\theta_0 = 1.6 + 0.2\log\left[\frac{EI}{E_m R^3}\right]$$

At the ultimate limit state, the compressive stress, $\sigma = T_f/A$, shall not exceed the factored failure compressive stress, f_b, calculated as follows:

(a) for $R \le R_e$:

$$f_b = \phi_t F_m \left[F_y - \frac{\left(F_y KR\right)^2}{12Er^2\rho}\right]$$

(b) for $R > R_e$:

$$f_b = \frac{3\phi_t \rho F_m E}{\left[\frac{KR}{r}\right]^2}$$

where

(i) ϕ_t for compressive strength is obtained from Clause 7.5.3;

(ii) F_m = 1.0 for structures with single conduits

$$= \left[0.85 + \frac{0.3S}{D_h}\right] \le 1.0 \text{ for structures with multiple conduits}$$

where S is the least transverse clear spacing between adjacent conduits and D_h corresponds to the largest conduit in the structure and is as shown in Figure 7.1. The value of F_m shall be assumed to be 1.0 for upper portions of soil-metal structures with deep corrugations;

(iii) $R_e = \frac{r}{K}\left[\frac{6E\rho}{F_y}\right]^{0.5}$

(iv) $\rho = \left[1000\frac{(H+H')}{R_c}\right]^{0.5} \le 1.0$

(v) $K = \lambda\left[\frac{EI}{E_m R^3}\right]^{0.25}$

(vi) E_m for the side and bottom portions of the conduit wall shall be the same as E_s, but for the upper quarter of the conduit wall, it shall be calculated as follows:

$$E_m = E_s\left[1 - \left[\frac{R_c}{R_c + 1000[H+H']}\right]^2\right]$$

When the conduit wall is supported by a combination of compacted soil and CLSM, the value of E_m shall be based on the lower value of E_s for the two materials; and

(vii) λ for the upper segments of the conduit wall of all structures except single-radius part-arches with rise-to-span ratios of less than 0.4 shall be calculated as follows:

$$\lambda = 1.22\left[1.0 + 1.6\left[\frac{EI}{E_m R_c^3}\right]^{0.25}\right]$$

For all other cases, λ shall be 1.22.

7.6.3.3 Wall strength in bending and compression

7.6.3.3.1 Wall strength during construction

For soil-metal structures with shallow or deep corrugations, the Plans shall specify the maximum axle load, A_c, of the construction equipment to be used above the conduit. The combined effects of the bending moment and axial thrust arising from the unfactored dead load and the specified construction equipment shall not exceed the factored plastic moment capacity of the section at all stages of construction, where the combined bending moment and axial thrust are calculated as follows:

$$\left[\frac{P}{P_{Pf}}\right]^2 + \left|\frac{M}{M_{Pf}}\right| \le 1$$

where

P = $T_D + T_C$ (for H_c smaller than the minimum depth of cover required by Clause 7.6.4.1, P shall be assumed to be zero)

P_{Pf} = $\phi_c AF_y$

M = $M_1 + M_B + M_C$

where

M_1 = $k_{M1} R_B \gamma D_h^3$

M_B = $-k_{M2} R_B \gamma D_h^2 H_c$

M_C = $k_{M3} R_L D_h L_c$

where

k_{M1} = $0.0046 - 0.0010 \log_{10}(N_F)$ for $N_F \le 5000$

= 0.0009 for $N_F > 5000$

k_{M2} = $0.018 - 0.004 \log_{10}(N_F)$ for $N_F \le 5000$

= 0.0032 for $N_F > 5000$

k_{M3} = $0.120 - 0.018 \log_{10}(N_F)$ for $N_F \le 100\,000$

= 0.030 for $N_F > 100\,000$

R_B = $0.67 + 0.87[(D_v/2D_h) - 0.2]$ for $0.2 \le D_v/2D_h \le 0.35$

= $0.80 + 1.33[(D_v/2D_h) - 0.35]$ for $0.35 < D_v/2D_h \le 0.50$

= D_v/D_h for $D_v/2D_h > 0.5$

R_L = $[0.265 - 0.053 \log_{10}(N_F)]/(H_c/D_h)^{0.75} \le 1.0$

L_c = A_c/k_4

M_{Pf} = $\phi_{hc} M_P$

$\left|\frac{M}{M_{Pf}}\right|$ = absolute value of the ratio M/M_{Pf}

k_4 shall be interpolated from the values specified in Table 7.6 and N_F shall be calculated as follows:

$N_F = E_s(1000D_h)^3/EI$

where E_s is as specified in Table 7.5.

Table 7.6
Values for k_4 for calculating equivalent line loads
(See Clauses 7.6.3.3.1, 7.6.3.3.2, and 7.7.3.1.3.)

	k_4, m		
Depth of cover, m	**Two wheels per axle**	**Four wheels per axle**	**Eight wheels per axle**
0.3	1.3	1.5	2.6
0.6	1.6	2.0	2.8
0.9	2.1	2.7	3.2
1.5	3.7	3.8	4.1
2.1	4.4	4.4	4.5
3.0	4.9	4.9	4.9
4.6	6.7	6.7	6.7
6.1	8.5	8.5	8.5
9.1	12.2	12.2	12.2

7.6.3.3.2 Wall strength of completed structure

For completed soil-metal structures with deep corrugations, the combined effects of the bending moment and axial thrust at the ultimate limit state shall not exceed the factored plastic moment capacity of the section, where the combined bending moment and axial thrust are calculated as follows:

$$\left[\frac{T_f}{P_{Pf}}\right]^2 + \left|\frac{M_f}{M_{Pf}}\right| \leq 1.0$$

where T_f is calculated in accordance with Clause 7.6.3.1.1 and P_{Pf}, M_f, and M_{Pf} are calculated as follows:

$P_{Pf} = \phi_h A F_y$

$M_f = |\alpha_D M_1 + \alpha_D M_D| + \alpha_L M_L (1 + DLA)$

where

$M_1 = k_{M1} R_B \gamma D_h^3$

$M_D = -k_{M2} R_B \gamma D_h^2 H_e$

where

H_e = smaller of H and $D_h/2$

$M_L = \frac{k_{M3} R_U D_h A_L}{k_4}$

where k_{M1}, k_{M2}, k_{M3}, and R_B are obtained from the equations in Clause 7.6.3.3.1, A_L is the weight of the second axle of the CL-W Truck, and k_4 is obtained by interpolation from Table 7.6 for H up to 3.0 m. For H greater than 3.0 m, k_4 shall be assumed to be 4.9 m. R_U shall be calculated as follows:

$$R_U = \frac{0.265 - 0.053\log_{10} N_F}{(H/D_h)^{0.75}} \leq 1.0$$

M_{Pf} = $\phi_h M_P$

$\left|\frac{M_f}{M_{Pf}}\right|$ = absolute value of the ratio M_f/M_{Pf}

7.6.3.4 Connection strength

The factored strength of longitudinal connections of conduit walls, $\phi_j S_s$, shall not be less than T_f, in which the fifth percentile strength, S_s, may be evaluated experimentally or obtained from Approved test data or published standards.

7.6.3.5 Maximum difference in plate thickness

The difference in the thicknesses of the plates meeting at a longitudinal connection shall not exceed 1 mm if the thinner plate has a thickness of less than 3.1 mm, or exceed 1.5 mm if one of the plates has a thickness between 3.1 and 3.5 mm.

7.6.3.6 Radius of curvature

The radius of curvature of the conduit wall, R, at any location shall not be less than $0.2R_c$ unless Approved. The ratio of the radii of mating plates at a longitudinal connection shall not be more than 8.

7.6.4 Additional design requirements

7.6.4.1 Minimum depth of cover

For soil-metal structures with shallow corrugations, unless the conduit wall is designed using an Approved method other than one specified in this Section, the minimum depth of cover, H_{min}, in metres, as shown in Figure 7.3, shall be the largest of

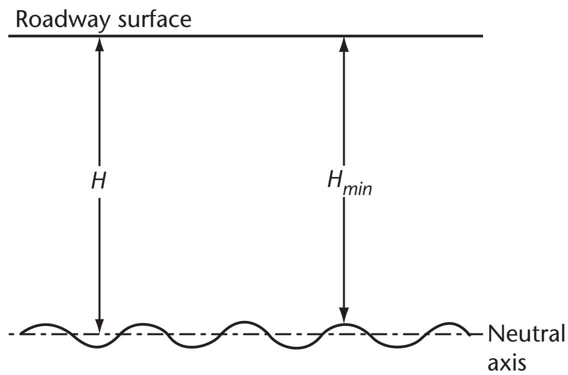
(a) 0.6

(b) $\frac{D_h}{6}\left[\frac{D_h}{D_v}\right]^{0.5}$

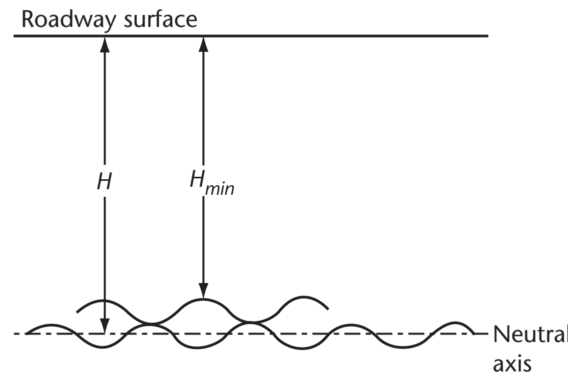
(c) $0.4\left[\frac{D_h}{D_v}\right]^2$

where D_h and D_v are as shown in Figure 7.1.

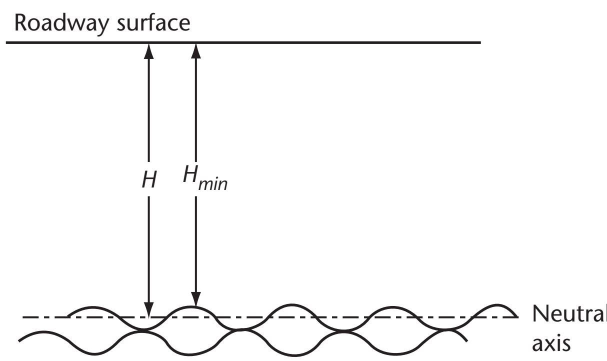
For soil-metal structures with deep corrugations, the minimum depth of cover shall be the smaller of 1.5 m and the minimum depth of cover for structures with shallow corrugations but the same conduit size.



Single corrugated plate



Single corrugated plate partially stiffened with sections of corrugated plate or other stiffeners



Continuously stiffened double corrugation

Figure 7.3
Depth of cover, H and H_{min}, for soil-metal structures and metal box structures
(See Clauses 7.2, 7.6.4.1, and 7.7.4.1.)

7.6.4.2 Foundation treatment for pipe-arches

Foundations below the haunches of pipe-arches shall be treated as follows:

(a) for dense to very dense cohesionless foundations and for stiff to hard cohesive foundations, no treatment shall be required;
(b) for soft to firm cohesive foundations, reinforcement shall be provided as shown in Figure 7.4; and
(c) for loose to compact cohesionless foundations, reinforcement shall be provided as shown in Figure 7.4 or by in-situ compaction.

Geotechnical engineering judgment shall be used to determine the state of the foundation indicated in Items (a) to (c).

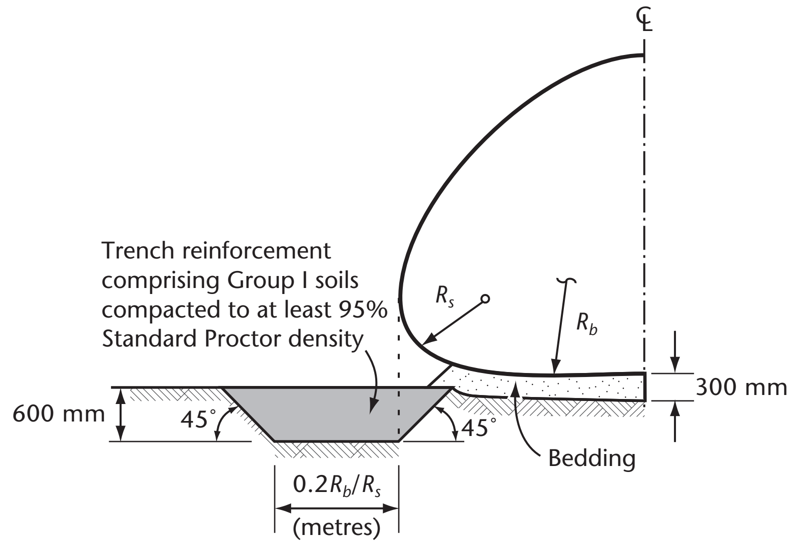


Figure 7.4
Trench reinforcement for the foundation of pipe-arches
(See Clause 7.6.4.2.)

7.6.4.3 Durability
The durability of the structure shall be ensured for the specified design life with respect to the environment to which it will be exposed, in accordance with the relevant requirements of Section 2.

7.6.5 Construction

7.6.5.1 General
The Plans shall specify the construction procedures and quality controls to be used.

7.6.5.2 Deformation during construction
For all conduit shapes, the upward or downward crown deflection shall not exceed 2% of the rise unless Approved. Longitudinal and transverse alignment shall be maintained. If struts or cables are used to maintain the conduit shape during assembly or backfilling, they shall be removed before they restrict the downward movement of the crown.

7.6.5.3 Foundations
When the foundation exhibits non-uniform characteristics, their effects shall be assessed and treated if necessary to ensure acceptable behaviour of the conduit.

7.6.5.4 Bedding
The bedding shall consist of free-draining, well-graded granular material, and be preshaped in the transverse direction to accommodate the curved invert. A 200 mm thickness of the bedding layer that is in direct contact with the invert shall be left uncompacted.

7.6.5.5 Assembly and erection

Bolts at longitudinal connections shall be arranged in accordance with one of the two arrangements shown in Figure 7.5. When arrangement (b) is used, the bolts in the row closest to a visible edge of the mating plate shall be in the valleys and those in the other row shall be on the ridges. The torque on the bolts prior to backfilling shall be between 200 and 340 N•m. Before backfilling, at least 5% of the bolts used in each circumferential and longitudinal connection shall be tested after assembly.

The test bolts shall be randomly selected and the installation shall be considered acceptable if the torque requirement is met in at least 90% of the bolts tested.

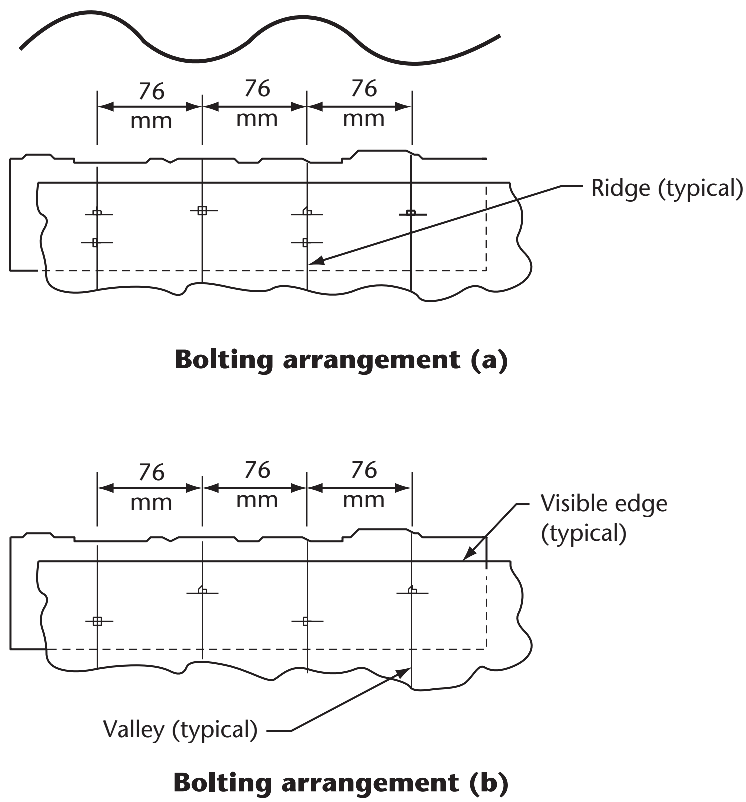


Figure 7.5
Longitudinal seam bolting arrangements
(See Clause 7.6.5.5.)

7.6.5.6 Structural backfill

7.6.5.6.1 Extent of structural backfill

The structural backfill in single-conduit structures under different fill conditions shall extend transversely at least the length specified in Table 7.7 on each side beyond the spring lines of the conduit, and vertically up to the minimum depth of cover required by Clause 7.6.4.1.

For multi-conduit structures, structural backfill shall be provided between the adjacent conduits and shall extend transversely beyond the outer conduits at least the applicable distance specified in Table 7.7 for single-conduit structures. In the vertical direction, the structural backfill shall extend up to the minimum depth of cover required by Clause 7.6.4.1.

Table 7.7
Minimum transverse distance of backfill in single-conduit soil-metal structures
(See Clause 7.6.5.6.1.)

Backfill condition	Minimum transverse distance beyond each spring line, m
Structure constructed in trench in which the natural soil is as good as, or better than, the engineered soil	Smaller of 2.0 m and $D_h/2$
Structure constructed in trench in which the natural soil is poorer than the engineered soil	Smaller of 5.0 m and $D_h/2$, but not less than the smaller of rise and $D_v/2$
Structure constructed on embankment	Smaller of 5.0 m and $D_h/2$, but not less than the smaller of rise and $D_v/2$

7.6.5.6.2 Material for structural backfill

The material for structural backfill shall be boulder free and shall be selected from the Group I or II soils specified in Table 7.4, with compaction corresponding to the modulus of soil stiffness used in the design. The backfill shall be placed and compacted in layers not exceeding 200 mm of compacted thickness, with each layer compacted to the required density prior to the addition of the next layer. The difference in levels of structural backfill on the two sides of a conduit at any transverse section shall not exceed 200 mm. The structural backfill within 300 mm of the conduit walls shall be free of stones exceeding 75 mm in any dimension. Heavy equipment shall not be allowed within 1 m of the conduit walls.

The structural backfill adjacent to the conduit wall and to within the frost penetration depth shall be free of frost-susceptible soils.

CLSM, if used, shall be considered part of the structural backfill.

7.6.6 Special features

Soil-metal structures may be designed with structural or soil modifications or both. However, for compliance with the requirements of Clause 7.6, the properties of the affected components resulting from such modifications shall be determined from laboratory tests or field observations.

7.6.7 Site supervision and construction control

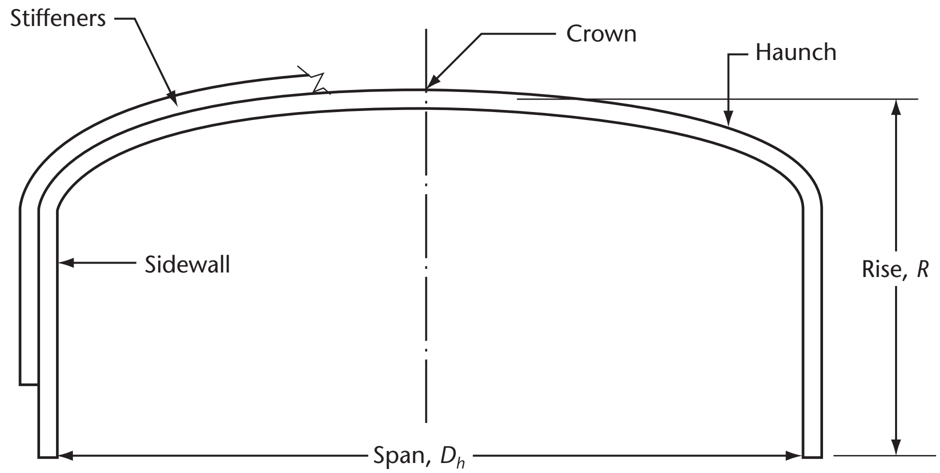
The Plans shall specify
(a) the requirements for testing of soil compaction;
(b) that the supervision of the construction of soil-metal structures shall be undertaken by an Engineer who is experienced in the design and construction of such structures; and
(c) the following procedures for inspection, as applicable:
   (i) for structures with spans between 3.0 and 6.0 m, that the work shall be inspected by the Engineer or a designated representative at the completion of the bedding, the erection of the conduit walls, the placement of the backfill under the haunches, the placement of the backfill up to the spring lines, the placement of the backfill up to the crown, and the placement of the backfill up to the level of minimum cover;
   (ii) for structures with spans between 6.0 and 8.0 m, that the inspections in Item (i) shall be conducted and that the construction shall be inspected daily by the Engineer or a designated representative until the backfill has reached the minimum depth of cover; and
   (iii) for structures with spans greater than 8 m or for which special techniques are used in accordance with Clause 7.6.6, that all stages of construction shall be inspected on a full-time basis by the Engineer.

7.7 Metal box structures

7.7.1 General

The soil-structure interaction requirements of Clauses 7.7.2 to 7.7.7 shall apply to the design of steel and aluminum box structures with the dimensional limits shown in Figure 7.6 and a depth of cover up to 1.5 m. For metal box structures beyond these limits, soil-structure interaction shall not be considered and the structure shall be designed in accordance with the relevant requirements of Section 10. For spans greater than 8 m or rises greater than 3.2 m, the forces in the structure shall be calculated using rigorous methods of analysis that take into account the beneficial effects of soil-structure interaction.

The rigorous methods of analysis may also be used for other metal box structures in lieu of the methods specified in Clauses 7.7.3.1.2 and 7.7.3.1.3.



Element	Minimum, m	Maximum, m
Rise, R	0.8	3.2
Span, D_h	2.7	8.0

Figure 7.6
Metal box structure dimensional limits
(See Clauses 7.2 and 7.7.1.)

7.7.2 Structural materials

7.7.2.1 Structural metal plates

Structural metal plates shall satisfy the requirements of Clause 7.6.2.1.

7.7.2.2 Soil materials

The soil properties shall be determined in accordance with Clause 7.6.2.3.

7.7.3 Design criteria

7.7.3.1 Design criteria for crown and haunches

7.7.3.1.1 General

The factored crown and haunch bending moments, M_{cf} and M_{hf}, induced by factored dead and live loads shall be calculated for ULS Combination 1 of Table 3.1 as follows:

$M_{cf} = \alpha_D M_{cD} + \alpha_L M_{cL}(1 + DLA)$

$M_{hf} = \alpha_D M_{hD} + \alpha_L M_{hL}(1 + DLA)$

where M_{cD} and M_{hD} are calculated in accordance with Clause 7.7.3.1.2 and M_{cL} and M_{hL} are calculated in accordance with Clause 7.7.3.1.3.

7.7.3.1.2 Dead loads

The intensities of bending moments at the crown and the haunch due to dead loads, M_{cD} and M_{hD}, shall be obtained as fractions of M_D and calculated as follows:

$M_{cD} = \kappa M_D$

$M_{hD} = (1 - \kappa)M_D$

where

κ = crown moment coefficient

= $0.70 - 0.0328D_h$

$$M_D = k_1\gamma D_h^3 + k_2\gamma\left[H - \left[0.3 + \frac{d_c}{2000}\right]\right]D_h^2$$

where

k_1 = $0.0053 - 0.00024\,(3.28D_h - 12)$

k_2 = 0.053

7.7.3.1.3 Live loads

The intensities of bending moments at the crown and the haunch due to live loads, M_{cL} and M_{hL}, shall be obtained as fractions of M_L and calculated as follows:

$M_{cL} = \kappa M_L$

$M_{hL} = (1 - \kappa)k_R M_L$

where

κ = crown moment coefficient

= $0.70 - 0.0328D_h$

k_R = $0.425H + 0.48 \le 1.0$

M_L = $C_1 k_3 L_L D_h$

where

C_1 = 1.0 for single axles

= $0.5 + \dfrac{D_h}{15.24} \le 1.0$ for multiple axles

$$k_3 = \frac{0.08}{\left[\dfrac{H}{D_h}\right]^{0.2}} \quad \text{for } D_h \le 6.0 \text{ m}$$

$$= \frac{\left[0.08 - 0.002(3.28D_h - 20)\right]}{\left[\frac{H}{D_h}\right]^{0.2}} \text{ for } 6 \text{ m} < D_h < 8 \text{ m}$$

$L_L = A_L / k_4$

where A_L is the weight of a single axle of the CL-W Truck for $D_h < 3.6$ m, or the combined weight of the two closely spaced axles of the CL-W Truck for $D_h \geq 3.6$ m, and k_4 is a factor for calculating the line load, as specified in Table 7.6

7.7.3.1.4 Flexural capacity at the ultimate limit state

At the ultimate limit state, neither the factored crown moment, M_{cf}, nor the factored haunch moment, M_{hf}, shall exceed the factored plastic moment, M_{Pf}, calculated as follows:

$M_{Pf} = \phi_h M_P$

where

ϕ_h = resistance factor for plastic hinge, as specified in Clause 7.5.3

M_P = plastic moment of the section

7.7.3.1.5 Fatigue resistance

Longitudinal bolted seams shall not be located in the vicinity of the crown nor in areas of maximum live load moments at haunches. For spans greater than 8.0 m, consideration shall be given to the fatigue resistance of the bolted seams.

7.7.3.2 Design criteria for connections

For conduit walls designed for only bending moments, the factored moment resistance of longitudinal connections, $\phi_j S_M$, shall not be less than M_{Pf}. For sidewalls designed for both axial thrust and bending moments, the factored axial strength of longitudinal connections, $\phi_j S_s$, shall not be less than T_f. The fifth percentile strengths, S_M and S_s, may be evaluated experimentally or obtained from Approved test data or from published standards. Connections shall be designed at the ultimate limit state for the larger of
(a) the calculated moment due to factored loads at the connection; and
(b) 75% of the factored resistance of the member, $\phi_h M_p$.

7.7.4 Additional design considerations

7.7.4.1 Depth of cover

The minimum depth of cover, H_{min}, shown in Figure 7.3, shall be 0.3 m.

7.7.4.2 Durability

The requirements of Clause 7.6.4.3 shall be satisfied.

7.7.5 Construction

7.7.5.1 Structural backfill

7.7.5.1.1 Extent of structural backfill

The extent of structural backfill shall be as shown in Figure 7.7.

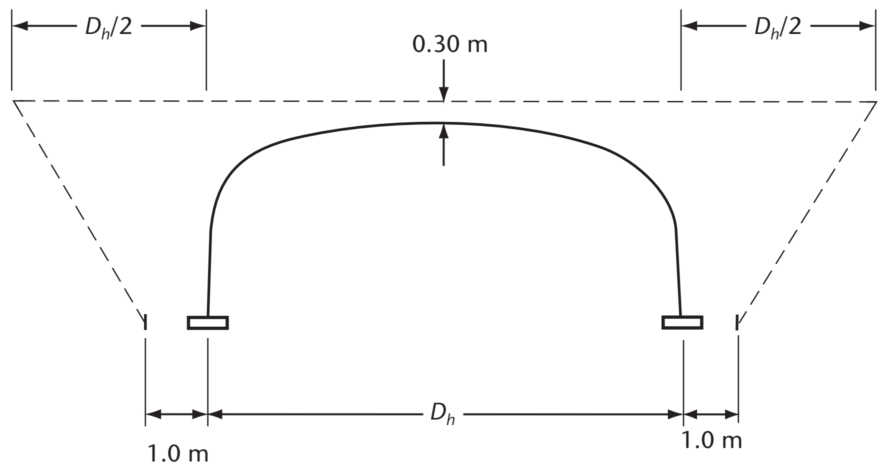


Figure 7.7
Minimum extent of structural backfill for metal box structures
(See Clause 7.7.5.1.1.)

7.7.5.1.2 Materials for structural backfill

The materials for structural backfill shall satisfy the requirements of Clause 7.6.5.6.2.

7.7.5.2 Deformation during construction

The upward or downward crown deflection during construction shall not exceed 1% of the span unless a greater deflection is Approved.

7.7.6 Special features

The use of special features to improve the structural performance, and their effects on the requirements of Clauses 7.7.1 to 7.7.5, shall be subject to Approval.

7.7.7 Site supervision and construction control

The site supervision and construction control of metal box structures shall be in accordance with Clause 7.6.7.

7.8 Reinforced concrete buried structures

7.8.1 Standards for structural components

For reinforced concrete buried structures, the materials, methods of material testing, and construction practices for concrete shall be in accordance with Clause 8.4.1.1, and those for reinforcing bars and meshes with Clauses 8.4.2.1.1 and 8.4.2.2. In addition, the manufacturing Standards applicable to generic precast buried concrete structures shall be as specified in Table 7.8. For precast segmental structures, including non-standard arches and three-sided boxes with flat and curved tops (some of which can be proprietary products), the manufacturing Standards shall be those specified in Table 7.8 (adapted as necessary) and such other Standards as are applicable (also adapted as necessary).

Table 7.8
Standards for precast buried concrete structures
(See Clause 7.8.1.)

Category of structure	Standard
Arch pipe	ASTM C 506M
Box sections	ASTM C 1433M or ASTM PS 62
Circular pipe	CAN/CSA-A257.1 and CAN/CSA-A257.2
Elliptical pipe	ASTM C 507M
Manholes and catchbasins	CAN/CSA-A257 Series

7.8.2 Standards for joint gaskets for precast concrete units
Elastomeric gaskets used for sealing precast concrete units shall comply with CAN/CSA-A257.3.

7.8.3 Installation criteria

7.8.3.1 Backfill soils
The extent of the soil included in the design of the structure shall be as specified in Clauses 7.8.3.5 and 7.8.3.6 for the applicable structure, with the classification of placed soils being as specified in Table 7.9.

Table 7.9
Classification of placed soils
(See Clauses 7.8.3.1 and 7.8.3.5.2.)

Soil group*	Description	Unified Soil Classification symbols†
I	Sand and gravel	SW, SP, GW, GP
II	Sandy silt	GM; SM; ML; GC and SC with less than 20% passing #200 sieve
III	Silty clay	CL; MH; GC and SC with more than 20% passing #200 sieve

**See Table 7.4.*
†According to ASTM D 2487.

7.8.3.2 Minimum depth of cover for structures with curved tops
For concrete structures with curved-top segments designed in accordance with the empirical methods specified in this Section, the minimum depth of cover shall be as follows:
(a) for structures below unpaved and flexible pavements: the greater of 300 mm and one-fourth the radius of curvature of the top segment; and
(b) for structures below rigid pavements: 150 mm plus the thickness of the pavement.

7.8.3.3 Compaction
Unless otherwise Approved, the measure of the compaction of placed soils shall be the Standard or Modified Proctor density in accordance with ASTM D 698 and ASTM D 1557, respectively.

7.8.3.4 Frost penetration
For concrete structures in climates where frost can penetrate embedment soils, frost-susceptible soils shall not be used adjacent to the conduit wall within the depth of frost penetration unless, for design purposes, they are considered to be uncompacted.

7.8.3.5 Standard installations for circular precast concrete pipes

7.8.3.5.1 General

Four types of installation for circular precast concrete pipes are specified in Table 7.10. Pipes in these installations shall be analyzed in accordance with the empirical method specified in Clause 7.8.5.2. The extent of the different zones of the backfill shall be as shown in Figures 7.8. and 7.9.

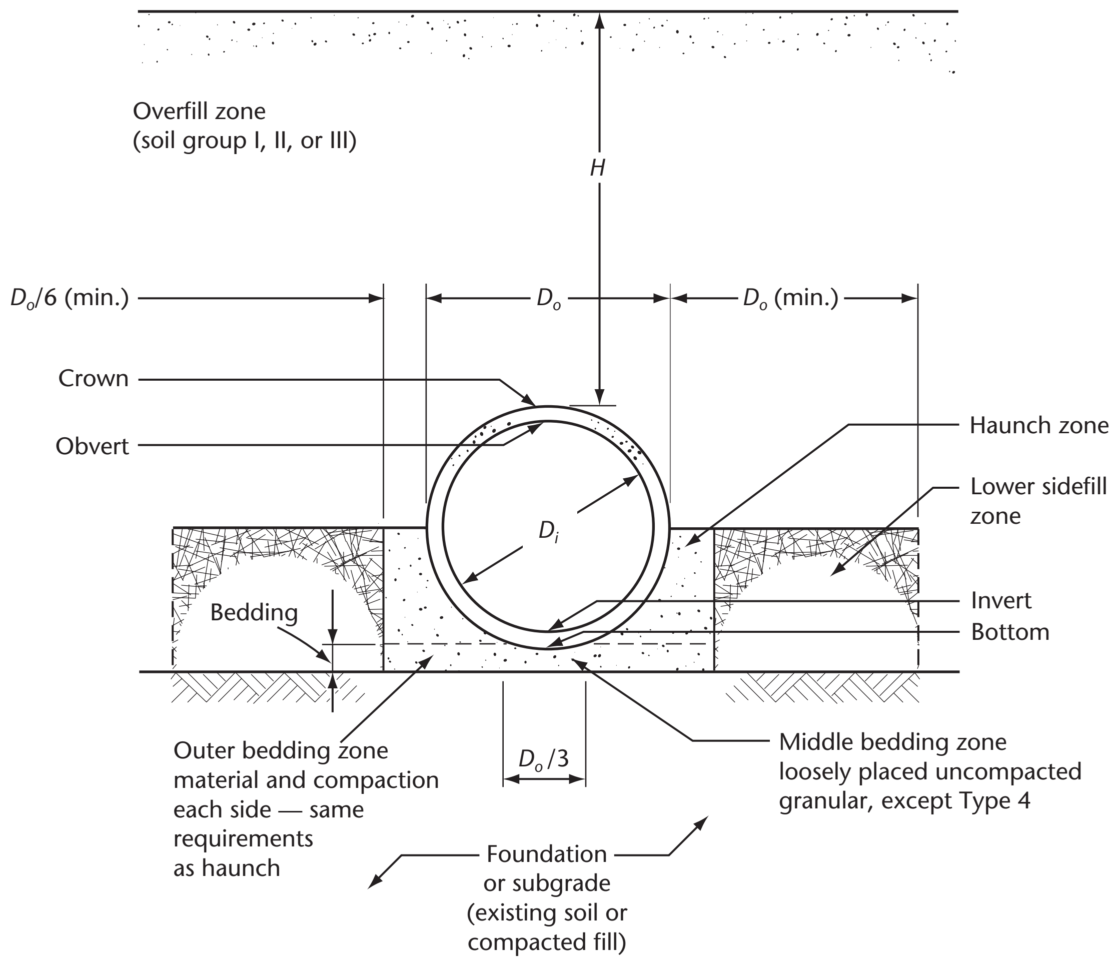


Figure 7.8
Terminology and standard installations for circular precast concrete pipes on embankments
(See Clauses 7.2, 7.8.3.5.1, 7.8.3.5.2, and 7.8.15.6.1.)

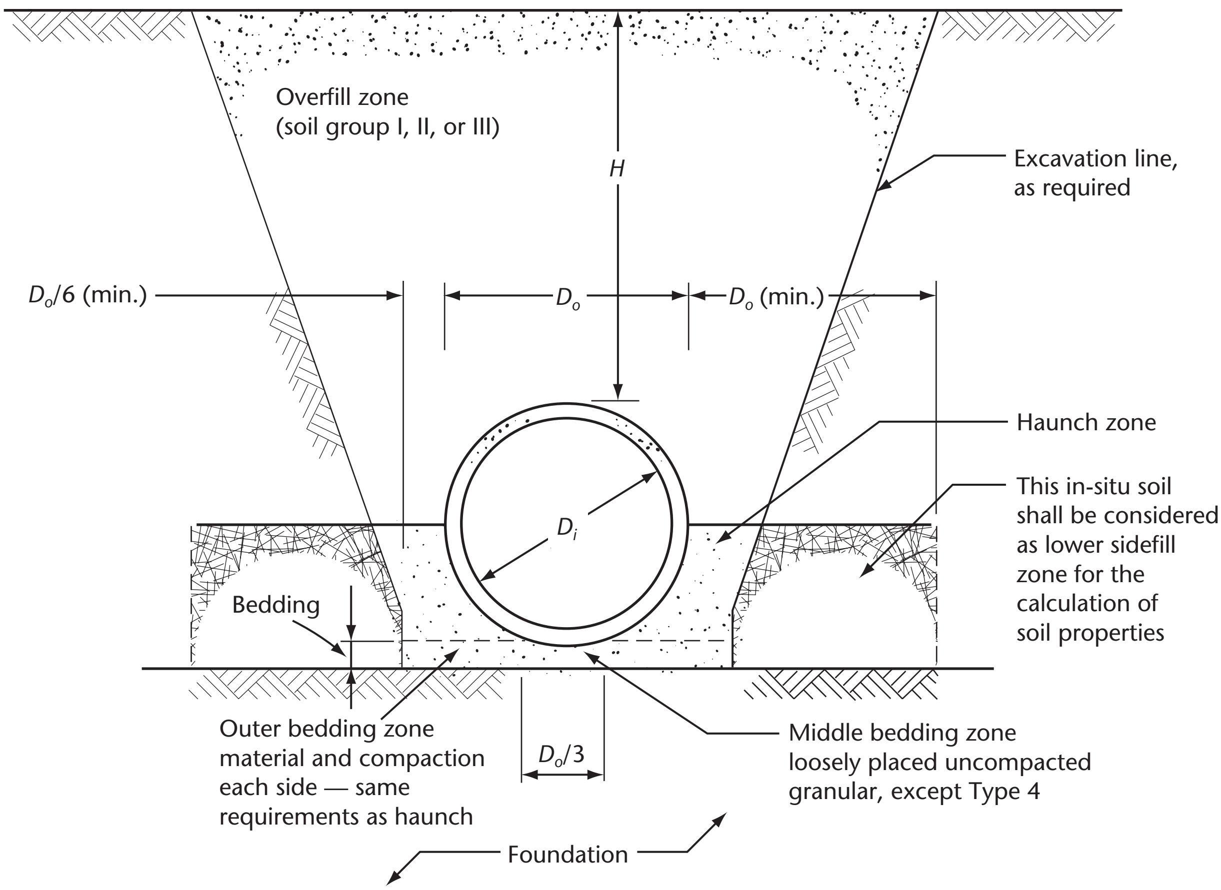


Figure 7.9
Terminology and standard installations for circular precast concrete pipes in trenches
(See Clauses 7.2, 7.8.3.5.1, 7.8.3.5.2, and 7.8.15.6.1.)

Table 7.10
Soils and compaction requirements for standard installations for circular precast concrete pipes
(See Clauses 7.8.3.5.1 and 7.8.3.5.2.)

Installation type	Minimum bedding thickness		Equivalent minimum Standard Proctor compaction		
	Soil foundations	Rock foundations	Soil group	Haunch and outer bedding zones	Lower sidefill zone
C1	D_o / 24, but not less than 75 mm	D_o / 12, but not less than 150 mm	I	95%	90%
			II	Not permitted	95%
			III	Not permitted	100%
C2	D_o / 24 , but not less than 75 mm	D_o / 12, but not less than 150 mm	I	90%	85%
			II	95%	90%
			III	Not permitted	95%
C3	D_o / 24, but not less than 75 mm	D_o / 12, but not less than 150 mm	I	85%	85%
			II	90%	90%
			III	95%	95%
C4	No bedding needed	D_o / 12, but not less than 150 mm	I	No compaction needed	No compaction needed
			II	No compaction needed	No compaction needed
			III	85%	85%

7.8.3.5.2 Additional requirements for standard trench and embankment installations

The following additional requirements shall apply to standard trench and embankment installations:
(a) the soil in the haunch zone, as shown in Figures 7.8 and 7.9, shall be one of the engineered soil groups identified in Table 7.9;
(b) the soil in the lower sidefill zones and in the overfill zone, as shown in Figures 7.8 and 7.9, shall be
 (i) engineered soil in accordance with Table 7.9 and meet the requirements of Table 7.10; or
 (ii) an in-situ soil of equivalent stiffness;
(c) the extent of bedding shall be as shown in Figures 7.8 and 7.9;
(d) the properties of the soil within a distance D_o measured laterally from the conduit wall shall be used for the design of the structure; and
(e) the soil in the outer bedding, haunch, and lower sidefill zones shall be compacted to at least the same degree as the soil in the overfill zone.

7.8.3.5.3 Additional requirements for standard trench installations only

The following additional requirements shall apply to standard trench installations only:
(a) the trench width shall be sufficient to permit the proper use of equipment for compacting the backfill in the haunch zone; and
(b) in-situ soils that have cuts from 0 to 10° of vertical shall be considered equivalent to Group I soils compacted to 90% of Standard Proctor density.

7.8.3.6 Standard installations for precast and cast-in-place concrete boxes

7.8.3.6.1 General

Two types of installation for precast and cast-in-place concrete boxes are specified in Table 7.11. Box structures in these installations shall be analyzed in accordance with the empirical method specified in Clause 7.8.5.3. The extent of the different zones of the backfill shall be as shown in Figures 7.10 and 7.11.

Table 7.11
Soils and compaction requirements for standard installations for concrete boxes
(See Clauses 7.8.3.6.1 and Figures 7.10 and 7.11.)

Installation type	Soil group	Equivalent minimum Standard Proctor compaction in sidefill and outer bedding zones
B1	I	90%
	II	95%
	III	Not permitted
B2	I	80%
	II	85%
	III	95%

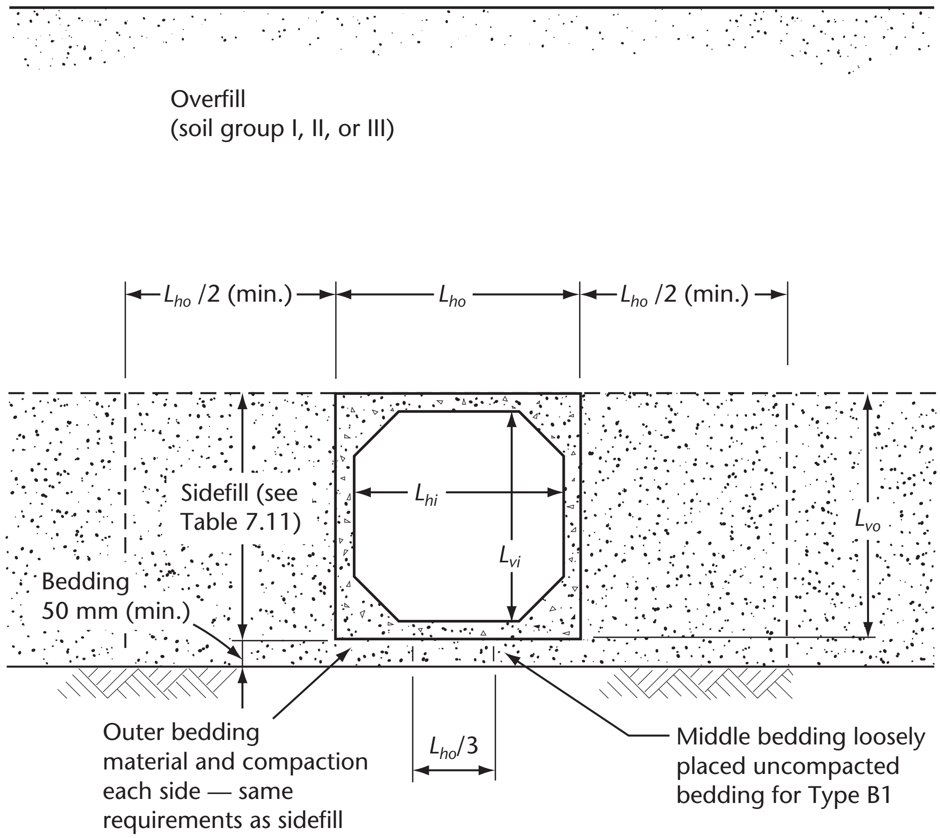


Figure 7.10
Standard installations for concrete box sections on embankments
(See Clauses 7.2, 7.8.3.6.1, and 7.8.15.6.1.)

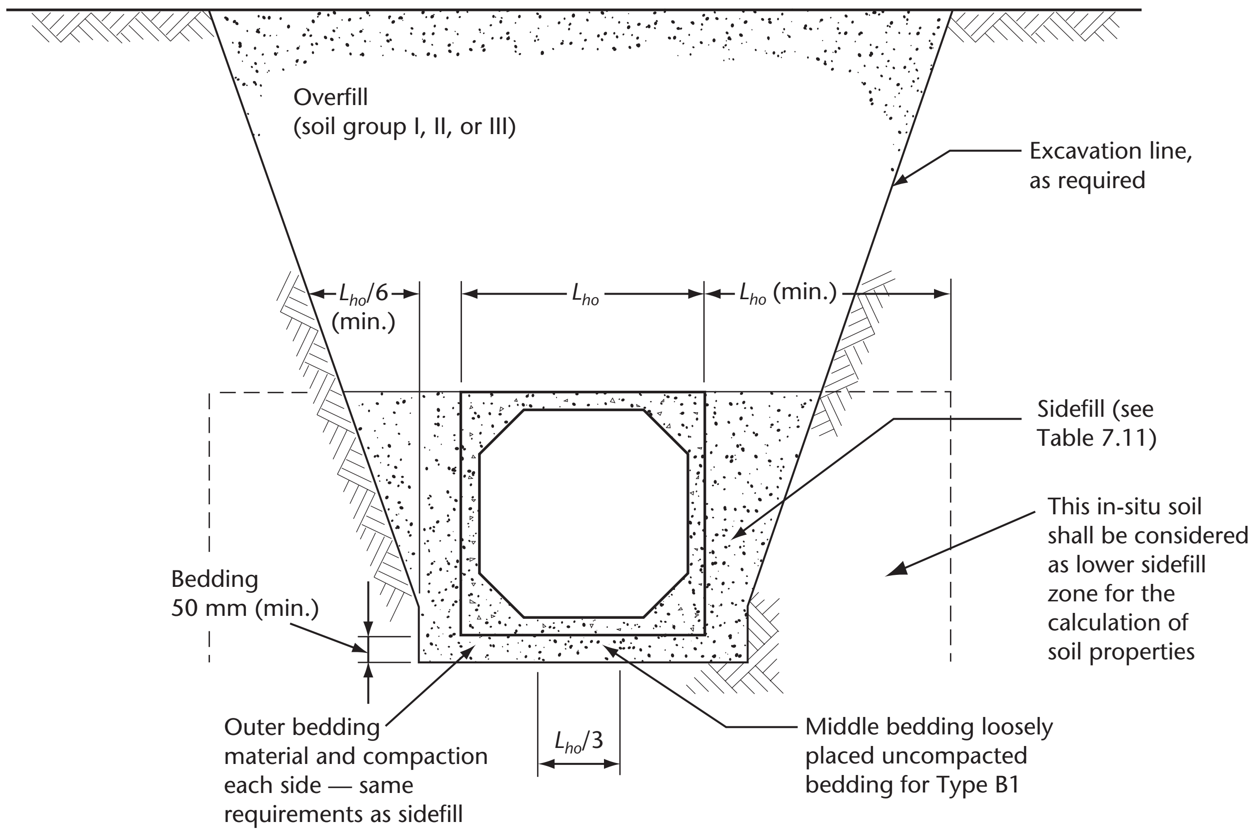


Figure 7.11
Standard installations for concrete box sections in trenches
(See Clauses 7.2, 7.8.3.6.1, and 7.8.15.6.1.)

7.8.3.6.2 Additional requirements for standard trench and embankment installations

The following additional requirements shall apply to standard trench and embankment installations:

(a) the soil in the outer bedding and sidefill zones shall be compacted to at least the same degree as the soil in the overfill zone; and

(b) the soil in the middle bedding zone shall be of the same material as that in the outer bedding zone, but shall not be compacted.

7.8.3.6.3 Additional requirements for standard trench installations only

The following additional requirements shall apply to standard trench installations only:

(a) the trench width shall be sufficient to permit the proper use of equipment for compacting the backfill; and

(b) in-situ soils that have cuts from 0 to 10° of vertical shall be considered equivalent to Group I soils compacted to 90% of Standard Proctor density.

7.8.3.7 Non-standard installations

Buried concrete structures may be placed in non-standard installations if they are designed in accordance with Approved methods based on soil-structure interaction.

7.8.4 Loads and load combinations

7.8.4.1 Load combinations

Consistent with the load combinations of Section 3, the following load combinations shall be considered for the ultimate limit state design:

(a) self-weight of the structure, earth load, live load, and water load, together with the minimum value in the range of lateral earth pressure;
(b) the combination specified in Item (a), except with the maximum value in the range of lateral earth pressure;
(c) the combination specified in Item (a), except that the live load is adjacent to structures with vertical sides instead of over the structure; and
(d) self-weight of the structure, earth, and earthquake load.

7.8.4.2 Earth load

7.8.4.2.1 General

Earth load shall be determined from the unit weight and height of overfill soil over the top of the structure and its effects shall be determined by an analysis of soil-structure interaction based on the characteristics of the installation.

7.8.4.2.2 Earth load on circular pipe in standard installations

The total vertical earth load acting on a buried pipe, W_e, shall be obtained by multiplying the weight of the column of earth over the outside diameter of the pipe, W, by the vertical arching factor, λ_v, for the specified standard installation type.

The total horizontal earth load acting on the buried pipe, P_h, shall be obtained by multiplying the weight of the column of earth over the outside diameter of the pipe, W, by the horizontal arching factor, λ_h, for the specified standard installation type.

The values of λ_v and λ_h for each standard installation type shall be as specified in Table 7.12.

Table 7.12
Vertical and horizontal arching factors for circular concrete pipes in standard installations
(See Clauses 7.8.4.2.2, 7.8.5.2.3, and 7.8.5.2.4.)

Installation type	Vertical arching factor, λ_v	Horizontal arching factor, λ_h
C1	1.35	0.45
C2	1.40	0.40
C3	1.40	0.37
C4	1.45	0.30

7.8.4.2.3 Earth load on box sections in standard installations

The vertical and horizontal earth loads shall be determined by multiplying the weight of earth over the top of the box section by the vertical and horizontal arching factors, λ_v and λ_h, respectively, as specified in Table 7.13.

Table 7.13
Vertical and horizontal arching factors for box sections in standard installations
(See Clauses 7.8.4.2.3 and 7.8.5.3.2.)

Installation type	Vertical arching factor, λ_v	Horizontal arching factor, λ_h	
		Minimum	Maximum
B1	1.20	0.30	0.50
B2	1.35	0.25	0.50

7.8.4.3 Live load

Live load shall be the applicable CL-W Truck load specified in Section 3 and shall include the dynamic load allowance specified in Clause 3.8.4.5.2.

7.8.4.4 Earthquake loads

The additional force effects due to earthquake loads shall be accounted for by multiplying the force effects due to self-weight and earth load, obtained in accordance with Clauses 7.8.6 and 7.8.7.1, by the vertical acceleration ratio, A_V, specified in Clause 7.5.5.1.

7.8.5 Earth pressure distribution from loads

7.8.5.1 General

Earth pressures acting on buried concrete structures shall be determined from a soil-structure interaction analysis for the soil/pipe installation. Earth pressure distributions developed for circular pipe and box sections in standard installations shall be as specified in Clauses 7.8.5.2 and 7.8.5.3, respectively.

7.8.5.2 Circular pipe in standard installations

7.8.5.2.1 Pipe weight

For the analysis of the effects of its self-weight, the pipe shall be assumed to be supported at the bottom over an arc length subtending an angle of 30° centred at the pipe invert. A radial pressure distribution at the pipe support shall be assumed to be sinusoidal, with the peak at the centre and zero at the edges.

7.8.5.2.2 Earth load

For the analysis of horizontal and vertical effects of earth load, a unit length of pipe shall be assumed to be subjected to the earth pressure distribution shown in Figure 7.12.

The various components of the Figure 7.12 force diagram shall be obtained by multiplying the earth load, W, by the factors specified in Table 7.14 for the applicable standard installation type.

The pressure distributions shall be taken to vary linearly or parabolically and the magnitude of their components shall be obtained by multiplying W/D_o by the relevant factors specified in Table 7.15 for the relevant standard installation type. The locations of the forces and peak values of pressures shall be determined by multiplying the length factors specified in Table 7.16 by D_o.

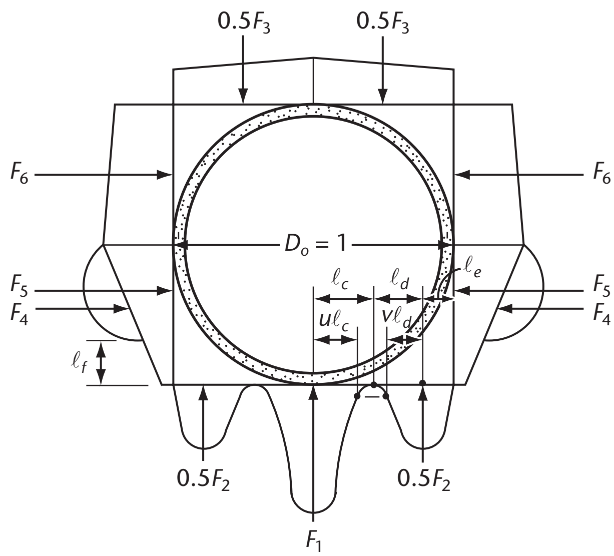


Figure 7.12
Earth pressure distribution for standard installations of circular concrete pipes (force diagram)
(See Clauses 7.8.5.2.2–7.8.5.2.4.)

Table 7.14
Force factors for earth loads
(See Clause 7.8.5.2.2.)

Installation type	F_1	F_2	F_3	F_4	F_5	F_6
C1	0.62	0.73	1.35	0.19	0.08	0.18
C2	0.85	0.55	1.40	0.15	0.08	0.17
C3	1.05	0.35	1.40	0.10	0.10	0.17
C4	1.45	0.00	1.45	0.00	0.11	0.19

Table 7.15
Earth pressure factors
(See Clause 7.8.5.2.3 and 7.8.5.2.4.)

Installation Type	p_a	p_b	p_c	p_d
C1	1.40	0.40	2.87	1.85
C2	1.45	0.40	3.51	1.48
C3	1.45	0.36	4.26	0.99
C4	1.45	0.30	4.58	0.00

Table 7.16
Length factors for earth pressures
(See Clause 7.8.5.2.2.)

Installation type	ℓ_c	ℓ_e	ℓ_f	u	v
C1	0.18	0.08	0.05	0.80	0.80
C2	0.19	0.10	0.05	0.82	0.70
C3	0.20	0.12	0.05	0.85	0.60
C4	0.25	0.00	—	0.90	—

Note: $\ell_d = 0.5 - \ell_c - \ell_e$.

7.8.5.2.3 Water loads
For determining the effects of water loads, a unit length of the pipe shall be assumed to be flowing full, and the non-dimensional earth pressure distribution on the bottom of the pipe shall be as shown in Figure 7.12 and specified in Table 7.15. To obtain the actual bottom pressures for an installation type, the force ratios F_1 and F_2 shall be multiplied by the total weight of water divided by the applicable vertical arching factor, λ_v, obtained from Table 7.12. Lateral pressure shall be neglected.

7.8.5.2.4 Live load
For the analysis of live load effects, a unit length of pipe shall be assumed to be subjected to a uniform pressure at the top of the pipe, determined by distributing the applied wheel load through the pavement (if any) and earth above the pipe over a distance specified in Clause 6.9.6. The reacting earth pressure on the bottom of the pipe shall be determined using the non-dimensional pressure distribution on the bottom of the pipe shown in Figure 7.12 and specified in Table 7.15. To obtain the actual bottom pressures for an installation type, the force ratios F_1 and F_2 shall be multiplied by the total live load acting on the pipe divided by the applicable vertical arching factor, λ_v, obtained from Table 7.12. Lateral pressure shall be neglected.

7.8.5.3 Box sections in standard installations

7.8.5.3.1 Box weight
For the analysis of the effects of its self-weight, the concrete box shall be assumed to be uniformly supported over its entire width.

7.8.5.3.2 Earth load
Earth pressures on box sections shall be assumed to be uniformly distributed vertical pressures, σ_v, and linearly varying horizontal pressures, σ_h, calculated as follows:

(a) $\sigma_v = \lambda_v w_c$

(b) $\sigma_h = \lambda_h w_c$

where the earth pressure arching factors, λ_v and λ_h, are as specified in Table 7.13 for the two standard installations, and w_c is the weight of a column of unit area of fill above the reference point. The maximum and minimum values of λ_h shall be used to obtain the maximum positive and negative moments in the conduit walls. The reaction pressure on the bottom of the box shall be assumed to be uniformly distributed.

7.8.5.3.3 Live load

Earth pressure on the buried box section due to live load shall be considered to be uniformly distributed above the box section over an area determined by distribution of the applied live load through the pavement (if any) and earth above the pipe as specified in Clause 6.9.6. The reacting earth pressure on the bottom of the box shall be assumed to be uniformly distributed. Lateral pressure due to live load for the load combinations specified in Items (a) and (b) of Clause 7.8.4.1 shall be neglected.

For the load combination specified in Item (c) of Clause 7.8.4.1, lateral pressure from an approaching wheel load shall be taken as shown in Figure 7.13.

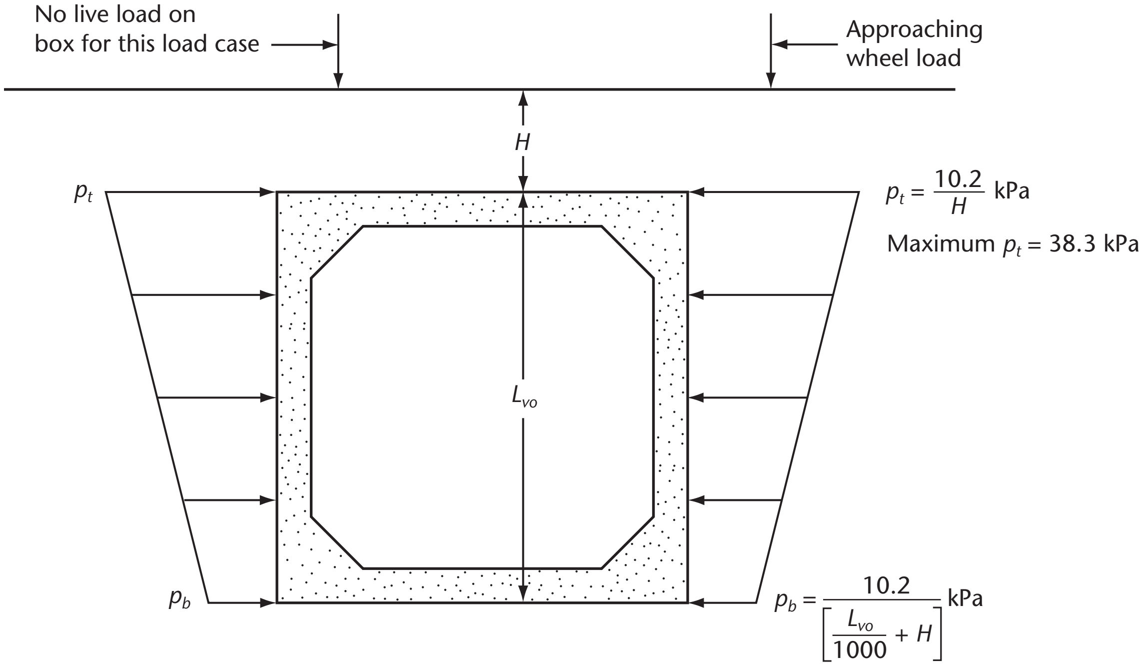


Figure 7.13
Lateral earth loads and pressure distribution on concrete box sections due to approaching wheel loads
(See Clause 7.8.5.3.3.)

7.8.6 Analysis

An analysis for moments, thrusts, and shears shall be performed for buried concrete structures subject to the load combinations of Clause 7.8.4. For pipe and box structures, the earth pressure distributions shall be as specified in Clause 7.8.5.

7.8.7 Ultimate limit state

7.8.7.1 Additional factors

The load factors for self-weight, earth, water, live, and earthquake loads shall be in accordance with Clause 7.5.2. In addition, the following requirements shall be satisfied:

(a) An installation factor of 1.1, in addition to the other load factors, shall be included in the multiplication to obtain the factored load effects due to earth load on the pipe and conduit shapes with curved bottoms.

(b) In the calculation of flexural tension, shear, and radial tension, where compressive thrust reduces the required strength for combined bending and thrust compared to bending alone, the load factors for compressive thrust caused by self-weight load, earth load, and live load shall be taken as 1.0 in lieu of the minimum load factors specified in Table 3.2.

7.8.7.2 Resistance factors
Resistance factors shall be as specified in Clause 7.5.3.

7.8.8 Strength design

7.8.8.1 Flexure

7.8.8.1.1 General
The proportioning of conduit walls subject to combined flexure and axial compression shall be in accordance with Clause 8.8. In addition, the following shall apply at locations where any flexural reinforcement is terminated:
(a) The minimum area of the remaining reinforcement shall have sufficient development length.
(b) At least 33% of the maximum inside reinforcement in slabs and walls shall be continuous in each component of a structure. At least 25% of the maximum outside reinforcement in slabs and walls shall be continuous in each component of a structure.
(c) The design yield strength, f_y, shall not exceed 500 MPa for bars or 550 MPa for welded wire fabric if the design yield strength is not greater than 85% of the breaking strength.
(d) The concrete strength, f_c', used in calculating design resistances shall not exceed 45 MPa.

7.8.8.1.2 Maximum flexural reinforcement without stirrups or ties
When stirrups or ties are not used, A_{si} shall not exceed the following limits, which are based on considerations of radial tensile strength:

$$A_{si} \leq 0.111 b r_s \sqrt{f_c'} \left[\frac{\phi_c}{\phi_s}\right] \frac{F_{rt}}{f_y}$$

where

$$F_{rt} = \frac{(3600 - D_i)^2}{16.8 \times 10^6} + 0.80 \text{ for } 1800 \text{ mm} < D_i \leq 3600 \text{ mm}$$

$$= 0.8 \text{ for } D_i > 3600 \text{ mm}$$

7.8.8.2 Design for shear

7.8.8.2.1 Circular, elliptical, and arch pipe without stirrups or ties
When stirrups or ties are not used, the conduit wall shall be designed so that for each region requiring flexural tensile reinforcement at the inside or outside of the wall, the shear strength of the concrete, V_c, shall be greater than the factored shear force, V_u, at any section in each region.

M_{nu} shall be calculated as follows:

$$M_{nu} = M_u - N_u \frac{(4h - d)}{8}$$

At sections where $M_{nu}/V_u \phi_c d$ is greater than or equal to 3.0, the following value of V_c shall apply:

$$V_c = V_b$$

where

$$V_b = 0.083 b \phi_c d \sqrt{f_c'} (1.1 + 63\rho) \left[\frac{F_d F_N}{F_c}\right]$$

where

$f_c' \leq 45$ MPa

$\rho = \dfrac{A_s}{bd} < 0.02$

$F_d = 0.8 + \dfrac{41}{d} \leq 1.3$

$F_N = 1 + \dfrac{N_u}{14bh} \geq 1.0$ for compressive thrust (N_u positive)

$= 1 + \dfrac{N_u}{3.5bh} \leq 1.0$ for tensile thrust (N_u negative)

$F_c = 1 + \dfrac{d}{2r}$ for tension on the inside of the pipe

$= 1 - \dfrac{d}{2r}$ for tension on the outside of the pipe

At sections where $M_{nu}/V_u\phi_c d$ is less than 3.0, the following value of V_c shall apply:

$$V_c = \frac{4V_b}{\dfrac{M_{nu}}{V_u d} + 1} \leq 0.25\phi_c bd\sqrt{f_c'}$$

where V_b is as specified in this Clause.

7.8.8.2.2 Box sections and segmental structures without stirrups or ties

7.8.8.2.2.1

For concrete box culverts, the shear strength of the slab need not be checked if the following conditions are satisfied:

(a) the centre-to-centre spacing of the vertical walls is less than or equal to 4.0 m;
(b) the slab thickness is greater than or equal to 175 mm; and
(c) the reinforcement ratio of bottom steel bars in the direction of the span is not less than 0.3%.

When stirrups or ties are not used in box sections and segmental structures, and the conditions specified in Items (a) to (c) are not satisfied, the shear strength shall be determined at critical sections, taking into consideration the fact that sections located less than a distance *d* from the face of a support can be designed for the same shear, V_u, as that calculated at a distance *d*, if the support reaction, in the direction of the applied shear, introduces compression into the end regions of the member and no concentrated load occurs between the face of the support and the location of the critical section at *d*.

Also, the tips of haunches with an inclination 45° or steeper shall be taken as the face of the support instead of the face of the sidewall.

7.8.8.2.2.2

The shear strength specified in Clause 7.8.8.2.2.1 shall be determined

(a) in accordance with Clauses 8.9.3 and 8.9.4, except that for sections within 2*d* of the face of the support, Clause 8.10 shall be used instead of Clause 8.9; or
(b) using an alternative procedure for single-cell box sections, for which the shear strength without stirrups or ties is taken in accordance with Clause 7.8.8.2.1, with $F_c = 1.0$, provided that the following conditions are satisfied:
    (i) the load distribution over the top and bottom slabs is uniform;

(ii) the combined area of inner and outer reinforcement at d from support satisfies

$$A_{si} + A_{so} \geq \frac{2V_u}{\phi_s f_y}$$

where V_u is taken at a distance d from the support;

(iii) the area of the reinforcement that resists tension produced by the bending moment at a distance d from the face of the support or haunch tip is calculated as follows:

$$A_s \geq \frac{\left[0.5V_u + \frac{M_u}{0.9d}\right]}{\phi_s f_y}$$

where M_u is taken at a distance d from the support or the tip of the haunch; and

(iv) inner and outer reinforcement extend into the support wall or haunch with sufficient anchorage to develop the minimum required reinforcement areas in Items (ii) and (iii) and the outer reinforcement extends beyond $2d$ from the support, with sufficient anchorage to develop the minimum required outer reinforcement area.

7.8.8.2.3 Stirrup reinforcement for shear and radial tension

If V_c is less than V_u at any section, stirrups shall be designed so that

$$A_{vs} = \frac{1.1 s_v}{f_v \phi_c d}\left[(V_u F_c - V_c) + \frac{(M_u - 0.45 N_u \phi_c d)}{r_s}\right]$$

where

V_c $\leq$ $0.166\phi_c bd\sqrt{f_c'}$

f_v = maximum strength that can be developed by the stirrups and is less than or equal to f_y

The following requirements shall also apply:

(a) For pipes, the maximum spacing between the stirrups in the circumferential direction shall be $s_v \leq 0.75\phi_c d$.

(b) For box and segmental structures, the maximum spacing between the stirrups in the circumferential direction shall be as specified in Clause 8.14.6.

(c) For curved members, the maximum spacing between the stirrups in the longitudinal direction shall be the same as the spacing of the circumferential reinforcing wires or bars.

(d) For straight members, the maximum spacing between the stirrups in the longitudinal direction shall be $1.5d$.

(e) Stirrups shall be provided in all locations where V_u is greater than V_c plus an additional minimum distance equal to the conduit wall thickness, h, beyond these locations.

(f) When stirrups are required for shear or radial tension at the invert or the crown regions of curved conduit walls, they shall extend on each side beyond the calculated arc length requiring stirrups for an additional arc length of at least $0.5\ell_\theta$, where

$$\ell_\theta = \frac{\pi\theta}{180}(D_i 6 + 2t_b) + h$$

If stirrups are also needed at the spring line regions, as can occur in very-high-loading conditions, they shall be spaced at s_v and shall extend around the entire circumference.

(g) The stirrups required at a point of critical shear in a region adjacent to a support shall be extended to the face of the support members. In box sections and other structures with 45° or steeper haunches, the stirrups used in the slab shall be extended to a point one-third the slab thickness, h, from the start of the haunch toward the support. Stirrup anchorage shall be in accordance with Clause 8.15.1.5 or as demonstrated by Approved tests.

7.8.9 Serviceability limit state

7.8.9.1 Control of cracking

The crack-control requirements of Clauses 8.12.2 and 8.12.3 shall be satisfied, except for structures with principal reinforcement consisting of 10M bars or smaller spaced at intervals of 100 mm or less, for which the crack-control factor, F_{cr}, calculated as follows, shall not exceed 0.85:

$$F_{cr} = \frac{B_1}{5250\phi_s}\left[f_s - \frac{0.083C_1 h^2 \sqrt{f_c'}}{\rho d^2}\right]$$

where

$$B_1 = \left[\frac{25t_b s_1}{2n}\right]^{1/3}$$

where

(a) $t_b \geq 25$ mm and $t_b \leq 0.33h$ or 75 mm, whichever is less;

(b) 50 mm $\leq s_1 \leq$ 102 mm; and

(c) $n = 1$ when tension reinforcement is a single layer and $n = 2$ when tension reinforcement is made of multiple layers.

The crack-control coefficient, C_1, for different types of reinforcement shall be as specified in Table 7.17.

Table 7.17
Crack-control coefficient, C_1
(See Clause 7.8.9.1.)

Type of reinforcement	C_1
Smooth wire or plain bars	1.0
Welded smooth wire fabric with 200 mm maximum spacing of longitudinal wires, deformed wire, or welded deformed wire fabric	1.5
Deformed bars or any reinforcement with stirrups anchored to it	1.9

7.8.9.2 Corrosion protection

Primary corrosion protection of reinforcement shall be provided by controlling crack widths in accordance with Clause 7.8.9.1 and by providing sufficient concrete cover in accordance with Clause 8.11.2.2.

7.8.10 Fatigue limit state

Reinforcement stress ranges in the top slabs of box sections and similar structures with depths of cover less than 0.6 m shall comply with the requirements of Clause 8.5.3.1, except that cross-wire welds in welded wire fabric reinforcement shall not be deemed to be tack welds.

7.8.11 Minimum reinforcement

7.8.11.1 Parallel to span

Each of the inside and outside layers of reinforcement parallel to the span shall provide a minimum area of reinforcement of $0.002bh$, but not less than that required for shrinkage and temperature in accordance with Clause 7.8.11.2.

7.8.11.2 Perpendicular to span

Reinforcement for shrinkage and temperature effects normal to the principal reinforcement shall be provided in conduits where principal reinforcement extends in one direction only. The minimum area and maximum spacing of shrinkage and temperature reinforcement shall be in accordance with Table 7.18.

Table 7.18
Shrinkage and temperature reinforcement
(See Clauses 7.8.11.2 and 7.8.12.2.)

Type of structure	Minimum depth of earth cover, m	Number of layers	Location	Minimum area of reinforcement, mm^2/m		Maximum spacing, mm
				Inside face	Outside face	
Precast concrete, maximum length 6 m	≥ 0.6	1	Above principal reinforcement on inside face	300	—	250
	< 0.6	2	Near each face	300	300	250
Other	≥ 0.6	2	Near each face	500	250	250
	< 0.6	2	Near each face	500	500	250

7.8.12 Distribution reinforcement

7.8.12.1 Design of reinforcement

The following requirements for design of reinforcement shall apply:

(a) The top slabs of box sections, and other structures with flat top slabs with a depth of cover less than 0.6 m, shall be provided with distribution reinforcement in accordance with Clause 8.18.7, to be placed near the inside of the bottom face of the slab.

(b) Where not overlaid by a cast-in-place reinforced concrete slab, top slabs of precast concrete box sections, and other precast concrete structures with flat top slabs with a depth of cover less than 0.6 m, shall have additional distribution reinforcement equal to at least one-half the amount of distribution reinforcement required by Item (a), placed near the outside of the top face of the slab.

Note: *The area of shrinkage and temperature reinforcement required by Clause 7.8.11.2 may also be used to satisfy the requirements for distribution reinforcement in this Clause.*

7.8.12.2 Minimum area of distribution reinforcement

The minimum area of distribution reinforcement perpendicular to the principal transverse reinforcement shall be as specified in Table 7.18.

7.8.13 Details of the reinforcement

7.8.13.1 General

Subject to Clause 7.8.13.2, the details of the reinforcement shall be in accordance with Clauses 8.14 and 8.15.

7.8.13.2 Precast concrete pipe and box sections

The following details of the reinforcement shall be in accordance with the requirements specified in Appendix A of ASCE 15:

(a) development of principal reinforcement for welded splices, lapped splices, and anchorage at cut-offs;

(b) anchorage of stirrups located in regions where the outside reinforcement is subjected to flexural tension; and
(c) joint reinforcement.

7.8.14 Joint shear for top slab of precast concrete box sections with depth of cover less than 0.6 m

The top slab joint between adjacent precast concrete units shall be capable of transferring a minimum unfactored shear load of 60 000 N/m unless the joints in the top slab are covered by a cast-in-place reinforced concrete slab at least 150 mm thick. If individual shear connectors are used, their centre-to-centre spacing shall not be greater than 800 mm, with a minimum of two shear connectors per joint.

7.8.15 Construction

7.8.15.1 Foundations

7.8.15.1.1 General

The foundation shall comprise moderately firm to hard in-situ soil, stabilized soil, or compacted fill materials.

7.8.15.1.2 Soft soil

When unsuitable or unstable material is encountered, the foundation shall be stabilized so as to meet the installation design requirements of Clause 7.8.3. Foundation soils for a minimum of one conduit inside width on each side of the conduit shall be at least as stiff as the foundation soil below the conduit.

7.8.15.1.3 Rock

Precast concrete pipe and other conduits with curved bottoms shall not be placed directly on a rock foundation. For pipes, the minimum bedding thickness over rock shall be the greater of 150 mm or $D_o/12$. Precast concrete box sections and other conduits with flat bottoms shall be placed on a flat granular bedding at least 75 mm thick.

7.8.15.1.4 Control of water

Groundwater levels shall be controlled to avoid disturbing fine sand or silty soil foundations and to comply with the installation requirements specified in Clause 7.8.3.

7.8.15.2 Subgrade for cast-in-place structures

7.8.15.2.1 Undisturbed foundation

Firm to hard in-situ foundation soils shall be undisturbed. Soils on top of the foundation shall be compacted to the same stiffness as the undisturbed in-situ soil to maintain uniform support along the length of the conduit.

Foundation soils that could be disturbed by the construction process shall be protected.

7.8.15.2.2 Control of line and grade

Line and grade shall be maintained to allow construction of structures at the specified location and with the specified minimum wall thickness. Low spots shall be filled with concrete or with soil compacted to the same stiffness as the undisturbed in-situ soil.

7.8.15.3 Bedding for precast concrete structures

7.8.15.3.1 Uniform support and control of grade

The bedding shall be constructed as required for the specific installation by Clause 7.8.3 in order to distribute the load-bearing reaction uniformly on the pipe barrel or structure base and to maintain the required conduit grade.

7.8.15.3.2 Compaction

The bedding layers shall be compacted as specified for the installation design in Clause 7.8.3. For pipes designed as Type C1, C2, or C3 in accordance with Clause 7.8.3.5, or as Type B1 in accordance with Clause 7.8.3.6, the bedding layer shall be placed as uniformly as possible but shall be loosely placed and uncompacted under the middle third of the conduit wall. For all structures, the outer bedding or any bedding that may be under the lower side areas shall be compacted to at least the same requirements as apply to the outer bedding or lower side areas, whichever are more stringent.

7.8.15.3.3 Maximum aggregate size

The maximum aggregate size for bedding shall not exceed 25 mm unless the bedding has a thickness of 150 mm or greater, in which case the maximum aggregate size shall not exceed 38 mm.

7.8.15.3.4 Bell holes

Bell holes shall be excavated in the bedding or foundation when pipe with expanded bells is installed so that the pipe is supported by the barrel and not by the bells.

7.8.15.4 Placement and joining of precast structures

7.8.15.4.1 Control of line and grade

Structures shall be installed to the line and grade shown on the Plans. Joining shall be in accordance with the manufacturer’s recommendations. Before the precast section is joined, it shall be brought to correct alignment and the top positioned.

7.8.15.4.2 Adjustments in alignment

If the precast section being installed is misaligned, the section shall be completely disconnected, the alignment corrected, and the section rejoined. Alignments shall not be adjusted by exerting force on the barrel of the section or by lifting and dropping the section.

7.8.15.5 Structural backfill

7.8.15.5.1 Type and compaction

Soils placed below and adjacent to a precast structure shall be of the type and compaction level specified in Clause 7.8.3.5 or 7.8.3.6, as applicable, for the particular location of the soils in the backfill zones. The soils shall be placed and compacted uniformly so as to distribute the load-bearing reaction uniformly to the bedding over the full length of the structure. Within 0.3 m of the conduit wall, the aggregate size shall be less than or equal to 38 mm.

7.8.15.5.2 Concrete pipe in standard installations

For precast concrete pipes designed as standard installations in accordance with Clause 7.8.3.5, the haunch and lower sidefill zones shall be constructed using the soil type and minimum compaction level corresponding to the particular standard installation type.

7.8.15.5.3 Box sections in standard installations
For box sections designed as standard installations in accordance with Clause 7.8.3.6, the sidefill zones shall be constructed using the soil type and minimum compaction level corresponding to the particular standard installation type.

7.8.15.5.4 Testing
When the design requires compliance with the soil type and compaction requirements of this Section, such compliance shall be verified by appropriate observations and tests performed by an Approved geotechnical Engineer.

7.8.15.6 Sidefill soils

7.8.15.6.1 Constructed soils
Soils in the sidefill zones identified in Figures 7.8 to 7.11 shall be of a type and have the minimum compaction specified in Clause 7.8.3.5 or 7.8.3.6, as applicable, or the minimum compaction of the overfill soils, whichever is greater. Constructed soils shall not contain debris, organic matter, frozen materials, or large stones of a diameter greater than one-half the thickness of the compacted layers being placed or 100 mm, whichever is smaller.

Soils shall be deposited uniformly on each side of the structure in order to prevent lateral displacement.

7.8.15.6.2 In-situ soils
In-situ soils that are located in the sidefill zones of trenches whose walls have a slope greater than 10° from the vertical and are less stiff than the constructed overfill soils shall be removed and replaced with compacted soils whose stiffness is at least that of the overfill soils.

7.8.15.7 Overfill soils

7.8.15.7.1 Type, compaction, and unit weight
Overfill soils shall be constructed as specified in this Section. The compaction shall not be greater than the compaction or equivalent stiffness of soils in the sidefill zone and foundation. The average unit weight shall not exceed the design unit weight of overfill soil.

7.8.15.7.2 Structures below pavements
Overfill in trenches and in other locations where pavements require control of differential settlement shall be of a type and compaction level that can control pavement differential settlement within acceptable limits for the particular type of pavement.

7.8.15.8 Trenches

7.8.15.8.1 General
The walls of trenches shall be maintained in a stable condition so as to permit safe construction operations and compliance with applicable safety standards.

7.8.15.8.2 Width control
When required by the installation design, trench width shall be controlled within the limits shown on the Plans. If no width limits are shown, the trench width shall be sufficient to facilitate compliance with this Section’s requirements for compaction of soils in the haunch zone.

7.8.15.8.3 Sheathing removal
Unless sheathing is to be left in place, it shall be pulled out in vertical increments in order to permit placement and compaction of fill material for the full width of the trench.

7.8.15.8.4 Trench shields and boxes
When trench shields or boxes are moved, the previously installed structure shall not be disturbed and any void left by the trench box shall be filled with soil, compacted as specified in this Section.

7.8.15.9 Protection from construction equipment overload

7.8.15.9.1 Limitation of construction loads
The load imposed on an installed structure by construction equipment shall be limited to a load that does not exceed the design strength of the buried structure. For box sections and similar structures, the effects of approaching wheel loads adjacent to the sides of the structure, as well as the effects of loads above the structure, shall be considered.

7.8.15.9.2 Extent of overfill for support of construction loads
In an embankment installation, the full overfill depth required to support construction equipment loads shall extend at least one structure width or 3.0 m, whichever is greater, beyond each side of the installed structure so as to protect the structure from excessive loading and to prevent possible lateral displacement of the structure. The overfill may be ramped beyond this width in order to facilitate passage of the construction equipment over an installed structure. If a large volume of construction traffic needs to cross an installed structure, the point of crossing shall be changed from time to time, in accordance with engineering judgment, to minimize the possibility of lateral displacement.

7.8.15.10 Site supervision and construction control
The Plans shall require that the Engineer designated by the Owner as responsible for inspection of the construction shall be experienced in the design and construction of soil installations for buried concrete structures. Construction shall be inspected for compliance with the compaction and testing requirements of Clauses 7.8.15.3 and 7.8.15.5 to 7.8.15.7. Records (including recorded observations) covering at least the following construction processes shall be provided:

(a) the condition of the foundation before installation of bedding (to include observations of in-situ soils below and adjacent to the structure);
(b) the type and compaction of bedding soil (including avoidance of bedding compaction near pipe inverts when such control is specified by the installation design);
(c) the type and compaction of embedment soils, especially below the pipe haunches and immediately adjacent to the conduit structures;
(d) for embankment installations, the type and compaction of embankment in the region adjacent to the height of the conduit;
(e) for trench installations, the width of the trench at the top and bottom of the conduit, the slope of the trench wall, and the type and stiffness of in-situ material in the trench and wall;
(f) the type and compaction of overfill soils above the conduit; and
(g) the type and compaction of the pavement sub-base, if any, and the type of pavement.

Section 8 — Concrete structures

Section 8
Concrete structures

8.1 Scope

This Section specifies requirements for the design of structural components that are made of precast or cast-in-place normal-density, low-density, or semi-low-density concrete and reinforced with prestressed or non-prestressed steel. The components covered by this Section can be prestressed with pretensioned steel, grouted post-tensioned steel, or both.

8.2 Definitions

The following definitions apply in this Section:

Adhesive anchor — an anchor inserted into a hole drilled in hardened concrete and held in place by epoxy resin or another adhesive.

Anchor — a bolt, stud, or reinforcing bar embedded in concrete.

Anchorage —
(a) in post-tensioning, a device used to anchor a tendon to a concrete member;
(b) in pretensioning, a device used to anchor a tendon until the concrete has reached a predetermined strength; and
(c) for reinforcing bars, a length of reinforcement, mechanical anchor, or hook, or a length of reinforcement combined with a mechanical anchor or a hook.

Anchorage blister — a protrusion in a web, flange, or flange-web junction for placement of tendon anchorage fittings.

Anchorage system — an anchor or assemblage of anchors.

At jacking — at the time of tensioning tendons.

Attachment — a structure external to concrete that transmits loads to an anchor.

At transfer — at the time immediately after transfer.

Bonded tendon — a tendon that is bonded to concrete directly or by grouting.

Cast-in-place anchor — an anchor that is in its final location at the time of placing of concrete.

Closure — a cast-in-place concrete segment used to complete a span in segmental construction.

Concrete cover — the least distance between the surface of reinforcing bars, strands, post-tensioning ducts, anchorages, or connections and the surface of concrete.

Creep — time-dependent deformation of concrete under sustained load.

Curvature friction — the friction resulting from the curvature of the specified profile of post-tensioning tendons.

Decompression — a condition at which the concrete compressive stress induced by prestress, at a specified point in a section, is reduced to zero by the tensile stress due to applied loads.

Deep beam — a member with a span-to-depth ratio of less than 2.0, where for continuous spans a effective span is taken as the distance between points of contraflexure due to dead load.

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

Deviator — a protrusion in a web, flange, or web-flange junction cast at appropriate locations in a span to control the geometry or to provide a means for changing the direction of external tendons.

Duct — an opening in concrete for internal post-tensioning tendons.

Edge distance — the minimum distance between the anchor centreline and the free edge of the concrete.

Effective depth — the distance from the extreme compression fibre to the centroid of the tensile force.

Effective prestress — the stress or force remaining in the tendons or the concrete after all losses have occurred.

Embedment depth — the distance from the bearing surface of the anchor in tension to the surface of the concrete.

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

Equivalent embedment length — the length of embedded straight reinforcement that can develop the same strength as that which can be developed by a standard hook or mechanical anchorage.

External tendon — a post-tensioning tendon placed outside a web or flange (usually inside a box girder cell).

Grouted anchor — an anchor grouted into a hole drilled in hardened concrete.

Internal tendon — a post-tensioning tendon embedded in a member.

Jacking force — the force applied to stress tendons. For pretensioning, the specified jacking force excludes the force to compensate for anchorage slip and temperature correction. For post-tensioning, the specified jacking force includes an allowance to compensate for anchorage slip.

Low-density concrete — concrete with an air-dry density not greater than 1850 kg/m^3 (determined in accordance with ASTM C 567).

Multi-beam decks — deck systems consisting of precast components placed side-by-side.

Normal-density concrete — concrete having a fresh density between 2150 and 2500 kg/m^3 (determined in accordance with CAN/CSA-A23.2).

Post-tensioning — a method of prestressing in which the tendons are stressed after the concrete has reached a predetermined strength.

Precast components — concrete components that are cast in a location other than their final position and manufactured and erected in accordance with CAN/CSA-A23.4.

Prestressed concrete — reinforced concrete with an average effective prestress of at least 1.50 MPa.

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

Reinforcement — steel in the form of reinforcing bars, wires, wire fabric, or tendons.

Relaxation — the time-dependent reduction of stress in tendons at constant strain.

Secondary prestressing effects — the effects caused by restraint of deformation resulting from the prestressing force.

Segmental girder — a girder made up of individual components post-tensioned together to act as a monolithic unit under loads.

Semi-low-density concrete — concrete with an air-dry density greater than 1850 kg/m^3 but less than 2150 kg/m^3 (determined in accordance with ASTM C 567).

Shear lug — a plate or bar that transmits shear forces to concrete.

Sheath — a tube-like component for forming a duct for internal post-tensioning and for containing tendons and grout for external post-tensioning.

Skew angle — the angle formed by subtracting the acute angle of the parallelogram from 90° in a slab panel in the form of a parallelogram.

Slab — a component with a width at least four times the effective depth.

Spacing — the distance between centrelines of adjacent reinforcing bars, wires, tendons, or anchors.

Specified strength of concrete — the 28-day compressive strength of concrete as specified on the Plans and determined in accordance with CAN/CSA-A23.2.

Specified strength of tendon — the tensile strength or breaking load of a tendon per unit area as specified on the Plans and determined in accordance with CSA G279.

Spiral — continuously wound bar or wire in the form of a cylindrical helix.

Stress range — the algebraic difference, at the fatigue limit state, between the maximum and minimum stresses for reinforcing bars or the increase in tension for tendons.

Tendon — a high-strength steel element used to impart prestress to concrete.

Tension stiffening — the stiffening effect on a member due to the contribution of the uncracked concrete between cracks.

Transfer — the act of transferring force in tendons to concrete.

Transfer length — the length over which a prestressing force is transferred to concrete by bond in a pretensioned component.

Transverse reinforcement — reinforcement used to resist shear, torsion, or lateral forces in a structural component (typically deformed bars bent into U, L, or rectangular shapes and located not parallel to longitudinal reinforcement).
Note: *The term "stirrups" is usually applied to transverse reinforcement in flexural components and the term "ties" to transverse reinforcement in compression components.*

Wall-type compression component — a component with a rectangular cross-section having a width-to-depth ratio of 4 or greater.

Wobble friction — the friction caused by the unintended deviation of a post-tensioning sheath or duct from its specified profile.

Yield strength — the specified minimum yield strength of reinforcement.

8.3 Symbols

The following symbols apply in this Section:

A_b = area of an individual reinforcing bar, mm^2; bearing area of a post-tensioned anchor, mm^2

A_{br} = bearing area of an anchor or shear lug, mm^2

A_c = area of core of a spirally reinforced compression member measured out-to-out of spirals, mm^2

A_{cp} = area enclosed by the outside perimeter of a concrete cross-section, including the area of holes, if any, mm^2

A_{cs} = effective cross-sectional area of a compressive strut, mm^2

A_{ct} = area of concrete on the flexural tension side of a member, mm^2

A_{cv} = area of concrete resisting shear transfer, mm^2

A_g = gross cross-sectional area, mm^2

A_o = area enclosed by shear flow path, including the area of holes, if any, mm^2

A_{oh} = area enclosed by the centreline of exterior closed transverse torsion reinforcement, including the area of voids, if any, mm^2

A_{ps} = area of tendons on the flexural tension side of a member, mm^2

A_s = area of reinforcing bars on the flexural tension side of a member, mm^2

A_s' = area of reinforcing bars on the flexural compression side of a member, mm^2

A_{ss} = area of reinforcement in the strut, mm^2

A_{st} = area of reinforcement in the tie, mm^2; total area of longitudinal reinforcing bars, mm^2

A_t = area of closed transverse torsion reinforcement, mm^2

A_{tr} = area of reinforcement within ℓ_d that crosses the potential bond-splitting crack, mm^2

A_v = area of transverse shear reinforcement perpendicular to the axis of a member within a distance s, mm^2

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

A_w = area of an individual wire to be developed or spliced, mm^2

A_1 = loaded area, mm^2

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area and does not overlap similar areas from adjacent loaded areas, mm^2

ANC = loss of prestress due to slip of post-tensioning tendon at anchorage, MPa

a = depth of an equivalent rectangular stress block, mm (see Clause 8.8.3); lateral dimension of the anchorage device measured parallel to the larger dimension of the cross-section, mm; maximum size of aggregate, mm; difference between mean concrete strength and specified strength f_c' at 28 days

a_g = specified nominal size of coarse aggregate, mm

B_r = factored bearing resistance of a concrete component, N

b = width of the compression face of member, mm; lateral dimension of the anchorage device measured parallel to the smaller dimension of the cross-section, mm

b_o = perimeter of the critical section for slabs and footings, mm

b_v = effective web width within depth d_v, mm (see Clause 8.9.1.6)

b_w = web width, mm

C_m = factor relating the actual moment diagram to an equivalent uniform moment diagram

CR = loss of prestress due to creep of concrete, MPa

c = distance from extreme compression fibre to neutral axis, mm; cohesion for interface shear transfer, MPa; distance from centroidal axis of a pile to the extreme fibre in tension or compression, mm

c_d = cover to a post-tensioning duct, mm

d = effective depth (being the distance from the extreme compression fibre to the centroid of the tensile force), mm

d_a = nominal diameter of an anchor, mm

d_b = nominal diameter of a bar, wire, or prestressing strand, mm

d_{ba} = nominal diameter of a reinforcing bar anchoring a strut, mm

d_{bs} = distance from the loaded surface to the centroid of the bursting force, mm

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

d_d = nominal diameter of a post-tensioning duct, mm

d_{eff} = effective length of a shear plane at post-tensioning ducts, mm

d_p = distance from the extreme compression fibre to the centroid of the tendons, mm

d_v = effective shear depth, mm

E_c = modulus of elasticity of concrete, MPa

$E_{c,28}$ = modulus of elasticity of concrete at 28 days, MPa

$E_c(t_0)$ = modulus of elasticity of concrete at time of loading, MPa

E_{ci} = modulus of elasticity of concrete at transfer, MPa

E_p = modulus of elasticity of tendons, MPa

E_s = modulus of elasticity of reinforcing bars, MPa

EI = flexural stiffness, N•mm^2

ES = loss of prestress due to elastic shortening of concrete, MPa

e = base of Napierian logarithms; eccentricity, mm

F_f = factored tensile force on an anchor, N

F' = reduced force effect due to creep

F_ℓ = lateral force per unit length due to the multi-strand effect in a curved tendon, N/mm

$F_{\ell c}$ = required tensile force in longitudinal reinforcement on the flexural compression side of a member, N

$F_{\ell t}$ = required tensile force in longitudinal reinforcement on the flexural tension side of a member, N

F_{pu} = total specified strength of tendons, N

F_r = factored tensile resistance of an anchor, N; distributed thrust per unit length in the plane of the curved tendon, N/mm

F_s = force in tendons, N

FR = loss of prestress due to friction at a point x metres from the jacking end, MPa

f'_c = specified compressive strength of concrete, MPa

$\sqrt{f'_c}$ = square root of the specified compressive strength of concrete, which after being multiplied by an empirical constant with suitable units is expressed in megapascals

f_{ca} = compressive stress in concrete immediately behind an anchorage device, MPa

f_{cds} = concrete stress at the centre of gravity of tendons due to all dead loads except the dead load present at transfer at the same section or sections for which f_{cir} is calculated (the stress being positive when tensile), MPa

f_{ce} = axial concrete stress that can be taken as f_{pc} for prestressed members and N_f/A_g for non-prestressed members (the stress being positive when compressive), MPa

f'_{ci} = compressive strength of concrete at transfer, MPa

$\sqrt{f'_{ci}}$ = square root of the compressive strength of concrete at transfer, which after being multiplied by an empirical constant with suitable units is expressed in megapascals

f_{cir} = concrete stress at the centre of gravity of tendons due to the prestressing effect at transfer and the self-weight of the member at sections of maximum moment, MPa

f_{cr} = cracking strength of concrete, MPa

f_{cri} = cracking strength of concrete at transfer, MPa

f_{cu} = limiting compressive stress in a strut, MPa; crushing strength of concrete, MPa

f_l = calculated stress in concrete due to specified live load, MPa

f_{pc} = compressive stress in concrete after all prestress losses have occurred, either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroidal axis lies in the flange, MPa; for two-way action, the average of the values of compressive stress in concrete for the two directions, after all prestress losses, at the centroid of the section, MPa

f_{po} = stress in prestressed reinforcement when stress in the surrounding concrete is zero, MPa

f_{ps} = stress in tendons at the ultimate limit state, MPa

f_{pu} = specified tensile strength of prestressing steel, MPa

f_{py} = yield strength of prestressing steel (may be taken as $0.90f_{pu}$ for low-relaxation strands, $0.85f_{pu}$ for smooth high-strength bars, and $0.80f_{pu}$ for deformed high-strength bars), MPa

f_s = tensile stress in reinforcing bars, MPa

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

f_{si} = stress in pretensioning strand just prior to transfer, MPa

f_{sj} = stress in prestressing steel at jacking, MPa

f_{st} = stress in prestressing steel at transfer, MPa

f_{su} = specified tensile strength of anchor steel, MPa

f_{tl} = tensile stress in concrete at the serviceability limit state, MPa

f_w = stress in reinforcement under conditions causing cracking, calculated on a cracked section

f_y = specified yield strength of reinforcing bars, MPa

h = overall thickness of a component, mm; the lateral dimension of the cross-section in the direction considered, mm; overall thickness of a deck slab, including the precast panel if present, mm

h_a = height of a strut at the outside edge of bearing, as shown in Figure 8.4(b), mm

h_o = a notional thickness that is a function of λ and r_v, mm

h_s = height of strut, as shown in Figure 8.4(c), mm

I_{cr} = moment of inertia of a cracked section, transformed to concrete, mm^4

I_e = effective moment of inertia, mm^4

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

I_s = moment of inertia of the reinforcement about the centroidal axis of component cross-section, mm^4

K = wobble friction coefficient per metre length of a prestressing tendon

K_{cr} = factor used to calculate prestress loss due to creep of concrete

K_{tr} = transverse reinforcement index (see Clause 8.15.2.2)

k = effective length factor for compression members

k_b = parameter used in calculating crack width (to account for the type of force causing the cracking) (see Clause 8.12.3.2)

k_p = factor dependent on the type of prestressing steel specified in Clause 8.8.4.2

k_1 = bar location factor

k_2 = coating factor

k_3 = bar size factor

ℓ = length, mm

ℓ_a = length of a reinforcing bar anchoring a strut, as shown in Figure 8.4(a), mm

ℓ_b = length of bearing, mm

ℓ_d = development length, mm

ℓ_{dh} = development length of a standard hook in tension measured from the critical section to the outside end of the hook, mm

ℓ_e = effective length, mm

ℓ_{hb} = basic development length of a standard hook in tension measured from the critical section to the outside end of the hook, mm

ℓ_u = unsupported length of a compression member, mm

M_a = maximum moment in a member at the stage for which the deformation is being calculated, N•mm; allowable flexural moment on a pile without axial load at the serviceability limit state, N•mm

M_c = magnified moment used for proportioning slender compression members, N•mm

M_{cr} = cracking moment, N•mm

M_f = factored moment at a section, N•mm

M_r = factored flexural resistance of a section in bending, N•mm

M_{rx} = factored flexural resistance of a section about the x-axis, N•mm

M_{ry} = factored flexural resistance of a section about the y-axis, N•mm

M_s = flexural moment at a section under consideration at the serviceability limit state load, N•mm

M_x = component about the x-axis of the moment due to factored loads, N•mm

M_y = component about the y-axis of the moment due to factored loads, N•mm

M_1 = value of the smaller end moment at the ultimate limit state due to factored loads acting on a compression member (to be taken as positive if the member is bent in single curvature and negative if it is bent in double curvature), N•mm

M_2 = value of the larger end moment at the ultimate limit state due to factored loads acting on a compression member (always taken as positive), N•mm

N = total number of post-tensioning tendons; unfactored permanent load normal to the interface area (taken as positive for compression and negative for tension), N

N_f = factored axial load normal to the cross-section occurring simultaneously with V_f, including the effects of tension due to creep and shrinkage, N

n = modular ratio (= E_s / E_c or E_p / E_c); number of anchorages in a row; projection of a base plate beyond the wedge hole or wedge plate, as applicable, mm; number of bars or wires being developed along the potential plane of bond splitting

P_a = allowable axial load on a prestressed concrete pile without flexure at the serviceability limit state, N

P_c = buckling load, N

P_f = factored axial load at a section at the ultimate limit state, N

P_o = factored axial resistance of a section in pure compression, N

P_r = factored axial resistance of a section in compression with minimum eccentricity, N

P_{rx} = factored axial resistance in compression corresponding to M_{rx}, N

P_{rxy} = factored axial resistance in compression with biaxial loading, N

P_{ry} = factored axial resistance in compression corresponding to M_{ry}, N

P_s = axial load at a pile section at the serviceability limit state, N

p_c = outside perimeter of a concrete section, mm

p_h = perimeter of closed transverse torsion reinforcement measured along its centreline, mm

R = reduction factor for laterally unsupported piles (see Clause 8.23.7.2.2); radius of curvature of a tendon, mm

REL_1 = loss of prestress due to relaxation of prestressing steel before transfer, MPa

REL_2 = loss of prestress due to relaxation of prestressing steel after transfer, MPa

RH = annual mean relative humidity, %

r = radius of gyration of a gross cross-section, mm

r_v = volume per unit length of a concrete section divided by the corresponding surface area in contact with freely moving air, mm

S = time-dependent factor for calculating deformations caused by sustained loads; effective span length of slab, m; spacing of the supporting beams for slabs, m

S_e = unsupported length of the edge-stiffening beams in deck slabs, m

SH = loss of prestress due to shrinkage of concrete, MPa

s = spacing of reinforcing bars, mm; spacing of stirrups measured parallel to the longitudinal axis of a component, mm; centre-to-centre spacing of multiple anchorages, mm; vertical spacing of ties, mm; maximum centre-to-centre spacing of transverse reinforcement within ℓ_d, mm

s_d = clear spacing between ducts in the plane perpendicular to the tendon deviation, mm

s_{rm} = average spacing of cracks

s_w = spacing of wires to be developed or spliced, mm

s_z = crack spacing parameter dependent on crack control characteristics of longitudinal reinforcement, mm

s_{ze} = equivalent value of s_z that accounts for influence of aggregate size, mm

T_{bs} = bursting force behind a post-tensioning anchor, N

T_{cr} = torsional cracking resistance, N•mm

T_f = factored torsional moment at a section, N•mm

T_r = factored torsional resistance provided by shear flow, N•mm

t = age of concrete after casting, days; time, days; thickness of a section, mm; average thickness of a bearing plate, mm

t_a = thickness of an anchor head, mm

t_d = maximum projection of an anchor head, mm

t_0 = age of concrete at the time of loading or from when the influence of shrinkage is calculated, days

V_c = factored shear resistance provided by tensile stresses in concrete, N

V_{cd} = resistance of concrete in the plane of the tendon curvature, N

V_f = factored shear force at a section, N

V_p = component in the direction of the applied shear of all of the effective prestressing forces crossing the critical section factored by ϕ_p (taken as positive if resisting the applied shear), N

V_r = factored shear resistance, N

V_s = factored shear resistance provided by shear reinforcement, N

v = nominal shear stress, MPa; shear resistance of shear friction plane, MPa

w = crack width, mm

x = distance from the jacking end in post-tensioning, m; bonded length of pretensioned strand up to the inside edge of the bearing area, mm; length of reinforcing bar extending beyond the inner edge of the node region, mm

α = vector sum of angular changes in elevation and plan of a prestressing tendon profile from the jacking end to any point x, radians; angle of inclination of transverse reinforcement to the longitudinal axis of a member, degrees; angle of inclination of a tendon force with respect to the centreline of a member (positive if the anchor force points toward the centroid of the section and negative if the anchor force points away from the centroid of the section), degrees

α_1 = ratio of average stress in a rectangular compression block to the specified concrete strength

β = factor used to account for the shear resistance of cracked concrete (see Clauses 8.9.3.4 and 8.9.3.6 to 8.9.3.8)

β_{RH} = coefficient describing the effect of relative humidity on shrinkage in concrete

β_c = parameter used in calculating crack width

β_d = ratio of the maximum factored axial dead load to the total factored load used in Clause 8.8.5.3(f)

β_f = coefficient used in calculation of creep coefficient

β_s = coefficient describing the development with time of shrinkage in concrete

β_t = coefficient used in calculation of creep coefficient

β_1 = factor in Clause 8.8.3(f)

γ_c = mass density of concrete, kg/m^3

Δ_{fs} = total loss of prestress, MPa

Δ_{fs1} = loss of prestress at transfer, MPa

Δ_{fs2} = loss of prestress after transfer, MPa

δ = moment magnification factor for compression members

ε_c = concrete creep strain

ε_{cs} = time-varying strain in concrete due to shrinkage

ε_{cs0} = notional shrinkage coefficient

$\varepsilon_{c\sigma}$ = total time-varying strain in concrete due to constant stress

ε_e = concrete elastic strain calculated using the modulus of elasticity based on the concrete strength at 28 days

ε_s = tensile strain in a tie; tensile strain in reinforcing bars; concrete shrinkage strain

ε_{sm} = average strain in reinforcement

ε_x = longitudinal strain (see Clause 8.9.3.8)

ε_1 = principal tensile strain, taken as a positive quantity, in cracked concrete due to factored loads

θ = angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of a member, degrees; angle of skew of a bridge, degrees

θ_s = smallest angle between a compressive strut and the adjoining tensile tie, degrees

κ = correction factor for closely spaced anchorages (see Clause 8.16.2.2.6)

λ_1 = parameter dependent on the density of concrete and used to determine the friction coefficient, μ

μ = friction coefficient

ρ = the ratio A_s/bd

ρ' = the ratio A_s'/bd

ρ_c = ratio of reinforcement in the effective tension area of concrete

ρ_s = ratio of the volume of spiral reinforcement to the total volume of the core, out-to-out of spirals, of spirally reinforced compression members

ρ_v = the ratio A_{vf}/A_{cv}; ratio of the area of vertical shear reinforcement to the gross concrete area of a horizontal section

σ = compressive stress across a shear-friction plane, MPa

σ_c = stress in concrete
ϕ = creep coefficient
ϕ_{RH} = coefficient used in calculation of creep coefficient
ϕ_c = resistance factor for concrete (see Clause 8.4.6)
ϕ_p = resistance factor for tendons (see Clause 8.4.6)
ϕ_s = resistance factor for reinforcing bars (see Clause 8.4.6)
ψ = ratio of creep strain ε_c to elastic strain ε_e

8.4 Materials

8.4.1 Concrete

8.4.1.1 Compliance with CAN/CSA-A23.1/CAN/CSA-A23.2
Materials, methods of material testing, and construction practices shall, unless otherwise specified in this Section, comply with CAN/CSA-A23.1/CAN/CSA-A23.2.

8.4.1.2 Concrete strength
Unless otherwise Approved, the specified strength of concrete, f_c', shall be a minimum of 30 MPa for non-prestressed members and a minimum of 35 MPa for prestressed members. However, concrete with strengths greater than 85 MPa shall be used only if Approved.

The concrete strength shall be shown on the Plans.

8.4.1.3 Thermal coefficient
In the absence of more accurate data, the thermal coefficient of linear expansion of concrete shall be taken as 10×10^{-6}/°C.

8.4.1.4 Poisson's ratio
Unless determined by Approved physical tests, Poisson's ratio for elastic strains shall be taken as 0.2.

8.4.1.5 Shrinkage

8.4.1.5.1 General
The design values of shrinkage strains in normal-density concrete shall be determined as follows:
(a) in accordance with Clause 8.4.1.5.2; or
(b) based on data obtained from physical tests on the same mix of concrete that is to be used in construction.

The choice of method shall take into consideration the sensitivity of structural behaviour to shrinkage strain as well as the possible consequences of calculated shrinkage strains being significantly different from actual strains.

The design values of shrinkage strains in low-density and semi-low-density concrete shall be determined on the basis of data obtained from physical tests on the same mix of concrete that is to be used in construction.

8.4.1.5.2 Calculation of shrinkage strain
Except as permitted in Clause 8.4.1.5.1(b), the strain, ε_{cs}, due to shrinkage that develops in an interval of time, $t - t_0$, shall be calculated as follows:

$$\varepsilon_{cs}(t - t_0) = \varepsilon_{cs0}\beta_s(t - t_0)$$

where

ε_{cs0} = notional shrinkage coefficient

= $\beta_{RH}\left[160+50\left[9-\frac{f_c'+a}{10}\right]\right]\times 10^{-6}$

where

$$\beta_{RH} = -1.55\left[1-\left[\frac{RH}{100}\right]^3\right]$$

a = difference between mean concrete strength and specified strength, f_c', at 28 days (in the absence of data from the concrete that is to be used, a may be taken as 10 MPa)

where

RH = annual mean relative humidity, %, as shown in Figure A3.1.3

$\beta_s(t-t_0)$, which describes the development of shrinkage with time, shall be calculated as follows:

$$\beta_s(t-t_0) = \sqrt{\frac{t-t_0}{350\left[\frac{2r_v}{100}\right]^2+(t-t_0)}}$$

8.4.1.6 Creep

8.4.1.6.1 General

The design values of creep strains in normal-density concrete shall be determined as follows:

(a) in accordance with Clause 8.4.1.6.2; or

(b) based on data obtained from physical tests on the same mix of concrete that is to be used in construction.

The choice of method shall take into consideration the sensitivity of structural behaviour to creep strain as well as the possible consequences of calculated creep strains being significantly different from actual strains.

The design values of creep strains in low-density and semi-low-density concrete shall be determined on the basis of data obtained from physical tests on the same mix of concrete that is to be used in construction.

8.4.1.6.2 Calculation of time-varying strain due to stress

Except as permitted in Clause 8.4.1.6.1(b), for structural components with serviceability limit state compressive stresses less than $0.4f_c'$, the total time-varying strain, $\varepsilon_{c\sigma}(t,t_0)$, due to a constant stress, $\sigma_c(t_0)$, applied at time t_0 shall be calculated as follows:

$$\varepsilon_{c\sigma}(t,t_0) = \sigma_c(t_0)\left[\frac{1}{E_c(t_0)}+\frac{\phi(t,t_0)}{E_{c,28}}\right]$$

where

$E_c(t_0)$ = modulus of elasticity of concrete at time of loading

$\phi(t,t_0)$ = creep coefficient as specified in Clause 8.4.1.6.3

$E_{c,28}$ = modulus of elasticity of concrete at 28 days

The principle of superposition may be used to calculate strains due to a time-varying stress.

8.4.1.6.3 Creep coefficient

The creep coefficient, $\phi(t,t_0)$, shall be calculated as follows:

$$\phi(t,t_0) = \phi_{RH}\beta_f\beta_t\beta_c(t-t_0)$$

where

$$\phi_{RH} = 1+\frac{1-(RH)/(100\%)}{0.46\left[(2r_v)/100\right]^{1/3}}$$

where

RH = annual mean relative humidity, %, as shown in Figure A3.1.3

$$\beta_f = \frac{5.3}{\left[(f_c'+a)/10\right]^{0.5}}$$

where

a = difference between mean concrete strength and specified strength, f_c', at 28 days (in the absence of data from the concrete that is to be used, a may be taken as 10 MPa)

$$\beta_t = \frac{1}{0.1+(t_0)^{0.2}}$$

$$\beta_c(t-t_0) = \left[\frac{t-t_0}{\beta_H+t-t_0}\right]^{0.3}$$

where

$$\beta_H = 150\left[1+\left[1.2\frac{RH}{100\%}\right]^{18}\right]\frac{2r_v}{100\text{ mm}}+250$$

but shall not be taken larger than 1500

8.4.1.7 Modulus of elasticity

In the absence of more accurate data, the modulus of elasticity of concrete, E_c, shall be taken as

$$\left(3000\sqrt{f_c'}+6900\right)(\gamma_c/2300)^{1.5}$$

8.4.1.8 Cracking strength

8.4.1.8.1

The cracking strength, f_{cr}, shall be taken as

(a) $0.4\sqrt{f_c'}$ for normal-density concrete;

(b) $0.34\sqrt{f_c'}$ for semi-low-density concrete; and

(c) $0.30\sqrt{f_c'}$ for low-density concrete.

8.4.1.8.2

The cracking strength at transfer, f_{cri}, shall be taken as

(a) $0.4\sqrt{f_{ci}'}$ for normal-density concrete;

(b) $0.34\sqrt{f_{ci}'}$ for semi-low-density concrete; and

(c) $0.30\sqrt{f_{ci}'}$ for low-density concrete.

8.4.2 Reinforcing bars and deformed wire

8.4.2.1 Reinforcing bars

8.4.2.1.1 Specification

All reinforcing bars shall meet the requirements of CAN/CSA-G30.18. Grade R bars shall meet the following additional requirements:

(a) the minimum elongation at rupture in a 200 mm gauge length shall be 12% for 25M bars and smaller and 10% for 30M bars and larger; and

(b) the pin diameter for the 180° bend tests shall be
   (i) $4d_b$ for 25M bars and smaller;
   (ii) $6d_b$ for 30M and 35M bars; and
   (iii) $8d_b$ for 45M and 55M bars.

8.4.2.1.2 Welding

Where welding of the reinforcing bars is permitted, the reinforcing bars shall be Grade W.

8.4.2.1.3 Yield strength

The specified yield strength, f_y, of reinforcing bars shall be between 300 and 500 MPa and shall be shown on the Plans.

8.4.2.1.4 Stress-strain relationship

Reinforcing bars may be assumed to exhibit a bilinear stress-strain relationship with a slope, E_s, equal to 200 000 MPa prior to the yield point and a slope of zero beyond the yield point.

8.4.2.1.5 Reinforcing bar diameters

The bar designation number may be taken as the nominal diameter of a reinforcing bar in millimetres.

8.4.2.2 Steel wires and welded wire fabric

Steel wires shall comply with the applicable requirements of CSA G30.3 and CSA G30.14. Deformed wire that complies with CSA G30.14 shall not be smaller than MD25.

Welded wire fabric shall comply with the applicable requirements of CSA G30.5 and CSA G30.15.

The minimum elongation of welded wire fabric, as measured over a gauge length of at least 100 mm and including at least one cross-wire, shall be 4%.

8.4.3 Tendons

8.4.3.1 General

Tendons shall take the form of high-tensile-strength, low-relaxation strand or high-strength bars and shall meet the requirements of CSA G279. For pretensioned construction, tendons shall be Size Designation 9, 13, or 15 strands. Coated strands shall not be used unless Approved.

8.4.3.2 Stress-strain relationship

The stress-strain relationship used shall be representative of the tendons to be used in construction.

8.4.3.3 Modulus of elasticity

The modulus of elasticity of tendons, E_p , shall be based on representative stress-strain curves, when available. In the absence of such data, the following values shall be used:

(a) seven-wire high-strength strand:
   (i) Size 9, 13, or 15: 200 000 MPa; and
   (ii) Size 16: 195 000 MPa; and

(b) high-strength bar: 205 000 MPa.

8.4.4 Anchorages, mechanical connections, and ducts

8.4.4.1 Anchorages for post-tensioning tendons

When tested in an unbonded condition, anchorages for post-tensioning tendons shall develop at least 95% of the specified tensile strength of the tendons without exceeding the anticipated set. After tensioning and seating, anchorages shall sustain applied loads without slippage, distortion, or other changes that result in loss of prestress. The dimensions and details of the anchorages, including any reinforcement immediately behind the anchorages, shall be based on the specified strength of the tendon and the specified strength of the concrete at transfer.

Anchorages for external unbonded post-tensioning tendons shall also meet Approved dynamic tests.

8.4.4.2 Anchorages for reinforcing bars

Mechanical anchorage devices shall be capable of developing the yield strength of the reinforcing bars without damage to the concrete.

8.4.4.3 Mechanical connections for post-tensioning tendons

When tested in an unbonded condition, couplers for post-tensioning tendons shall develop 95% of the specified tensile strength of the tendons without exceeding the anticipated set. Couplers for external unbonded post-tensioning tendons shall also meet Approved dynamic tests.

Couplers and their components shall be enclosed in housings. The housings shall be long enough to permit the necessary movements and shall be provided with fittings to allow complete grouting. Couplers shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplers shall not be used at points of sharp tendon curvature or in the vicinity of points of maximum moments.

8.4.4.4 Mechanical connections for reinforcing bars

Mechanical connections for reinforcing bars shall develop, in tension or compression (as required), the greater of 120% of the specified yield strength of bars or 110% of the mean yield strength of the actual bars used to test the mechanical connection.

The total slip of the reinforcing bars within the splice sleeve of the connector after loading in tension to $0.5f_y$ and relaxing to $0.05f_y$ shall not exceed the following measured displacements between gauge points straddling the splice sleeve:

(a) for bars sizes up to and including 45M: 0.25 mm; and
(b) for 55M bars: 0.75 mm.

8.4.4.5 Ducts

8.4.4.5.1 General

Sheaths for internal post-tensioning ducts shall be made of bright steel, galvanized steel, or plastic. The sheaths shall be corrugated and shall be non-reactive with concrete, tendons, and grout. The shape of corrugations shall be such that the sheaths can be completely filled with grout. Sheaths for external post-tensioning shall be made of plastic.

8.4.4.5.2 Size

For single-strand or bar tendons, the inside diameter of the sheaths for post-tensioning ducts shall be at least 6 mm larger than the nominal diameter of the strand or bar. For multiple-strand tendons, the inside cross-sectional area of the sheath shall be at least twice the cross-sectional area of the prestressing tendon.

The inside diameter of a circular sheath or an equivalent diameter of a non-circular sheath shall not exceed 40% of the least gross concrete thickness at the duct.

8.4.4.5.3 Steel sheaths

Sheaths shall be watertight under an internal pressure of 350 kPa. Rigid steel sheaths shall have a wall thickness of at least 0.6 mm and shall permit bending of the sheath to a minimum inside radius of curvature of 9 m without distress. Semi-rigid steel sheaths shall have a wall thickness of at least 0.25 mm and shall permit the bending of the sheath to a minimum inside radius of curvature of 3.5 m without distress.

8.4.4.5.4 Plastic sheaths

Unless otherwise Approved, plastic sheaths, including their splices, shall be made of high-density polyethylene conforming to ASTM D 3350 Cell Classification 324420C, shall be vapour tight, and shall remain vapour tight after tendon installation and stressing. The polyethylene sheath shall be manufactured in accordance with ASTM D 2239.

Plastic sheaths shall not be used when the radius of curvature of the tendon is less than 10 m. The sheaths shall be capable of bending to the specified minimum radius of curvature without local buckling or damage. The sheath wall thickness shall be such that for the specified minimum radius of curvature the remaining wall thickness, after a tendon movement of 750 mm under a tendon stress of 80% of the specified strength, will not be less than 1 mm. For curved sheaths, the radial force exerted by a single strand on the sheath wall shall not exceed 40 kN/m.

The stiffness of plastic sheaths shall be such that

(a) for sheaths with an inside diameter of 50 mm or less, a 3 m length supported at the ends will not deflect, under its own weight, more than 75 mm at room temperature (i.e., not less than 20 °C);
(b) for sheaths with an inside diameter of more than 50 mm, a 6 m length supported at the ends will not deflect, under its own weight, more than 75 mm at room temperature; and
(c) the sheath shall not dent more than 3 mm under a point load of 445 N applied through a 10M reinforcing bar between the corrugation ribs at room temperature.

Sheaths and their splices for external post-tensioning shall be smooth, seamless, and capable of withstanding a grouting pressure of at least 1000 kPa.

8.4.4.5.5 Vents and drains

Ducts shall be provided with vents and drains at appropriate locations.

8.4.4.5.6 Ducts at deviators

Within deviators, the sheaths for post-tensioning tendons shall consist of

(a) galvanized steel pipe in accordance with ASTM A53/A53M, Type E, Grade B, with a wall thickness not less than 3 mm, and bent to conform to the tendon alignment; or
(b) an Approved sheath detail.

8.4.4.6 Anchor bolts and studs

Anchor bolts and studs shall comply with Section 10.

8.4.5 Grout

8.4.5.1 Post-tensioning

Unless otherwise Approved, grout for post-tensioning ducts shall comply with CAN/CSA-A23.1 and have a compressive strength of at least 35 MPa at 28 days.

8.4.5.2 Other applications

Grout for other applications shall be Approved.

8.4.6 Material resistance factors

The material resistance factors specified in Table 8.1 shall be used to calculate the factored resistance.

Table 8.1
Material resistance factors
(See Clause 8.4.6.)

Material	Material resistance factor
Concrete	$\phi_c = 0.75$
Reinforcement	
Reinforcing bars, wire, and wire fabric	$\phi_s = 0.90$
Prestressing strands	$\phi_p = 0.95$
High-strength bars	$\phi_p = 0.90$
Anchor bolts and studs	In accordance with Section 10

8.5 Limit states

8.5.1 General

Bridge components and retaining walls shall be proportioned to satisfy the requirements at the serviceability limit states, fatigue limit state, and the ultimate limit states.

8.5.2 Serviceability limit states

8.5.2.1 General

The cracking, deformation, stress, and vibration serviceability limit states shall be considered.

8.5.2.2 Cracking

The requirements of Clause 8.12 shall be met, except for tensile surfaces of components that are permanently covered with 600 mm or more of earth.

8.5.2.3 Deformation

The requirements of Clause 8.13 shall be met and attention shall be given to short- and long-term deformations that could affect the function of the structure.

8.5.2.4 Stress

The stresses in a component shall not exceed the values specified in Clauses 8.7.1, 8.8.4.6, and 8.23.7, as applicable.

8.5.2.5 Vibration

The requirements of Section 3 with respect to vibration of the structure shall be met.

8.5.3 Fatigue limit state

8.5.3.1 Reinforcing bars

Except for reinforcement in deck slabs designed in accordance with Clause 8.18.4, the following requirements shall apply:

(a) The stress range in straight bars shall not exceed 125 MPa.

(b) The stress range at anchorages, connections, and bends shall not exceed 65 MPa.

(c) Unless otherwise Approved, tack welding of reinforcing bars shall not be permitted. For bars containing complete joint penetration groove welds that meet the requirements of CSA W186, the stress range in the vicinity of welds shall not exceed 100 MPa. For other types of welded splices, the stress range shall not exceed 65 MPa.

8.5.3.2 Tendons

The stress range in strands in corrugated steel ducts or for pretensioning strands shall not exceed 125 MPa for radii of curvature of 10 m or more and 70 MPa for radii of curvature of 3.5 m or less. Linear interpolation shall be used for intermediate radii.

The stress range in strands in corrugated plastic ducts shall not exceed 125 MPa. The stress range in deformed and smooth high-strength bars shall not exceed 70 and 90 MPa, respectively. The stress range in tendons at couplers shall not exceed 70 MPa.

8.5.4 Ultimate limit states

8.5.4.1 General

The ultimate limit states to be considered shall be those of strength and stability.

8.5.4.2 Strength

Structural components shall be proportioned so that the factored resistances are equal to or greater than the effects of factored loads.

8.5.4.3 Stability

The structure as a whole and its components shall be proportioned to resist sliding, overturning, uplift, and buckling. The effects of the eccentricity of loads shall be considered.

8.6 Design considerations

8.6.1 General

Except as permitted by Clause 8.6.2.6, load effects shall be determined by elastic analysis, while still retaining equilibrium and strain compatibility.

The strut-and-tie model specified in Clause 8.10 may be used to proportion reinforcement and concrete sections in areas near supports, concentrated loads, and abrupt changes in cross-sections.

8.6.2 Design

8.6.2.1 General

Components shall be proportioned for all load stages that can be critical during the life of the structure, including construction.

8.6.2.2 Member stiffness

Any reasonable assumption may be adopted for computing the axial, flexural, shear, and torsional stiffnesses, provided that the assumption is used consistently throughout the analysis.

8.6.2.3 Imposed deformations

Imposed deformations due to elastic shortening, shrinkage, temperature change, creep, movement of supports, and other causes shall be considered.

The effects on adjoining elements of a structure due to deformations caused by prestressing shall be considered.

The restraining forces produced in the adjoining elements may be reduced to take account of the effects of creep. The reduced restraining forces in the adjoining elements due to the prestress in a component, F', may be calculated as follows:

$$F' = F(1-e^{-\Psi(t,t_0)})$$

where

$\Psi(t,t_0)$ = ratio of creep strain at time t for the loading applied at time t_0

In the absence of a more accurate procedure, the shrinkage stresses shall be reduced by 60% to account for creep.

8.6.2.4 Stress concentrations
Stress concentrations induced by prestressing, other loads, or restraints shall be considered.

8.6.2.5 Secondary effects due to prestress
Secondary effects in statically indeterminate structures induced by prestress shall be considered. The factored secondary effects shall be included with the factored load effects.

8.6.2.6 Redistribution of force effects
When a statically indeterminate structure is constructed in stages, the redistribution of the permanent loads and prestressing effects due to creep shall be taken into account.

Non-linear analysis may be used to determine the redistribution of load effects due to concrete cracking and material non-linearity in statically indeterminate structures. For continuous beams, in lieu of such analysis, the negative moments at the ultimate limit states obtained by linear elastic analysis may be decreased or increased by not more than $20(1 - 2.26c/d)$%, provided that c/d is less than or equal to 0.28 and the positive moments are adjusted accordingly.

8.6.2.7 Directional change of tendons

8.6.2.7.1 Thrusts in plane of tendons
Thrusts produced by directional change of tendons shall be investigated and resisted by the concrete or by reinforcing bars. The design forces shall be taken as the specified strength of the tendons.

The magnitude of the thrust in the plane of the tendon deviation shall be calculated as shown in Figure 8.1.

The resistance per unit length provided by the concrete cover in the plane of the tendon curvature, V_{cd}, may be taken equal to $0.40\phi_c d_{eff} f_{cr}$, where d_{eff} is the lesser of $2(b_w - d_d/2)$ and $2(c_d + d_d/4 + \sum s_d/2)$ when s_d is greater than or equal to d_d, and is equal to $2(c_d + d_d/4)$ when s_d is less than d_d.

Where the resistance provided by the concrete cover is less than the thrust, fully anchored tie-backs to resist the total thrust shall be provided.

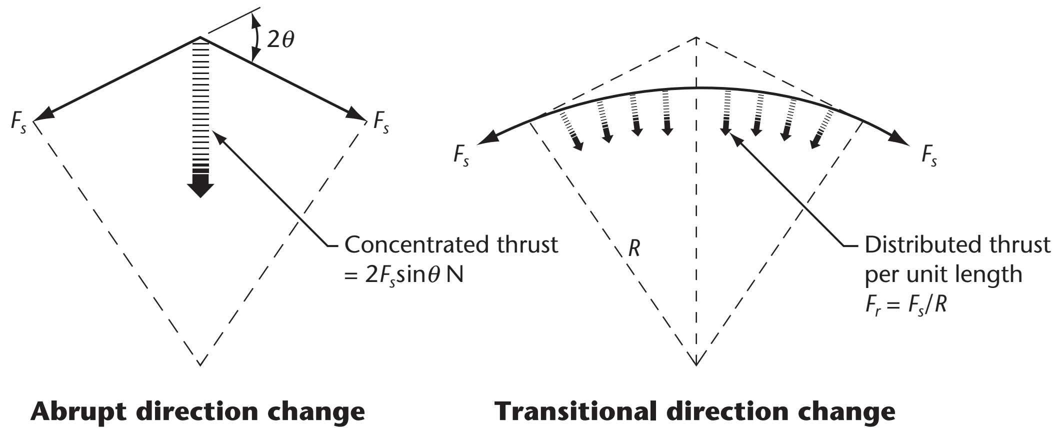


Figure 8.1
Magnitude of thrust
(See Clause 8.6.2.7.1.)

8.6.2.7.2 Multi-strand tendons

The lateral force, F_ℓ, exerted by the bunching of strands of multi-strand post-tensioning tendons at the inside of the curved ducts shall be calculated as $F_s/\pi R$.

Where the resistance provided by the concrete cover is less than the lateral forces, local confining reinforcement, which should be in the form of spirals, shall be provided throughout the curved tendon segment to resist the lateral forces.

8.6.2.7.3 Webs and flanges of box girders

The flexure in the webs or flanges of box girders due to the forces in the plane of the tendon curvature or deviation may be calculated using an elastic frame analysis.

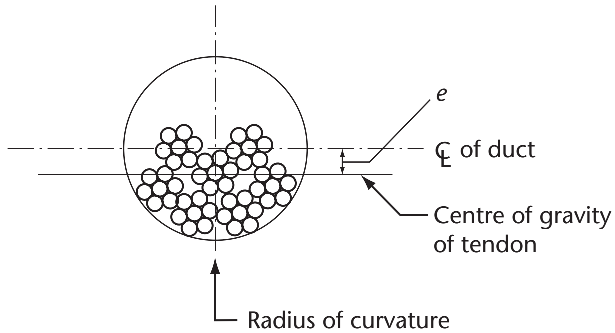
Confinement reinforcement shall be provided around the ducts at each segment face for post-tensioning ducts located in the bottom flange of variable-depth segmental girders whose bottom flange consists of chords between segment joints. The reinforcement shall consist of at least two rows of 10M bars at both sides of each duct and shall extend the full depth of the flange minus the thickness of the top and bottom covers.

8.6.2.7.4 Stress in reinforcement

The stress in the reinforcement to resist tension shall not exceed 240 MPa and the spacing of the reinforcement shall not exceed 250 mm.

8.6.2.7.5 Centre of gravity of tendons in ducts

The eccentricity of curved tendons with respect to the duct shall be determined as shown in Figure 8.2.



Duct diameter, mm	*e*, mm
75 or less	12
Over 75 to 100	20
Over 100	25

Figure 8.2
Eccentricity of curved tendons
(See Clause 8.6.2.7.5.)

8.6.3 Buckling

Consideration shall be given to the buckling of precast components during handling and erection and to the buckling of thin webs and flanges. The effects of lateral eccentricity of loads shall be taken into account in determining the spacing of lateral restraints. However, unless a stability analysis is carried out, the spacing shall not exceed the lesser of $50b$ and $200(b^2/d)$ for beams. For cantilevers with lateral restraint only at the support, the clear distance from the end of the cantilever to the face of the support shall not exceed the lesser of $25b$ and $100(b^2/d)$.

8.7 Prestressing

8.7.1 Stress limitations for tendons

Tendons shall be stressed to provide a minimum effective prestress of $0.45f_{pu}$. The stress in the tendons shall not exceed the values specified in Table 8.2.

Table 8.2
Prestressing tendon stress limits
(See Clause 8.7.1.)

	Tendon type		
		High-strength bar	
	Low-relaxation strand	Smooth	Deformed
At jacking			
Pretensioning	$0.78f_{pu}$	—	—
Post-tensioning	$0.80f_{pu}$	$0.76f_{pu}$	$0.75f_{pu}$
At transfer			
Pretensioning	$0.74f_{pu}$	—	—
Post-tensioning			
At anchorage and couplers	$0.70f_{pu}$	$0.70f_{pu}$	$0.66f_{pu}$
Elsewhere	$0.74f_{pu}$	$0.70f_{pu}$	$0.66f_{pu}$

8.7.2 Concrete strength at transfer
The force in the tendons shall not be transferred to the concrete until the compressive strength of the concrete is at least 25 MPa for pretensioned components and at least 20 MPa for post-tensioned components.

8.7.3 Grouting
After completion of post-tensioning, all internal and external ducts shall be grouted and load shall not be applied to or removed from the components until the grout has reached a compressive strength of at least 20 MPa.

8.7.4 Loss of prestress

8.7.4.1 General
In the calculation of the prestress losses, the following shall be considered:
(a) anchorage slip and friction;
(b) elastic shortening of concrete;
(c) relaxation of tendons;
(d) creep of concrete;
(e) shrinkage of concrete; and
(f) any other special circumstances.

In the calculation of time-dependent losses due to creep and shrinkage of concrete and relaxation of tendons, the interdependence of these phenomena, as well as the influence of non-prestressed reinforcement, shall be considered.

For segmental construction, for components of low- or semi-low-density concrete, and where a more accurate estimate of losses is required, the calculation of prestress losses shall be based on a method supported by proven data.

For multi-stage construction and multi-stage prestressing, the prestress losses shall be calculated by taking into consideration the elapsed time between each stage.

In lieu of a more detailed analysis, the prestress losses at transfer, Δ_{fs1}, and after transfer, Δ_{fs2}, for components constructed using normal-density concrete and single-stage prestressing shall be calculated in accordance with Clauses 8.7.4.2 and 8.7.4.3, respectively. The total loss considered, Δ_{fs}, shall be taken as $\Delta_{fs1} + \Delta_{fs2}$.

8.7.4.2 Losses at transfer

8.7.4.2.1 General
In lieu of a more accurate method, the total losses at transfer, Δ_{fs1}, shall be taken as $ANC + FR + REL_1 + ES$.

8.7.4.2.2 Anchorage slip
The magnitude of the anchorage slip, *ANC*, shall be as required to control the stress in the tendons at transfer or as recommended by the manufacturer of the anchorage, whichever is greater. The magnitude of the slip shall be shown on the Plans.

8.7.4.2.3 Friction loss
The loss due to friction between tendons and the sheath, *FR*, at a distance *x* from the jacking end shall be calculated as $f_{sj}(1 - e^{-(Kx + \mu\alpha)})$.

The values of K and μ shall be based on test data for the materials specified and shall be shown on the Plans. In the absence of such data, the values of K and μ specified in Table 8.3 may be used.

Table 8.3
Friction factors
(See Clause 8.7.4.2.3.)

	Strand		Smooth bar		Deformed bar	
Sheath type	***K***	***μ***	***K***	***μ***	***K***	***μ***
Internal ducts						
Rigid steel	0.002	0.18	—	—	—	—
Semi-rigid steel over 75 mm outside diameter	0.003	0.20	—	—	—	—
Semi-rigid steel up to 75 mm outside diameter	0.005	0.20	0.003	0.20	0.003	0.30
Plastic	0.001	0.14	—	—	—	—
External ducts						
Straight plastic	0.000	—	—	—	—	—
Rigid steel pipe deviators	0.002	0.25	—	—	—	—

8.7.4.2.4 Relaxation of tendons
In pretensioned components, the relaxation loss, REL_1, in low-relaxation tendons initially stressed in excess of $0.50f_{pu}$ shall be calculated as follows:

$$REL_1 = \frac{\log(24t)}{45}\left[\frac{f_{sj}}{f_{py}} - 0.55\right]f_{sj}$$

8.7.4.2.5 Elastic shortening
The loss due to elastic shortening, *ES*, shall be calculated as follows:

(a) pretensioned components:

$$ES = \frac{E_p}{E_{ci}}f_{cir}$$

(b) post-tensioned components:

$$ES = \left[\frac{N-1}{2N}\right]\frac{E_p}{E_{ci}}f_{cir}$$

8.7.4.3 Losses after transfer

8.7.4.3.1 General
The losses after transfer, Δ_{fs2}, shall be taken as $CR + SH + REL_2$.

If the ratio A_s/A_{ps} is equal to or less than 1.0, the losses after transfer due to creep, shrinkage, and relaxation of tendons may be calculated in accordance with Clauses 8.7.4.3.2, 8.7.4.3.3, and 8.7.4.3.4, respectively. Otherwise, a more detailed analysis shall be performed.

8.7.4.3.2 Creep
In lieu of a more accurate method, prestress losses due to creep, CR, may be calculated as follows:

$$CR = \left[1.37 - 0.77(0.01RH)^2\right] K_{cr} \frac{E_p}{E_c}(f_{cir} - f_{cds})$$

where

RH = annual mean relative humidity, %, as shown in Figure A3.1.3

K_{cr} = 2.0 for pretensioned components and 1.6 for post-tensioned components

8.7.4.3.3 Shrinkage
In lieu of a more accurate method, the loss of prestress due to shrinkage, SH, may be calculated as $(117 - 1.05RH)$ for pretensioned components and $(94 - 0.85RH)$ for post-tensioned components.

8.7.4.3.4 Relaxation of tendons
In lieu of a more accurate method, loss of prestress due to relaxation after transfer, REL_2, may be calculated as follows for low-relaxation strand:

$$REL_2 = \left[\frac{f_{st}}{f_{pu}} - 0.55\right]\left[0.34 - \frac{CR + SH}{1.25 f_{pu}}\right]\frac{f_{pu}}{3} \geq 0.002 f_{pu}$$

For high-strength bars, the relaxation loss, REL_2, shall be based on Approved test data. In the absence of such data, REL_2 shall be taken as 20 MPa.

8.8 Flexure and axial loads

8.8.1 General
The requirements of Clauses 8.8.2 to 8.8.7 shall apply with respect to the proportioning of concrete components subjected to flexure or axial loads or both.

8.8.2 Assumptions for the serviceability and fatigue limit states
In addition to the conditions of equilibrium and compatibility of strains, the following shall apply to calculations for the serviceability and fatigue limit states:

(a) Concrete may be assumed to resist tension at sections that are uncracked, except as specified in Clause 8.8.6.

(b) The stress in the concrete shall be assumed to be directly proportional to strain.

(c) Strain in the concrete shall be assumed to vary linearly over the depth of the section, except for deep beams, where a non-linear distribution of strain shall be considered.

(d) Strain changes in bonded reinforcement shall be assumed to be equal to strain changes in the surrounding concrete.

(e) The transformed area of bonded reinforcement may be included in the calculation of section properties. Before grouting, the loss of concrete area due to post-tensioning ducts, coupler sheaths, or transition trumpets shall be considered, except where such loss of area is insignificant. The modular ratio, n, shall not be taken as less than 6. An effective modular ratio of $2n$ may be used to transform the compression reinforcement for stress computations corresponding to permanent loads.

8.8.3 Assumptions for the ultimate limit states

In addition to the conditions of equilibrium and compatibility of strains, the calculations for the ultimate limit states shall be based on the material resistance factors specified in Clause 8.4.6 and the following shall apply to such calculations:

(a) Strain in the concrete shall be assumed to vary linearly over the depth of the section, except for deep beams, which shall satisfy the requirements of Clause 8.10.
(b) Strain changes in bonded reinforcement shall be assumed to be equal to strain changes in the surrounding concrete.
(c) The maximum usable strain at the extreme concrete compression fibre shall be assumed to be 0.0035 unless the concrete is confined and a higher value of strain can be justified. In the latter case, a strain compatibility analysis shall be used.
(d) Except for the strut-and-tie model of Clause 8.10, the stress in the reinforcement shall be taken as the value of the stress determined using strain compatibility based on a stress-strain curve representative of the steel reinforcement to be used, multiplied by ϕ_s or ϕ_p.
(e) The tensile strength of the concrete shall be neglected in the calculation of the factored flexural resistance.
(f) The relationship between concrete strain and the concrete compressive stress may be assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with the results of comprehensive tests. In this regard, an equivalent rectangular concrete stress distribution may be used, i.e., a concrete stress of $\alpha_1 \phi_c f_c'$ is uniformly distributed over an equivalent compression zone, bounded by the edges of the cross-section and a straight line parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain, where c is the shortest length between the fibre of maximum compressive strain and the neutral axis, $\alpha_1 = 0.85 - 0.0015 f_c' \geq 0.67$ and $\beta_1 = 0.97 - 0.0025 f_c' \geq 0.67$.

8.8.4 Flexural components

8.8.4.1 Factored flexural resistance

The factored flexural resistance shall be calculated in accordance with Clause 8.8.3.

8.8.4.2 Tendon stress at the ultimate limit states

The value of f_{ps} for components with bonded tendons shall be computed using a method based on strain compatibility and using stress-strain curves representative of the steel, except that if c/d_p is less than or equal to 0.5, the following expression may be used:

$$f_{ps} = f_{pu}(1 - k_p c/d_p)$$

where k_p is 0.3 for low-relaxation strands, 0.4 for smooth high-strength bars, and 0.5 for deformed high-strength bars, and the value of c shall be determined assuming a stress of f_{ps} in the tendons.

For components with unbonded tendons, f_{ps} shall be taken as f_{se} unless a detailed analysis accounting for deformations demonstrates that a higher value can be used.

External tendons shall be treated as unbonded tendons.

8.8.4.3 Minimum reinforcement

The total amount of reinforcement shall be such that the factored flexural resistance, M_r, of the component is at least 1.20 times the cracking moment. This requirement may be waived if the factored flexural resistance provided is at least one-third greater than the minimum resistance required for factored loads.

8.8.4.4 Cracking moment

A component shall be assumed to crack when the moment at a section is such that a tensile stress of f_{cr}, as specified in Clause 8.4.1.8, is induced in the concrete.

8.8.4.5 Maximum reinforcement

The amount of reinforcement provided shall be such that the factored flexural resistance, M_r, is developed with c/d not exceeding 0.5. This requirement may be waived if it is demonstrated to the satisfaction of the Regulatory Authority that the consequences of reinforcement not yielding are acceptable.

8.8.4.6 Prestressed concrete stress limitations

The stresses in the concrete shall not exceed the following:

(a) At transfer and during construction:
  - (i) compression: $0.60f'_{ci}$;
  - (ii) tension in components without reinforcing bars in the tension zone: $0.50f'_{ci}$. Where the calculated tensile stress exceeds $0.50f_{cri}$, reinforcing bars in which the tensile stress is assumed to be 240 MPa shall be provided to resist the total tensile force in the concrete, calculated on the basis of an uncracked section; and
  - (iii) tension at joints in segmental components:
    - (1) without reinforcing bars passing through the joint in the tension zone: zero; and
    - (2) with reinforcing bars passing through the joint in the tension zone: $0.50f_{cri}$.

    Where the calculated tensile stress is between zero and $0.50f_{cri}$, reinforcing bars in which the tensile stress is assumed to be 240 MPa shall be provided to resist the total tensile force in the concrete calculated on the basis of an uncracked section.

(b) At the serviceability limit states, if the tension in the concrete exceeds f_{cr}, Clause 8.12 shall apply. Tension shall not be permitted across the joints of segmental components unless bonded reinforcing bars pass through the joints in the tensile zone.

(c) In prestressed slabs with circular voids, the average compressive stress due to effective longitudinal prestress alone shall not exceed 6.5 MPa. In post-tensioned slabs with circular voids, the following shall apply:
  - (i) an effective transverse prestress shall be provided to give a compressive stress of 4.5 MPa in the concrete above the longitudinal voids; and
  - (ii) the thicknesses of the concrete above and below the voids shall not be less than 175 mm and 125 mm, respectively.

8.8.5 Compression components

8.8.5.1 General

The proportioning of cross-sections subject to combined flexure and axial compression shall be in accordance with Clause 8.8.3.

8.8.5.2 Slenderness effects

The proportioning of compression components shall be based on forces and moments determined from an analysis of the structure. Except as permitted by Clause 8.8.5.3, such an analysis shall include the influence of axial loads and variable moment of inertia on component stiffness and moments, the effect of deflections on the moments and forces, and the effects of the duration of the loads and prestressing forces.

8.8.5.3 Approximate evaluation of slenderness effects

In lieu of the requirements of Clause 8.8.5.2, the proportioning of non-prestressed compression components with a slenderness ratio, $k\ell_u/r$, less than 100, may be based on the following approximate procedure:

(a) The unsupported length, ℓ_u, of a compression component shall be taken as the clear distance between components capable of providing lateral support for the compression component.

(b) For components braced against side-sway, the effective length factor, k, shall be taken as 1.0 unless an analysis shows that a lower value can be used. For components not braced against side-sway, the effective length factor, k, shall be determined with due consideration of end restraint and the effects of cracking and reinforcement on relative stiffness, and shall not be taken as less than 1.0.

(c) The radius of gyration, r, shall be calculated for the gross concrete section.

(d) For components braced against side-sway, the effects of slenderness may be neglected when the slenderness ratio, $k\ell_u/r$, is less than $[34 - 12(M_1/M_2)]$.

(e) For components not braced against side-sway, the effects of slenderness may be neglected when $k\ell_u/r$ is less than 22.

(f) Components in structures that do not undergo appreciable lateral deflections shall be proportioned using the factored axial load at the ultimate limit state and a magnified moment, M_c, calculated as follows:

$$M_c = \delta M_2$$

where

$$\delta = \frac{C_m}{1 - \left[\dfrac{P_f}{0.75P_c}\right]} \geq 1.0$$

where

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

In lieu of a more accurate calculation, EI may be taken as

$$EI = \frac{0.2E_cI_g + E_sI_s}{1 + \beta_d}$$

or, conservatively, as $0.25E_cI_g$.

For components braced against side-sway, and without transverse loads between supports for the loading case under consideration, C_m may be taken as

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

For all other cases, C_m shall be taken as 1.0.

(g) If calculations show that there is no moment at both ends of a compression component or that the calculated end eccentricities are less than $(15 + 0.03h)$ mm, M_2 shall be based on a minimum eccentricity of $(15 + 0.03h)$ mm about each principal axis separately.

When calculated end eccentricities are less than $(15 + 0.03h)$ mm, calculated end moments shall be used to evaluate M_1/M_2. However, if calculations show that there is essentially no moment at both ends of a compression component, the ratio M_1/M_2 shall be taken as equal to 1.0.

(h) For eccentrically prestressed components, consideration shall be given to the effect of lateral deflection due to prestressing in determining the magnified moment.

(i) For components in structures that undergo appreciable lateral deflections resulting from combinations of vertical load or combinations of vertical and lateral loads, M_1 and M_2 shall be determined using a second-order analysis.

8.8.5.4 Maximum factored axial resistance

For components with spiral reinforcement, the factored axial resistance, P_r, shall be less than or equal to $0.80P_o$, and for components with tie reinforcement, shall be less than or equal to $0.75P_o$.

8.8.5.5 Biaxial loading

In lieu of an analysis based on stress and strain compatibility for a loading condition of biaxial bending, non-circular components subjected to biaxial bending may be proportioned approximately in accordance with the following:

(a) When the required factored axial resistance is equal to or greater than $0.10\phi_c f_c' A_g$:

$$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} - \frac{1}{P_o}$$

(b) When the required factored axial resistance is less than $0.10\phi_c f_c' A_g$:

$$\frac{M_x}{M_{rx}} + \frac{M_y}{M_{ry}} \le 1$$

8.8.5.6 Reinforcement limitations

The maximum area of prestressed and non-prestressed longitudinal reinforcement shall be such that

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \le 0.08$$

and

$$\frac{A_{ps} f_{ps}}{A_g f_c'} \le 0.30$$

The minimum area of prestressed and non-prestressed longitudinal reinforcement shall be such that

$$\frac{A_s f_y}{A_g f_c'} + \frac{A_{ps} f_{pu}}{A_g f_c'} \ge 0.135$$

When the proportioning of compression components is controlled by considerations other than applied loading, the minimum area of longitudinal reinforcement shall be that required for a component with a reduced effective area of concrete capable of resisting the factored loads.

The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement. The minimum size of bar shall be 15M and the spacing shall not exceed 300 mm.

8.8.5.7 Transverse reinforcement

Transverse reinforcement shall be provided in accordance with Clause 8.14.4.

8.8.5.8 Hollow rectangular components

The wall slenderness ratio of a hollow rectangular cross-section, calculated as the larger internal plan dimension of the section divided by the wall thickness, shall not exceed 35.

The resistance of a section with a wall slenderness ratio greater than 15 shall be reduced at a rate of 2.5% for each unit increase in the wall slenderness ratio above 15, to a maximum reduction of 25% at a wall slenderness ratio of 25. The reduction shall remain at this level up to a wall slenderness ratio of 35.

Two layers of longitudinal and transverse reinforcement shall be provided in each wall of the cross-section, with one layer near each face of the wall and the two layers having approximately equal areas.

The spacing of the longitudinal reinforcement shall comply with the requirements for walls and slabs in Clause 8.14.2.1.

The transverse reinforcement shall comply with the requirements of Clause 8.14.4.3.

Cross-ties shall be provided between layers of reinforcement in each wall. The cross-ties shall have a standard 135° hook at one end and a standard 90° hook at the other end and shall be located to enclose each longitudinal and transverse bar at a spacing not to exceed 600 mm.

8.8.6 Tension components

For components in which the applied loading induces tensile stresses throughout the cross-section, the load shall be assumed to be resisted by the reinforcement alone when the tensile stress under serviceability limit state loads exceeds $0.6f_{cr}$. The requirements of Clause 8.12 shall apply.

The amount of reinforcement shall be such that the factored axial tensile resistance is at least 1.20 times the load inducing a tensile stress of f_{cr} in the concrete.

Components subjected to eccentric tension loading that induces both tensile and compressive stresses in the cross-section shall comply with Clauses 8.8.2 to 8.8.4 and 8.12.

8.8.7 Bearing

8.8.7.1 Factored bearing resistance

The factored bearing resistance of concrete without transverse reinforcement shall be taken as $0.85\phi_c f_c' A_1$.

8.8.7.2 Bearing area

The bearing area of a concrete component shall be taken as the loaded area A_1, except that

(a) when the supporting surface is wider on all sides than the loaded area, the factored bearing resistance may be multiplied by $\sqrt{A_2/A_1}$, but not by a value greater than 2; and

(b) when the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest prismoid contained wholly within the support, having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

8.8.7.3 Bursting and spalling

When the factored applied load exceeds that based on the bearing area permitted by Clause 8.8.7.2, adequate provision shall be made to resist the bursting and spalling forces in accordance with Clause 8.16.

8.9 Shear and torsion

8.9.1 General

8.9.1.1 Consideration of torsion

Torsional effects shall be considered in regions where the factored torsional moment, T_f, is greater than $0.25T_{cr}$, where

$$T_{cr} = 0.80\phi_c f_{cr} \frac{A_{cp}^2}{p_c}\left[1+\frac{f_{ce}}{0.80\phi_c f_{cr}}\right]^{0.5}$$

8.9.1.2 Regions requiring transverse reinforcement

Except for solid slabs, walls, and footings, transverse reinforcement shall be provided in all regions where V_f is greater than $(0.20\phi_c f_{cr} b_v d_v + 0.5\phi_p V_p)$ and T_f is greater than $0.25T_{cr}$.

8.9.1.3 Minimum amount of transverse reinforcement

When calculations show that transverse shear reinforcement is required, A_v shall not be less than $0.15f_{cr}(b_v s/f_y)$.

8.9.1.4 Design yield strength of transverse reinforcement
The design yield strength of tendons used as transverse reinforcement shall be taken as the effective prestress plus 400 MPa, but shall not be taken greater than f_{py}.

8.9.1.5 Effective shear depth
The effective shear depth, d_v, shall be taken as the greater of $0.72h$ or $0.9d$, where d is taken as the distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement in the half-depth of the section containing the flexural tension zone.

8.9.1.6 Effective web width
The effective web width, b_v, shall be taken as the minimum web width within the depth d_v. In determining b_v at a particular level, one-half the diameters of ungrouted ducts or one-quarter the diameters of grouted ducts at that level shall be subtracted from the web width. For solid circular sections, b_v may be taken as the diameter.

8.9.1.7 Variable-depth components
The resolved force components of inclined flexural compression and flexural tension in variable-depth components shall be taken into account when calculating shear resistance.

8.9.1.8 Reduced prestress within transfer length
In pretensioned members, the reduction in prestress within the transfer length of prestressing tendons shall be considered when calculating 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 strands and 100 diameters for single wires.

8.9.2 Design procedures

8.9.2.1 Flexural regions
When it is reasonable to assume that plane sections remain plane, components shall be proportioned for shear and torsion using either the sectional design model specified in Clause 8.9.3 or the strut-and-tie model specified in Clause 8.10. In addition, the applicable requirements of Clause 8.9.1 shall be satisfied.

8.9.2.2 Regions near discontinuities
When the plane sections assumption of flexural theory is not applicable, components shall be proportioned for shear and torsion using the strut-and-tie model specified in Clause 8.10. In addition, the applicable requirements of Clause 8.9.1 shall be satisfied.

8.9.2.3 Interface regions
Interfaces between elements such as webs and flanges, between dissimilar materials, and between concretes cast at different times or at potential or existing major cracks shall be proportioned for shear transfer in accordance with Clause 8.9.5.

8.9.2.4 Slabs, walls, and footings
With the exception of deck slabs, slab-type components subjected to concentrated loads shall be proportioned for shear in accordance with Clause 8.9.4 or 8.10.

8.9.2.5 Detailed analysis
In lieu of the methods specified in Clauses 8.9.2.1 to 8.9.2.4, the resistance of components in shear or in shear combined with torsion may be determined by satisfying the applicable conditions of equilibrium and compatibility of strains, using appropriate stress-strain relationships for reinforcement and for diagonally cracked concrete.

8.9.3 Sectional design model

8.9.3.1 Sections near supports
Where the reaction force introduces compression into the end region of a component, the critical section for shear near the support shall be located at a distance of d_v from the face of the support.

8.9.3.2 Required shear resistance
Components subjected to shear shall be proportioned so that V_f is less than V_r.

8.9.3.3 Factored shear resistance
The factored shear resistance, V_r, shall be calculated as $V_c + V_s + V_p$. However, $V_c + V_s$ shall not exceed $0.25\phi_c f_c' b_v d_v$.

8.9.3.4 Determination of V_c
V_c shall be calculated as $2.5\beta\,\phi_c\,f_{cr}\,b_v\,d_v$. However, f_{cr} shall not be greater than 3.2 MPa.

8.9.3.5 Determination of V_s
V_s shall be determined as follows:

(a) For components with transverse reinforcement perpendicular to the longitudinal axis, V_s shall be calculated as follows:

$$V_s = \frac{\phi_s f_y A_v d_v \cot\theta}{s}$$

(b) For components with transverse reinforcement inclined at an angle to the longitudinal axis and in the direction that will intersect diagonal cracks caused by the shear, V_s shall be calculated as follows:

$$V_s = \frac{\phi_s f_y A_v d_v \left(\cot\theta + \cot\alpha\right)\sin\alpha}{s}$$

8.9.3.6 Determination of β and θ for non-prestressed components (simplified method)
For non-prestressed components not subjected to axial tension, and provided that the specified yield strength of the longitudinal reinforcement does not exceed 400 MPa and the design concrete strength does not exceed 60 MPa, the value of the angle of inclination, θ, shall be taken as 42° and the value of β shall be determined as follows:

(a) For sections with at least the minimum amount of transverse reinforcement required by Clause 8.9.1.3, β shall equal 0.18.

(b) For sections containing no transverse reinforcement located in footings where the distance from the point of zero shear to the face of the column, pedestal, or wall is less than $3d_v$, β shall equal 0.18.

(c) For other sections not containing transverse reinforcement but having a specified nominal maximum size of coarse aggregate not less than 20 mm, β shall equal $230/(1000 + d_v)$.

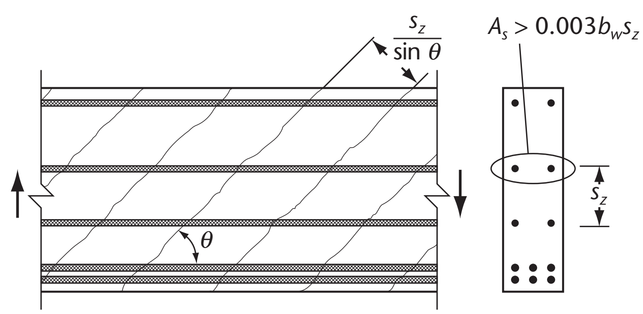
Alternatively, for sections containing no transverse reinforcement, β may be determined for all aggregate sizes as equal to $230/(1000 + s_{ze})$, where the equivalent crack spacing parameter, s_{ze}, is $35s_z/(15 + a_g)$. However, s_{ze} shall not be taken as less than $0.85s_z$. As shown in Figure 8.3, the crack spacing parameter, s_z, shall be taken as d_v or as the distance between layers of distributed longitudinal reinforcement where each intermediate layer of such reinforcement has an area at least equal to $0.003b_w s_z$.

8.9.3.7 Determination of β and θ (general method)

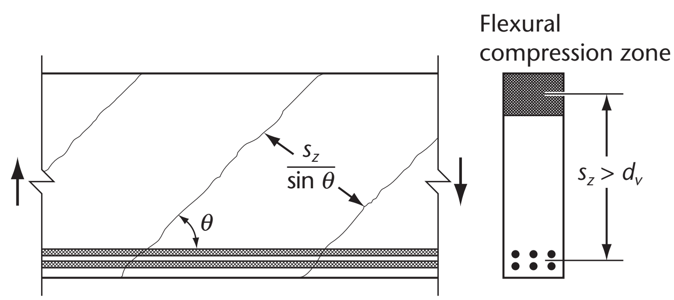
The value of β shall be calculated as follows:

$$\beta = \left[\frac{0.4}{(1+1500\varepsilon_x)}\right]\left[\frac{1300}{(1000+s_{ze})}\right]$$

For sections containing at least the minimum transverse reinforcement required by Clause 8.9.1.3, s_{ze} shall be taken as 300 mm; otherwise, s_{ze} shall be calculated in accordance with Clause 8.9.3.6. The value of a_g in Clause 8.9.3.6 shall be taken as zero if f'_c is greater than 70 MPa and shall be linearly equal to zero as f'_c goes from 60 to 70 MPa. The angle of inclination, θ, shall be calculated as $(29 + 7000\varepsilon_x)(0.88 + s_{ze}/2500)$.



(a) Component without transverse reinforcement but with well-distributed longitudinal reinforcement



(b) Component without transverse reinforcement and with concentrated longitudinal reinforcement

Figure 8.3
Influence of reinforcement on spacing of diagonal cracks
(See Clause 8.9.3.6.)

8.9.3.8 Determination of ε_x

In lieu of more accurate calculations, ε_x shall be calculated as follows:

$$\varepsilon_x = \frac{M_f/d_v + V_f - V_p + 0.5N_f - A_{ps}f_{po}}{2\left(E_sA_s + E_pA_{ps}\right)}$$

Evaluation of this equation shall be based on the following:

(a) V_f and M_f are positive quantities and M_f shall not be less than $(V_f - V_p)d_v$.

(b) N_f shall be taken as positive for tension and negative for compression. For rigid frames and rectangular culverts, the value of N_f used to determine ε_x may be taken as twice the compressive axial thrust calculated by elastic analysis.

(c) A_s and A_{ps} are the areas of reinforcing bars and prestressing tendons in the half-depth of the section containing the flexural tension zone.

(d) f_{po} may be taken as $0.7f_{pu}$ for bonded tendons outside the transfer length and f_{pe} for unbonded tendons.

(e) In calculating A_s, the area of bars that terminate less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.

(f) If the value of ε_x is negative, it shall be taken as zero or recalculated with the denominator replaced by $2(E_sA_s + E_pA_{ps} + E_cA_{ct})$. However, ε_x shall not be less than -0.20×10^{-3}.

(g) 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 θ and β.

(h) 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 the equation shall be doubled.

(i) θ and β may be determined from Clause 8.9.3.7 using a value of ε_x that is greater than that calculated from the equation in this Clause. However, ε_x shall not be greater than 3.0×10^{-3}.

8.9.3.9 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.

8.9.3.10 Extension of longitudinal reinforcement

At every section, the longitudinal reinforcement shall be designed to resist the additional tensile forces caused by shear as specified in Clauses 8.9.3.11 and 8.9.3.12. Alternatively, for members not subjected to significant tension or torsion, these requirements may be satisfied by extending the flexural tension reinforcement a distance of $d_v\cot\theta$ beyond the location required by flexure alone.

8.9.3.11 Longitudinal reinforcement on the flexural tension side

Longitudinal reinforcement on the flexural tension side shall be proportioned so that at all sections the factored resistance of the reinforcement, taking account of the stress that can be developed in this reinforcement, is greater than or equal to $F_{\ell t}$, calculated as follows:

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

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 this equation, d_v may be taken as the flexural lever arm at the factored resistance.

8.9.3.12 Longitudinal reinforcement on the flexural compression side

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 $F_{\ell c}$, calculated as follows:

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

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

8.9.3.13 Compression fan regions

In regions adjacent to maximum moment locations, the cross-sectional area of longitudinal reinforcement on the flexural tension side of the member need not exceed the cross-sectional area required to resist the maximum moment acting alone. This exception shall apply only when 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.

8.9.3.14 Anchorage of longitudinal reinforcement at exterior supports

At exterior direct-bearing supports, the longitudinal reinforcement on the flexural tension side for the member shall be capable of resisting a tensile force of $(V_f - 0.5V_s - V_p)\cot\theta + 0.5N_f$, where 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.

8.9.3.15 Transverse reinforcement for combined shear and torsion

For sections subjected to combined shear and torsion, the transverse reinforcement provided shall be at least equal to the sum of that required for shear and that required for the coexisting torsion.

8.9.3.16 Transverse reinforcement for torsion

The amount of transverse reinforcement required for torsion shall be such that T_r is greater than or equal to T_f.

8.9.3.17 Factored torsional resistance

The value of T_r shall be calculated as follows:

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

where A_o is taken as $0.85A_{oh}$ and θ is as specified in Clause 8.9.3.6 or 8.9.3.7.

8.9.3.18 Cross-sectional dimensions to avoid crushing for combined shear and torsion

The cross-sectional dimensions to avoid crushing for combined shear and torsion shall be as follows:

(a) For box sections:

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

If the wall thickness of the box section is less than A_{oh}/p_h, the second term in this expression 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:

$$\sqrt{\left[\frac{V_f - V_p}{b_v d_v}\right]^2 + \left[\frac{T_f p_h}{1.7A_{oh}^2}\right]^2} \le 0.25\phi_c f_c'$$

8.9.3.19 Determination of ε_x for combined shear and torsion

If β and θ and are determined using Clause 8.9.3.7, the value of ε_x for a section subjected to torsion shall be determined by the equation specified in Clause 8.9.3.8, but the term $(V_f - V_p)$ in Clause 8.9.3.8(a) shall be replaced by

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

8.9.3.20 Proportioning longitudinal reinforcement for combined shear and torsion

The longitudinal reinforcement shall be proportioned to satisfy the requirements of Clauses 8.9.3.11 and 8.9.3.12, except that the term $(V_f - 0.5V_s - V_p)$ in those clauses shall be replaced by

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

8.9.4 Slabs, walls, and footings

8.9.4.1 Critical sections for shear

In determining the shear resistance of slabs, walls, and footings in the vicinity of concentrated loads or reactions, the more severe of the following two actions shall govern:

(a) beam action, with a critical section extending in a plane across the entire width and located at a distance, d, from the face of the concentrated load or reaction area, or from any change in slab thickness; and

(b) two-way action, with a critical section perpendicular to the plane of the slab and located so that its perimeter, b_o, is a minimum, but need not approach closer than $0.5d$ to the perimeter of the concentrated load or reaction area. Shear resistance shall also be investigated at critical sections located at a distance not closer than $0.5d$ from any change in slab thickness and located such that the perimeter, b_o, is a minimum.

8.9.4.2 Beam action

For beam action, the shear resistance shall be calculated in accordance with Clause 8.9.3.3.

8.9.4.3 Two-way action

For two-way action, the shear resistance shall be such that V_r is greater than V_f, where

$$V_r = (\phi_c f_{cr} + 0.25 f_{pc}) b_o d + V_p$$

8.9.5 Interface shear transfer

8.9.5.1 General

A crack shall be assumed to occur along the shear plane and the relative displacement shall be considered to be resisted by cohesion and friction maintained by the shear-friction reinforcement crossing the crack. In lieu of more detailed calculations, the shear resistance of the plane, v, may be calculated as $\phi_c (c + \mu\sigma)$, but v shall not exceed $0.25\phi_c f_c'$ or 6.5 MPa. c and μ shall be as specified in Clause 8.9.5.2 and σ shall be as specified in Clause 8.9.5.3.

8.9.5.2 Values of c and μ

8.9.5.2.1

The following values shall be taken for c and μ in Clause 8.9.5.1:

(a) For concrete placed against hardened concrete, with the surface clean and free of laitance but not intentionally roughened, c shall equal 0.25 MPa and μ shall equal $0.60\lambda_1$.

(b) For concrete placed against hardened concrete, with the surface clean and free of laitance and intentionally roughened to a full amplitude of about 5 mm and a spacing of about 15 mm, c shall equal 0.50 MPa and μ shall equal $1.00\lambda_1$.

(c) For concrete placed monolithically, c shall equal 1.00 MPa and μ shall equal $1.40\lambda_1$.

The values of λ_1 shall be as specified in Clause 8.9.5.2.2.

8.9.5.2.2 Values of λ_1

The values of λ_1 shall be as follows:

(a) normal-density concrete: 1;

(b) semi-low-density concrete: 0.85; and

(c) low-density concrete: 0.75.

8.9.5.3 Value of σ

The value of σ in Clause 8.9.5.1 shall be calculated as follows:

$$\sigma = \rho_v f_y + \frac{N}{A_{cv}}$$

where

$$\rho_v = \frac{A_{vf}}{A_{cv}}$$

8.9.5.4 Anchorage of shear-friction reinforcement

The shear-friction reinforcement shall be capable of developing the specified yield strength of the reinforcement on both sides of the shear-friction plane.

8.10 Strut-and-tie model

8.10.1 General

Strut-and-tie models may be used to determine internal force effects near supports and the points of application of concentrated loads. Strut-and-tie models shall be considered for the design of deep footings and pile caps or other situations in which the distance between the centres of applied load and the supporting reaction is less than twice the component thickness.

8.10.2 Structural idealization

The strength of concrete structures, components, or regions shall be investigated by idealizing them 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 compressive struts and tensile ties.

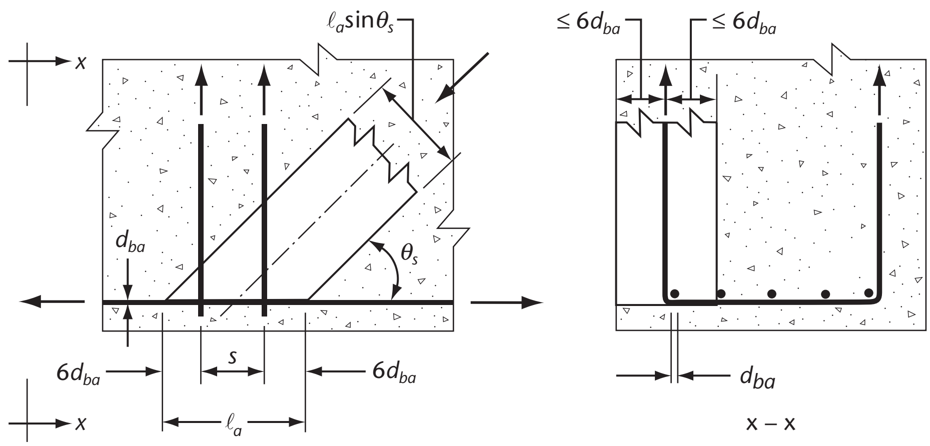
8.10.3 Proportioning of a compressive strut

8.10.3.1 Strength of strut

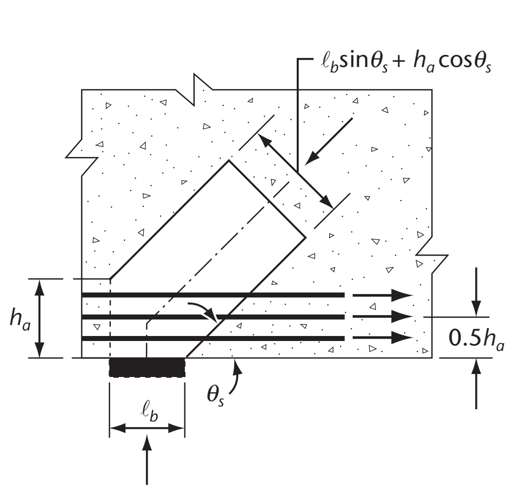
The dimensions of the strut shall be large enough to ensure that the calculated compressive force in the strut does not exceed $\phi_c A_{cs} f_{cu}$, where A_{cs} and f_{cu} are determined in accordance with Clauses 8.10.3.2 and 8.10.3.3, respectively.

8.10.3.2 Effective cross-sectional area of strut

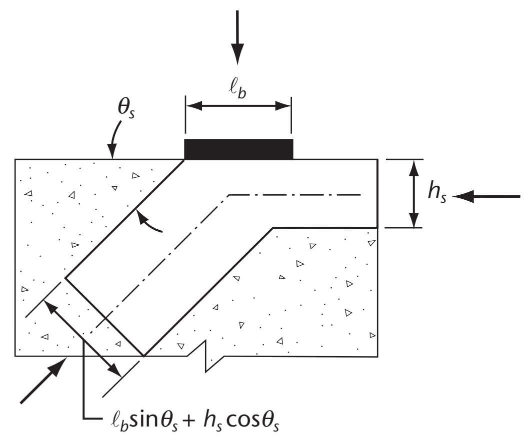
The value of A_{cs} shall be calculated by considering both the influence of the anchorage conditions at the ends of the strut, as shown in Figure 8.4, and the available concrete area.



(a) Strut anchored by reinforcement



(b) Strut anchored by bearing and reinforcement



(c) Strut anchored by bearing and strut

Figure 8.4
Influence of anchorage conditions on effective cross-sectional area of strut
(See Clause 8.10.3.2.)

8.10.3.3 Limiting compressive stress in strut

The value of f_{cu} shall be calculated as follows:

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \le \alpha_1 f'_c$$

where ε_1 is calculated as $\varepsilon_s + (\varepsilon_s + 0.002)\cot^2\theta_s$, in which θ_s is the smallest angle between the compressive strut and the adjoining tensile ties and ε_s is the tensile strain in the tensile tie inclined at θ_s to the compressive strut.

8.10.3.4 Reinforced strut

If the compressive strut contains reinforcement that is parallel to the strut and has been detailed to develop its yield stress in compression, the calculated force in the strut shall not exceed $\phi_c f_{cu} A_{cs} + \phi_s f_y A_{ss}$. The strut shall be reinforced with lateral ties in accordance with Clause 8.14.4.3.

8.10.4 Proportioning of a tension tie

8.10.4.1 Strength of tie

The cross-sectional area of the reinforcement in a tension tie shall be large enough to ensure that the calculated tensile force in the tie does not exceed $\phi_s f_y A_{st} + \phi_p f_{py} A_{ps}$, where A_{st} is the cross-sectional area of the reinforcing bars in the tie and A_{ps} is the cross-sectional area of the tendons in the tie.

8.10.4.2 Anchorage of tie

The tension tie reinforcement shall be anchored so that it is capable of resisting the calculated tension in the reinforcement at the inner edge of the node region. For straight bars extending a distance x beyond the inner edge of the node region, where x is less than ℓ_d, the calculated stress shall not exceed $f_y(x/\ell_d)$, where ℓ_d is calculated in accordance with Clause 8.15.2.

8.10.5 Proportioning of node regions

8.10.5.1 Stress limits in node regions

Unless special confining reinforcement is provided, the calculated concrete compressive stress in the node regions shall not exceed the following (with α_1, as specified in Clause 8.8.3):

(a) $\alpha_1 \psi_c f'_c$ in node regions bounded by compressive struts and bearing areas;
(b) $0.88\alpha_1 \psi_c f'_c$ in node regions anchoring a tension tie in only one direction; and
(c) $\alpha_1 f'_c$ in node regions anchoring tension ties in more than one direction.

8.10.5.2 Satisfying stress limits in node regions

The stress limits in node regions may be considered satisfied if the following two conditions are met:

(a) the bearing stress in the node regions produced by concentrated loads or reactions does not exceed the stress limits specified in Clause 8.10.5.1; and
(b) the tensile tie reinforcement is uniformly distributed over an effective area of concrete at least equal to the tensile tie force divided by the stress limits specified in Clause 8.10.5.1.

8.10.6 Crack control reinforcement

Except for slabs and footings, components or regions that have been designed in accordance with Clauses 8.10.1 to 8.10.5 shall contain an orthogonal grid of reinforcing bars near each face. The spacing of this reinforcement shall not exceed 300 mm. The ratio of reinforcement area to gross concrete area shall not be less than 0.003, but the reinforcement need not be more than 1500 mm^2/m in each face and in each direction. If located within the tension tie, the crack control reinforcement may also be considered tension tie reinforcement.

8.11 Durability

8.11.1 Deterioration mechanisms

The deterioration mechanisms to be considered for concrete components shall include, but not be limited to, the following:
(a) carbonation-induced corrosion without chloride;
(b) chloride-induced corrosion due to seawater;
(c) chloride-induced corrosion from sources other than seawater;
(d) freeze-thaw deterioration;
(e) alkali aggregate reaction;
(f) chemical attack; and
(g) abrasion.

8.11.2 Protective measures

8.11.2.1 Concrete quality

8.11.2.1.1 General

The maximum water to cementing materials ratio by mass requirements for structural concrete shall be as specified in Table 8.4 for the applicable combination of deterioration mechanisms and environmental exposures.

For structural concrete not covered by Table 8.4, the maximum water to cementing materials ratio shall be 0.50 unless otherwise Approved.

Table 8.4
Maximum water to cementing materials ratio
(See Clause 8.11.2.1.1.)

Deterioration mechanism	Environmental exposure	Maximum ratio*†‡
Chloride-induced corrosion	Marine	
	Airborne salts	0.45
	Tidal and splash spray	0.45
	Submerged	0.40
	Other than marine	
	Wet, rarely dry	0.40
	Dry, rarely wet	0.40
	Cyclic, wet/dry	0.40
Freeze-thaw attack§	Unsaturated	0.45
	Saturated	0.40
Carbonation-induced corrosion without chloride	Wet, rarely dry	0.50
	Dry, rarely wet	0.50
	Cyclic, wet/dry	0.45

**Unless otherwise Approved.*
†*Water to cementing materials ratio by mass. Cementing materials include Portland cement, silica fume, fly ash, and slag.*
‡*The ratio shall be independently verified on the submitted concrete mix design and concrete materials. Quality control and quality assurance measures shall be taken to ensure uniformity of concrete production so that water/cement limits are maintained throughout production. Such measures shall include measurements of slump, air content, unit weight, and strength.*
§*Air content shall be in accordance with CAN/CSA-A23.1. The minimum air content shall be 5.5% for concrete in saturated conditions unless otherwise Approved.*

8.11.2.1.2 Concrete composition

The concrete composition shall be such that the concrete

(a) satisfies all specified performance criteria;
(b) contains durable materials;
(c) can be placed, compacted, and cured to form a dense cover to the reinforcement;
(d) is free of harmful internal reactions, e.g., alkali-aggregate reactions;
(e) withstands the action of freezing and thawing, including the effects of de-icing salts (where applicable);
(f) withstands external exposures, e.g., weathering, gases, liquids, and soil; and
(g) withstands mechanical attacks, e.g., abrasion.

8.11.2.1.3 Concrete placement

The maximum and minimum allowable concrete placement temperatures to ensure durable concrete shall be shown on the Plans.

8.11.2.1.4 Compaction

The methods used for mixing, placing, and compacting the fresh concrete shall be shown on the Plans to ensure that

(a) the constituents are distributed uniformly in the mixture;
(b) the concrete is well consolidated; and
(c) the reinforcement, pretensioning strands, and post-tensioning ducts are not damaged by vibrating operations.

8.11.2.1.5 Cold joints

The concrete surface at a cold joint shall be rough cleaned, abrasive blast cleaned, or both. Coated bars at cold joints shall be protected during abrasive blast cleaning.

8.11.2.1.6 Slip-form construction

The slip-form construction for reinforced concrete components shall not be permitted unless Approved.

8.11.2.1.7 Finishing

The methods to be used for finishing the surface of the concrete to ensure a durable surface shall be shown on the Plans.

8.11.2.1.8 Curing

The methods to be used for curing the concrete to ensure durability shall be shown on the Plans.

8.11.2.1.9 Exposure to chlorides

Chlorides shall not be added to fresh concrete and the concrete components shall not be exposed to chlorides until the concrete has attained the specified minimum strength.

8.11.2.2 Concrete cover and tolerances

The minimum concrete cover and tolerances for steel reinforcement, pretensioning strands, and post-tensioning ducts shall not be less than the values specified in Table 8.5 for the applicable environmental exposure. The minimum cover and tolerances for anchorages and mechanical connections shall be those specified for reinforcing steel in Table 8.5. The applicable concrete covers and tolerances shall be shown on the Plans.

8.11.2.3 Corrosion protection for reinforcement, ducts, and metallic components

Unless otherwise Approved, steel reinforcement, anchorages, and mechanical connections specified for use within 75 mm of a surface exposed to moisture containing de-icing chemicals shall have an Approved protective coating, be protected by other Approved methods of corrosion protection or prevention, or be of non-corrosive materials. Exposed inserts, fasteners, and plates shall be protected from corrosion by Approved methods. Sheaths for internal post-tensioning ducts specified for use within 100 mm of a surface subject to moisture containing de-icing chemicals shall be made of non-corroding material or with an Approved coating. The ends of pretensioning strands shall be protected by Approved methods when they are not encased in concrete.

8.11.2.4 Sulphate-resistant cements

Sulphate-resistant cement shall be specified for concrete in deep foundation units, footings, buried structures made of reinforced concrete, or other substructure components exposed to soils or water to an extent sufficient to cause a strong sulphate attack on concrete. Protection against sulphate attack shall be in accordance with CAN/CSA-A23.1.

8.11.2.5 Alkali-reactive aggregates

Aggregates for concrete shall be tested for susceptibility to alkali aggregate reaction. The evaluation and use of aggregates susceptible to alkali aggregate reaction shall be in accordance with CAN/CSA-A23.1 and CAN/CSA-A23.2-27A.

8.11.2.6 Drip grooves

Continuous drip grooves shall be formed on the underside of the bridge deck. The grooves shall be located close to the fascia and shall have minimum dimensions for depth and width of 20 mm and 50 mm, respectively. At expansion joints without joint armouring, the end of the concrete deck slab shall be provided with a drip groove. If joint armouring is provided, it shall cover the end of the deck slab and extend at least 50 mm below the concrete in order to form a drip projection.

8.11.2.7 Waterproofing

Unless otherwise Approved, concrete decks that are expected to be salted for winter maintenance or are exposed to a marine environment shall be waterproofed with an Approved waterproofing system.

Table 8.5
Minimum concrete covers and tolerances
(See Clause 8.11.2.2.)

Environmental exposure	Component	Reinforcement/ steel ducts	Concrete covers and tolerances: Cast-in-place concrete, mm	Concrete covers and tolerances: Precast concrete, mm
De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray	(1) Top of bottom slab for rectangular voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	40 ± 10 — 60* ± 10	40 ± 10 55 ± 5 60* ± 10
	(2) (a) Top surface of buried structure with less than 600 mm fill†	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	50 ± 10 65 ± 5 70* ± 10
	(b) Bottom slab of buried structure	Reinforcing steel Pretensioning strands	70 ± 20 —	55 ± 10 70 ± 5
	(3) Top surface of structural component, except (1) and (2) above‡	Post-tensioning ducts Longitudinal Transverse ($d_d \leq 60$ mm) Transverse ($d_d > 60$ mm)	 130* ± 15 90* ± 15 130* ± 15	 120* ± 10 80* ± 10 120* ± 10
	(4) Soffit of precast deck form	Reinforcing steel Pretensioning strands	— —	40 ± 10 38 ± 3
	(5) Soffit of slab less than 300 mm thick or soffit of top slab of voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	50 ± 10 — 70* ± 10	45 ± 10 60 ± 5 65* ± 10
	(6) Soffit of slab 300 mm thick or thicker or soffit of structural component, except (4) and (5) above	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 10 — 80* ± 10	50 ± 10 65 ± 5 70* ± 10
	(7) Vertical surface of arch, solid or voided deck, pier cap, T-beam, or interior diaphragm	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 10 — 90* ± 10	60 ± 10 75 ± 5 80* ± 10
	(8) Inside vertical surface of buried structure or inside surface of circular buried structure	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	50 ± 10 65 ± 5 70* ± 10
	(9) Vertical surface of structural component, except (7) and (8) above	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	55 ± 10 70 ± 5 75* ± 10
	(10) Precast T-, I-, or box girder	Reinforcing steel Pretensioning strands Post-tensioning ducts	— — —	35 +10 or –5 50 ± 5 55* ± 10

(Continued)

Table 8.5 (Continued)

Environmental exposure	Component	Reinforcement/ steel ducts	Concrete covers and tolerances: Cast-in-place concrete, mm	Concrete covers and tolerances: Precast concrete, mm
No de-icing chemicals; no spray or surface runoff containing de-icing chemicals; no marine spray	(1) Top of bottom slab for rectangular voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	40 ± 10 — 60* ± 10	40 ± 10 55 ± 5 60* ± 10
	(2) Top surface of buried structure with less than 600 mm fill† or top surface of bottom slab of buried structure	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	40 ± 10 55 ± 5 60* ± 10
	(3) Top surface of structural component, except (1) and (2) above‡	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	50 ± 10 70 ± 5 70 ± 10
	(4) Soffit of precast deck form	Reinforcing steel Pretensioning strands	— —	40 ± 10 38 ± 3
	(5) Soffit of slab less than 300 mm thick or soffit of top slab of voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	40 ± 10 — 60* ± 10	40 ± 10 55 ± 5 60* ± 10
	(6) Soffit of slab 300 mm thick or thicker or soffit of structural component, except (4) and (5) above	Reinforcing steel Pretensioning strands Post-tensioning ducts	50 ± 10 — 70* ± 10	40 ± 10 55 ± 5 60* ± 10
	(7) Vertical surface of arch, solid or voided deck, pier cap, T-beam, or interior diaphragm	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 10 — 80* ± 10	50 ± 10 65 ± 5 70* ± 10
	(8) Inside vertical surface of buried structure or inside surface of circular buried structure	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	40 ± 10 55 ± 5 60* ± 10
	(9) Vertical surface of structural component, except (7) and (8) above	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	50 ± 10 70 ± 5 70* ± 10
	(10) Precast T-, I-, or box girder	Reinforcing steel Pretensioning strands Post-tensioning ducts	— — —	30 +10 or –5 45 ± 5 50* ± 10

(Continued)

Table 8.5 (Concluded)

Environmental exposure	Component	Reinforcement/ steel ducts	Concrete covers and tolerances: Cast-in-place concrete, mm	Concrete covers and tolerances: Precast concrete, mm
Earth or fresh water	(1) Footing, pier, abutment, or retaining wall	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	55 ± 10 75 ± 5 80* ± 10
	(2) Concrete pile	Reinforcing steel Pretensioning strands Post-tensioning ducts	— — —	40 ± 10 55 ± 5 60* ± 10
	(3) Caisson with liner	Reinforcing steel Post-tensioning ducts	60 ± 20 80* ± 15	— —
	(4) Buried structure with more than 600 mm of fill†	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	40 ± 10 55 ± 5 60* ± 10
Swamp, marsh, salt water, or aggressive backfill	(1) Footing, pier, abutment, or retaining wall	Reinforcing steel Pretensioning strands Post-tensioning ducts	80 ± 20 — 100* ± 15	65 ± 10 85 ± 10 90* ± 10
	(2) Concrete pile	Reinforcing steel Pretensioning strands Post-tensioning ducts	— — —	50 ± 10 65 ± 5 70* ± 10
	(3) Caisson with liner	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	— — —
	(4) Buried structure with more than 600 mm of fill†	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	55 ± 10 70 ± 5 80* ± 10
Cast against and permanently exposed to earth	(1) Footing	Reinforcing steel	100 ± 25	—
	(2) Caisson	Reinforcing steel Post-tensioning ducts	100 ± 25 120 ± 15	— —
Various	Components other than those covered elsewhere in this Table	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20§ — 90* ± 15§	55 ± 10§ 70 ± 5§ 80* ± 10§

**Or $0.5d_d$, whichever is greater.*
†Buried structures with less than 600 mm of fill shall have a distribution slab.
‡For concrete decks without waterproofing and paving, increase the concrete cover by 10 mm to allow for wearing of the surface concrete.
§Or as Approved.

8.11.3 Detailing for durability

8.11.3.1 Reinforcement detailing

Reinforcement shall be spaced or grouped to facilitate the placing and compaction of concrete.

8.11.3.2 Confining reinforcement cage
Pretensioned and post-tensioned tendons shall be confined in an outer reinforcement cage, where practical.

8.11.3.3 Debonding of pretensioned strands
Pretensioned strands shall not be debonded at the ends of girders unless the ends are protected by Approved methods.

8.12 Control of cracking

8.12.1 General
The requirements of Clauses 8.12.2 to 8.12.6 shall apply with respect to the distribution of reinforcing bars and tendons to control cracking.

8.12.2 Distribution of reinforcement
Bonded reinforcing bars and, where applicable, tendons, shall be uniformly distributed within the tensile zone as close to the extreme tension fibre as cover and spacing requirements permit. Reinforcing bars shall also be provided at the side faces of beams in accordance with Clause 8.12.4.

8.12.3 Reinforcement

8.12.3.1 Maximum crack width
Crack widths at serviceability limit states shall not exceed the values specified in Table 8.6 for the applicable type of structural component and exposure.

Table 8.6
Maximum crack width
(See Clause 8.12.3.1.)

Type of structural component	Type of exposure	Maximum crack width, mm
Non-prestressed	De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray; swamp; marsh; salt water; aggressive backfill	0.25
	Other environmental exposures	0.35
Prestressed	De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray; swamp; marsh; salt water; aggressive backfill	0.15
	Other environmental exposure	0.20

8.12.3.2 Calculation of crack width
Crack width, w, shall be taken as $k_b \beta_c s_{rm} \varepsilon_{sm}$.

k_b shall be taken as 1.2 for components with epoxy-coated reinforcing steel and 1.0 for all other components.

When cracking is caused by load, β_c shall be taken as 1.7.

When cracking is caused by superimposed deformations, β_c shall be taken as 1.7 for cross-sections with a minimum dimension exceeding of 800 mm and 1.3 for cross-sections with a minimum dimension of

300 mm or less. Linear interpolation may be used to calculate β_c for cross-sections with a minimum dimension between these limits.

s_{rm} shall be calculated as follows (in millimetres):

$$s_{rm} = 50 + 0.25k_c \frac{d_b}{\rho_c}$$

k_c shall be taken as 0.5 for bending and 1.0 for pure tension.

ρ_c is the ratio A_s/A_{ct}, where A_{ct} is the effective tension area of the concrete cross-section and A_s is the area of reinforcement contained within A_{ct}. The depth of A_{ct} shall be taken as the lesser of

(a) 2.5 times the distance from the extreme tensile fibre of the cross-section to the centroid of tensile reinforcement; and

(b) one-third the distance from the neutral axis of the cross-section to the extreme tensile fibre.

ε_{sm} shall be calculated as follows:

$$\varepsilon_{sm} = \frac{f_s}{E_s}\left[1 - \left[\frac{f_w}{f_s}\right]^2\right]$$

where f_s is stress in reinforcement at the serviceability limit state and f_w is stress in reinforcement under the conditions causing initial cracking. Both f_s and f_w shall be calculated on the basis of a cracked section.

8.12.4 Crack control in the side faces of beams

Note: *This Clause does not apply to prestressed components in which the minimum prestress is such that the cracks due to the application of live load remain closed under permanent load effects.*

Where the overall depth of a beam exceeds 750 mm, longitudinal reinforcement with a total area not less than $0.01b_wd$ shall be evenly distributed over both faces of the web over a distance of 70% of the overall depth from the tension face, at a spacing of not more than 200 mm. The value of b_w used to calculate the area of reinforcement need not be greater than 250 mm. This reinforcement may be included in strength calculations if a strain compatibility analysis is conducted to determine the stresses in the individual bars.

8.12.5 Flanges of T-beams

For flanges of T-beams subjected to flexural tension exceeding f_{cr}, the reinforcing bars shall be uniformly distributed over an effective flange width as specified in Clause 5.8.2 or over a flange width equal to 10% of the span, whichever is smaller. If the effective flange width exceeds 10% of the span, additional longitudinal reinforcement shall be provided in the outer portions of the flange.

8.12.6 Shrinkage and temperature reinforcement

Reinforcement for shrinkage and temperature crack control normal to the principal reinforcement shall be provided in structural components where the principal reinforcement extends in one direction only. At all sections where it is required, such reinforcement shall be developed in accordance with Clause 8.15.2.

The minimum area of shrinkage and temperature reinforcement in each face and in each direction shall be 500 mm^2/m and the spacing of the bars shall not exceed 300 mm.

8.13 Deformation

8.13.1 General

Dimensional changes, deflections, and rotations occurring immediately upon the application of loads shall be determined in accordance with elastic methods using the value of E_c at the time of loading and taking into consideration the effects of cracking and reinforcement.

8.13.2 Dimensional changes

Dimensional changes due to loads, temperature, shrinkage, and creep shall be determined using the data specified in Clauses 8.4.1.3, 8.4.1.5, and 8.4.1.6.

8.13.3 Deflections and rotations

8.13.3.1 General

Deflections and rotations shall be calculated in accordance with one of the methods specified in Clauses 8.13.3.2 to 8.13.3.4.

8.13.3.2 Refined method

Determination of deflection and rotation of a member by a refined method shall make allowance for the the following, as applicable:

(a) shrinkage and creep properties of the concrete;
(b) relaxation of prestressing steel;
(c) expected load history; and
(d) effects of cracking and tension stiffening.

8.13.3.3 Simplified method

Deflections and rotations may be calculated using the effective moment of inertia, I_e, as follows:

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

For prestressed concrete, the value of M_{cr}/M_a to be used in calculating deflections and rotations due to live load shall be taken as

$$\frac{M_{cr}}{M_a} = 1 - \frac{\left(f_{tl} - f_{cr}\right)}{f_l}$$

For continuous spans, the effective moment of inertia may be taken as the average for the critical positive and negative moment sections.

For prismatic members, the effective moment of inertia may be taken as the value at midspan for simple spans and at the support for cantilevers.

8.13.3.4 Total deflection and rotation

In lieu of a more refined analysis, the sum of the total instantaneous and long-term deflection and rotation for flexural non-prestressed components may be obtained by multiplying, respectively, the instantaneous deflection and rotation caused by the sustained load by the factor

$$\left[1 + \frac{S}{1 + 50\rho'}\right]$$

where ρ' shall be taken as the value at midspan for simple and continuous spans and at the support for cantilevers. The factor S for duration of sustained loads shall be taken as follows:

(a) three months: 1;
(b) six months: 1.2;
(c) 12 months: 1.4; and
(d) five years or more: 2.

If necessary, linear interpolation may be used for durations of less than five years.

In lieu of a more refined analysis, the long-term deflection and rotation of flexural prestressed components may be estimated by multiplying, respectively, the instantaneous deflection and rotation due to loads and prestress by appropriate factors. The total deflection and rotation may be estimated by adding the instantaneous and the long-term deflection and rotation, respectively.

8.14 Details of reinforcement and special detailing requirements

8.14.1 Hooks and bends

8.14.1.1 Standard hooks

The standard hooks specified in Clauses 8.14.1.2 and 8.15.5 shall consist of
(a) a semi-circular 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 twelve bar diameters at the free end of the bar; or
(c) for stirrup and tie anchorage only, either a 90° or a 135° bend plus an extension of at least six bar diameters at the free end of the bar.

8.14.1.2 Minimum bend diameter

The diameter of a bend measured on the inside of a bar for standard hooks, except for stirrup and tie hooks, shall not be less than the applicable value specified in Table 8.7.

Table 8.7
Minimum bend diameter, mm
(See Clause 8.14.1.2.)

	Type of reinforcement			
Bar	300R	400R or 500R	400W or 500W	Epoxy coated
10M	60	70	60	80
15M	90	100	90	120
20M	—	120	100	160
25M	—	150	150	200
30M	—	250	200	240
35M	—	300	250	350
45M	—	450	400	450
55M	—	600	550	550

8.14.1.3 Stirrups and tie hooks

The inside diameter of bends and 90° and 135° hooks for stirrups and ties shall not be less than four bar diameters for uncoated bars and eight bar diameters for epoxy-coated bars.

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

8.14.2 Spacing of reinforcement

8.14.2.1 Reinforcing bars

8.14.2.1.1

For cast-in-place concrete, the clear distance between parallel bars in a layer or a ring shall not be less than
(a) 1.5 times the nominal diameter of the bars;
(b) 1.5 times the maximum size of the coarse aggregate; and
(c) 40 mm.

8.14.2.1.2

For precast concrete, the clear distance between parallel bars in a layer or a ring shall not be less than
(a) the nominal diameter of the bars;
(b) 1.33 times the maximum size of the coarse aggregate; and
(c) 25 mm.

8.14.2.1.3

For parallel reinforcing bars placed in two or more layers, with a clear distance between layers of not more than 150 mm, the bars in the upper layers shall be placed directly above those in the lower layers (except in deck slabs). The clear distance between layers shall not be less than
(a) 25 mm; and
(b) the nominal diameter of the bars.

8.14.2.1.4

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

8.14.2.1.5

In walls and slabs, primary flexural reinforcement shall be spaced not farther apart than
(a) 1.5 times the thickness of the component; and
(b) 450 mm.

The maximum spacing of hoops, spirals, ties and shrinkage, and temperature reinforcement shall satisfy Clauses 8.12.6, 8.14.3, and 8.14.4.

8.14.2.2 Tendons

8.14.2.2.1 Pretensioning

The centre-to-centre spacing between pretensioning strands at the ends of the members shall not be less than 50 mm.

Pretensioning strands may be bundled, provided that a minimum of 50 mm spacing is maintained at the end of the member. Groups of up to eight strands may be bundled to touch one another in a vertical plane. The number of strands bundled in any other manner shall not exceed four.

The clear distance between groups of bundled strands shall not be less than 1.33 times the maximum size of the aggregate or 25 mm, whichever is greater.

8.14.2.2.2 Post-Tensioning

The clear distance between post-tensioning ducts shall not be less than 40 mm.

For groups of ducts in the same horizontal plane, the clear horizontal distance between each group shall not be less than 100 mm. A group shall contain not more than three ducts.

For groups of ducts in two or more horizontal planes, the clear horizontal distance between adjacent groups shall not be less than 100 mm. A group shall contain not more than two ducts in the same horizontal plane.

For precast or cast-in-place segmental construction, the clear horizontal distance between groups of ducts may be reduced to 75 mm.

8.14.3 Transverse reinforcement for flexural components

Where compression reinforcement for flexural components is required by analysis, the reinforcement shall be confined by closed stirrups. The stirrups shall be at least 10M when the longitudinal bars are 30M or smaller and at least 15M when the longitudinal bars are larger than 30M. Welded wire fabric of equivalent area may be used for closed stirrups. The spacing of the stirrups shall not exceed

(a) 16 times the diameter of the longitudinal bar;
(b) the least dimension of the component; and
(c) 300 mm.

For hollow rectangular components that meet the requirements of Clause 8.8.5.8, the spacing of the stirrups shall also not exceed 1.25 times the wall thickness. For specified concrete strength exceeding 60 MPa, the spacing of the stirrups shall be reduced by 25%. None of the longitudinal bars shall be farther than 150 mm from the leg of a confining stirrup.

Closed stirrups may be formed in one piece by overlapping the hooks of standard stirrups around a longitudinal bar, or formed in one or two pieces lap-spliced with a minimum lap of $1.3\ell_d$.

8.14.4 Transverse reinforcement for compression components

8.14.4.1 General

The longitudinal reinforcement for wall-type compression components need not be enclosed by lateral ties if the reinforcement area is not greater than 0.01 times the gross concrete area or when analysis shows that longitudinal reinforcement is not required as compression reinforcement.

8.14.4.2 Spirals

Spiral reinforcement for compression components shall consist of evenly spaced continuous spirals held firmly in place by attachment to the longitudinal reinforcement and by spacers.

The spirals shall be of a size that permits handling and placing without distortion from the specified dimensions.

Anchorage of spiral reinforcement shall be provided by one and one-half extra turns of spiral bar at each end of the spiral unit embedded in the footing and the component supported above the footing, or by a 90° bend around a longitudinal reinforcing bar plus an extension of at least 24 bar diameters into the core.

Splices in spiral bars shall be provided by one of the following means:

(a) complete joint penetration groove welds meeting the requirements of CSA W186;
(b) mechanical connections meeting the requirements of Clause 8.4.4.4;
(c) ends of spiral bars anchored around a longitudinal reinforcing bar with extensions of at least 24 bar diameters into the core; or
(d) an Approved method.

Spiral reinforcement shall extend over the full length of the compression component. The maximum centre-to-centre spacing shall not exceed six times the diameter of the longitudinal bars or 150 mm, whichever is less. The clear spacing shall not be less than 25 mm and not less than 1.33 times the maximum size of the coarse aggregate.

The ratio of spiral reinforcement, ρ_s, shall not be less than the following value:

$$\rho_s = 0.45\left[\frac{A_g}{A_c} - 1\right]\frac{f_c'}{f_y}\left[0.5 + 1.25\frac{P_f}{\phi_c f_c' A_g}\right]$$

8.14.4.3 Ties

In tied compression components, all bars shall be enclosed by ties. The size and spacing of these ties shall meet the requirements for stirrups in Clause 8.14.3, except that the spacing may be increased for compression components that have a larger cross-section than required by the conditions of loading, in which case the maximum spacing shall not exceed 450 mm. Welded wire fabric of equivalent area may be used for ties.

Ties shall be arranged so that every corner bar and alternate longitudinal bar has 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. Ties shall be located vertically not more than half a tie spacing above the footing or from other support, and not more than half a tie spacing below the lowest horizontal reinforcement in the components supported above.

8.14.5 Reinforcement for shear and torsion

8.14.5.1 Transverse reinforcement

Transverse reinforcement shall consist of one of the following forms:

(a) stirrups perpendicular to the axis of the component or at an angle of 45° or more to the longitudinal tension reinforcement, with the inclined stirrups oriented to intercept potential cracks;

(b) well-anchored tendons that are detailed and constructed to minimize seating and time-dependent losses and are perpendicular to the axis of the component or at an angle of 45° or more to the longitudinal tension reinforcement, with the inclined tendons oriented to intercept potential diagonal cracks;

(c) spirals; or

(d) welded wire fabric, with the wires perpendicular to the axis of the component.

Transverse reinforcement for shear shall be anchored in accordance with Clause 8.15.1.5.

8.14.5.2 Torsional reinforcement

Torsional reinforcement shall consist of longitudinal reinforcement and one of the following forms of transverse reinforcement:

(a) closed stirrups perpendicular to the axis of the component and anchored with 135° hooks;

(b) a closed cage of welded wire fabric perpendicular to the axis of the component; or

(c) spirals.

8.14.6 Maximum spacing of reinforcement for shear and torsion

If V_f is less than or equal to $(0.10\phi_c f_c' b_v d_v + V_p)$ and T_f is less than or equal to $0.25T_{cr}$, the spacing of the transverse reinforcement, s, measured in the longitudinal direction, shall not exceed the lesser of 600 mm or $0.75d_v$.

If V_f exceeds $(0.10\phi_c f_c' b_v d_v + V_p)$, or if T_f exceeds $0.25T_{cr}$, s shall not exceed the lesser of 300 mm or $0.33d_v$.

The spacing of the transverse reinforcement, s, measured in the longitudinal direction shall not exceed the lesser of

(a) 600 mm or $0.75d_v$ if the nominal shear stress is less than $0.10\phi_c f_c'$; and

(b) 300 mm or $0.33d_v$ if the nominal shear stress equals or exceeds $0.10\phi_c f_c'$.

The spacing of longitudinal bars for torsion distributed around the perimeter of the stirrups shall not exceed 300 mm. At least one longitudinal bar with a diameter not less than 0.06 times the spacing of the stirrups and not smaller than 15M shall be placed inside each corner of the closed stirrups. The corner bars shall be anchored in accordance with Clause 8.15.2 or 8.15.5.

8.15 Development and splices

8.15.1 Development

8.15.1.1 General

The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by one or more of embedment length, end anchorage, and a hook or mechanical device. Hooks or mechanical devices may be used in developing the strength of the bars in tension only.

Tension reinforcement may be anchored by extending it into the compression zone or bending it and making it continuous with the reinforcement on the opposite face of the member.

Reinforcement shall extend beyond the point at which it is theoretically no longer required to resist flexure in accordance with the requirements of Clause 8.9.3.10.

The value of $\sqrt{f_c'}$ in Clause 8.4.1.8 used to compute f_{cr} in Clauses 8.15.2.2, 8.15.2.3, 8.15.3.1, 8.15.5.2, and 8.15.7.2 shall not exceed 8.0.

8.15.1.2 Positive moment reinforcement

At least 33% of the positive moment reinforcement in simply supported members and 25% of the positive moment reinforcement in continuous members shall extend along the same face of the member into the support. Such reinforcement shall extend at least 150 mm beyond the centreline of the exterior support and shall satisfy the requirements of Clause 8.9.3.10.

When a flexural member is part of the lateral-load-resisting system, the positive moment reinforcement required to be extended into the support shall be anchored so as to develop the yield strength in tension at the face of the support.

8.15.1.3 Negative moment reinforcement

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

At least 33% of the total reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflection not less than the effective depth of the member, $12d_b$, or 0.06 of the clear span, whichever is greatest.

8.15.1.4 Special members

Adequate end anchorage shall be provided for tension reinforcement in flexural members where stress in the reinforcement is not directly proportional to moment. Such members include, but are not limited to, sloped, stepped, or tapered footings, brackets, deep beams, and members in which the tension reinforcement is not parallel to the compression face.

8.15.1.5 Anchorage of transverse reinforcement

Transverse reinforcement provided for shear shall extend as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit.

Transverse reinforcement provided for shear shall be anchored at both ends by one of the following:

(a) For 15M and smaller bars and MD200 and smaller wire, a standard hook, as specified in Clause 8.14.1.1, around longitudinal reinforcement.

(b) For 20M and 25M stirrups, a standard hook, as specified in Clause 8.14.1.1, 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 single U-stirrups,
   (i) two longitudinal wires running at a 50 mm spacing along the member at the top of the U; or
   (ii) one longitudinal wire located not more than $0.25d$ from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first. 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 $0.25d$ 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.

(e) A mechanical anchor capable of developing the yield strength of the bar.

Pairs of U-stirrups or ties placed so as to form a closed unit shall be considered properly spliced when lapped for a length of $1.3\ell_d$. In components with a depth of at least 450 mm, such splices having A_bf_y not more than 40 kN per leg may be considered adequate if the stirrup legs extend the full available depth of the component.

Between the anchored ends, each bend in the continuous portion of a transverse single U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

8.15.2 Development of reinforcing bars and deformed wire in tension

8.15.2.1 General

The development length, ℓ_d, of reinforcing bars and deformed wire in tension shall be determined from Clause 8.15.2.2 or 8.15.2.3, but shall not be less than 300 mm, except as specified in Clause 8.15.2.5.

8.15.2.2 Development length

The development length, ℓ_d, of reinforcing bars and deformed wire in tension shall be calculated as follows:

$$\ell_d = 0.45 \frac{k_1 k_2 k_3}{(d_{cs} + K_{tr})} \left[\frac{f_y}{f_{cr}}\right] A_b$$

where

K_{tr} = $0.45 \dfrac{A_{tr} f_y}{10.5 sn}$

where

s = maximum centre-to-centre spacing of transverse reinforcement within a distance ℓ_d and the factor 10.5 is expressed in millimetres per newton

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

8.15.2.3 Simplified development length

The development length, ℓ_d, of reinforcing bars and deformed wire in tension may be taken from Table 8.8 if the clear cover and clear spacing of the bars being developed are at least d_b and $1.4d_b$, respectively.

Table 8.8
Minimum development length of reinforcing bars and deformed wire in tension
(See Clause 8.15.2.3.)

Cases	Minimum development length, ℓ_d
Components containing minimum stirrups or ties (Clause 8.9.1.3 or 8.14.4.3) within ℓ_d or slabs and walls with a clear spacing of not less than $2d_b$ between bars being developed	$0.18 k_1 k_2 k_3 \frac{f_y}{f_{cr}} d_b$
Other cases	$0.24 k_1 k_2 k_3 \frac{f_y}{f_{cr}} d_b$

8.15.2.4 Modification factors

The following modification factors shall be used in calculating the development length specified in Clauses 8.15.2.2 and 8.15.2.3:

(a) Bar location factor, k_1:
  (i) 1.3 for horizontal reinforcement placed so that more than 300 mm of fresh concrete is cast in the component below the development length or splice; and
  (ii) 1.0 for other cases.

(b) Coating factor, k_2:
  (i) 1.5 for epoxy-coated reinforcement with a clear cover less than $3d_b$ or a clear spacing between bars being developed less than $6d_b$;
  (ii) 1.2 for all other epoxy-coated reinforcement; and
  (iii) 1.0 for uncoated reinforcement.

(c) Bar size factor, k_3:
  (i) 0.8 for 20M and smaller bars and deformed wires; and
  (ii) 1.0 for 25M and larger bars.

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

8.15.2.5 Modification factors for 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 proportioned in accordance with Clause 8.17.

8.15.3 Development of reinforcing bars in compression

8.15.3.1

The development length, ℓ_d, for reinforcing bars in compression shall be calculated as follows:

$$\ell_d = \frac{0.10 f_y d_b}{f_{cr}}$$

but shall not be less than $0.044 f_y d_b$ and not less than 200 mm. The units of the constant 0.044 shall be taken as the reciprocal of MPa.

8.15.3.2

The development length, ℓ_d, may be multiplied by one or both of the applicable modification factors specified in Table 8.9. The cumulative value shall be not less than 0.6.

Table 8.9
Modification factors for development length
(See Clause 8.15.3.2.)

Condition	Modification factor
Reinforcement exceeding that required by analysis	(A_s required)/(A_s provided)
Reinforcement enclosed within spirals at least 6 mm in diameter and with a pitch of not more than 100 mm, or within 10M ties in accordance with Clause 8.14.4.3 and spaced not more than 100 mm on centre	0.75

8.15.4 Development of pretensioning strand

Pretensioning strand shall be bonded beyond the critical section for a development length of not less than ℓ_d, calculated as follows:

$$\ell_d = 1.5\frac{f_{si}}{f'_{ci}}d_b - 117 + 0.18\left(f_{ps} - f_{se}\right)d_b$$

Where bonding of the strand does not extend to the ends of the component and tension occurs at the serviceability limit state within the development length, ℓ_d, a development length of $2\ell_d$ shall be used. The number of strands where the bonding does not extend to the ends of the member shall not exceed 25% of the total number of strands.

8.15.5 Development of standard hooks in tension

8.15.5.1 General

The development length, ℓ_{dh}, for reinforcing bars in tension terminating in a standard hook shall be calculated as the product of the basic development length, ℓ_{hb}, specified in Clause 8.15.5.2 and the applicable modification factor or factors specified in Table 8.10. The development length ℓ_{dh} shall be not less than $8d_b$ or 150 mm, whichever is greater.

8.15.5.2 Basic development length

The basic development length for a hooked bar, ℓ_{hb}, shall be calculated as $40d_b/f_{cr}$.

8.15.5.3 Factors modifying hook development length

The basic development length, ℓ_{hb}, shall be multiplied by the applicable modification factor or factors specified in Table 8.10.

Table 8.10
Modification factors for hook development length
(See Clauses 8.15.5.1 and 8.15.5.3.)

Condition	Modification factor
Bars with f_y other than 400 MPa	$f_y/400$
35M or smaller bars where the side cover normal to plane of the hook is greater than 60 mm; 90° hooks where the cover on the bar extension beyond the hook is greater than 50 mm	0.7
35M or smaller bars where the hook is confined by at least three ties or stirrups with a spacing not greater than $3d_b$ along a length at least equal to the inside diameter of the hook, where d_b is the diameter of the hooked bar	0.8
Reinforcement exceeding that required by analysis, provided that anchorage or development to attain f_y is not specifically required	(A_s required)/(A_s provided)
Epoxy-coated reinforcement	1.2

For bars being developed by a standard hook at the ends of components where both the side cover and the top or bottom cover over the hook are less than 60 mm, the hook shall be enclosed within at least three ties or stirrups with a spacing of not greater than $3d_b$ along a length at least equal to the inside diameter of the hook, where d_b is the diameter of the hooked bar. For this case, the factor of 0.8 in Table 8.10 shall not apply.

8.15.6 Combination development length

The development length, ℓ_d, may consist of a combination of the equivalent embedment length of a hook or mechanical anchorage plus the additional embedment length of the reinforcement measured from the point of tangency of the hook.

8.15.7 Development of welded wire fabric in tension

8.15.7.1 Deformed wire fabric

The development length, ℓ_d, of welded deformed wire fabric measured from the point of critical section to the end of the wire shall be calculated as the product of the development length specified in Clause 8.15.2.2 or 8.15.2.3 and the applicable wire fabric factor specified in this Clause, but ℓ_d shall not be less than 200 mm except for lap splices, which shall be in accordance with Clause 8.15.9.5.

For welded deformed wire fabric with at least one cross-wire within the development length not less than 50 mm from the point of critical section, the wire fabric factor shall be the greater of $(f_y - 240)/f_y$ and $5d_b/s_w$, but need not be taken greater than 1.0.

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

8.15.7.2 Smooth wire fabric

The yield strength of welded smooth wire fabric shall be considered developed by embedment of two cross-wires, with the closer cross-wire not less than 50 mm from the point of critical section.

However, the development length, ℓ_d, measured from the point of critical section to the outermost cross-wire shall not be less than $1.30A_w f_y / s_w f_{cr}$, modified by the ratio for reinforcement exceeding that required by analysis, but shall not be taken less than 150 mm, except for the calculation of lap splices in accordance with Clause 8.15.9.6.

8.15.8 Mechanical anchorages

Reinforcement may be developed by a mechanical anchorage device of the type specified in Clause 8.4.4.2.

8.15.9 Splicing of reinforcement

8.15.9.1 Lap splices

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

Bars spliced by non-contact lap splices in flexural members shall not be spaced transversely farther apart than

(a) 0.20 times the required lap splice length; and
(b) more than 150 mm.

8.15.9.2 Welded splices

A 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 mean yield strength representative of the bars to be used in the test of the welded splice.

8.15.9.3 Splices of deformed bars and deformed wire in tension

Lap splices of deformed bars and deformed wire in tension shall be classified as Class A or Class B in accordance with Table 8.11. The minimum length of lap shall be $1.0\ell_d$ for Class A splices and $1.3\ell_d$ for Class B splices, but not less than 300 mm. In this regard, the development length, ℓ_d, shall be calculated in accordance with Clause 8.15.2.1, but without the modification factors for excess reinforcement specified in Clause 8.15.2.5.

Table 8.11
Classification of lap splices in tension
(See Clause 8.15.9.3.)

	Maximum percentage of A_s spliced within required splice length	
(A_s provided)/(A_s required)	**50**	**100**
≥ 2	Class A	Class B
< 2	Class B	Class B

Splices in components subjected to axial tension shall be staggered by at least 800 mm and shall be welded or made by means of a mechanical connection.

8.15.9.4 Splices of deformed bars in compression

The minimum length of lap for compression lap splices shall be $(0.133f_y - 24)d_b$ but not less than 300 mm. In tied reinforced compression members where ties throughout the lap splice length have an effective area not less than $0.0015hs$, the minimum lap splice length may be taken as $0.83(0.133f_y - 24)d_b$, but the lap length shall not be less than 300 mm. Tie legs perpendicular to the dimension h shall be used in determining the effective area.

In spirally reinforced compression members, the minimum lap splice length of bars within a spiral may be taken as $0.75(0.133f_y - 24)d_b$, but the lap length shall not be less than 300 mm. Welded splices and mechanical connections used in compression shall meet the requirements of Clauses 8.4.4.4 and 8.15.9.2.

8.15.9.5 Splices of welded deformed wire fabric in tension

The minimum length of lap for lap splices of welded deformed wire fabric, measured between the ends of each fabric sheet, shall be the greater of $1.3\ell_d$ and 200 mm, where ℓ_d is the development length in accordance with Clause 8.15.7.1. The overlap measured between outermost cross-wires of each fabric sheet shall not be less than 50 mm.

The minimum length of lap for lap splices of welded deformed wire fabric with no cross-wires within the lap splice length shall be determined as for deformed wire.

8.15.9.6 Splices of welded smooth wire fabric in tension

The minimum length of lap for lap splices of welded smooth wire fabric shall be as follows:

(a) When the area of reinforcement provided is less than twice that required by analysis at the splice location, the length of the overlap measured between the outermost cross-wires of each fabric sheet shall not be less than
   (i) one spacing of cross-wires plus 50 mm;
   (ii) $1.5\ell_d$; and
   (iii) 150 mm.

(b) When the area of reinforcement provided is at least twice that required by analysis at the splice location, the length of overlap measured between the outermost cross-wires of each fabric sheet shall not be less than
   (i) $1.5\ell_d$; and
   (ii) 50 mm.

The development length, ℓ_d, shall be in accordance with Clause 8.15.7.2.

8.15.9.7 Special requirements for columns

Where the bar stress due to factored loads is compressive, lap splices shall comply with Clause 8.15.9.4.

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

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

Where welded splices or mechanical connections 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 at least 750 mm apart.

8.16 Anchorage zone reinforcement

8.16.1 General

Anchorage zones shall be reinforced to resist tensile, bursting, and spalling forces induced by concentrated loads due to prestressing or load effects from attachments. Design forces due to prestressing shall be taken as the specified strength of the tendons.

8.16.2 Post-tensioning anchorage zones

8.16.2.1 General

For design purposes, the anchorage zone shall be considered to comprise two regions: the general zone, to which the requirements of Clause 8.16.2.2 apply, and the local zone, to which the requirements of Clause 8.16.2.3 apply.

For anchorage zones at the end of a component or segment, the transverse dimensions may be taken as the depth or width of the section, but not larger than the longitudinal dimension of the component or segment. The longitudinal extent of the anchorage zone in the direction of the tendon shall not be taken as

(a) less than the greater of the transverse dimensions of the anchorage zone; or
(b) more than 1.5 times the greater of the transverse dimensions of the anchorage zone.

For intermediate anchorages, the anchorage zone shall be considered to extend in the direction of the tendon for a distance not less than the greater of the transverse dimensions of the anchorage zone.

8.16.2.2 General zone

8.16.2.2.1 General

The dimensions of the general zone shall be taken as identical to those of the overall anchorage zone specified in Clause 8.16.2.1. Overall details for the general zones, including the location of the tendons and anchorage devices, general zone reinforcement, and the stressing sequence, shall be shown on the Plans.

8.16.2.2.2 Design methods

One of the following methods may be used in designing the anchorage zone reinforcement:

(a) the strut-and-tie model;
(b) an elastic stress analysis; and
(c) the approximate method, where applicable.

The effect of the stressing sequence and of the three-dimensional effects due to concentrated jacking forces shall be considered.

8.16.2.2.3 Design principles

8.16.2.2.3.1 Compressive stresses
The compressive stresses in the concrete behind the anchorage device shall be investigated at a distance measured from the concrete bearing surface. This distance shall be not less than the depth to the end of the local confinement reinforcement and not less than the smaller lateral dimension of the anchorage device.

The compressive stress shall not exceed $0.75\phi_c f'_{ci}$ except in areas that could be extensively cracked at the ultimate limit state or where large inelastic rotations are expected, in which case the compressive stress shall be limited to $0.65\phi_c f'_{ci}$.

8.16.2.2.3.2 Bursting
Resistance to bursting forces shall be provided by non-prestressed or prestressed reinforcement in the form of spirals, closed ties, or anchored transverse ties and shall meet the following requirements:
(a) reinforcement shall extend over the full width of the component and be anchored as close to the outer faces of the component as cover requirements permit;
(b) reinforcement shall be distributed behind the loaded surface along both sides of the tendon for a distance that is the lesser of $2.5d_{bs}$ and 1.5 times the corresponding lateral dimension of the section;
(c) the centroid of the bursting reinforcement shall be at a distance d_{bs} from the loaded surface; and
(d) spacing of reinforcement shall not exceed the lesser of 24 bar diameters and 300 mm.

8.16.2.2.3.3 Spalling and longitudinal edge tension
The spalling force shall not be taken as less than 2% of the prestressing force.

Resistance to spalling forces shall be provided by non-prestressed or prestressed reinforcement located close to the longitudinal and transverse edges of the concrete and shall meet the following requirements:
(a) spalling reinforcement shall extend over the full available width and depth of the component;
(b) spalling reinforcement between multiple anchorage devices shall tie the anchorage devices together; and
(c) longitudinal edge tension reinforcement and spalling reinforcement for eccentric anchorage devices shall be continuous. The reinforcement shall be extended along the tension face over the full length of the anchorage zone and along the loaded face from the longitudinal edge to the other side of the eccentric anchorage device or group of anchorage devices.

For multiple anchorages with centre-to-centre spacing of more than 0.4 times the depth of the section, the spalling force shall be determined by analysis.

8.16.2.2.4 Application of the strut-and-tie model to the design of anchorage zones
The flow of forces in the anchorage zone may be approximated by an appropriate strut-and-tie model in accordance with Clause 8.10.

All forces acting on the anchorage zone shall be considered in the selection of the strut-and-tie model, which shall follow a load path from the anchorages to the end of the anchorage zone.

8.16.2.2.5 Elastic stress analysis
Analyses based on elastic material properties, equilibrium, and compatibility of strains may be used for design of anchorage zones.

When the compressive stresses in the concrete behind the anchorage device are determined from a linear elastic stress analysis, local stresses shall be averaged over an area equal to the bearing area of the anchorage device.

8.16.2.2.6 Approximate method

The concrete compressive stresses behind the anchorage device, location and magnitude of the bursting force, and edge tension forces may be estimated in accordance with this Clause if

(a) the component has a rectangular cross-section and its length is not less than the largest transverse dimension of the cross-section;

(b) the component has no discontinuities within or behind the anchorage zone;

(c) the minimum edge distance between the anchor centreline and the free edge of the concrete is not less than 1.5 times the corresponding lateral dimension of the anchorage device;

(d) only one anchorage device or one group of closely spaced anchorage devices is located in the anchorage zone; and

(e) the angle of inclination, α, of the tendon is between –5° and +20°.

The concrete compressive stress, f_{ca}, behind the anchorage device may be calculated as follows:

$$f_{ca} = \kappa \frac{0.6F_{pu}}{ab\left[1 + a\left[\frac{1}{b} - \frac{1}{t}\right]\right]}$$

where

t = smaller transverse dimension of the cross-section of the component

If $a \leq s < 2a$, then

$$\kappa = 1 + \left[2 - \frac{s}{a}\right]\left[0.3 + \frac{n}{15}\right]$$

If $s \geq 2a$, then $\kappa = 1.0$.

If a group of anchorages are closely spaced in two directions, the product of the correction factors, κ, for each direction shall be used.

The bursting forces, T_{bs}, may be calculated as follows:

$$T_{bs} = 0.25\Sigma F_{pu}\left[1 - \frac{a}{h}\right] + 0.5\left(\Sigma F_{pu} \sin\alpha\right)$$

The distance of the centroid of the bursting force, d_{bs}, from the loaded surface may be calculated as follows:

$$d_{bs} = 0.5(h - 2e) + 0.5e\sin\alpha$$

where

e = eccentricity of the anchorage device or group of devices with respect to the centroid of the section, always taken positive

The longitudinal edge tension force may be determined from an analysis of a section located at one-half the depth of the section away from the loaded surface (taken as a beam subject to combined flexure and axial load). The spalling force may be taken as equal to the longitudinal edge tension force, but not less than that specified in Clause 8.16.2.2.3.3.

8.16.2.3 Local zone

8.16.2.3.1 Dimensions of the local zone

The local zone shall be the region surrounding each anchorage device. It may be taken as a prism with transverse dimensions equal to the greater of

(a) the relevant plan dimensions of the anchorage device plus twice the distance from the edge of the device to the edge of the concrete (as recommended by the supplier of the anchorage device); and

(b) the outer dimension of any required confining reinforcement plus the concrete covers.

The local zone shall extend along the tendon axis for a distance equal to the largest of the transverse dimensions of the prism.

8.16.2.3.2 Design

The design of the local zone shall be meet the requirements of Clause 8.5.4 and be based on the results of Approved acceptance tests. The dimensions of the anchorage device and the reinforcement in the local zone supplementary to the reinforcement in the general zone shall be determined by the supplier of the anchorage device. This responsibility of the supplier shall be specified on the Plans.

8.16.3 Pretensioning anchorage zones

8.16.3.1 End blocks

End blocks shall not be required where all tendons are pretensioned strands.

8.16.3.2 Reinforcement

Stirrups with an area of at least $0.08F_{pu}/\phi_s f_y$ shall be distributed uniformly over a distance equal to $0.25h$ from the end of the component. The end stirrup shall be placed as close to the end of the component as cover requirements permit.

For the distance h from the end of beams (other than box beams), reinforcement shall be provided to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, not be less than 10M deformed bars, and have a spacing not exceeding 150 mm.

For box beams, transverse reinforcement shall be provided and anchored into the webs of the girder.

8.16.4 Inclined anchorages

When the anchorage force is inclined to the axis of the component, reinforcement or post-tensioning shall be provided to resist the component of the force perpendicular to the axis of the component.

8.16.5 Intermediate anchorages

For intermediate anchors located within the normal thickness of the section, consideration shall be given to the tensile stresses developed ahead of the anchor. The force effects may be determined using the strut-and-tie model of Clause 8.10. Alternatively, a minimum amount of reinforcement may be provided to resist a tension of 50% of the specified strength of the anchored tendon, or a minimum steel area of 0.6% of the concrete area, whichever is greater.

8.16.6 Anchorage blisters

Anchorage blisters shall be proportioned for shear and flexure between the anchorage blisters and the web or flange interface in accordance with Clause 8.9.5 or 8.10. The shear transfer reinforcement shall be distributed linearly from a maximum at the anchor to a minimum at the point of tangency with the web or flange.

Local flexure induced into the web or the flange by the anchorage blister shall be investigated and provided for by reinforcement. Anchorage blisters projecting from one surface only shall be restricted to a size consistent with the web or flange thickness.

Where possible, anchorage blisters shall be located near the flange-web interface.

The minimum amount of reinforcement provided shall be sufficient to transfer at least 25% of the tendon force into the concrete ahead of the anchorage blister. When calculations indicate tensile stresses ahead of the anchorage blister, the reinforcement shall be sufficient to resist at least 50% of the specified strength of the tendon.

8.16.7 Anchorage of attachments

Note: *This Clause specifies requirements for the design of anchors for attaching appurtenances or transferring loads.*

8.16.7.1 General

Anchors shall be proportioned to transfer load effects, including effects of eccentricities and deformations.

The anchorage system shall be proportioned in such a manner that yielding of a steel portion of the system will take place before failure of the concrete unless it can be shown that the factored resistance of the anchor is at least 1.5 times the applied factored load.

Cast-in-place anchors shall be proportioned in accordance with Clauses 8.16.7.2, 8.16.7.3, and 8.16.7.6. Grouted and adhesive anchors shall be proportioned in accordance with Clauses 8.16.7.2, 8.16.7.3, 8.16.7.6, and 8.16.7.7.

8.16.7.2 Transfer of tensile load from anchor to concrete

8.16.7.2.1

Load transfer from the anchor to the concrete shall be achieved by one of the following:

(a) an anchor head at the base of the device transmitting the tensile force;

(b) a deformed reinforcing bar with a hook or development length in accordance with Clause 8.14 or 8.15; or

(c) an Approved method.

8.16.7.2.2

For a device with an anchor head, the factored pull-out resistance of the concrete shall be based on a uniform factored tensile strength of $0.75\phi_c f_{cr}$ acting on an effective stress area.

The effective stress area shall be taken as the projected base area of the prismoids radiating toward the concrete surface from the bearing edge of each anchor head at an angle of inclination of 45°. The total effective stress area shall be limited by the overlapping base areas of the prismoids, the intersection of the prismoids with the concrete surfaces, the area of the anchor heads, and the overall thickness of the concrete.

The edge distance of the anchors shall not be less than 100 mm or $0.33d_a(f_{su}/f_{cr})^{0.5}$, whichever is greater.

8.16.7.2.3

Clause 8.8.7 shall apply to the bearing resistance of concrete at an anchor head or shear lug, except for anchor heads in which the following requirements are satisfied:

(a) the minimum gross area of the anchor head is at least 2.5 times the cross-sectional area of the component transmitting tensile force;

(b) t_a is at least equal to t_d;

(c) the bearing area of the anchor head is approximately evenly distributed around the perimeter of the component transmitting tensile force; and

(d) the specified yield strength of the component transmitting tensile force does not exceed 420 MPa.

8.16.7.2.4

If the factored tensile resistance of the concrete is less than the factored tensile force, reinforcement capable of resisting the factored tensile force shall be provided across the potential failure surface.

8.16.7.3 Transfer of shear load from anchor to concrete

Transfer of shear from the anchor to the concrete may be by bearing or by shear friction.

When shear is transmitted by bearing of anchors or shear lugs on the concrete, the factored bearing resistance of the concrete, B_r, per anchor or lug, shall be $1.4\phi_c f'_c A_{br}$.

For an anchor bolt or headed stud, the bearing area, A_{br}, shall be assumed to have a width equal to the diameter d_a and a depth equal to one-quarter of the embedment depth, but not more than $5d_a$.

For a shear lug, the bearing area, A_{br}, shall be taken as the projected area of the shear lug perpendicular to the direction of the applied shear force.

The embedment depth of the anchor device shall be adequate to develop a factored tensile resistance at least equal to the factored shear force being transferred, except that for anchors transferring load by shear friction, the factored tensile resistance shall not be less than V_f/μ.

When the shear is transmitted by shear friction, shear lugs shall be considered effective only if they are perpendicular to the shear force and are located in a zone where compression is developed between the attachment and the concrete.

When the shear acts toward a free edge, the factored tensile resistance of concrete shall be based on a uniform tensile stress of $0.75\phi_c f_{cr}$ acting on the effective area as follows:

(a) for anchor bolts, studs, and bars, the effective stress area shall be that defined by the intersection of half of the 90° pyramid and the free edge;

(b) for shear lugs, the effective stress area shall be defined by the intersection of projected 45° planes from the edges of the shear lug and the free edge. The bearing area of the shear lug shall be excluded from the effective stress area; and

(c) where shear friction is employed, the effective stress area shall be defined by the intersection of the projected 45° planes and the free edge.

If the factored shear resistance of concrete is less than the factored shear force, reinforcement capable of resisting the factored shear force shall be provided across the potential failure surface.

8.16.7.4 Reinforcement

The reinforcement shall be proportioned for the factored force to be transferred by the anchor and shall be detailed to develop the required force on both sides of the potential failure surface.

8.16.7.5 Compressive resistance of concrete

The compressive resistance of concrete shall be determined in accordance with Clause 8.8.7.1. When compression exists over the entire base plate area, the bearing pressure on the concrete may be assumed to be uniform over an area equal to the width of the base plate multiplied by a length equal to the length of the base plate minus twice the eccentricity of the factored load normal to the base plate.

The moment resisted by the anchors shall be taken as the couple formed by the tensile resistance of the anchor determined in accordance with Clause 8.16.7.6.3 or 8.16.7.6.4, as applicable, and by the compressive resistance of the concrete determined in accordance with Clause 8.8.7.1.

8.16.7.6 Design requirements for anchors

8.16.7.6.1 General

Anchors shall have a minimum diameter of 15 mm.

8.16.7.6.2 Tensile resistance of bolts and studs

The factored tensile resistance of an anchor bolt or stud shall be as specified in Clause 10.19.2.1.

8.16.7.6.3 Tensile resistance of reinforcing bars

The factored tensile resistance, f_r, of an anchor made of a reinforcing bar shall be taken as $\phi_s f_y A_b$.

8.16.7.6.4 Shear resistance

8.16.7.6.4.1

When shear force is transferred through shear friction, the factored shear resistance of the anchorage system, V_r, shall be calculated as μF_r. The values of μ may be taken as follows:

(a) as-rolled steel plate against concrete or grout where the plate is embedded a full plate thickness below the concrete surface: 0.8;

(b) as-rolled steel plate against concrete or grout, with the contact plane coinciding with the concrete surface: 0.6; and

(c) as-rolled steel plate against grout, with the contact plane exterior to the concrete surface: 0.5.

8.16.7.6.4.2
When shear force is transferred through bearing, the shear resistance of the anchor shall be taken as the smallest of
(a) the factored shear resistance of an anchor bolt or stud as specified in Clause 10.19.2.2;
(b) $V_r = 0.8\phi_s f_y A_b$ (for an anchor made of a reinforcing bar); and
(c) B_r as specified in Clause 8.16.7.3.

8.16.7.6.5 Combined tension and shear
An anchor bolt or a stud required to develop resistance to combined tension and shear through bearing shall be proportioned in accordance with Clause 10.19.2.3.

An anchor made of a reinforcing bar required to develop resistance to combined tension and shear through bearing shall be proportioned in such a manner that $(V_f/V_r)^2 + (F_f/F_r)^2 \leq 1.0$.

For an anchor required to develop resistance to combined axial tension and shear through shear friction, the cross-sectional area shall be at least the sum of
(a) the area required by Clause 8.16.7.6.2 or 8.16.7.6.3; and
(b) the area required by Clause 8.16.7.6.4.

8.16.7.6.6 Combined tension and bending
An anchor required to develop resistance to combined tension and bending shall be proportioned to meet the requirements of Clause 10.19.2.4.

8.16.7.7 Additional requirements for grouted and adhesive anchors

8.16.7.7.1 General
Grouts and adhesives shall be formulated, mixed, and placed in accordance with Approved procedures established by tests.

Randomly selected grouted and adhesive anchors shall be tested in accordance with Clause 8.16.7.7.2 to a minimum of 110% of the factored load effect to verify load transfer capabilities. The tests may be waived if acceptable test and installation data are available.

Grouted and adhesive anchors installed in tensile zones of a concrete member shall be capable of sustaining the factored resistance in cracked concrete.

8.16.7.7.2 Testing of anchors
Tests shall be carried out by an Approved testing agency or by the anchor manufacturer using an Approved procedure and shall be certified by a suitably qualified person. Test reports shall present the testing program, procedures, results, and conclusions.

Tests shall be representative of the anchorage system with regard to the embedment depth, spacing, edge distance, load application, concrete type and strength, grouts or adhesives used, and expected environmental conditions. A minimum of five tests shall be carried out for each applicable combination of variables.

Tests conducted in tension shall use the embedment depth needed to attain the full capacity of the anchor. During testing of the tensile strength of an anchor, the testing device shall not apply compression to the concrete surface within a circle that is concentric with the anchor and has a diameter of four times the anchor embedment depth.

8.17 Seismic design and detailing
Seismic design and detailing shall meet the requirements of Section 4.

8.18 Special provisions for deck slabs

8.18.1 Design methods

Clause 8.18 applies to deck slabs supported on girders, stringers, or floor beams of concrete or steel. When proportioned in accordance with the empirical design method of Clause 8.18.4, these deck slabs need not be analyzed, except for negative transverse moments due to loads on the deck slab overhang and the barrier walls, and for longitudinal moments in continuous-span bridges, and the requirements of Clauses 8.8, 8.9, 8.12, and 8.13 shall be waived. As an alternative to the empirical method of Clause 8.18.4, flexural design methods may be used for the design of deck slabs.

8.18.2 Minimum slab thickness

Unless otherwise Approved or as specified in Clause 8.18.4.4, the slab thickness shall be such as to accommodate the minimum covers specified in Clause 8.11.2.2, with the clear distance between the top and bottom transverse reinforcement being at least 55 mm. The slab thickness shall not be less than 175 mm.

8.18.3 Allowance for wear

An additional thickness of 10 mm at the top surface of exposed concrete decks shall be provided to allow for wear.

8.18.4 Empirical design method

8.18.4.1 General

The empirical design method is applicable to that portion of the deck slab which is of nearly uniform thickness and bounded by the exterior supporting beams, provided that the applicable conditions of Clauses 8.18.4.2 to 8.18.4.4 exist, the applicable requirements of Clauses 8.18.4.3.2, 8.18.5, and 8.18.6 are met, and the following conditions exist:

(a) the deck slab is composite with the supporting beams, which are parallel to each other, and the lines of supports for the beams are also parallel to each other;
(b) the ratio of the spacing of the supporting beams to the thickness of the slab is less or equal to 18.0. The spacing of the supporting beams used in calculating this ratio is taken parallel to the direction of the transverse reinforcement;
(c) the spacing of the supporting beams does not exceed 4.0 m and the slab extends sufficiently beyond the external beams to provide full development length for the bottom transverse reinforcement; and
(d) longitudinal reinforcement in the deck slab in the negative moment regions of continuous composite beams is provided for in accordance with Clause 8.19.4 and Section 10, if applicable.

When the supporting beams or their lines of supports are not parallel to themselves, engineering judgment shall be used to determine whether the empirical design method for the design of the deck slab is to be adopted.

8.18.4.2 Cast-in-place deck slabs

For the empirical design method to apply, a full-depth cast-in-place deck slab shall satisfy the following conditions in addition to those of Clause 8.18.4.1:

(a) As shown in Figure 8.5, the deck slab contains two orthogonal assemblies of reinforcement, near the top and bottom of the slab, respectively, with ρ in each direction in each assembly being at least 0.003, except as specified in Item (c). For calculating ρ, the effective depth of concrete, d, is assumed to be the distance between the top of the slab and the centroid of the lower reinforcement assembly.
(b) When the slab is supported on parallel beams, the reinforcement bars closest to the top and bottom of the slab are laid perpendicular to the axes of the supporting beams or are laid on a skew parallel to the lines of beam supports.
(c) The reinforcement ratio, ρ, may be reduced to 0.002 where deck slabs with the reduced reinforcement can be satisfactorily constructed and the reduction of ρ below 0.003 is Approved.

(d) Where the transverse reinforcing bars are placed on a skew, the reinforcement ratio for these bars is not less than $\rho/\cos^2\theta$, where θ is the skew angle.
(e) Where the unsupported length of the edge stiffening beam, S_e, exceeds 5 m, the reinforcement ratio, ρ, in the exterior regions of the deck slab is increased to 0.006, as shown in Figure 8.6.
(f) The spacing of the reinforcement in each direction and in each assembly does not exceed 300 mm.

8.18.4.3 Cast-in-place deck slabs on precast panels

8.18.4.3.1 Composite deck slabs

For the empirical design method to apply, a cast-in-place deck slab on precast panels shall satisfy the following conditions in addition to those of Clause 8.18.4.1:
(a) The precast panel is at least 90 mm thick and contains pretensioned or non-prestressed reinforcement.
(b) For precast pretensioned panels, the reinforcement ratio, ρ, for transverse tendons is a minimum of 0.001. For reinforced concrete precast panels, ρ for transverse reinforcement is a minimum of 0.003. In both cases, ρ is calculated for d equal to the effective depth of the composite slab, and the spacing of the pretensioned strands or reinforcing bars is not more than 300 mm.
(c) For precast pretensioned panels, the strands from a panel continue over the girder flanges into the next panel or are embedded in the cast-in-place concrete for a distance equal to the strand development length specified in Clause 8.15.4. Alternatively, the strands are anchored in the cast-in-place concrete by mechanical devices to develop their strength.
(d) For reinforced concrete precast panels, the reinforcement is anchored by a hook or similar device in the cast-in-place concrete.
(e) As shown in Figure 8.7, the cast-in-place concrete contains one orthogonal assembly of reinforcement near the top of the slab, with ρ equal to 0.003 for reinforcement in each direction. Unless otherwise Approved, the slab contains additional longitudinal reinforcement lying directly on the precast panel, with ρ equal to 0.003. In both cases, ρ is calculated for d equal to the effective depth of the composite slab.
(f) The bottom longitudinal reinforcement placed directly on the panels does not have lap splices over the panel joints.

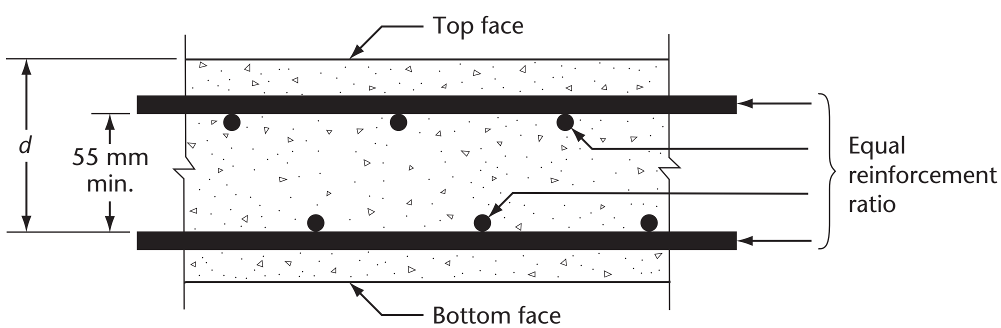
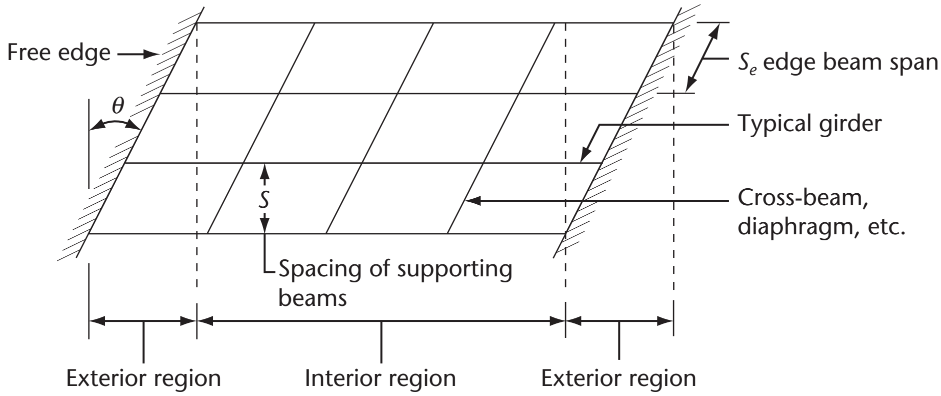


Figure 8.5
Reinforcement in cast-in-place deck slab
(See Clause 8.18.4.2.)



Note: *For the orthogonal arrangement of reinforcing bars in exterior regions, minimum* $\rho = 0.003$ *for* $S_e \leq 5$ m *and* 0.006 *for* $S_e > 5$ m. *For the orthogonal arrangement of reinforcing bars in interior regions, minimum* $\rho = 0.003$.

Figure 8.6
Reinforcement for cast-in-place deck slabs designed using the empirical method
(See Clause 8.18.4.2.)

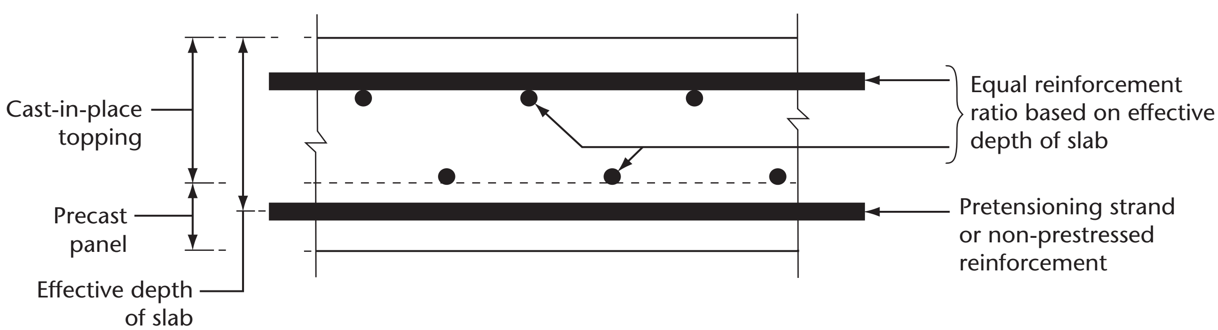


Figure 8.7
Reinforcement for cast-in-place deck slabs on precast panels
(See Clause 8.18.4.3.1.)

8.18.4.3.2 Partial-depth precast panels

For the empirical design method to apply, partial-depth precast panels acting compositely with the cast-in-place topping and the supporting beams shall be designed to satisfy the following conditions in addition to those of Clause 8.18.4.1:

(a) The design takes handling and construction methods into account.
(b) The effective span is taken as the distance between the edges of flanges of the supporting beams plus 150 mm.
(c) The thickness of the panel is not more than $0.55h$.
(d) The pretensioning strands or reinforcing bars are located at the mid-depth of the panel.
(e) In addition to the transverse strands or reinforcing bars, the panel contains 10M longitudinal reinforcing bars at a maximum spacing of 400 mm or a reinforcement mesh with a cross-sectional area of 230 mm^2/m width in the longitudinal direction of the bridge.
(f) For pretensioned panels, the compressive and tensile stresses in the concrete during construction do not exceed $0.6f_c'$ and f_{cr}, respectively (assuming the strand-placing tolerances specified in Clause 8.11.2.2).

(g) The effective span of a precast panel with only non-prestressed reinforcement does not exceed 2.0 m.
(h) The deflection of a panel during construction does not exceed
 (i) 15 mm; and
 (ii) 1/240 of the effective span of the panel.
(i) The top surface of a panel is clean and free of laitance and intentionally roughened to a full amplitude of about 2 mm at about 15 mm centres.
(j) The ends of a panel are supported on the beams in such a manner that, after placement of the concrete topping, a continuous bedding support at least 75 mm wide is provided over the full length of the beams, and such support is within 25 mm of the edges of the beam flanges.
(k) For pretensioned panels, the transfer and development length of the strands accounts for the anticipated conditions during construction.

8.18.4.4 Full-depth precast panels

For the empirical design method to apply, the full-depth precast panels shall satisfy the following conditions in addition to those of Clause 8.18.4.1 and, as applicable, Clause 8.18.4.2:
(a) the panels cover the full width of the bridge;
(b) the depth of the panels is not less than 190 mm;
(c) at their transverse joints, the panels are joined together by grouted shear keys and are longitudinally post-tensioned with a minimum effective prestress of 1.7 MPa;
(d) the ducts for longitudinal post-tensioning are located at the mid-depth of the panels, and openings (also known as blockouts) are provided at the joints to accommodate splices for tendons;
(e) blockouts are provided in the panels at locations where the panels are to be connected to the beams for composite action;
(f) initially, the panels are supported on the beams by means of temporary levelling devices, with the blockouts for connections to beams for composite action and the gap between the panels and beams being filled with grout after completion of post-tensioning; and
(g) the grout used in the shear keys has a minimum strength of 35 MPa at 24 h.

8.18.5 Diaphragms

The decks slabs of all continuous-span bridges shall have cross-frames or diaphragms extending throughout the cross-section at intermediate support lines. Steel I-girders supporting deck slabs designed in accordance with the empirical design method of Clause 8.18.4 shall have intermediate cross-frames or diaphragms at a spacing of not greater than 8.0 m centre-to-centre.

Deck slabs on box girders shall have intermediate diaphragms or cross-frames at a spacing not exceeding 8.0 m centre-to-centre between the boxes. Alternatively, deck slabs may contain reinforcement over the internal webs additional to that required by the empirical method (to provide for the global transverse bending due to eccentric loads).

8.18.6 Edge stiffening

The transverse free edges of all deck slabs shall be stiffened by composite edge beams and shall be proportioned for the effects of wheel loads. Where the unsupported length of an edge stiffening beam, S_e, is less than or equal to 5 m and the slab is designed in accordance with Clause 8.18.4, the details as shown in any one of the diagrams of Figure 8.8 may be considered satisfactory.

8.18.7 Distribution reinforcement

The distribution reinforcement for slabs analyzed using elastic methods in accordance with Section 5 shall be placed transverse to the main reinforcement.

The amount of distribution reinforcement for the main reinforcement parallel to traffic, as a percentage of the main reinforcement, shall be $55/(S)^{0.5}$, up to a maximum of 50%.

When the main reinforcement is perpendicular to traffic, the amount of the distribution reinforcement shall be $120/(S)^{0.5}$, up to a maximum of 67%. In the outer quarter of the span it may be reduced to one-half of the calculated amount.

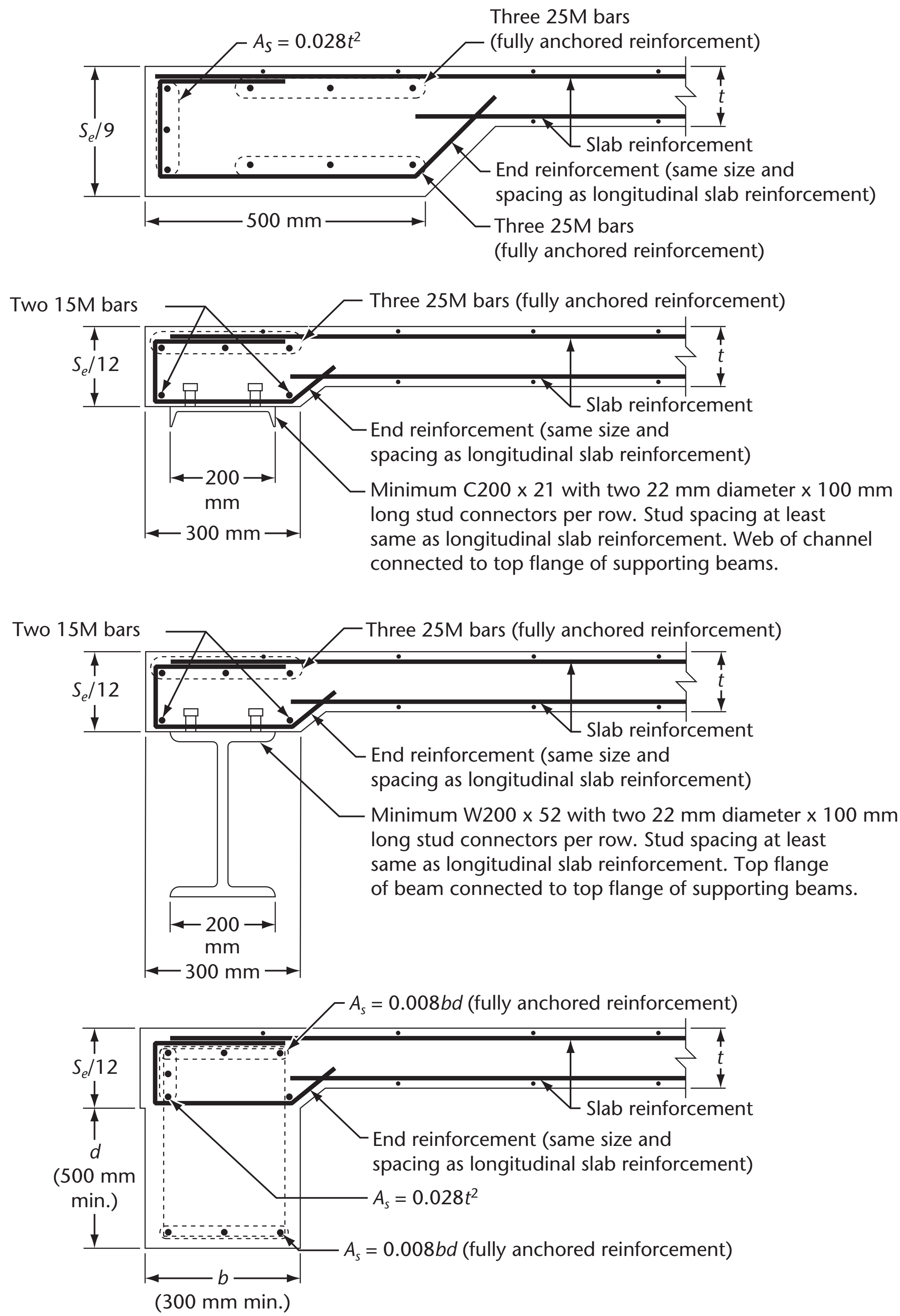


Figure 8.8
Edge stiffening at transverse free edges
(See Clause 8.18.6.)

8.19 Composite construction

Note: *This Clause applies to flexural components constructed in separate placements and interconnected in such a manner that they respond to loads as an integral unit.*

8.19.1 General

Precast concrete units shall be proportioned to support all loads applied before the cast-in-place concrete attains a strength of $0.75f_c'$.

8.19.2 Flexure

When the components used in composite construction have different specified strengths, stresses at the serviceability limit state shall be calculated on the basis of the respective moduli of elasticity.

Differential shrinkage between cast-in-place and precast concrete shall be considered in the design of composite components at the serviceability limit state. A differential shrinkage strain of 100×10^{-6} shall be assumed unless more accurate data are available.

The factored resistance of a composite section shall be calculated in the manner used for a monolithically cast unit.

8.19.3 Shear

The factored shear resistance of a composite section shall be calculated in accordance with Clause 8.9.3.3. Interface shear shall be investigated and provided for in accordance with Clause 8.9.5.

8.19.4 Semi-continuous structures

8.19.4.1 General

The effects of creep and shrinkage shall be considered when structural continuity is assumed in calculating live load and superimposed dead load effects in bridges composed of simply supported girders that are precast, prestressed, and made continuous by providing tensile reinforcement in the cast-in-place deck slabs and diaphragms over the girder supports.

8.19.4.2 Positive moments

When the age of girders at the time of introducing continuity can be predicted and controlled, the positive moment reinforcement over the supports shall be proportioned for structural continuity to resist the moments due to creep, shrinkage, temperature change, and live load in remote spans. The effects of deformation and settlement of piers shall also be considered. The stress in the reinforcement shall be limited to 240 MPa at serviceability limit states.

When the age of girders at the time of introducing continuity cannot be predicted and controlled, the superimposed dead load and live load moments shall be determined from an analysis that accounts for the lack of positive moment continuity. Minimum positive moment reinforcement having an area at least 1.50 times the nominal depth of the precast component shall be provided in the bottom flanges over the supports (with the units of both the multiplier of 1.50 and the depth in millimetres).

The reinforcement shall be adequately embedded in the bottom flange of the girders beyond the strand transfer length and anchored into the diaphragm over the continuity supports.

8.19.4.3 Negative moments

The negative moment at the supports shall be calculated based on the assumption of full structural continuity. The effect of precompression due to prestress in the girders shall be neglected in calculating the negative flexural resistance within the strand transfer length.

The ratio of the continuity reinforcement, ρ, in the deck slab shall not exceed 0.5 times the ratio that would produce balanced strain conditions for the composite section.

8.20 Concrete girders

8.20.1 General

In box girders and T-girders, there shall be full transfer of shear forces at the interface of the girder webs and the top and bottom flanges. Proportioning for interface shear shall be in accordance with Clause 8.9.5.

Changes in the thickness of the web of a girder shall be achieved by tapering for a minimum distance of twelve times the difference in web thickness.

8.20.2 Effective flange width for T- and box girders

The effective flange width shall be as specified in Clause 5.8.2.

8.20.3 Flange thickness for T- and box girders

8.20.3.1 Top flange

The thickness of the top flange shall be as specified in Clause 8.18.2 and not less than the following:

(a) for cast-in-place T- and box girders: 0.05 times the clear distance between the webs; and

(b) for precast T- and box girders:
   (i) 125 mm; and
   (ii) 0.03 times the clear distance between the webs.

Where the top flanges of precast T- and box girders act compositely with a cast-in-place concrete topping, the flange thickness limit shall be that for cast-in-place girders and shall be based on the total thickness.

8.20.3.2 Bottom flange

The thickness of the bottom flange shall not be less than the following:

(a) for cast-in-place girders:
   (i) 150 mm; and
   (ii) 0.06 times the clear distance between the webs; and

(b) for precast girders:
   (i) 100 mm; and
   (ii) 0.03 times the clear distance between the webs.

8.20.3.3 Fillets

For cast-in-place girders, fillets with dimensions of at least 100 × 100 mm shall be provided at the intersections of all interior surfaces. They may, however, be omitted at the junction of the web and bottom flange of a box girder.

8.20.4 Isolated girders

Isolated girders in which the T-form is used for providing additional compression area shall have a flange thickness at least equal to 0.3 times the web width or 100 mm, whichever is greater.

8.20.5 Top and bottom flange reinforcement for cast-in-place T- and box girders

In each flange, reinforcement with a minimum area of 0.004 times the flange area shall be placed parallel to the girder span (for prestressed components, however, a minimum reinforcement of 0.003 times the flange area shall be used). Such reinforcement shall be distributed near both surfaces of the flange. The spacing of the reinforcement shall not exceed 300 mm.

In each flange, reinforcement with a minimum area of 0.005 times the transverse cross-sectional area of the flange based on the least corresponding flange thickness shall be placed transverse to the girder span. Such reinforcement shall be distributed near both surfaces of the flange. The maximum spacing of the reinforcement shall be 300 mm.

All transverse reinforcement in the bottom flange of box girders shall extend over the width of the girder and shall be adequately anchored.

8.20.6 Post-tensioning tendons

Ducts for post-tensioning shall be located within the stirrups in webs and, where applicable, between layers of transverse reinforcing in flanges and slabs. The effect of grouting pressure in the ducts shall be considered. Curved tendons shall meet the requirements of Clause 8.6.2.7.

In the top and bottom flanges of box sections where ducts for post-tensioning are spaced closer than 300 mm, the top and bottom reinforcement mats shall be tied together with vertical reinforcement consisting of 10M hairpin bars with a spacing not exceeding 300 mm in each direction.

8.20.7 Diaphragms

Diaphragms shall be provided at abutments and piers. The diaphragms shall be proportioned to transfer loads to the supports and to allow for future jacking of the girders. Intermediate diaphragms shall be provided if required for improving load distribution or for stability during construction.

8.21 Multi-beam decks

Multi-beam decks consisting of precast units placed side by side shall have a means for live load shear transfer between the units. Shear transfer may be achieved by

(a) a 150 mm thick concrete structural slab. The transverse shear in the slab shall be calculated in accordance with Section 5 and the concrete slab shall be reinforced to resist this shear in accordance with Clause 8.9.5;

(b) grouted shear keys in combination with lateral post-tensioning providing a prestress of not less than 1.7 MPa, after all losses, over a compressed depth of joint not less than 175 mm; or

(c) an Approved means capable of live load shear transfer between the units.

8.22 Segmental construction

8.22.1 General

Clause 8.22 applies to post-tensioned girders made of match-cast or cast-in-place concrete segments. The cross-section may consist of single or multi-cell box segments or beam-type segments. The box segments may be transversely prestressed and the beam-type segments may be pretensioned. The erection and construction loads shall be as specified in Section 3.

Stresses due to the changes in the structural system, in particular the effects of the application of a load to one system and its removal from a different system, shall be accounted for. Redistribution of force effects due to creep shall be taken into account and allowance made for possible variations in the creep rate and magnitude.

8.22.2 Additional ducts and anchorages

8.22.2.1 General

Provision shall be made for the introduction of additional post-tensioning to compensate for excessive friction losses during construction and for future strengthening of the bridge.

8.22.2.2 During construction

Segmental box girder bridges with internal tendons shall have additional anchorages and ducts capable of accommodating tendons with a capacity equal to at least 5% of the positive and negative moment post-tensioning forces, respectively. The ducts shall be located symmetrically about the bridge centreline and the anchorages shall be distributed uniformly at three segment intervals along the length of the bridge. At least one additional duct per web with adequate anchorage shall be provided.

For continuous bridges, the additional positive moment ducts and anchorage capacity need not be provided along 25% of the span length on either side of an intermediate support. All additional ducts not used during construction shall be grouted at the same time as other ducts in the span.

8.22.2.3 Future strengthening

Provision shall be made for access, anchorages, deviators, and openings along the box girder cells to permit addition of external tendons located symmetrically about the bridge centreline for future strengthening. In this regard, provision shall be made for at least 10% of the positive moment post-tensioning forces and at least 10% of the negative moment post-tensioning forces.

8.22.3 Diaphragms

Diaphragms shall be provided at abutments, piers, and locations of abrupt angular changes of the soffit of the girders. Provision shall be made in the diaphragms for openings for access, future strengthening, and utilities.

8.22.4 Deviators for external tendons

8.22.4.1 Design and detailing

Deviators shall consist of deviation blocks or diaphragms. The design of the deviators shall be based on the specified strength of the tendons. Localized flexural effects in the web and flange shall be considered.

Reinforcement shall be provided in the form of reinforcing bars anchored in the web and flange. The development length shall be measured from the tendon axis and the reinforcement shall be mechanically anchored around longitudinal reinforcing bars.

8.22.4.2 Localized effects

The transverse force effects at the deviation blocks due to unsymmetrical geometry and sequence of post-tensioning shall be considered and shall be resisted by post-tensioning or by reinforcing bars proportioned for a stress not exceeding 240 MPa.

8.22.5 Coupling of post-tensioning tendons

Not more than 50% of the tendons in a member shall be coupled at the same section. The distance between couplers of adjacent tendons shall not be less than the segment length and not less than twice the segment depth.

8.22.6 Special provisions for various bridge types

8.22.6.1 Precast segmental

8.22.6.1.1 General

Precast segmental bridges shall be designed to be erected in accordance with one of the following methods:

(a) balanced cantilever;
(b) span-by-span; or
(c) progressive placement.

The minimum age of the segments at the time of erection shall be 14 d unless otherwise Approved.

8.22.6.1.2 Joints

Precast segments shall be match cast and erected with epoxied joints. The minimum thickness of epoxy shall be 2 mm on each surface if applied to both surfaces or 3 mm if applied to one surface.

A minimum compressive stress of 350 kPa shall be provided over the entire cross-sectional area between precast segments by temporary post-tensioning until the permanent tendons are fully stressed.

8.22.6.1.3 Shear keys
At the joints, shear keys incorporating corrugations shall be providing in the webs. The spacing of the corrugations shall be four times their depth. The corrugations shall be not less than 30 mm deep and shall extend for as much of the web width and depth as practicable. Interface shear resistance shall be calculated in accordance with Clause 8.9.5.

Keys in the top and bottom flanges for alignment of segments during erection shall also be provided. These may be large single-element keys.

8.22.6.2 Cast-in-place segmental

8.22.6.2.1 General
Cast-in-place segmental bridges shall be designed to be constructed on falsework in accordance with the balanced cantilever method, span-by-span construction, or incremental launching.

8.22.6.2.2 Closure segments
The length of a closure segment shall be such as to permit coupling of the duct sheaths and jacking of the tendons in the completed cantilevers.

8.22.6.2.3 Joints
The contact surfaces between cast-in-place segments shall be clean, free of laitance, and intentionally roughened. Longitudinal reinforcing bars in the segments shall extend across the joints.

8.22.6.3 Balanced cantilever construction
This Clause shall apply to both precast and cast-in-place cantilever construction.

Longitudinal tendons may be anchored in the webs, in the slab, or in blisters built out from the web or slab. A minimum of two longitudinal tendons shall be anchored in each segment.

Continuity tendons shall be anchored at least one segment beyond the point where they are theoretically required for stresses.

The segment lengths, construction loads, and sequence of construction assumed in the design shall be shown on the Plans.

8.22.6.4 Span-by-span construction
Provision shall be made in the design of span-by-span construction for accumulated construction force effects due to the change in the structural system as the construction progresses.

8.22.6.5 Incrementally launched construction

8.22.6.5.1 General
Tensile stresses under all stages of launching shall not exceed the limits specified in Clause 8.8.4.6(a)(iii)(2).

Provision shall be made to resist the frictional forces on the substructure during launching and to restrain the superstructure if the structure is launched down a gradient. For determining the critical frictional forces, the friction on launching bearings shall be assumed to vary between zero and 4%, whichever is critical. The upper value may be reduced to 3.5% if pier deflections and launching jack forces are monitored during construction.

8.22.6.5.2 Force effects due to construction tolerances
The force effects due to the permissible construction tolerances specified in Table 8.12 shall be superimposed on those arising from gravity loads.

Table 8.12
Construction tolerances
(See Clause 8.22.6.5.2.)

Condition	Tolerance, mm
In the longitudinal direction between bearings of adjacent supports	5
In the transverse direction between two adjacent bearings	3
Between the fabrication area and the launching equipment in the longitudinal and transverse directions	3
Lateral deviation at the outside of the webs	3

The horizontal force acting on the lateral guides of the launching bearings shall be not less than 1/100 of the vertical support reaction.

For stresses during construction, one-half of the force effects due to construction tolerances and one-half of the force effects due to temperature as specified in Section 3 shall be superimposed on those arising from gravity loads.

8.22.6.5.3 Design details

Piers and superstructure diaphragms at piers shall be designed to permit jacking of the superstructure during all launching stages and for the installation of permanent bearings. Frictional forces during launching shall be considered in the design of the substructure.

Local stresses that could develop at the underside of the web during launching shall be investigated. The following dimensional requirements shall be satisfied:

(a) launching bearing pads shall not be placed closer than 80 mm to the outside of the web;
(b) concrete cover between the soffit and post-tensioning ducts shall not be less than 150 mm; and
(c) bearing pressures at the web/soffit corner shall be investigated and the effects of ungrouted ducts and any eccentricity between the intersection of the centrelines of the web and the bottom slab and the centreline of the bearing shall be considered.

The straight tendons required to resist forces during launching should be placed in the top and bottom flanges. For T-sections, the bottom tendons shall be located in the lower one-third of the web.

The faces of construction joints shall be intentionally roughened or provided with shear keys in accordance with Clause 8.22.6.1.3. The reinforcement in both directions at all concrete surfaces across the joint and extending up to at least 2 m on each side of the joint shall be 15M bars at 200 mm centres.

8.22.7 Precast segmental beam bridges

8.22.7.1 General

Precast beam-type segments shall, where practicable, be pretensioned to resist the applicable dead and construction loads so that the tensile stress during construction is limited to $0.6f_{cr}$.

8.22.7.2 Joints

Joints between the segments shall be epoxied or cast in place.

Epoxied joints shall be formed between match-cast surfaces. The match-cast effect in spliced pretensioned girders shall be created by casting against precision-made steel bulkheads. The joints shall meet the requirements of Clause 8.22.6.1.2.

Cast-in-place joints shall be wide enough to permit the coupling of duct sheaths and placing of concrete. The strength of concrete in the joints shall be compatible with that of the adjacent girder concrete.

8.22.7.3 Shear keys
Large single-element shear keys shall be provided for match-cast splices. For cast-in-place splices, the ends of the beams at the joints shall be artificially roughened.

8.23 Concrete piles

8.23.1 General
The design of concrete piles shall meet the requirements of this Section and Section 6.

8.23.2 Specified concrete strength
Unless otherwise Approved, the minimum concrete strength shall be 30 MPa for cast-in-place piles and 35 MPa for precast piles.

8.23.3 Handling
Account shall be taken of the handling and transportation of precast piles. An allowance for impact of 50% of the weight of the pile shall be made in proportioning the pile.

8.23.4 Splices
The shape and size of a splice shall be such as not to affect the performance of the pile. The strength of a splice shall be at least equal to the strength of the pile in compression, tension, and flexure. The slack in mechanical splices shall be less than 0.5 mm in either compression or tension.

8.23.5 Pile dimensions
The minimum diameter or side dimension shall be 200 mm for precast piles and 400 mm for cast-in-place piles.

Prestressed concrete piles may be solid or hollow. The minimum wall thickness for hollow piles shall be 125 mm.

8.23.6 Non-prestressed concrete piles

8.23.6.1 General
Non-prestressed concrete piles shall meet the requirements of Clauses 8.8.3 and 8.8.5.

8.23.6.2 Reinforcement details

8.23.6.2.1 Cast-in-place
The reinforcement details for cast-in-place concrete piles shall meet the requirements of Clauses 8.14 and 8.15.

8.23.6.2.2 Precast
For precast concrete piles, the area of longitudinal reinforcement shall not be less than 0.015 and shall not be more than 0.08 of the cross-sectional area of the pile. Longitudinal reinforcement shall be enclosed within spirals that meet the requirements of CSA G30.3.

For piles up to 600 mm in diameter, the spiral wire shall have a diameter of at least 5 mm. At the end of a pile, the spiral shall have a pitch of 25 mm for five turns followed by a pitch of 75 mm for 16 turns. For the remainder of the pile, the spiral shall have a pitch of not more than 150 mm.

For piles more than 600 mm in diameter, the spiral wire shall have a diameter of at least 6 mm. At the ends of a pile, the spiral shall have a pitch of 40 mm for four turns followed by a pitch of 50 mm for 16 turns. For the remainder of the pile, the spiral shall have a pitch of not more than 100 mm.

8.23.7 Prestressed concrete piles

8.23.7.1 Effective prestress

Prestressing steel shall be placed and stressed to provide an effective prestress of between 3 and 5 MPa for piles up to 12 m long and between 5 and 8 MPa for piles longer than 12 m. The effective axial prestress shall not exceed $0.20f_c'$.

8.23.7.2 Concrete stress limitations

8.23.7.2.1 Handling

Stresses during handling shall not exceed $0.60f_c'$ in compression and f_{cr} in tension.

8.23.7.2.2 Under loads

The stresses at serviceability limit state loads acting on a pile shall be such that

(a) no tension develops; and

(b) $(P_s/P_a + M_s/M_a) < 1.0$

where

P_a = $(0.33f_c' - 0.27f_{pc})A_g$ for laterally supported piles

= $R(0.33f_c' - 0.27f_{pc})A_g$ for laterally unsupported piles with $l_e/r < 120$

where

R = $(1.23 - 0.008l_e/r) < 1.0$

l_e = 1.0ℓ for piles hinged at both ends

= 0.8ℓ for piles fixed at one end

= 0.6ℓ for piles fixed at both ends

M_a = $f_{pc}(I_g/c)$

8.23.7.3 Factored resistance

The basic assumptions of Clause 8.8.3 and the requirements of Clause 8.8.5 shall be used in calculating the resistance of piles.

8.23.7.4 Sections within development length

The effect of the transfer length on the stresses at serviceability limit states and the development length on the factored resistance shall be investigated.

8.23.7.5 Reinforcement details

The full length of tendons shall be enclosed within spiral wire meeting the requirements of CSA G30.3. Spirals shall be provided in accordance with Clause 8.23.6.2.2.