Section 9 — Wood structures

Section 9
Wood structures

9.1 Scope

This Section applies to structural wood components and their fastenings.

9.2 Definitions

The following definitions apply in this Section:

Beam and stringer (grading term) — sawn wood with a smaller dimension of at least 114 mm and a larger dimension more than 51 mm greater than the smaller dimension, graded for use in bending with the load applied to the narrow face.

Bearing block — a short wood block with its grain parallel to the applied post-tensioning force, used to distribute the forces in a stress-laminated wood bridge with an external post-tensioning system.

Butt joint — the discontinuities in a laminated wood deck where the ends of two laminates meet.

Crib — a configuration of horizontal members with alternating layers (usually perpendicular to one another) connected to form a closed box.

Dimension lumber — sawn wood 38 to 102 mm thick.

Direct bearing area — the area of outside lamination over which the post-tensioning is assumed to be applied.

Direct bearing pressure — the average pressure that is assumed to be applied to the direct bearing area by the post-tensioning force.

Distribution bulkhead — a steel section used to distribute the post-tensioning force.

Drift pin — a steel pin used to connect wood members.

Duration of load — a period of continuous application of a specified load or the summation of the time periods of intermittent applications of the same load.

External post-tensioning system — a system that transversely post-tensions a longitudinally laminated wood deck using two bars at each anchorage, one above and one below the deck.

Framed bent — a line of wood columns suitably braced.

Glued-laminated timber (Glulam) — structural wood that is manufactured in accordance with CSA O122 and is produced by gluing together a number of laminates with essentially parallel grains.

Grade — the designation of the quality of a wood element.

Header — a horizontal member of a crib whose longitudinal axis runs perpendicular to the long side of the crib and provides anchorage to the stretchers.

Incising — the process of cutting many small slits into the surface of the wood before pressure preservative treatment.

Internal post-tensioning system — a system for transversely post-tensioning a longitudinally laminated wood deck using a single bar at each anchorage (the bar being situated at the neutral axis of the wood deck).

Joist (grading term) — sawn wood that is 38 to 89 mm thick, at least 114 mm wide, and intended to be loaded on its narrow face.

Laminate — dimension lumber used in a laminated wood deck or beam.

Laminated veneer lumber — structural wood that is manufactured in accordance with ASTM D 5456 and consists of bonded wood veneer sheet elements with their wood fibres primarily oriented along the length of the member.

Laminated wood deck — a deck consisting of dimension lumber joined to form a continuous wood slab with the widths oriented vertically.

Load-sharing system — a system of members consisting of two or more essentially parallel members arranged or connected in such a way that they mutually support the load and deflect together by approximately the same amount.

Longitudinally laminated deck — a laminated wood deck in which the length of the laminates is oriented in the direction of the span of the bridge.

Nail-laminated deck — a laminated wood deck joined together only by the successive nailing of each lamination to the preceding one.

Parallel strand lumber — structural wood that is manufactured in accordance with ASTM D 5456 and consists of wood strand elements with their wood fibres primarily oriented along the length of the member.

Pile bent — a single line of free-standing piles, suitably braced and connected to form a pier.

Plank (grading term) — sawn wood that is 38 to 89 mm thick, at least 114 mm wide, and intended to be loaded on its wide face.

Post and timber (grading term) — sawn wood with a smaller dimension of at least 114 mm and a larger dimension not more than 51 mm greater than the smaller dimension, graded for use as a column.

Preservative treatment — impregnation under pressure with a wood preservative in accordance with the CSA O80 Series of Standards.

Sawn wood — wood that is the product of a sawmill and is not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing mill, and crosscutting to length.

Specified strength of sawn wood — the assigned strength for calculating resistance, as specified in Tables 9.12 to 9.17.

Stress-graded lumber — sawn wood that has been graded in accordance with the NLGA *Standard Grading Rules for Canadian Lumber.*

Stress-laminated wood deck — a laminated wood deck that is post-tensioned perpendicular to the deck laminates using high-strength steel bars.

Transversely laminated deck — a laminated wood deck in which the laminates are oriented approximately perpendicular to the direction of the span of the bridge.

Wood-concrete composite deck — a longitudinally laminated wood deck made composite with a reinforced concrete overlay.

Wood mudsill — a horizontal member that bears on soil and is used to distribute vertical loads.

Wood preservative — a chemical formulation that is toxic to fungi, insects, borers, and other wood-destroying organisms and meets the requirements of the CSA O80 Series of Standards.

Wood trestle — a wood bridge with pile bents or framed bents.

9.3 Symbols

The following symbols apply in this Section:

A = cross-sectional area of a member or the bearing area, mm^2

A_b = area of direct bearing on the edge lamination, mm^2

A_r = steel/wood ratio (A_s/A_w)

A_s = total cross-sectional area of post-tensioning steel at one anchorage, mm^2

A_w = product of the distance between two consecutive post-tensioning anchorages and the depth of the wood deck, mm^2

b = width of a member or lamination, mm

b_b = width of the distribution of the post-tensioning forces at the edge lamination, mm

C_c = slenderness ratio

C_k = intermediate slenderness factor

C_m = factor relating the actual moment diagram to an equivalent uniform moment diagram

C_s = slenderness factor

D_b = diameter of the butt of a pile, mm

D_e = width over which elements sharing load deform substantially uniformly, m

D_{eff} = effective diameter of a pile or other round compression member at 0.45 of the member length above the lower point of contraflexure, mm

D_h = diameter of a hole for post-tensioning, mm

D_t = diameter of the tip of a pile, mm

d = depth of a member or lamination, mm

d_c = depth of a channel bulkhead, mm

E_c = modulus of elasticity of concrete, MPa

E_s = modulus of elasticity of steel, MPa

E_{05} = 5th percentile of the modulus of elasticity, MPa

E_{50} = 50th percentile of the modulus of elasticity, MPa

e_b = unamplified eccentricity at the middle of the unsupported length due to a bow in a column, mm

e_o = unamplified eccentricity at the critical section, mm

f_{bu} = specified bending strength, MPa

f_{pu} = specified compressive strength parallel to grain, MPa; specified tensile strength of a prestressing tendon, MPa

f_{py} = specified yield strength of a prestressing tendon, as defined in CSA G279

$f_{q\ell}$ = limiting pressure perpendicular to the grain, MPa

f_{qu} = specified compressive strength perpendicular to the grain, MPa

f_{tg} = specified tensile strength parallel to the grain at the gross section for glued-laminated Douglas fir, MPa

f_{tn} = specified tensile strength parallel to the grain at the net section for glued-laminated Douglas fir, MPa

f_{tu} = specified tensile strength parallel to the grain, MPa

f_{vu} = specified shear strength for a 1.0 m^3 cube subjected to uniform shear, MPa

I = moment of inertia of a section, mm^4

I_b = moment of inertia of a pile at its butt, mm^4

k = effective length factor

k_b = modification factor for the effect of butt joints on the stiffness of laminated wood decks

k_c = slenderness factor

k_d = modification factor for duration of load

$k_{\ell s}$ = modification factor for lateral stability

k_m = modification factor for load sharing

k_{sb} = modification factor for the size effect for flexure

k_{sp} = modification factor for the size effect for compression parallel to the grain

k_{sq} = modification factor for the size effect for compression perpendicular to the grain

k_{st} = modification factor for the size effect for tension

k_{sv} = modification factor for the size effect for shear

k_u = ratio of effective length to total length of a pile

L = length of a component, mm

L_b = length of the distribution of the post-tensioning force along the edge lamination, mm

L_p = length of a steel anchorage plate, mm

L_u = laterally unsupported length of a component, mm

M_c = amplified moment used for proportioning slender compression members, N•mm

M_p = factored unamplified moment at the critical section of a pile, N•mm

M_r = factored resistance of a member in flexure, N•mm

M_x = amplified moment about the x-axis of a compression member, N•mm

M_{xr} = factored resistance in bending about the x-axis of a compression member, N•mm

M_y = amplified moment about the y-axis of a compression member, N•mm

M_{yr} = factored resistance in bending about the y-axis of a compression member, N•mm

M_0 = total factored maximum unamplified moment for columns other than tapered piles, N•mm

M_1 = value of the smaller end moment at the ultimate limit state due to factored loads acting on a compression member (positive if the member is bent in single curvature and negative if 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 positive), N•mm

N_b = a measure of the frequency of butt joints in laminated wood decks, being, for any 1.0 m wide band perpendicular to the laminates, the minimum number of laminates without joints adjacent to a laminate having a butt joint

N_f = assumed uniformly distributed normal pressure after losses, MPa

N_j = assumed uniformly distributed normal pressure at transfer, MPa

P = factored axial load, N

P_{cr} = factored Euler buckling load, N

P_r = factored resistance in compression of an axially loaded short column, N

R_r = factored resistance in bearing, N

S = section modulus, mm^3

S_b = section modulus of a transformed section of a composite wood-concrete deck with respect to the bottom fibres, mm^3

S_t = section modulus of a transformed section of a composite wood-concrete deck with respect to the reinforcing steel, mm^3

s = spacing of prestressing anchorages, m

T = total time for which a segment of a transversely laminated deck is not under stress

T_r = factored resistance in tension, N

t_p = thickness of an anchorage plate, mm

V = volume of a beam, m^3

V_f = factored shear load on a member, kN

V_r = factored shear resistance, N

w = width of a steel anchorage plate, mm

δ = moment amplification factor

η = factor used in computing P_{cr} for tapered piles

θ = angle between the plane of loading and the direction of the grain, degrees

ϕ = resistance factor for wood components

ϕ_s = resistance factor for steel components

9.4 Limit states

9.4.1 General

Structural components shall be proportioned to satisfy the requirements at the serviceability limit state in accordance with Clause 9.4.2 and at the ultimate limit state in accordance with Clause 9.4.3.

9.4.2 Serviceability limit states

The superstructure vibration limitation specified in Clause 3.4.4 and the deflection limitations at serviceability limit states specified in this Clause shall apply to wood components.

The deflection of a component shall not exceed 1/400 of the span of the component and shall be calculated using E_{50} obtained from Tables 9.12 to 9.17. Only live load shall be considered in accordance with SLS Combination 1 of Table 3.1, excluding dynamic load allowance, and the truck shall be placed as specified in Clause 3.8.4.1.

9.4.3 Ultimate limit states

Components shall be proportioned to have a factored resistance not less than the sum of the load effects due to the factored loads specified in Section 3.

9.4.4 Resistance factor

The resistance factor for wood components, ϕ, shall be as specified in Table 9.1.

Table 9.1
Resistance factor for wood components, ϕ
(See Clauses 9.4.4, 9.22.5.2, and 9.23.4.2.)

	Load effect				
Component type	Flexure	Shear	Compression parallel to grain	Compression perpendicular to grain	Tension parallel to grain
Sawn wood	0.9	0.9	0.8	0.8	0.9
Glued-laminated timber	0.9	0.9	0.8	0.8	0.9
Structural composite lumber	0.9	0.9	0.8	0.8	0.9
Piles	0.9	0.9	0.8	0.8	0.9

9.5 General design

9.5.1 Design assumption

In accordance with Section 5, only linear elastic analysis shall be used.

9.5.2 Spans

The span length of simply-supported components shall be taken as the distance face-to-face of supports plus one-half the required length of bearing at each end. For continuous members, the span shall be taken as the distance between centres of supports.

9.5.3 Load-duration factor

The value of factor k_d shall be taken as 0.7 when considering dead load alone, earth pressure alone, and dead load plus earth pressure only. For load combinations including wind and earthquake, the factor shall be taken as 1.15. For all other cases, k_d shall be taken as 1.0.

9.5.4 Size-effect factors

The values of size-effect factors shall be obtained from the following clauses:
(a) k_{sb}: Clause 9.6.2;
(b) k_{sv}: Clause 9.7.2;
(c) k_{sp}: Clauses 9.8.2.2 and 9.8.2.3;
(d) k_{sq}: Clause 9.10; and
(e) k_{st}: Clause 9.9.

9.5.5 Service condition

It shall be assumed that the properties specified in Tables 9.12 to 9.17 have been modified for the appropriate service condition.

9.5.6 Load-sharing factor

For systems of members in flexure and shear, and for tension members at the net section, the load-sharing factor, k_m, shall be obtained either directly or by linear interpolation from Table 9.2 for the number of load-sharing components, n. For members in compression not spaced more than 600 mm apart, k_m shall be taken as 1.1. For all other systems, k_m shall be taken as 1.0.

For moments and shears in flexural members, n shall not be greater than the number of components within the widths D_e and $0.8D_e$, respectively, where D_e is as specified in Table 9.3.

Table 9.2
Load-sharing factor for bending, shear, and tension for all species and grades
(See Clause 9.5.6.)

Number of load-sharing components, n	Load-sharing factor, k_m
2	1.10
3	1.20
4	1.25
5	1.25
6	1.30
10	1.35
15	1.40
20	1.40

Table 9.3
Values of D_e
(See Clause 9.5.6.)

Structure	D_e, m
Longitudinal nail-laminated deck	0.85
Transverse nail-laminated deck	0.40
Longitudinal stress-laminated deck	1.75
Transverse stress-laminated deck	0.75
Stringer of sawn timber stringer bridge	1.75
Longitudinal laminate of wood-concrete composite deck	1.60

9.5.7 Notched components

Notches or abrupt changes in section shall not be used unless a detailed assessment of the stress concentration effect has been made.

9.5.8 Butt joint stiffness factor

The stiffness of laminated wood decks shall be adjusted by a modification factor, k_b, to account for the effect of butt joints. For decks other than longitudinal nail-laminated decks, k_b shall be calculated as follows:

$$k_b = (N_b - 1)/N_b$$

where the frequency of butt joints is 1 in N_b, as specified in Clauses 9.21.2.2.5 and 9.22.2.2.2. For longitudinal nail-laminated wood decks other than wood-concrete composite decks, the value of k_b shall be calculated as follows:

$$k_b = 0.8(N_b - 1)/N_b$$

9.5.9 Treatment factor

The properties specified given in this Section incorporate adjustments for preservative treatment and incising. For wood treated with a fire retardant or other strength-reducing chemicals, the assumed properties shall be based on the documented results of tests that take into account the effects of time, temperature, and moisture content.

9.6 Flexure

9.6.1 Flexural resistance

The factored resistance, M_r, shall be calculated as follows:

$$M_r = \phi\, k_d\, k_{\ell s}\, k_m\, k_{sb}\, f_{bu}\, S$$

where f_{bu} is obtained from Tables 9.14 to 9.17, as applicable, and the values of k_d, $k_{\ell s}$, k_m, and k_{sb} are specified in Clauses 9.5.3, 9.6.3, 9.5.6, and 9.6.2, respectively.

9.6.2 Size effect

The value of k_{sb} for sawn wood members shall be obtained from Table 9.4. The value of k_{sb} for members other than sawn wood members shall be 1.0.

Table 9.4
Size-effect factors k_{sb} for flexure and k_{sv} for shear for all species and grades
(See Clauses 9.6.2, 9.7.2, and 9.22.5.2.)

	Larger dimension, mm						
Smaller dimension, mm	**89**	**140**	**184**	**235**	**286**	**337**	**≥ 387**
≤ 64	1.7	1.4	1.2	1.1	1.0	0.9	0.8
> 64 but < 114	1.7	1.5	1.3	1.2	1.1	1.0	0.9
≥ 114	—	1.3	1.3	1.2	1.1	1.0	0.9

9.6.3 Lateral stability

The value of $k_{\ell s}$ shall be obtained from Table 9.5, where b and d are, respectively, the width and depth of the beam or laminate, and C_s and C_k are calculated as follows:

$$C_s = \sqrt{\frac{L_u d}{b^2}}$$

$$C_k = \sqrt{\frac{E_{05}}{f_{bu}}}$$

where L_u is the laterally unsupported length of the component and f_{bu} and E_{05} are obtained from Tables 9.12 to 9.17, as applicable.

For laminated wood decks, or when the compression edge of a beam is effectively supported along its length, $k_{\ell s}$ shall be taken as 1.0.

When d/b is greater than 1.0, lateral support shall be provided at points of bearing to restrain torsional rotation.

A beam shall not have C_s greater than 30.0.

Table 9.5
Modification factor for lateral stability, $k_{\ell s}$
(See Clause 9.6.3.)

d/b	C_s	$k_{\ell s}$
≤ 1.0	—	1.0
> 1.0	≤ 10.0	1.0
> 1.0	> 10.0 but < C_k	$1 - 0.3(C_s/C_k)^4$
> 1.0	≥ C_k	$(0.70E_{05}) / (C_s^2 f_{bu})$

9.7 Shear

9.7.1 Shear resistance

The factored shear resistance, V_r, of a member of rectangular section shall be calculated as follows:

$$V_r = \phi\, k_d\, k_m\, k_{sv}\, f_{vu}\, A/1.5$$

where f_{vu} is obtained from Tables 9.12 to 9.17 and the values of k_d, k_m, and k_{sv} are as specified in Clauses 9.5.3, 9.5.6, and 9.7.2, respectively.

9.7.2 Size effect

The value of factor k_{sv} for sawn wood members shall be obtained from Table 9.4. The value of k_{sv} for glued-laminated timbers shall be $V^{-0.18}$.

9.7.3 Shear force and shear load

The factored shear resistance of a sawn member shall equal or exceed the factored shear force acting on the member (the shear effects of all loads acting within a distance from a support equal to the depth of the member need not be considered). The factored shear resistance of a glued-laminated member shall equal or exceed the factored shear load on the member, V_f, calculated as follows:

$$V_f = 0.82\left[\frac{1}{L}\int_0^L \left|V(x)\right|^5 dx\right]^{0.2}$$

where

$|V(x)|$ = absolute value of the total factored shear force at a section at distance x along the length of the member

9.7.4 Shear modulus

The value of the shear modulus shall be 0.065 times the modulus of elasticity, E_{50}, obtained from Tables 9.14 to 9.17.

9.7.5 Vertically laminated decks

Shear shall be neglected in vertically laminated decks.

9.8 Compression members

9.8.1 General

The proportioning of compression members shall satisfy the following:

$$\frac{P}{P_r} + \frac{M_c}{M_r} \leq 1.0 \text{ (for uniaxial bending)}$$

$$\frac{P}{P_r} + \frac{M_x}{M_{xr}} + \frac{M_y}{M_{yr}} \leq 1.0 \text{ (for biaxial bending)}$$

where

(a) the factored resistance in compression, P_r, is as specified in Clause 9.8.2.1;

(b) the factored resistance in flexure, M_r, is as specified in Clause 9.6.1;

(c) the factored resistances in flexure, M_{xr} and M_{yr}, for bending about the *x*- and *y*-axes, respectively, are calculated in the same manner as M_r;

(d) the amplified moment, M_c, is calculated in accordance with Clause 9.8.5.1 or 9.8.6 by taking into account the slenderness effects specified in Clause 9.8.3; and

(e) the amplified moments, M_x and M_y, acting about the *x*- and *y*- axes, respectively, are calculated in the same manner as M_c.

9.8.2 Compressive resistance parallel to grain

9.8.2.1 General

The factored compressive resistance parallel to the grain, P_r, shall be calculated as follows:

$$P_r = \phi\, k_m\, k_d\, k_{sp}\, k_c\, f_{pu}\, A$$

where k_{sp} is obtained from Clause 9.8.2.2 or 9.8.2.3, k_c is obtained from Clause 9.8.2.4, and f_{pu} is obtained from Tables 9.9 to 9.13.

9.8.2.2 Size factor for sawn wood in compression

The size factor, k_{sp}, for sawn wood in compression parallel to the grain shall be calculated as follows:

$$k_{sp} = 6.3(dL)^{-0.13} \leq 1.3$$

where

d = dimension in the direction of buckling, mm

L = unsupported length associated with the member dimension, mm

9.8.2.3 Size factor for glued-laminated timber in compression

The size factor, k_{sp}, for glued-laminated timber in compression parallel to the grain shall be calculated as follows:

$$k_{sp} = 0.68V^{-0.13} \leq 1.3$$

9.8.2.4 Slenderness factor

The slenderness factor, k_c, for members in compression parallel to the grain shall be calculated as follows:

$$k_c = \left[1 + \frac{k_m k_d k_{sp} f_{pu} C_c^3}{35 E_{05}}\right]^{-1.0}$$

where C_c is determined in accordance with Clause 9.8.3.3 and E_{05} is obtained from Tables 9.12 to 9.17.

9.8.3 Slenderness effect

9.8.3.1 Effective length

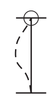
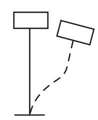
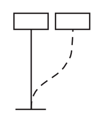
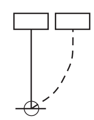
The effective length of a compression member shall be taken as kL_u and, for members other than piles, the following requirements shall apply:

(a) the unsupported length, L_u, shall be taken as the centre-to-centre distance of lateral supports capable of sustaining a lateral restraint force of at least $0.04P$, together with any other force that is generated by the effects of end moments and lateral loading;

(b) for compression members braced against side-sway, the effective length factor, k, shall be taken as 1.0 unless rigorous analysis confirms a lower value; and

(c) for compression members not braced against side-sway, the effective length factor, k, corresponding to the end-restraint condition of the member, shall be obtained from Table 9.6 or shall be determined by rigorous analysis. For the latter case, the value of k shall not be taken as less than 1.0.

Table 9.6
Minimum values of the effective length factor, *k*
(See Clauses 9.8.3.1 and 9.8.3.2.)

End restraint		Minimum value of effective length factor, *k*
Held in position and restrained against rotation at both ends		0.65
Held in position at both ends and restrained against rotation at one end		0.80
Held in position but free to rotate at both ends		1.00
Held in position and restrained against rotation at one end, and restrained against rotation, but not held in position, at the other end		1.20
Held in position and restrained against rotation at one end, and partially restrained against rotation, but not held in position, at the other end		1.50
Held in position at one end, but not restrained against rotation, and restrained against rotation, but not held in position, at the other end		2.00
Held in position and restrained against rotation at one end, but not held in position or restrained against rotation at the other end		2.00

9.8.3.2 Effective length of piles
When the finished pile projects above the ground and is not braced against buckling, the effective length shall be determined in accordance with Table 9.6 (using the value associated with the end restraint provided by the structure the pile supports) and in accordance with the following requirements:
(a) in firm ground, the lower point of contraflexure of the pile shall be taken at a depth below the ground level that is not greater than one-tenth of the exposed length of the pile;
(b) where the top stratum of the ground is soft clay or silt, the lower point of contraflexure of the pile shall be taken at a depth below the ground level that is not greater than one-half of the depth of penetration into this stratum or less than one-tenth of the exposed length of the pile; and
(c) a stratum of extremely soft soil, peat, or mud shall be treated as if it were water.

Where a pile is wholly embedded in soil, the effect of slenderness may be ignored.

9.8.3.3 Slenderness ratio

For simple compression members of constant rectangular section, the slenderness ratio, C_c, shall not exceed 50 and shall be taken as the greater of

$$C_c = \frac{\text{effective length, } kL_u\text{, associated with width}}{\text{member width}}$$

and

$$C_c = \frac{\text{effective length, } kL_u\text{, associated with depth}}{\text{member depth}}$$

For piles and other round compression members, the slenderness ratio, C_c, shall not exceed 50 and shall be calculated as follows:

$$C_c = kL_u/0.866D_{eff}$$

9.8.4 Amplified moments

At the ultimate limit state, the effect of lateral deflection in causing or amplifying bending due to axial loads shall be considered as follows:
(a) for members not braced against side-sway, when C_c is greater than 11.6; and
(b) for members braced against side-sway, when C_c is greater than $17.3 - 5.8M_1/M_2$.

9.8.5 Rigorous evaluation of amplified moments

9.8.5.1 General

When the approximate method of Clause 9.8.6 is not adopted, the amplified moment, M_c, shall be obtained by taking account of the effect of factored axial loads in amplifying the moments due to end eccentricities, bow, and lateral loads in the unsupported length, L_u. The unsupported length shall be determined in accordance with Clause 9.8.3.1 or 9.8.3.2, the end eccentricity in accordance with Clause 9.8.5.2 or 9.8.5.3, and the bow moments in accordance with Clause 9.8.5.4.

9.8.5.2 End eccentricity

All compression members, except piles, shall be analyzed for end eccentricity at each end. The eccentricity shall be taken as the greater of
(a) the eccentricity corresponding to the maximum end moment associated with the axial load; and
(b) 0.05 times the lateral dimension of the member in the plane of the flexure being considered.

The eccentricity corresponding to Item (b) shall be assumed to cause uniaxial bending with single curvature.

9.8.5.3 End eccentricity in piles

When lateral displacement of the pile butt is prevented, the moment, M_p, shall be determined at a section 0.55 times the effective length below the butt, and shall be calculated as the product of P and e_o, plus the effects of end moments and the moments due to lateral loads. The value of e_o shall be obtained from Table 9.7.

Table 9.7
A, S, and e_o* for piles at 0.55 of the effective length below the butt joint

(See Clause 9.8.5.3.)

			k_u				
D_b, mm	D_t, mm	Property	0.3	0.4	0.5	0.6	0.7
356	254	A	9.03	8.71	8.45	8.13	7.81
		S	3.82	3.62	3.44	3.28	3.10
		e_o	29	36	47	62	79
	229	A	8.77	8.45	8.06	7.68	7.35
		S	3.67	3.44	3.23	3.02	2.85
		e_o	29	35	46	60	77
	203	A	8.58	8.13	7.68	7.29	6.90
		S	3.54	3.28	3.02	2.79	2.56
		e_o	28	35	44	58	74
330	254	A	7.94	7.68	7.48	7.29	7.10
		S	3.15	3.02	2.90	2.79	2.67
		e_o	28	35	45	59	76
	229	A	7.68	7.42	7.16	6.90	6.65
		S	3.02	2.85	2.70	2.56	2.41
		e_o	28	34	44	57	73
	203	A	7.48	7.16	6.84	6.52	6.19
		S	2.90	2.70	2.52	2.34	2.18
		e_o	27	33	42	55	70
	178	A	7.29	6.90	6.52	6.13	5.74
		S	2.79	2.56	2.34	2.15	1.95
		e_o	27	32	41	53	67
305	229	A	6.71	6.52	6.32	6.13	5.94
		S	2.44	2.34	2.25	2.15	2.05
		e_o	27	33	42	54	70
	203	A	6.52	6.26	6.00	5.74	5.48
		S	2.34	2.21	2.08	1.95	1.84
		e_o	26	32	41	52	67
	178	A	6.32	6.00	5.68	5.42	5.16
		S	2.25	2.08	1.92	1.77	1.64
		e_o	26	31	39	50	64
279	203	A	5.61	5.42	5.23	5.10	4.90
		S	1.87	1.79	1.69	1.61	1.52
		e_o	26	31	39	50	64
	178	A	5.42	5.16	4.97	4.71	4.52
		S	1.77	1.67	1.56	1.46	1.36
		e_o	25	30	37	48	61
	152	A	5.24	4.95	4.67	4.41	4.16
		S	1.69	1.56	1.43	1.31	1.20
		e_o	25	29	36	46	58
254	152	A	4.41	4.21	3.99	3.81	3.61
		S	1.31	1.22	1.13	1.05	0.97
		e_o	24	28	34	43	55
229	152	A	3.61	3.52	3.37	3.25	3.11
		S	0.99	0.93	0.87	0.82	0.77
		e_o	23	27	33	41	52

*A is in mm^2 × 10^4, S is in mm^3 × 10^6, and e_o is in mm.

9.8.5.4 Bow moments

All compression members, except piles, shall be analyzed for bow moments midway between the points of lateral support due to an eccentricity, e_b, equal to $L_u/500$.

Bow moments shall be assumed to act in the same plane and with the same sense as the end moments derived from Clause 9.8.5.2.

9.8.6 Approximate evaluation of amplified moments

In the absence of a rigorous analysis, the amplified moments shall be obtained as follows:

(a) Compression members, except piles, shall be designed using the factored axial load at the ultimate limit state and a magnified moment, M_c, calculated as follows:

$$M_c = \delta M_0 \text{ (but not less than } M_2\text{)}$$

where

$$\delta = \frac{C_m}{1.0 - \dfrac{P}{\phi P_{cr}}}$$

where

$$P_{cr} = k_d k_m \frac{\pi^2 E_{05} I}{k L_u^2}$$

and k_d and k_m are obtained from Clauses 9.5.3 and 9.5.6, respectively.

(b) For members braced against side-sway and without lateral loads between supports, C_m shall be calculated as follows:

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

(c) For all cases not covered by Item (b), C_m shall be 1.0.

(d) For piles, the method specified in Item (a) shall be used, except that M_c shall be calculated as follows:

$$M_c = \delta M_p \text{ (but not less than } M_2\text{)}$$

where

$$\delta = \frac{C_m}{1.0 - \dfrac{P}{\phi P_{cr}}}$$

where

$$P_{cr} = k_d \frac{\eta E_{05} I_b}{(k L_u)^2}$$

and η is obtained from Table 9.8.

Table 9.8
η to be used in calculating P_{cr} for piles
(See Clause 9.8.6.)

		η for k_u equal to				
D_b, mm*	D_t, mm*	0.3	0.4	0.5	0.6	0.7
356	254	8.25	7.74	7.25	6.77	6.31
	229	7.87	7.25	6.66	6.09	5.55
	203	7.49	6.78	6.09	5.45	4.79
330	254	8.55	8.13	7.72	7.33	6.94
	229	8.13	7.59	7.07	6.56	6.08
	203	7.72	7.07	6.44	5.84	5.27
	178	7.33	6.56	5.84	5.18	4.52
305	229	8.44	7.99	7.55	7.13	6.71
	203	8.00	7.41	6.85	6.31	5.80
	178	7.56	6.85	6.18	5.55	4.95
279	203	8.33	7.84	7.37	6.91	6.47
	178	7.84	7.21	6.61	6.03	5.49
	152	7.36	6.61	5.89	5.22	4.59
254	152	7.64	6.96	6.82	5.70	5.12
229	152	7.99	7.41	6.85	6.82	5.80

**Within* ± 5 mm.

9.9 Tension members

The factored resistance of a tension member, T_r, shall be calculated as follows:

$$T_r = \phi\, k_d\, k_m\, k_{st}\, f_{tu}\, A$$

where k_d is as specified in Clause 9.5.3, k_{st} applies only to dimension lumber at the net section and is obtained from Table 9.9 for all species and grades, and k_m applies only at the net section and is as specified in Clause 9.5.6.

For all other cases, k_{st} and k_m shall be taken as 1.0.

Table 9.9
Size-effect factor, k_{st}, for tension at net section in dimension lumber
(See Clause 9.9.)

Depth, mm	Factor*
89	1.50
114	1.40
140	1.30
184	1.20
235	1.10
286	1.00
337	0.90
387 and larger	0.80

**Linear interpolation is permitted.*

9.10 Compression at an angle to grain

The factored resistance in bearing, R_r, for loads applied at an angle θ to the grain shall be calculated as follows:

$$R_r = \phi k_{sq} k_d \frac{A f_{pu} f_{qu}}{f_{pu} \sin^2 \phi + f_{qu} \cos^2 \phi}$$

where ϕ is 0.8, f_{pu} and f_{qu} are obtained from Tables 9.12 to 9.16, and k_d is as specified in Clause 9.5.3.

When the larger dimension or the diameter of the bearing area is less than 150 mm, no part of the bearing area is closer than 75 mm to the end of the member, and the bending moments at the bearing section do not exceed $0.4M_r$, k_{sq} shall be obtained from Table 9.10. For all other cases, k_{sq} shall be taken as 1.0.

Table 9.10
Size-effect factor for bearing, k_{sq}
(See Clause 9.10.)

Length of bearing, mm	Factor*
15	1.75
25	1.38
40	1.25
50	1.19
75	1.13
100	1.10
150 or more	1.00

**Linear interpolation is permitted.*

9.11 Sawn wood

9.11.1 Materials

9.11.1.1 Species and species combinations

Only the individual species and species combinations specified in Table 9.11 shall be used.

Table 9.11
Permitted species and species combinations for sawn wood
(See Clause 9.11.1.1.)

Species combinations	Treatable species included in species combination	Stamp identification
Douglas fir–Larch	Douglas fir	D.FIR-L(N)
Hem-Fir	Western hemlock* Amabilis fir	W.Hem(N) Am Fir(N)
Spruce-Pine-Fir	Lodgepole pine* Jack pine	L Pine(N) J Pine(N)
Northern species	Ponderosa pine Red pine Western red cedar	P Pine R Pine WR Cedar(N)

**Treatable with some difficulty.*

9.11.1.2 Grades of sawn wood

All wood shall be stress-graded in conformity with the NLGA *Standard Grading Rules for Canadian Lumber* and shall comply with CSA O141.

9.11.1.3 Identification of wood

All wood shall be identified by a grade stamp or certification of an association or independent grading agency approved by the Canadian Lumber Standards Accreditation Board as specified in CSA O141. When it is possible that preservative treatment could obscure the grade stamp, a certificate of inspection or other Approved evidence of grade shall be supplied by the treating company.

9.11.2 Specified strengths and moduli of elasticity

The specified strengths and moduli of elasticity for structural joists and planks shall be obtained from Table 9.12, for beam and stringer grades from Table 9.13, and for post and timber grades from Table 9.14.

Table 9.12
Specified strengths and moduli of elasticity for structural joists and planks, MPa

(See Clauses 9.5.5, 9.6.3, 9.8.2.1, 9.8.2.4, 9.10, 9.11.2, 9.22.5.1, 9.22.5.2, and 14.14.1.7.3.)

Species combination	Grade	Bending at extreme fibre, f_{bu}	Longitudinal shear, f_{vu}	Compression parallel to grain, f_{pu}	Compression perpendicular to grain, f_{qu}	Tension parallel to grain, f_{tu}	Modulus of elasticity	
							E_{50}	E_{05}
Douglas fir–Larch	SS	11.8	1.6	11.1	4.0	7.6	11 200	7 600
	No.1/No.2	7.1	1.6	8.2	4.0	4.1	9 800	6 300
Hem-Fir	SS	11.4	1.3	10.3	2.6	6.9	10 700	7 600
	No.1/No.2	7.9	1.3	8.7	2.6	4.4	9 800	6 700
Spruce-Pine-Fir	SS	11.8	1.2	8.5	3.0	6.1	9 400	6 700
	No.1/No.2	8.4	1.2	6.7	3.0	3.9	8 500	5 800
Northern species	SS	7.6	1.1	7.6	2.0	4.4	6 700	4 900
	No.1/No.2	5.4	1.1	6.1	2.0	2.9	6 300	4 500

Note: *These values are based on CAN/CSA-O86 ultimate strengths and the following conditions:*
(a) maximum dimension of 286 mm*;*
(b) least dimension of 89 mm *or less;*
(c) wet service conditions;
(d) standard term duration of load; and
(e) preservative treated and incised.

Table 9.13
Specified strengths and moduli of elasticity for beam and stringer grades, MPa

(See Clauses 9.5.5, 9.6.3, 9.8.2.1, 9.8.2.4, 9.10, 9.11.2, 9.23.4.4.6, 14.14.1.7.3, 16.12.2.1, 16.12.2.2, 16.12.3.1, and 16.12.3.2.)

Species combi-nation	Grade	Bending at extreme fibre, f_{bu}	Longitudinal shear, f_{vu}	Compression parallel to grain, f_{pu}	Compression perpendicular to grain, f_{qu}	Tension parallel to grain, f_{tu}	Modulus of elasticity	
							E_{50}	E_{05}
Douglas fir–Larch	SS	19.5	1.5	12.0	4.7	10.0	12 000	8 000
	No.1	15.8	1.5	10.0	4.7	7.0	12 000	8 000
	No.2	9.0	1.5	6.6	4.7	3.3	9 500	6 000
Hem-Fir	SS	14.5	1.2	9.8	3.1	7.4	10 000	7 000
	No.1	11.7	1.2	8.2	3.1	5.2	10 000	7 000
	No.2	6.7	1.2	5.4	3.1	2.4	8 000	5 500
Spruce-Pine-Fir	SS	13.6	1.2	8.6	3.6	7.0	8 500	6 000
	No.1	11.0	1.2	7.2	3.6	4.9	8 500	6 000
	No.2	6.3	1.2	4.7	3.6	2.3	6 500	4 500
Northern species	SS	12.8	1.0	6.6	2.3	6.5	8 000	5 500
	No.1	10.8	1.0	5.5	2.3	4.6	8 000	5 500
	No.2	5.9	1.0	3.5	2.3	2.2	6 000	4 000

Notes:

(1) *Beam and stringer grades have a smaller dimension of at least* 114 mm *and a larger dimension more than* 51 mm *greater than the smaller dimension.*

(2) *An approximate value for the modulus of rigidity may be estimated as 0.065 times the modulus of elasticity.*

(3) *With sawn members that are thicker than* 89 mm *and season slowly, care shall be taken to avoid overloading in compression before appreciable seasoning of the outer fibre has taken place. Alternatively, compression strengths for wet service conditions shall be used.*

(4) *The beam and stringer grades specified in this Table are not graded for continuity.*

(5) *The values in this Table are based on CAN/CSA-O86 ultimate strengths and the following conditions:*
- *(a)* 343 mm *larger dimension for bending and shear and* 292 mm *larger dimension for tension and compression parallel to grain;*
- *(b) wet service conditions; and*
- *(c) standard term duration of load.*

(6) *The specified strengths for beam and stringer grades are based on loads applied to the narrow face of the member. When beam and stringer grade members are subjected to loads applied to the wide face, the specified strength for bending at the extreme fibre and the specified modulus of elasticity shall be multiplied by the following factors:*

Grade	*Factor for f_{bu}*	*Factor for E_{50} or E_{05}*
SS	*0.88*	*1*
No. 1/No. 2	*0.77*	*0.9*

Table 9.14
Specified strengths and moduli of elasticity for post and timber grades, MPa
(See Clauses 9.5.5, 9.6.1, 9.6.3, 9.7.1, 9.7.4, 9.8.2.1, 9.8.2.4, 9.10, 9.11.2, 9.23.4.4.6, and 14.14.1.7.3.)

Species combination	Grade	Bending at extreme fibre, f_{bu}	Longitudinal shear, f_{vu}	Compression parallel to grain, f_{pu}	Compression perpendicular to grain, f_{qu}	Tension parallel to grain, f_{tu}	Modulus of elasticity	
							E_{50}	E_{05}
Douglas fir–Larch	SS	18.3	1.5	12.6	4.7	10.7	12 000	8 000
	No.1	13.8	1.5	11.1	4.7	8.1	10 500	6 500
	No.2	6.0	1.5	6.8	4.7	3.8	9 500	6 000
Hem-Fir	SS	13.6	1.2	10.3	3.1	7.9	10 000	7 000
	No.1	10.2	1.2	9.1	3.1	6.0	9 000	6 000
	No.2	4.5	1.2	5.6	3.1	2.8	8 000	5 500
Spruce-Pine-Fir	SS	12.7	1.2	9.0	3.6	7.4	8 500	6 000
	No.1	9.6	1.2	7.9	3.6	5.6	7 500	5 000
	No.2	4.2	1.2	4.9	3.6	2.6	6 500	4 500
Northern species	SS	12.0	1.0	6.8	2.3	7.0	8 000	5 500
	No.1	9.0	1.0	6.1	2.3	5.3	7 000	5 000
	No.2	3.9	1.0	3.7	2.3	2.5	6 000	4 000

Notes:

(1) *Post and timber grades have a smaller dimension of at least* 114 mm and *a larger dimension not more than* 51 mm *greater than the smaller dimension.*

(2) *Post and timber grades graded according to the rules for beam and stringer grades may be assigned beam and stringer strength.*

(3) *An approximate value for the modulus of rigidity may be estimated as 0.065 times the modulus of elasticity.*

(4) *With sawn members that are thicker than* 89 mm *and season slowly, care should be exercised to avoid overloading in compression before appreciable seasoning of the outer fibres has taken place.*

(5) *The values in this Table are based on CAN/CSA-O86 ultimate strengths and the following conditions:*

(a) 343 mm *larger dimension for bending and shear and* 292 mm *larger dimension for tension and compression parallel to grain;*

(b) *wet service conditions; and*

(c) *standard term duration of load.*

9.12 Glued-laminated timber

9.12.1 Materials

All structural glued-laminated timber shall be manufactured in accordance with CSA O122 by a plant certified in accordance with CSA O177.

9.12.2 Specified strengths and moduli of elasticity

The specified strengths and moduli of elasticity for glued-laminated Douglas fir timber shall be obtained from Table 9.15.

Table 9.15
Specified strengths and moduli of elasticity for glued-laminated Douglas fir timber, MPa
(See Clauses 9.5.5, 9.6.1, 9.6.3, 9.7.1, 9.7.4, 9.8.2.1, 9.8.2.4, 9.10, and 9.12.2.)

	CSA stress grade					
Type of stress	24f-E bending grade	24f-EX bending grade	20f-E bending grade	20f-EX bending grade	16c-E compression grade	18t-E tension grade
Bending moment positive, f_{bu}	27.5	27.5	23	23	12.6	21.9
Bending moment negative, f_{bu}	12.6	27.5	12.6	23	12.6	21.9
Longitudinal shear, f_{vu}	1.4	1.4	1.4	1.4	1.4	1.4
Compression parallel to grain, f_{pu}	26.4*	26.4*	26.4*	26.4*	26.4	26.4
Compression parallel to grain combined with bending, f_{pu}	26.4*	26.4	26.4*	26.4	26.4	26.4
Compression perpendicular to grain, f_{qu}	5.8	5.8	5.8	5.8	5.8	5.8
Axial tension at gross section, f_{tg}	13.4*	13.4	13.4*	13.4	13.4	15.7
Axial tension at net section, f_{tn}	17.9*	17.9	17.9*	17.9	17.9	20.1
Modulus of elasticity						
E_{50}	12 400	12 400	11 800	11 800	11 800	13 100
E_{05}	10 800	10 800	10 200	10 200	10 200	11 400

**The use of this stress grade for this primary application is not recommended.*

Notes:

(1) *Designers should check the availability of grades before specifying.*

(2) *The values in this Table are based on the following standard conditions:*

(a) semi-wet service conditions; and

(b) standard term duration of load.

9.12.3 Vertically laminated beams

The factored resistance in flexure for beams composed of vertical laminations shall be calculated as for load-sharing systems in sawn wood.

9.12.4 Camber

Glued-laminated beams shall be cambered by the sum of 1/600 of the span plus twice the calculated deflection due to the unfactored dead loads.

9.12.5 Varying depth

When there is a variation in the depth of a flexural member, the bevel of the laminates on the tension side shall not be steeper than 7% and the factored fibre stress shall not be less than 50% of the specified strength.

9.12.6 Curved members
The requirements of this Section shall apply only to glued-laminated members with a radius greater than 12 m. In such members the reduction in capacity due to curvature may be ignored.

9.13 Structural composite lumber

9.13.1 Materials
Structural composite lumber shall be laminated veneer lumber or parallel strand lumber manufactured from Douglas fir.

9.13.2 Specified strengths and moduli of elasticity
The specified strengths and moduli of elasticity shall be obtained from ASTM D 5456, as modified by the procedures specified in CAN/CSA-O86. Typical values for some representative products are specified in Table 9.16.

Table 9.16
Typical specified strengths and moduli of elasticity for structural composite lumber, MPa
(See Clauses 9.5.5, 9.6.1, 9.6.3, 9.7.1, 9.7.4, 9.8.2.1, 9.8.2.4, and 9.10.)

	Laminated veneer lumber	Parallel strand lumber
Type of stress		
Bending at extreme fibre, f_{bu}	32.1	33.2
Longitudinal shear — Parallel, f_{vu}*	3.3	3.3
Longitudinal shear — Perpendicular, f_{vu}*	2.0	2.4
Compression parallel to grain, f_{pu}	31.2	33.2
Compression perpendicular to grain — Parallel, f_{qu}*	8.6	8.6
Compression perpendicular to grain — Perpendicular, f_{qu}*	5.5	5.5
Axial tension parallel to grain, f_{tu}	20.0	27.5
Modulus of elasticity		
E_{50}	13 000	13 000
E_{05}	11 300	11 300

**To glueline for laminated veneer lumber and to wide face of strand for parallel strand lumber.*
Note: *These values are provided for illustrative purposes; the design values shall be obtained after verification of the structural properties and adjustment factors of the proprietary products.*

9.14 Wood piles

9.14.1 Materials
Wood pile materials shall comply with CSA CAN3-O56.

9.14.2 Splicing
Splicing of wood piles shall require Approval.

9.14.3 Specified strengths and moduli of elasticity
The specified strengths and moduli of elasticity for round wood piles shall be obtained from Table 9.17.

Table 9.17
Specified strengths and moduli of elasticity for round wood piles, MPa
(See Clauses 9.5.5, 9.6.1, 9.14.3, 9.14.4.2, 9.14.4.3, and 16.9.6.8.)

Species	Bending at extreme fibre, f_{bu}	Longitudinal shear, f_{vu}	Compression parallel to grain, f_{pu}	Compression perpendicular to grain, f_{qu}	Tension parallel to grain, f_{tu}	Modulus of elasticity	
						E_{50}	E_{05}
Douglas fir and western larch	20.1	1.4	17.0	5.2	13.6	11 000	7 000
Jack pine	18.1	1.5	14.2	3.5	11.6	7 000	5 000
Lodgepole and ponderosa pine	14.2	1.0	12.0	3.5	9.7	7 000	5 000
Red pine	13.6	1.2	10.6	3.5	9.0	7 000	5 000

Note: *These values are for wet service conditions and standard term duration of load.*

9.14.4 Design

9.14.4.1 General
In addition to meeting the requirements of Clauses 9.14.4.2 and 9.14.4.3, the design of wood piles and pile groups shall meet the requirements of Section 6.

9.14.4.2 Embedded portion
The portion of a pile permanently in contact with a soil mass that provides adequate lateral support shall be designed as a short column using the specified strengths in Table 9.17. The factored resistance of an end-bearing pile shall be calculated on the basis of the minimum cross-section. The factored resistance of a friction pile shall be calculated on the basis of the cross-section located one-third of the thickness of the supporting stratum above the tip.

9.14.4.3 Unembedded portion
The portion of a pile in contact with air, water, or a soil mass not providing adequate lateral support shall be designed as a tapered column in accordance with Clause 9.8 using the specified strengths in Table 9.17.

9.15 Fastenings

9.15.1 General
The design of fastenings shall be in accordance with CAN/CSA-O86. Glulam rivets shall not be used in bridge structures with a design life of more than two years. Truss nail plates shall not be used in bridge structures with a design life of more than two years, except as specified for wood-concrete composite decks in accordance with Clause 9.22.2.2.3.

9.15.2 Design
The design requirements and factored resistances for fastenings shall be in accordance with CAN/CSA-O86 and use the applicable modification factor for duration of load specified in Clause 9.5.3. The service condition factor for fastenings shall be determined from CAN/CSA-O86 for wood that is assumed to be seasoned at the time of fabrication and used in wet service conditions.

9.15.3 Construction
The construction details of fastenings shall be in accordance with CAN/CSA-O86.

9.16 Hardware and metalwork
All steel plates, shapes, and welded assemblies shall be designed in accordance with Section 10.

9.17 Durability

9.17.1 General
Except as specified in Clauses 9.17.2, 9.17.6, 9.17.9, and 9.17.12, or as otherwise Approved, all wood in permanent structures shall be preservative treated in accordance with the CSA O80 Series of Standards. One of the following preservatives shall be used:
(a) creosote;
(b) pentachlorophenol in Type A hydrocarbon solvent;
(c) copper naphthenate in Type A hydrocarbon solvent;
(d) chromated copper arsenate (CCA);
(e) ammoniacal copper zinc arsenate (ACZA);
(f) alkaline copper quaternary (ACQ) (if approved by Health Canada's Pest Management Regulatory Agency); or
(g) copper azole type B (CA-B) (if approved by Health Canada's Pest Management Regulatory Agency).

The net retention of preservatives shall be the minimum specified in the CSA O80 Series of Standards for the applicable conditions and wood species. Preservative treatment of laminated veneer lumber and parallel strand lumber (see Clause 9.13.1) shall be in accordance with the CSA O80 Series of Standards and Clauses 9.17.5 and 9.17.6.

All treated wood shall be substantially devoid of free surface preservative liquid and preservative deposits.

All treated wood shall be inspected by qualified personnel in accordance with the CSA O80 Series of Standards or the applicable AWPA Standards.

9.17.2 Pedestrian contact
Main structural members shall not be exposed to direct contact by pedestrians in a pedestrian walkway. For components subject to direct pedestrian contact, one of the following preservatives shall be used:
(a) chromated copper arsenate (CCA);
(b) ammoniacal copper zinc arsenate (ACZA);
(c) alkaline copper quaternary (ACQ) (if approved by Health Canada's Pest Management Regulatory Agency); or
(d) copper azole type B (CA-B) (if approved by Health Canada's Pest Management Regulatory Agency).

The net retention of preservatives shall be the minimum specified in the CSA O80 Series of Standards for the applicable conditions and wood species.

9.17.3 Incising
All sawn wood and glued-laminated members shall be incised before treatment in accordance with the CSA O80 Series of Standards. Members made of laminated veneer lumber and parallel strand lumber shall not be incised.

Glued-laminated members too large to be mechanically incised shall be incised by hand throughout the area of contact with caps, sills, or hold-down brackets in accordance with the CSA O80 Series of Standards.

The incising requirements shall be noted on the Plans.

9.17.4 Fabrication

Except for fabrication that cannot be accurately detailed before erection, all treated wood shall be cut to finished size. All surfacing, holes, notches, ring grooves, chamfering, daps, and other cuts shall be made before pressure preservative treatment.

Fabrication drawings shall detail the shape and fabrication requirements of members with the aim of eliminating or minimizing the need for field fabrication. Except when unavoidable, components shall not be cut to length in the field.

The fabrication requirements shall be noted on the Plans.

9.17.5 Pressure preservative treatment of laminated veneer lumber

Treatment shall be in accordance with CSA O80.9, with retentions as specified in Table 1 of CSA O80.9.

For the purpose of penetration sampling, three increment borer samples shall be taken from each member in a treating cylinder charge at the centreline of each side perpendicular to the veneers and approximately at the quarter-length points of the member. If a minimum of five of the six borings show preservative penetration in three outer veneers, the member shall be considered to have met the penetration requirement. Non-conforming members shall be re-treated.

9.17.6 Pressure preservative treatment of parallel strand lumber

Treatment shall be in accordance with AWPA C33.

9.17.7 Field treatment

The Plans shall specify that all cuts, bore holes, and other field fabrication exposing untreated wood surfaces shall be field treated. Creosote and copper naphthenate shall be the only permitted field preservatives. Creosote shall be the preferred preservative for structural members but shall not be used on components subject to direct pedestrian contact. Copper naphthenate may be used on field cuts of all bridge components. The instructions on the product label shall be adhered to and a minimum of two preservative coats shall be applied.

9.17.8 Treated round wood piles

Round wood piles shall be treated in accordance with CSA O80.3.

9.17.9 Untreated round wood piles

Untreated round wood piles used in permanent structures shall be clean-peeled and free from wood-destroying organisms. The cut-off shall be below a known permanent water level and the pile shall be completely embedded in soil.

9.17.10 Pile heads

After the final cut-off has been made, pile heads shall be given two saturation coats of creosote, followed by the application of a saturation coat of coal-tar pitch. There shall be an interval between applications sufficient to permit drying of each coat before the succeeding one is applied.

9.17.11 Protective treatment of hardware and metalwork

9.17.11.1 Wood treated with creosote, pentachlorophenol, or copper naphthenate

Except for nails, spikes, and sheet metal fastenings, all hardware and metalwork used in permanent structures shall be hot-dipped galvanized in accordance with CAN/CSA-G164. Nails and spikes shall be hot-dipped galvanized in accordance with CSA B111 and truss plates shall be galvanized in accordance with ASTM A 653/A 653M for the G90 coating class.

9.17.11.2 Wood treated with CCA, ACZA, ACQ, or CA-B

Because of the high copper and zinc content of this group of preservatives (particularly ACZA, ACQ, and CA-B), there is a risk of corrosion of metal items in contact with such preservatives. Accordingly, hot-dipped galvanized or (preferably) stainless steel fasteners, hardware, and metalwork are necessary. Except for nails, spikes, and sheet metal fastenings, all hardware and metalwork used in permanent structures shall be hot-dipped galvanized in accordance with CAN/CSA-G164. Nails and spikes shall be 304 or 316 stainless steel or hot-dipped galvanized to CSA B111. Sheet metal fastenings shall be 304 or 316 stainless steel or hot-dipped galvanized in accordance with ASTM A 653/A 653M for the G185 coating class.

9.17.11.3 Galvanized nuts

Galvanized nuts shall be retapped to allow for the increased diameter of the bolt due to galvanizing. Heat-treated alloy components and fastenings shall be protected by an Approved protective treatment.

9.17.12 Stress-laminated timber decking

Because of the need for dimensional stability, stress-laminated timber decking shall be treated with one of the following oil-borne preservatives:
(a) creosote;
(b) pentachlorophenol in Type A hydrocarbon solvent; or
(c) copper naphthenate in Type A hydrocarbon solvent.

Water-borne preservatives may also be used, provided that the decking is adequately sealed with an Approved product and measures are taken to ensure that prestress levels are maintained.

The net retention of preservatives shall be the minimum specified in the CSA O80 Series of Standards for the applicable conditions and wood species.

9.18 Wood cribs

9.18.1 General

Wood cribs shall be assumed to act as a unit and shall be designed to resist overturning and sliding.

Headers and stretchers shall be designed to resist the bending and shearing load effects and to provide adequate bearing.

Vertical spacing between members shall be small enough to retain the fill. The crib shall be closed-faced where an ice problem is anticipated.

9.18.2 Member sizes and assembly

Members for wood cribs shall have a minimum dimension of 184 mm. Stretchers shall be as long as practicable to achieve continuity. Joints in each tier of the crib shall be staggered with respect to joints in adjacent tiers.

9.18.3 Fastening

Members shall be connected by drift pins at least 19 mm in diameter and of sufficient length to extend completely through one tier and at least three-quarters of the way through the next member.

9.18.4 Load transfer to cribs

Load transfer from the superstructure to the top of the crib shall be effected by spreader beams or other bearing devices situated as near to the middle of the crib as possible.

9.19 Wood trestles

9.19.1 General

Piles for trestles shall be designed in accordance with Clauses 9.8 and 9.14. Tops of piles not otherwise encased shall be fitted with snug steel collars with a minimum cross-section of 5 × 75 mm. Caps, sills, and decks shall be securely fastened in accordance with Section 3 to resist uplift forces due to buoyancy.

9.19.2 Pile bents

Pile bents higher than 3.0 m shall be braced transversely in accordance with Clause 9.19.5. Longitudinal bracing shall be provided unless a detailed analysis shows that it can be omitted.

9.19.3 Framed bents

9.19.3.1 Supports

Framed bents shall be supported on piles, concrete pedestals, or, where appropriate, mudsills. All bents shall be braced transversely and longitudinally in accordance with Clause 9.19.5.

9.19.3.2 Sills

Sills shall be connected to piles or mudsills by drift pins that are at least 19 mm in diameter and extend at least 300 mm into the pile and at least 150 mm into the mudsill. Sills shall be connected to concrete pedestals by anchor bolts that are at least 19 mm in diameter and spaced at 1.8 m or less.

9.19.3.3 Post connections

Posts shall be connected to sill beams by clip angles, steel dowels, or drift pins that are at least 19 mm in diameter and extend at least 300 mm into the post and at least 150 mm into the sill.

9.19.4 Caps

Caps shall be connected to the piles or posts by steel drift pins that are at least 19 mm in diameter and extend at least 300 mm into the pile or post.

9.19.5 Bracing

9.19.5.1 Transverse bracing

Diagonal bracing shall be provided on each side of a bent and shall have a minimum cross-section of 75 × 200 mm. The bracing shall be adequately bolted to the posts, piles, caps, and sills. The bolts shall be at least 19 mm in diameter. Where multiple-storey bracing is required, horizontal bracing members of the same size as the diagonal bracing shall be placed between tiers.

9.19.5.2 Longitudinal bracing

The requirements for longitudinal bracing shall be determined from analysis. The diagonal braces shall have a minimum cross-section of 75 × 200 mm. The horizontal braces shall have a minimum cross-section of 150 × 200 mm.

9.20 Stringers and girders

9.20.1 Design details

The Plans shall specify that the stringers are to be sized to permit even bearing and to compensate for variations in stringer depths at supports.

All stringers and girders shall be securely fastened in accordance with Section 3 to resist buoyancy effects. Bolts or drift pins shall be at least 19 mm in diameter and shall extend at least 150 mm into the cap at each end of a wood stringer.

9.20.2 Diaphragms

Stringers and girders shall be provided with diaphragms at each support.

Unless otherwise Approved, intermediate diaphragms shall be provided at the midpoint for spans less than 12 m and at the one-third span point for spans 12 m or more.

Diaphragms shall be made of solid sawn wood, solid glued-laminated timber, or steel frames. Wood frame systems shall not be used.

9.21 Nail-laminated wood decks

9.21.1 General

Clauses 9.21.2 and 9.21.3 shall apply to wood decks composed of nail-laminated dimension lumber. Where the wood deck surface is exposed to traffic, the depth of the deck shall be increased by 15 mm to allow for wear.

9.21.2 Transversely laminated wood decks

9.21.2.1 General

The laminates shall be between 38 and 51 mm thick and have a minimum width of 89 mm. The difference in widths of the deck laminates shall not exceed 5 mm.

9.21.2.2 Assembly

9.21.2.2.1 Nailing

Common nails shall be used to fasten each lamination to the preceding one at intervals not exceeding 250 mm. The nails shall be driven alternately near the top and bottom edges. The nails shall be of sufficient length to pass through two laminates and at least halfway through the third. At least one nail shall be placed within 100 to 125 mm of the end of each lamination.

9.21.2.2.2 Deck support anchorage with wood stringers

Laminates shall be securely fastened to wood stringers by bolts, lag screws, lugs, or angles or by each of the laminates being toe-nailed with 100 mm nails as follows:

(a) one nail at every support for a stringer or girder spacing not exceeding 1.2 m; and
(b) two nails at every support for a stringer or girder spacing exceeding 1.2 m.

9.21.2.2.3 Deck support anchorage with steel stringers

Laminates shall be securely fastened to the top flanges of steel stringers by

(a) bolts;
(b) lag screws;
(c) plates;
(d) angles; or
(e) galvanized steel nailing clips that are least 2 mm thick (see Figure 9.1), spaced at 450 mm intervals along each side of the steel beam, and staggered by 225 mm.

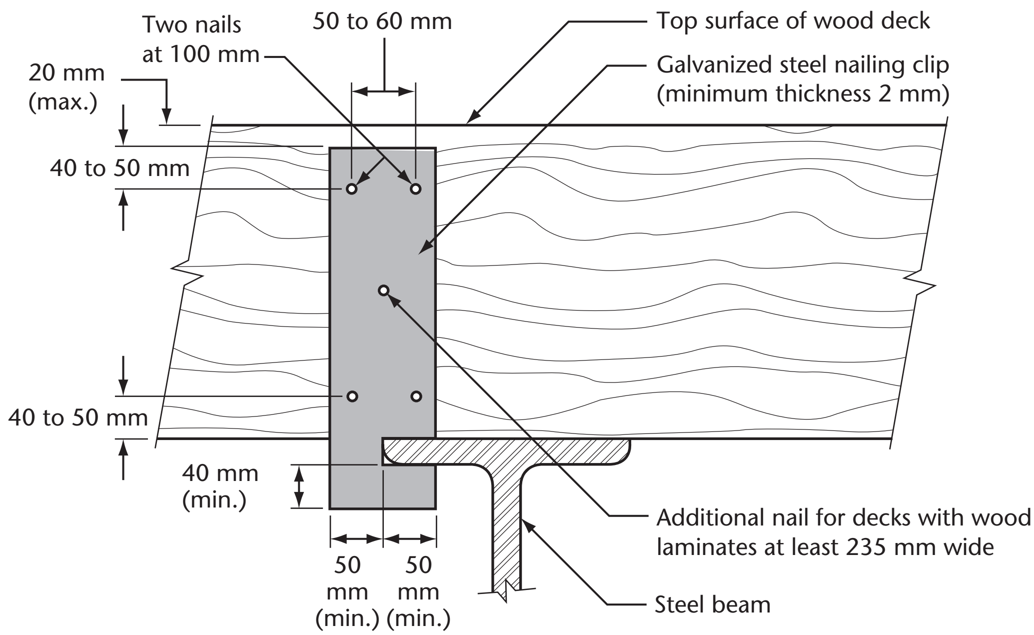


Figure 9.1
Connection of nail-laminated deck to steel beam
(See Clause 9.21.2.2.3.)

9.21.2.2.4 Laminate placement
Each laminate shall be vertical, tight against the preceding one, and bear evenly on all supports.

9.21.2.2.5 Butt joints
Butt joints shall be staggered in such a way that within any band with a width of 1.0 m measured along the laminate, a butt joint shall not occur in more than one laminate out of any three adjacent laminates.

9.21.3 Longitudinal nail-laminated wood decks
Longitudinal nail-laminated wood decks shall be used only when made composite with a concrete overlay in accordance with Clause 9.22 or when an Approved alternative method of providing load sharing among the laminates is used. Butt joints shall comply with Clause 9.21.2.2.5.

9.22 Wood-concrete composite decks

9.22.1 General
Clause 9.22.2 shall apply to nail-laminated wood decks that are longitudinally laminated and are made composite with a reinforced concrete overlay.

9.22.2 Wood base

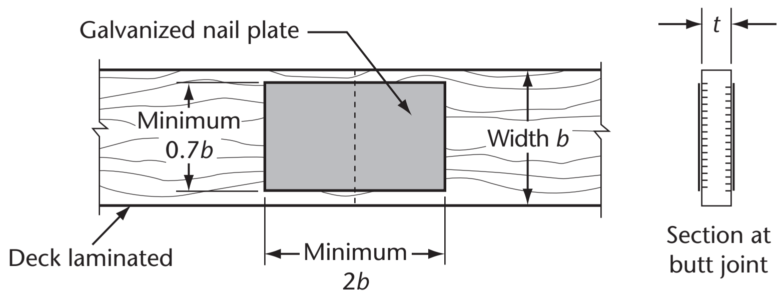
9.22.2.1 General
The wood base shall consist of longitudinally laminated dimension lumber that is 38 to 51 mm thick and 140 to 292 mm wide.

9.22.2.2 Assembly

9.22.2.2.1 General
The requirements of Clauses 9.21.2.2.1 and 9.21.2.2.4 shall apply.

9.22.2.2.2 Butt joints
Butt joints shall meet the requirements of Clause 9.21.2.2.5.

9.22.2.2.3 Spliced butt joints
Butt joints shall be provided with a connection detail as shown in Figure 9.2 or spliced in accordance with an Approved method. Steel nail plates shall be installed using a hydraulic press that applies uniform pressure or using an Approved alternative method.



Thickness of laminate, t, mm	Minimum base steel nominal thickness, mm
38–45	1.3
46–51	1.6

Figure 9.2
Spliced butt joint
(See Clause 9.22.2.2.3.)

9.22.2.2.4 Deck anchorage
The wood base shall be supported on wood-bearing members and the laminates shall be toe-nailed with 100 mm common nails as follows:
(a) one nail at
  (i) each support for each lamination that is continuous over the support; and
  (ii) each abutment;
(b) one nail in each butting lamination at joints over the supports; and
(c) additional attachment provided to account for the effects of buoyancy if the superstructure is expected to be submerged.

9.22.3 Concrete slab

9.22.3.1 Strength
The concrete shall have a minimum specified strength of 30 MPa.

9.22.3.2 Thickness
The minimum thickness of the concrete slab shall be 125 mm.

9.22.3.3 Reinforcement

The minimum reinforcement in the concrete slab shall consist of a mat of 10M bars placed at 180 mm centres in both directions. Where the deck is continuous over a support, the tensile steel shall be designed to provide the required factored flexural resistance. Additional reinforcement, when necessary, shall be placed on top of the mat. Concrete cover to the top of the deck shall be in accordance with Section 8. Concrete cover to the wood-concrete interface shall not be restricted, except that in the case of the form of construction shown in Figure 9.4, the reinforcement shall not bear directly on the wood base.

9.22.4 Wood-concrete interface

The wood base and the concrete slab shall be connected in such a manner as to prevent separation and to resist the factored horizontal shear forces between the two materials under repeated loads. This requirement shall be considered satisfied if one of the following methods is used:

(a) The wood base consists of laminates that alternate in width by at least 50 mm to form longitudinal grooves. The top surfaces of all laminates are dapped and the sides of the higher laminates are grooved as shown in Figure 9.3.

(b) The wood base consists of laminates of substantially equal width, with variations in width not exceeding 5 mm. The top surface of the laminates have transverse grooves 38 mm deep, 150 mm wide, and spaced approximately 600 mm centre to centre. Common spikes at least 50 mm longer than the width of the laminates are driven into alternate laminates to provide shear key reinforcement in accordance with Figure 9.4. Where the grooves in adjacent laminates are staggered by more than 50 mm, all laminates involved are provided with shear key reinforcing nails.

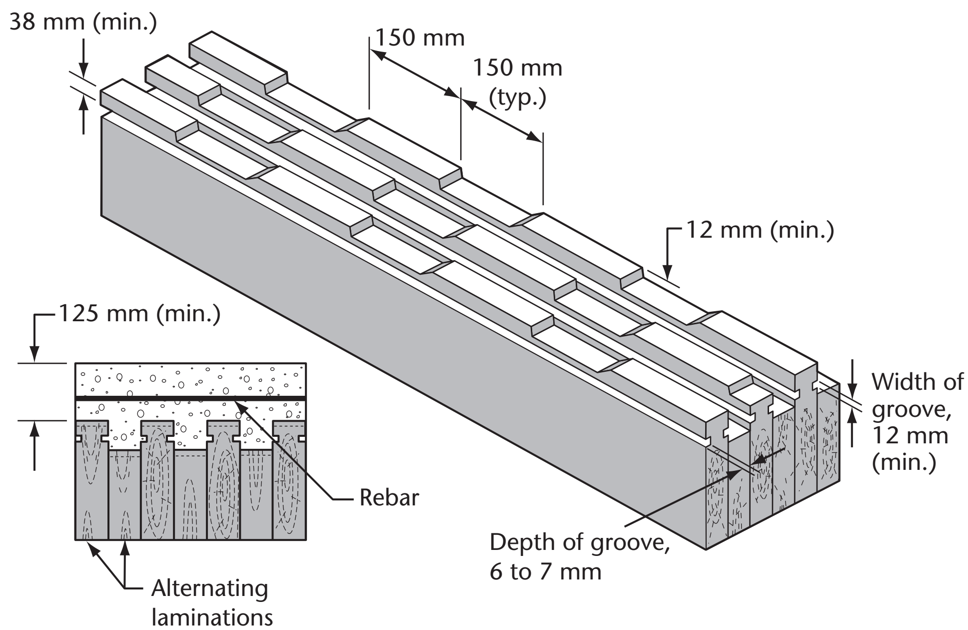


Figure 9.3
Details of wood-concrete interface
(See Clause 9.22.4.)

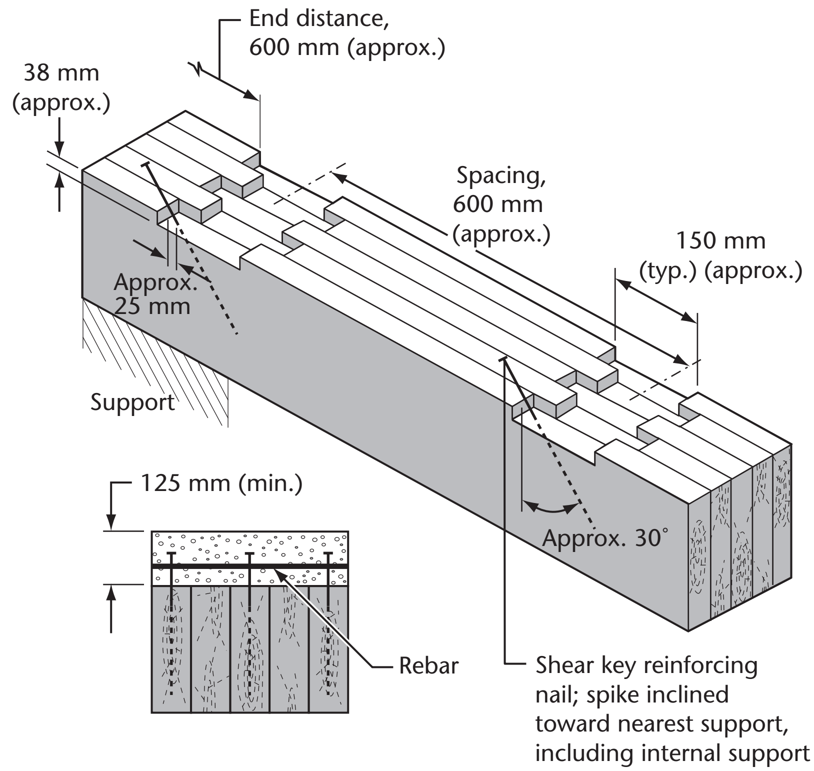


Figure 9.4
Alternative details of wood-concrete interface
(See Clauses 9.22.3.3 and 9.22.4.)

9.22.5 Factored moment resistance

9.22.5.1 General

The factored moment resistance of the composite section shall be calculated using the method of transformed sections. The modulus of elasticity for concrete, E_c, shall be obtained from Section 8, the modulus of elasticity for wood shall be E_{50} and obtained from Table 9.12, and the modulus of elasticity for steel, E_s, shall be taken as 200 000 MPa.

9.22.5.2 Factored positive moment resistance

Clause 9.22.5.1 shall be considered satisfied if the factored positive moment resistance, M_u, is calculated as follows:

$$M_u = \phi \, k_d \, k_m \, k_{sb} \, f_{bu} \, S_b$$

where ϕ is obtained from Table 9.1 for dimension lumber, f_{bu} is obtained from Table 9.12, k_d is in accordance with Clause 9.5.3, k_m is in accordance with Clause 9.5.6, and k_{sb} is obtained from Table 9.4.

The elastic section modulus, S_b, with respect to the bottom of the composite section shall be obtained by transforming the concrete into an equivalent area of wood. Only the net section of the wood base shall be considered; the capacity of spliced butt joints shall be ignored.

9.22.5.3 Factored negative moment resistance

Clause 9.22.5.1 shall be considered satisfied if the factored negative moment resistance, M_u, is taken as the smaller of

$$M_u = \phi\, k_d\, k_m\, k_{sb}\, f_{bu}\, S_b$$

and

$$M_u = \phi_s f_y \frac{E_{50} S_t}{E_s}$$

where, ϕ, f_{bu}, k_d, k_m, and k_{sb} are in accordance with Clause 9.22.5.2, ϕ_s is the resistance factor for steel in tension (see Clause 8.4.6), and f_y is the yield strength of the steel specified in Clause 10.5.

The elastic section moduli, S_b and S_t, with respect to the top and bottom of the composite section, respectively, shall be obtained by transforming the steel into an equivalent area of wood.

Where the method described in Clause 9.22.4(b) applies, only the portion of the wood below the bottom of the grooves shall be considered.

9.23 Stress-laminated wood decks

9.23.1 General

Clause 9.23.2 to 9.23.8 shall apply to vertically laminated wood decks that are post-tensioned perpendicular to the direction of laminates.

9.23.2 Post-tensioning materials

9.23.2.1 Post-tensioning steel

Post-tensioning steel shall be high-strength bars satisfying the requirements of CSA G279.

9.23.2.2 Anchorages

The dimensions and all details of the anchorages, including the details of the load distribution bulkhead, shall be subject to Approval. Anchorages shall be capable of developing 95% of the ultimate strength of the bars. After tensioning and seating, anchorages shall transmit applied loads without slippage, distortion, or other changes that would contribute to loss in bar force.

9.23.2.3 Couplers

Couplers shall be capable of developing 95% of the ultimate strength of the uncoated tendons.

9.23.2.4 Stress limitations

The stress in the post-tensioning steel shall not exceed f_{py}, $0.85f_{pu}$ at jacking, or $0.80f_{pu}$ at transfer. Where coating or galvanizing of the bar reduces the anchorage or coupler capacity, these maximum values shall be reduced accordingly.

9.23.3 Design of post-tensioning system

9.23.3.1 General

The post-tensioning system may be external or internal and shall be as shown in Figure 9.5 or 9.6.

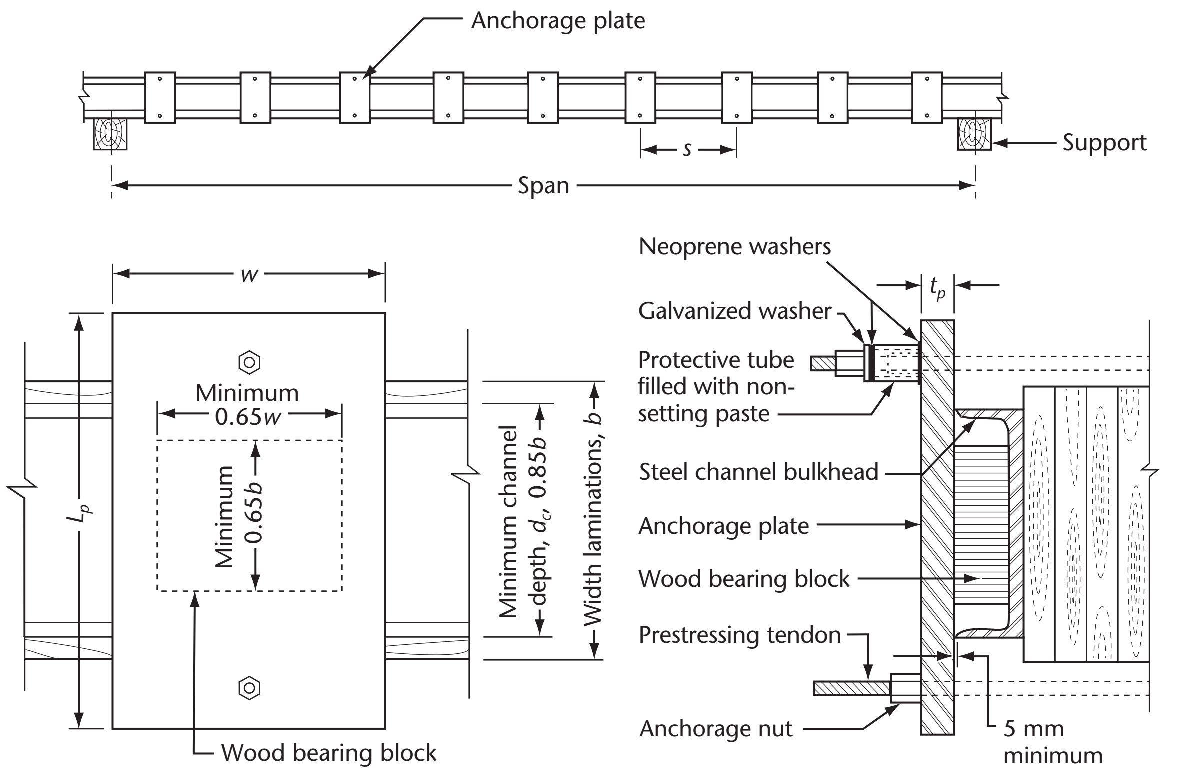


Figure 9.5
External post-tensioning system
(See Clauses 9.23.3.1, 9.23.4.4.2, 9.23.4.4.3, 9.23.4.4.6, and 16.9.3.)

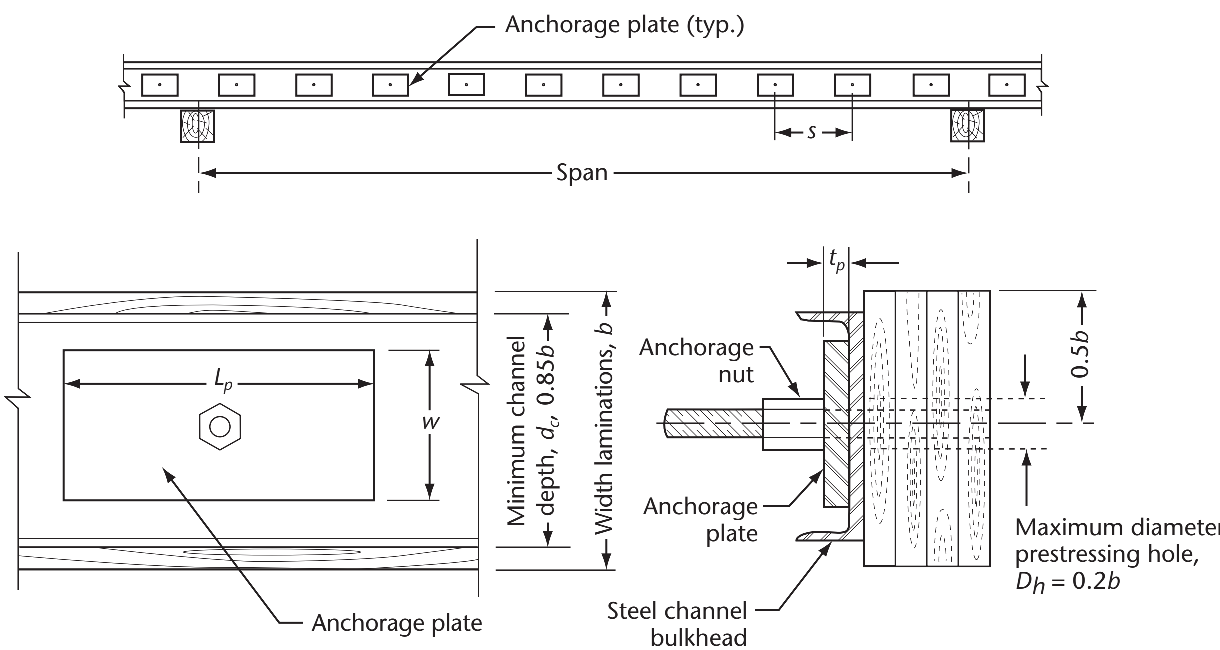


Figure 9.6
Internal post-tensioning system
(See Clauses 9.23.3.1, 9.23.4.4.2, 9.23.4.4.3, and 16.9.3.)

9.23.3.2 Steel/wood ratio

The steel/wood ratio, A_r, being A_s/A_w, shall not exceed 0.0016.

9.23.3.3 Distributed normal pressure on laminates

The maximum value of the normal pressure N_j at jacking, assumed to be uniformly distributed over an area $s \times b$, shall be $0.25f_{q\ell}$, where $f_{q\ell}$ is obtained from Table 9.18.

The minimum final pressure, N_f, assumed to be uniformly distributed, shall be taken as $0.4N_j$. N_f shall not be less than 0.35 MPa.

Table 9.18
Limiting pressure perpendicular to grain, $f_{q\ell}$, MPa
(See Clauses 9.23.3.3 and 9.23.4.2.)

Species or species combination	$f_{q\ell}$
Douglas fir or larch	6.2
Hem-Fir	5.5
Lodgepole pine	4.4
Jack pine	6.2
Red pine	6.2
White pine	4.4

9.23.3.4 Stressing procedure

The stressing of stress-laminated wood decks shall be accomplished by hydraulic jacks.

High-strength bars shall be stressed to the forces specified on the Plans. The tensioning shall be performed in the following sequence:

(a) the initial stressing, at the time of construction of the deck, shall consist of two stressing operations conducted not less than 12 h apart;
(b) the first restressing shall be conducted not less than two weeks after completion of the initial stressing; and
(c) the second restressing shall be conducted not less than four weeks after the first restressing.

The variation of the prestressing force from the specified values in each bar shall not exceed ± 5%.

The time between restressing operations shall not include any time during which the ambient temperature is below 0 °C.

9.23.4 Design of distribution bulkhead

9.23.4.1 General

Prestressed distribution bulkheads shall be of steel and shall extend along the full length of both edges of the decks.

9.23.4.2 Factored bearing resistance to post-tensioning forces

The factored resistance of the wood in bearing, R_r, due to post-tensioning forces shall be

$$R_r = \phi\, f_{q\ell}\, A_b$$

where ϕ is the value for compression perpendicular to grain obtained from Table 9.1, $f_{q\ell}$ is obtained from Table 9.18, and A_b is obtained from Clause 9.23.4.3.

The load factor for the post-tensioning force shall be taken as the maximum specified in Section 3 for secondary prestressing effects.

9.23.4.3 Bearing area for post-tensioning force

When the bulkhead satisfies the requirements of Clause 9.23.4.4, the bearing area, A_b, shall be

$$A_b = b_b L_b$$

where b_b and L_b are as specified in Clause 9.23.4.4.5.

9.23.4.4 Steel channel bulkhead

9.23.4.4.1 General

Clause 9.23.4.1 shall be considered satisfied if the distribution bulkhead consists of a steel channel and steel anchorage plates in accordance with Clauses 9.23.4.4.2 to 9.23.4.4.6.

9.23.4.4.2 Channel

The depth of the steel channel bulkheads, d_c, as shown in Figures 9.5 and 9.6 shall be at least 85% of the width of the edge lamination, *b,* but shall not exceed *b*. The minimum section properties of the channel shall be obtained from Table 9.19.

Table 9.19
Minimum section properties for steel channel bulkheads
(See Clause 9.23.4.4.2.)

Depth of laminated deck, mm	Minimum moment of inertia (about the minor axis) of the channel, mm^4	Minimum web thickness, mm
184	550 000	9.5
235	1 000 000	11.0
286	1 400 000	11.0

9.23.4.4.3 Anchorage plate

The ratio of the length, L_p, and width, *w*, of the anchorage plate shown in Figures 9.5 and 9.6 shall satisfy

$$1.0 \le \frac{L_p}{w} \le 2.0$$

The thickness of the anchorage plate, t_p, shall be not less than $L_p/12$.

9.23.4.4.4 Spacing of post-tensioning anchorages

The spacings between post-tensioning anchorages shall
(a) not exceed six times the depth of the wood deck;
(b) not be less than 2.5 times the depth of the wood deck;
(c) not exceed 1.50 m; and
(d) not be less than $15D_h$ in internal systems.

9.23.4.4.5 Effective bearing area

The width, b_b, of the direct bearing area on the edge lamination shall be taken as the height of the steel channel. The length, L_b, of the direct bearing area along the channel shall be taken as the length, L_p, of the anchorage plate plus twice the thickness of the web of the channel for an internal system, or the width, *w*, of the anchorage plate plus twice the width of the flange of the channel for an external system.

9.23.4.4.6 Web stiffening

In external post-tensioning systems, the web of the steel channel shall be stiffened beneath the anchorage plate to transmit prestress forces to the flanges and the web of the channel. This requirement shall be considered satisfied if

(a) a wood bearing block is provided in accordance with Figure 9.5, with the grain oriented parallel to the applied load;

(b) the cross-sectional dimensions of the wood bearing block are, respectively, not less than 65% of the depth of the channel and not less than 65% of the width, w, of the anchorage plate;

(c) the thickness of the wood bearing block is such that before stressing it protrudes at least 5 mm beyond the flanges of the channel; and

(d) the bearing pressure on the wood bearing block at jacking does exceed the applicable value of f_{pu} specified in Table 9.13 or 9.14.

9.23.5 Laminated decks

9.23.5.1 General

Clause 9.22.2.2.2 shall apply to all stress-laminated decks.

9.23.5.2 Lamination dimensions

Decking shall consist of laminated dimension lumber 38 to 76 mm thick, at least 184 mm wide for longitudinally laminated decks, and at least 140 mm wide for transversely laminated decks.

9.23.5.3 Holes in laminates for internal systems

The diameter of holes, D_h, drilled in laminated decking for an internal prestressing system shall not exceed 20% of the width of the laminates.

9.23.5.4 Nailing

Each laminate shall be fastened to the preceding one by nails driven in two rows, one near the top and one near the bottom edges of the laminates. The nails shall be staggered between the rows and within each row the spacing shall not exceed 500 mm. The nails shall be long enough to pass through at least two laminates, but shall not be longer than 152 mm. For power-driven nails, the specified spacing shall be adjusted in proportion to the cross-sectional area of the power nails and the same size standard spiral nails.

9.23.5.5 Support anchorage

Decks shall have a support anchorage system that can be either installed or engaged after the stressing of the post-tensioning bars specified in Clause 9.23.3.4.

During assembly, stress-laminated wood decks shall not be anchored to the supports, except as specified in Clause 9.23.5.7.

The deck support anchorage shall be designed to resist the factored force effects specified in Section 3 and shall meet the following minimum requirements:

(a) The deck shall be secured to each supporting member at intervals of not more than 1 m with the equivalent of two 19 mm diameter bolts (for decks up to 235 mm deep) or two 25 mm diameter bolts (for decks more than 235 mm deep).

(b) Where the spacing of the supporting members, measured parallel to the span of the deck, is less than 2 m, the spacing of the anchorages shall not be less than 2 m and the anchors shall be staggered by 1 m between adjacent supporting members.

9.23.5.6 Deck attachment

The deck shall not be attached to the supporting members, except as specified in Clause 9.23.5.5, until after the first restressing. When a deck requires restraint against buckling during stressing, the restraint shall not prevent free movement of the deck perpendicular to the laminates.

9.23.5.7 Transversely laminated decks

The length of decking perpendicular to the laminate length shall not exceed 40 times the width of the laminate unless restraint against buckling is provided or the deck is constructed in segments in accordance with Clause 9.23.5.8.

9.23.5.8 Segmental construction

When a deck is to be constructed in segments, each segment shall undergo the restressing specified in Clause 9.23.3.4 before being installed. The method of installation of the segments shall be such that the final assembled deck will be continuous.

When the method of installation requires the temporary release of stressing in a segment to facilitate installation, that segment shall then be stressed twice before any segments are attached to it. The first stressing shall be at the time of installation of the segment. The second stressing shall be performed not sooner than $4T$ after the first stressing, where T equals the total time for which the segment was not under stress.

9.23.6 Net section

When the factored flexural resistance of the deck is calculated, a section perpendicular to the laminates and incorporating butt joints shall be considered. For the post-tensioning bars, the effects of holes shall be ignored.

9.23.7 Hardware durability

All steel components of the post-tensioning system shall meet the requirements of Section 10. In addition, all bars of external systems shall be protected by a system equivalent to that shown in Figure 9.7. The protective tubing shall be sealed against moisture penetration by neoprene seals and shall have a collapsible connection to facilitate movement during stressing of the bars. The anchorages of internal and external systems shall be protected by a system equivalent to that shown in Figure 9.7. The anchorages of transversely laminated decks shall be protected from the effects of traffic.

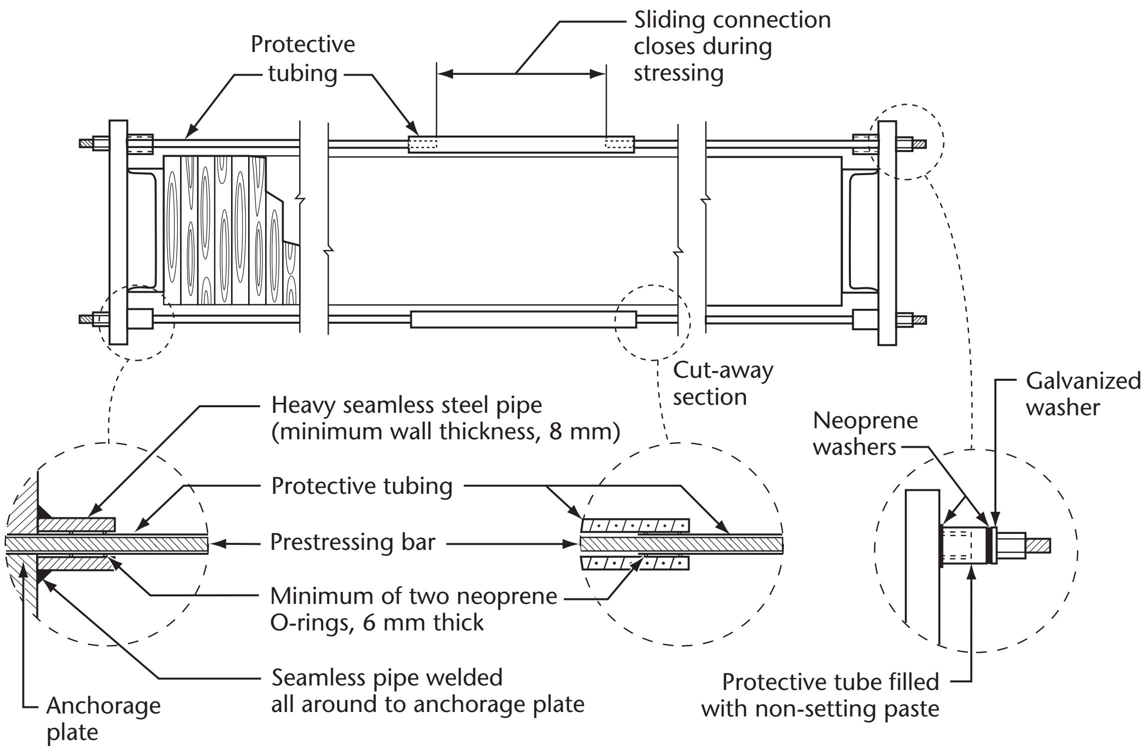


Figure 9.7
Protection for external post-tensioning system
(See Clause 9.23.7.)

9.23.8 Design details

9.23.8.1 Curbs and barriers
Curbs and barriers shall not be connected directly to the steel distribution bulkhead.

9.23.8.2 Containment of failed prestressing components
The steel distribution bulkhead shall be designed to restrain all post-tensioning components in the event of their failure. The restraint shall be removable to enable access to the anchorages.

9.24 Wearing course
A wearing course of untreated wood, plant-mix asphalt, asphalt planks, tar and chips, concrete, or an Approved material shall be used on all wood bridges other than those of wood-concrete composite construction.

9.25 Drainage

9.25.1 General
Positive drainage paths shall be provided to ensure drainage away from all primary components of the structure.

9.25.2 Deck

The crown and crossfall of the deck shall meet the requirements of Section 1. Where deck drains are not provided, the anchorage of the curb to the deck shall be such that a minimum vertical gap of 150 mm is provided between the curb and the wearing surface for an aggregate length equal to at least one-half the length of the deck.

Section 10 — Steel structures

Section 10
Steel structures

10.1 Scope

This Section specifies requirements for the design of structural steel bridges, including requirements for structural steel components, welds, bolts, and other fasteners required in fabrication and erection. Requirements related to the repeated application of loads and to fracture control and fracture toughness for primary tension and fracture-critical members are also specified.

10.2 Definitions

The following definitions apply in this Section:

Brittle fracture — a type of fracture in structural materials without prior plastic deformation that usually occurs suddenly.

Buckling load — the load at which a member or element reaches a condition of instability.

Camber — the built-in deviation of a bridge member from straight, when viewed in elevation.

Class — a designation of structural sections with regard to the width-to-thickness ratios of their constituent elements and their flexural-compressive behaviour.

Coating — an Approved protective system for steel, e.g., galvanizing, metallizing, a paint system, or coal tar epoxy.

Composite beam or girder — a steel beam or girder structurally connected to a concrete slab so that the beam and slab respond to loads as a unit.

Composite column — a column consisting of a steel tube filled with concrete, with or without internal reinforcement.

Critical net area — the area with the least tensile or tensile-shear resistance.

Element — a flat plate or plate-like component of a structural member.

Erection diagrams — drawings that show the layout and dimensions of a steel structure and from which shop details are made. They also correlate the fabricator's piece marks with locations on the structure.

Fatigue — initiation of microscopic cracks and propagation of such cracks into macroscopic cracks caused by the repeated application of load.

Fatigue limit — the level of stress range below which no fatigue crack growth is assumed to occur.

Fixed joint — a joint that allows rotation but not translation.

Flush — weld reinforcement not exceeding 1 mm in height that has a smooth, gradual transition with the surrounding plate (and involving grinding where necessary).

Fracture-critical members — members or portions of members, including attachments, in a single load path structure that are subject to tensile stress and the failure of which can lead to collapse of the structure.

Gauge — the distance between successive holes, measured at right angles to the direction of the force in the member.

Lateral torsional buckling — the buckling of a member involving lateral deflection and twisting.

Local buckling — the buckling of a plate element (as distinct from the buckling of the member as a whole).

Matching electrode — an electrode with an ultimate strength closest to and greater than the minimum specified ultimate strength of the base metal.

Notch toughness — the ability of steel to absorb tensile strain energy in the presence of a notch.

Post-buckling resistance — the ability of plate elements to resist additional load after initial elastic buckling.

Primary tension members — members or portions of members, including attachments (but not fracture-critical members or secondary components) that are subject to tensile stress.

Proposal — a constructor's submission of changes, when engineering design is required, that affects either the original design or the method of construction or shipping of a structure.

Prying action — an additional force introduced into fasteners as a result of deformation of the parts that they connect.

Single load path structure — a structure in which failure of a single structural component could lead to a total collapse.

Slenderness ratio — the effective length of a member divided by the radius of gyration, both with respect to the same axis.

Slip-critical connection — a connection where slippage cannot be tolerated, including connections subject to fatigue or to frequent load reversal or where the resulting deflections are unacceptable.

Smooth — a profile of weld reinforcement where any uneven surface has been ground away and the remaining metal profile merges gradually with the surrounding plate. In order to be regarded as smooth, weld reinforcements that remain after grinding are limited to 2 mm for plate thicknesses of 50 mm and less and 3 mm for plate thicknesses greater than 50 mm.

Snug-tight — the tightness of a bolt that is attained after a few impacts of an impact wrench or the full effort of a person using a spud wrench.

Stress range — the algebraic difference between the maximum and the minimum stresses caused by fatigue loading, where tensile stress has the opposite sign to compressive stress.

Stress range category — a category that establishes the level of stress range permitted in accordance with the classification of the detail and the number of the design stress cycles.

Tension-field action — the truss-like behaviour of a plate girder panel under shear force that develops after shear-buckling of the web and is characterized by diagonal tensile forces in the web and compressive forces in the transverse stiffeners.

Web crippling — the local failure of a web plate in the immediate vicinity of a concentrated load or reaction.

10.3 Abbreviations and symbols

10.3.1 Abbreviations

The following abbreviations apply in this Section:

CJP — complete joint penetration
FLS — fatigue limit state
PJP — partial joint penetration
SLS — serviceability limit state
ULS — ultimate limit state

10.3.2 Symbols

The following symbols apply in this Section:

A = area, mm^2

A' = area enclosed by the median line of the wall of a closed section, mm^2

A_b = cross-sectional area of a bolt, based on nominal diameter, mm^2

A_c = area of concrete in a tube pile, mm^2; transverse area of concrete between the longitudinal shear planes that define A_{cv}, mm^2

A_{ce} = area of concrete in compression in a composite column, mm^2

A_{cf} = area of compression flange of a steel section, mm^2

A_{cv} = critical area of longitudinal shear planes in the concrete slab, one on each side of the steel compression flange, extending from the point of zero moment to the point of maximum moment, mm^2

A_{de} = effective cross-sectional area of the deck, including longitudinal ribs, mm^2

A_f = area of bottom flange of box girders, including longitudinal stiffeners, mm^2; area of flanges of plate girder, mm^2

A_g = gross area, mm^2

A_m = area of fusion face, mm^2

A_n = critical net area, mm^2; total net area of a member tributary to the particular lap splice, including elements not directly connected, mm^2; tensile stress area, mm^2

A_{ne} = effective net cross-sectional area (equal to the sum of the critical net areas), mm^2

A'_{ne} = reduced effective net area of tension members accounting for shear lag effects, mm^2

A_r = area of reinforcing steel within the effective width of a concrete slab, mm^2

A_{rL} = area of longitudinal reinforcement within the concrete area A_c, mm^2

A_{rt} = area of transverse reinforcement crossing the longitudinal shear planes of A_{cv}, mm^2

A_s = area of steel section, mm^2; area of stiffener or pair of stiffeners, mm^2; tensile stress area of bolt, mm^2

A_{sc} = area of shear connector, mm^2

A'_{sc} = area of steel section in compression (see Clause 10.11.6.2.2), mm^2

A_{st} = tensile stress area, mm^2

A'_{st} = area of steel section in tension (see Clause 10.11.6.2.2), mm^2

A_w = web area or shear area, mm^2; size of effective throat area of weld, mm^2

$ADTT_f$ = single lane average daily truck traffic for fatigue

a = spacing of transverse stiffeners, mm; depth of compression block in a concrete slab, mm; transverse distance between centroids of groups of fasteners or welds connecting the batten to each main component, mm; clear distance between webs of a trough at deck level, mm

a' = the larger of e, the clear distance between stiffener troughs at deck level, and a, the clear distance between webs of a trough at deck level, mm

B = ratio in interaction equation for composite columns

B_r = factored bearing resistance of a member or component, N

B_1, B_2 = geometric coefficients for a laterally unsupported monosymmetric I-beam

b = half of width of flange of I-sections and T-sections, mm; full width of flange of channels, Z-sections, and stems of tees, mm; distance from free edge of plates to the first line of bolts or welds, mm; width of stiffener, mm; width of bottom flange plate between webs of box girder, mm

b_c = width of concrete at the neutral axis, mm (see Clause 10.9.5.5)

b_e = effective width of concrete slab, mm

b_f = width of widest flange of curved welded I-girders, mm

b_s = width of compression flange between longitudinal stiffeners, mm; distance from web to nearest longitudinal stiffener, mm

C = coefficient in formula for area of stiffener; coefficient in formula for moment resistance of unstiffened compression flanges of composite box girders

C_c = factored compressive resistance of concrete, N

C_e = Euler buckling load, N

C_{ec} = Euler buckling load of a concrete-filled hollow structural section, N

C_f = factored compressive force in a member or component at ULS, N

C_r = factored compressive resistance of a member or component, N; factored compressive resistance of steel acting at the centroid of the steel area in compression, N; factored compressive resistance of reinforcing steel, N

C_r' = factored compressive resistance of concrete area, A_c, of a column, N

C_{rc} = factored compressive resistance of a composite column, N

C_{rcm} = factored compressive resistance of composite column that can coexist with M_{rc} when all of the section is in compression, N

C_{rco} = factored compressive resistance of composite column of zero slenderness ratio, N

C_{rx} = factored compressive resistance of a member or component about the major axis, N

C_s = factored compressive force in steel of composite beam when the plastic neutral axis is in the steel section, N; coefficient in equation for moment resistance of stiffened compression flanges of composite box girders

C_w = warping torsional constant, mm^6

C_y = axial compressive force at yield stress, N

C_1, C_2 = limiting values of compressive resistance of slab, N

c_1 = coefficient related to the slip resistance of a bolted joint

D = stiffener factor; outside diameter of circular section, mm; diameter of rocker or roller, mm; weld leg size, mm

d = depth, mm; depth of beam or girder, mm; diameter of bolt or stud shear connector, mm; longitudinal distance centre-to-centre of battens, mm

d_c = depth of compression portion of web in flexure, mm

d_s' = distance from extreme compression fibre to centroid of reinforcing steel, mm

E_c = modulus of elasticity of concrete, MPa

E_s = modulus of elasticity of steel, MPa

e = edge distance, mm; lever arm between the factored compressive resistance, C_r, and the factored tensile resistance, T_r, of the steel, mm; clear distance between stiffener troughs at deck level, mm

e' = lever arm between the factored compressive resistance, C_r', and the factored tensile resistance, T_r, of the steel, mm

e_c = lever arm between the factored tensile resistance and the factored compressive resistance of the concrete, mm

e_r = lever arm between the factored tensile resistance and the factored compressive resistance of the reinforcing steel, mm

e_s = lever arm between the tensile resistance and the compressive resistance of the steel, mm

F_{cr} = shear buckling stress, MPa; buckling stress of plate in compression, MPa; lateral torsional buckling stress, MPa

F_e = critical torsional or flexural torsional elastic buckling stress, MPa

F_{ex} = elastic flexural buckling stress about the major axis, MPa

F_{ey} = elastic flexural buckling stress about the minor axis, MPa

F_{ez} = elastic torsional buckling stress, MPa

F_s = ULS shear stress, MPa

F_{sr} = fatigue stress range resistance, MPa

F_{srt} = constant amplitude threshold stress range, MPa

F_{st} = factored force in stiffener at ULS, N

F_t = tension field component of post-buckling stress, MPa

F_u = specified minimum tensile strength, MPa

F_y = specified minimum yield stress, yield point, or yield strength, MPa

F_{yc} = yield strength of a column, MPa

f_b = calculated bending stress, MPa

f_c' = specified compressive strength of concrete, MPa

f_g = axial global tensile stress in a deck induced by flexure and axial tension in the main longitudinal girders, MPa

f_s = coexisting shear stress due to warping torsion, MPa

f_{sr} = calculated FLS stress range at the detail due to passage of the CL-W Truck or of a tandem set of axles, MPa

f_{vg} = the simultaneous global shear stress in the deck, MPa

f_w = warping normal stress, MPa

f_y = specified minimum yield strength of reinforcing steel, MPa

G_s = shear modulus of elasticity of structural steel, MPa

g = transverse spacing between fastener gauge lines, mm; distance from heel of connection angle to first gauge line of bolts in outstanding legs, mm

H = coefficient for flexural torsional buckling

h = clear depth of web between flanges, mm; width of rectangular hollow section, mm; height of shear connector, mm; height of stiffener, mm; height of trough, mm

h' = length of inclined portion of a rib web, mm

h_c = clear depth of column web, mm

h_n = variable used to calculate M_{rc} of a circular hollow structural section

h_p = depth of subpanel of a girder, mm

I = moment of inertia, mm^4

I_s = moment of inertia of longitudinal compression flange stiffener, mm^4

I_t = moment of inertia of transverse compression flange stiffener, mm^4; moment of inertia of transformed section, mm^4

I_x = major axis moment of inertia, mm^4

I_y = minor axis moment of inertia of the whole cross-section, mm^4

I_{y1}, I_{y2} = moment of inertia of upper and lower flanges, respectively, about the y-axis of symmetry, mm^4

J = St. Venant torsional constant, mm^4

j = coefficient used in determining moment of inertia of stiffeners

K = effective length factor

K_x, K_y, K_z = effective length factor with respect to x-, y-, or z-axis

k = distance from outer face of flange to toe of flange-to-web fillet, mm

k_s = coefficient related to the slip resistance of a bolted joint; plate buckling coefficient

k_v = shear buckling coefficient

k_1, k_2 = buckling coefficients

L = length, mm; span length between simple connections at girder ends, mm; connection length in direction of loading, equal to the distance between the first and last bolts in bolted connections and to the overall length of the weld pattern in welded connections, mm; laterally unsupported distance from one braced location to an adjacent braced location, mm; length of roller or rocker, mm; length of cut-out in a closed cross-section member measured parallel to the longitudinal axis of the member, mm; length of a compression flange between points of lateral restraint, mm

L_c = length of channel shear connector, mm

L_n = length of segment parallel to the force, mm

ℓ = length in which warping restraint is developed, mm

M_L = bending moment in beam or girder at SLS due to live load, N•mm

M_d = bending moment in beam or girder at SLS due to dead load, N•mm

M_f = factored bending moment in member or component at ULS, N•mm

M_{fb} = factored bending moment in transverse beam at ULS, N•mm

M_{fd} = factored bending moment in beam or girder at ULS due to dead load, N•mm

M_{fl} = factored bending moment in beam or girder at ULS due to live load, N•mm

M_{fr} = factored bending moment in longitudinal rib at ULS, N•mm

M_{fsd} = factored bending moment in beam or girder at ULS due to superimposed dead load, N•mm

M_{fw} = factored bending moment in plane of a girder flange due to torsional warping, N•mm

M_{fx} = factored bending moment in member or component about the x-axis of the cross-section at ULS, N•mm

M_{fy} = factored bending moment in member or component about the y-axis of the cross-section at ULS, N•mm

M_{f1} = smaller factored end moment of beam-column at ULS, N•mm

M_{f2} = larger factored end moment of beam-column at ULS, N•mm

M_p = plastic moment resistance (= ZF_y), N•mm

M_r = factored moment resistance of member or component, N•mm

M_{rb} = factored moment resistance of transverse beam, N•mm

M_{rc} = factored moment resistance of composite column, N•mm

M_{rr} = factored moment resistance of longitudinal rib, N•mm

M_{rx} = factored moment resistance of member or component about the x-axis of the cross-section, N•mm

M'_{rx} = reduced factored moment resistance of curved non-composite I-girder, N•mm

M_{ry} = factored moment resistance of member or component about the y-axis of the cross-section, N•mm

M_{sd} = bending moment in beam or girder at SLS due to superimposed dead load, N•mm

M_u = critical elastic moment of a laterally unbraced beam, N•mm

M_y = yield moment, N•mm

m = number of faying surfaces or shear planes in a bolted joint (equal to one for bolts in single shear and two for bolts in double shear)

N = length of bearing of an applied load, mm; number of shear connectors

N_a = number of additional shear connectors per beam at point of contraflexure

N_c = specified number of design stress cycles

N_d = number of design stress cycles experienced for each passage of the design truck (see Table 10.5)

n = number of equally spaced longitudinal stiffeners in box girders; number of parallel planes of battens; number of bolts; modular ratio, E_s/E_c; number of studs arranged transversely across a flange at a given location; coefficient for axial buckling resistance

P = factored force to be transferred by shear connectors, N

p = pitch of threads, mm; pitch between bolts, mm; reduction factor for multi-lane fatigue loading

Q = moment of area, about the neutral axis of the composite section, of the transformed compressive concrete area in positive moment regions or in negative moment regions that are prestressed, mm^3; for non-prestressed sections in negative moment regions, moment of the transformed area of reinforcement embedded in the concrete, mm^3

Q_f = factored torsional moment in a member at ULS, N•mm

Q_r = factored torsional resistance, N•mm

q_r = factored shear resistance of shear connectors, N

q_{sr} = range of interface shear, N

R = radius of curvature of girder web, mm; transition radius as shown in Example 12 of Figure 10.6

R_s = vertical force for proportioning connection of transverse stiffener to longitudinal stiffener in box girders, at ULS, N

R_v = reduced normal stress factor, taking coexisting warping shear stresses into account

R_w = vertical force for proportioning connection of transverse stiffener to web in box girders, at ULS, N

R_1, R_2 = non-dimensional width-to-thickness demarcation ratios between yielding, inelastic buckling, and elastic buckling of compression flange; radius of roller or rocker and of groove of supporting plate, respectively, mm

r = radius of gyration, mm

r_c = radius of gyration of the concrete area, mm

r_x = radius of gyration of a member about its strong axis, mm

r_y = radius of gyration of a member about its weak axis, mm

r_0 = centroidal radius of gyration (see Clause 10.9.3.2), mm

S = elastic section modulus of steel section, mm^3; short-term load, N

S' = elastic modulus of composite section comprising the steel section, reinforcement, and prestressing steel within the effective width of the slab with respect to the flange or reinforcing steel under consideration, mm^3

S_e = effective section modulus, mm^3

S_h = elastic modulus of longitudinal stiffener with respect to the base of the stiffener, mm^3

S_n, S_{3n} = elastic modulus of section comprising the steel beam or girder and the concrete slab, calculated using a modular ratio of n or $3n$, respectively, mm^3

S_t = section modulus of transverse stiffener, mm^3

s = centre-to-centre spacing between successive fastener holes in the line of load, mm; centre-to-centre spacing of each group of shear studs, mm

T = tension in bolt at SLS, N; total load on column, N

T_f = factored tensile force in member or component at ULS, N

T_r = factored tensile resistance of a steel section, member, or component, of reinforcing steel, or of the effective width of a deck, including the longitudinal ribs, N

T_s = factored tensile resistance of steel section or component, N; minimum service temperature, °C

T_t = Charpy V-notch test temperature, °C

T_u = specified minimum tensile resistance, taken as follows:

(a) for parallel wire strands, the product of the sum of the areas of the individual wires and the specified minimum tensile strength of the wires, N; and

(b) for helical strands and wire ropes, the specified minimum tensile resistance established by test, taking into account the actual configuration such as socketing and bending over cable bands, N

t = thickness, mm; average thickness of channel shear connector flange, mm; thickness of flange, mm; thickness of end connection angles, mm; thickness of stiffener, mm

t_b = thickness of beam flange, mm; thickness of bottom flange, mm

t_c = thickness of concrete slab, mm; thickness of column flange, mm

t_{de} = effective thickness of deck plate, taking into account the stiffening effect of the surfacing, mm

t_r = thickness of rib, mm

t_t = thickness of top flange, mm

U = factor to account for moment gradient and for second-order effects of axial force acting on the deformed member

V = shear in a bolt or bolts at SLS, N

V_H = horizontal shear between troughs in orthotropic deck bridge due to shear, V_{LL+I}, due to live load and impact, as specified in Table 10.8, N

V_{LL+I} = shear due to live load and impact, N

V_f = factored shear force at ULS, N

V_r = factored shear resistance of member or component, N; shear range, N

V_s = slip resistance at SLS, N

V_{sr} = range of shear force at FLS resulting from passage of CL-W Truck, N

V_u = longitudinal shear in concrete slab of a composite beam, N

W = load level in CL-W, kN

w = web thickness, mm; thickness of channel shear connector web, mm; width of plate, mm

w_c = thickness of column web, mm

w_n = length of a segment, normal to a force, mm

X = curvature correction factor for transverse stiffener requirements

X_u = ultimate strength of weld metal, as rated by electrode classification number, MPa

x = subscript relating to the strong axis of a member

$\bar{x}$ = distance perpendicular to axis of member from the fastener plane to the centroid of the portion of the area of the cross-section under consideration, mm

x_0 = x-coordinate of shear centre with respect to centroid, mm

Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel

Y_o = distance from the neutral axis to the extreme outer fibre, mm

y = subscript relating to the weak axis of a member; design life, years

y_b = distance from centroid of a steel section to bottom fibre of a steel beam or girder, mm

y_b' = distance from centroid of the lower portion of a steel section under tension or compression to bottom fibre of a beam or girder, mm

y_{bc} = distance from plastic neutral axis of a composite section to bottom fibre of a steel beam or girder, mm

y_t = distance from centroid of a steel section to top fibre of a steel beam or girder, mm

y_t' = distance from centroid of the upper portion of a steel section under tension or compression to top fibre of a steel beam or girder, mm

y_{tc} = distance from plastic neutral axis of a composite section to top fibre of a steel beam or girder, mm

y_0 = y-coordinate of shear centre with respect to centroid, mm

Z = plastic section modulus of a steel section, mm^3; curvature parameter

Z_{sr} = allowable range of interface shear in an individual shear connector, N

β = value derived from a recursive equation, radians (see Clause 10.9.5.5)

β_x = coefficient of monosymmetry

γ = fatigue life constant

Δ_{DL} = camber at any point along the length of a span

Δ_m = maximum value of Δ_{DL}, mm

Δ_r = additional camber for horizontally heat-curved beams, mm

θ = angle of inclination of web plate of box girders to the vertical, degrees; angle of weld axis to line of action of force, degrees

κ = ratio of the smaller factored moment to the larger factored moment at opposite ends of an unbraced length (positive for double curvature and negative for single curvature)

λ = non-dimensional slenderness parameter in column formula; slenderness parameter

λ_c = slenderness parameter for concrete portion of composite column

λ_e = equivalent slenderness parameter

ρ = factor modifying contribution of steel to compressive resistance of composite column

ρ_b, ρ_w = curvature correction factors

τ, τ' = factors modifying contributions of steel and concrete, respectively, to compressive resistance of composite column

ϕ_b = resistance factor for bolts

ϕ_{be} = resistance factor for beam web bearing, end

ϕ_{bi} = resistance factor for beam web bearing, interior

ϕ_{br} = resistance factor for load bearing in bolted connections

ϕ_c = resistance factor for concrete

ϕ_r = resistance factor for reinforcement

ϕ_s = resistance factor for steel

ϕ_{sc} = resistance factor for shear connectors

ϕ_{tc} = resistance factor for steel cables in tension

ϕ_w = resistance factor for welds

ψ = ratio of total cross-sectional area to that of both flanges

ω_1 = coefficient used to determine equivalent uniform bending effect in beam-columns

ω_2 = coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient

10.4 Materials

10.4.1 General

Clauses 10.4.2 to 10.4.7 shall apply unless deviations from their requirements are Approved.

The fracture toughness of steel shall meet the requirements of Clause 10.23.3.

Plates provided from coils shall be used only if it can be demonstrated that the levelling process used in manufacturing produces plate with longitudinal residual stresses that are balanced about mid-thickness. In addition, after levelling, plates shall conform to the flatness tolerances specified in CSA G40.20, and the elongation and impact properties, after testing in accordance with CSA G40.20, shall be to the satisfaction of the Engineer.

10.4.2 Structural steel

Structural steel shall conform to CSA G40.21. The modulus of elasticity of structural steel, E_s, shall be taken as 200 000 MPa and the shear modulus of elasticity of structural steel, G_s, shall be taken as 77 000 MPa.

Weathering steel members shall be of Type A atmospheric corrosion-resistant steel as specified in CSA G40.21.

Fracture-critical members and primary tension members shall be of type AT, type WT, or type QT steels as specified in CSA G40.21.

10.4.3 Cast steel

Cast steel shall comply with ASTM A 27/A 27M, ASTM A 148/A 148M, or ASTM A 486/A 486M.

10.4.4 Stainless steel

Stainless steel shall comply with ASTM A 167.

10.4.5 Bolts

Bolts shall comply with ASTM A 325, ASTM A 325M, ASTM A 490, or ASTM A 490M. Bolts less than M16 or 5/8 inch in diameter shall not be used in structural applications.

10.4.6 Welding electrodes

Except as permitted by Clause 10.23.4.5, welding electrodes shall comply with CAN/CSA-W48 and shall be one of the following:

(a) carbon-steel-covered electrodes for shielded metal arc welding;
(b) low-alloy-steel-covered basic electrodes for shielded metal arc welding;
(c) solid carbon steel filler metals for gas-shielded arc welding;
(d) carbon steel electrodes for flux- and metal-cored arc welding;
(e) fluxes and solid carbon steel electrodes for submerged arc welding; or
(f) fluxes and composite carbon steel electrodes for submerged arc welding.

10.4.7 Stud shear connectors

Stud shear connectors shall comply with ASTM A 108 (Grade 1015, 1018, or 1020).

The mechanical properties of bar stock after drawing or of full-diameter finished studs, as determined by tests conducted in accordance with ASTM A 370, shall comply with the following requirements:

(a) minimum tensile strength: 410 MPa;
(b) minimum yield strength by 0.2% offset method: 350 MPa;
(c) minimum elongation in 50 mm: 20%; and
(d) minimum reduction of area: 50%.

10.4.8 Cables

10.4.8.1 Bright wire

Bright wire shall comply with ASTM A 510.

10.4.8.2 Galvanized wire
Galvanized wire shall comply with ASTM A 641/A 641M.

10.4.8.3 Bridge strand and wire rope
Bridge strand shall comply with ASTM A 586. Wire rope shall comply with ASTM A 603.

10.4.9 High-strength bars
High-strength bars shall comply with CSA G279.

10.4.10 Galvanizing and metallizing
Galvanizing shall comply with CAN/CSA-G164 and CSA G189. Zinc metallizing shall comply with CSA G189.

10.4.11 Identification

10.4.11.1 Identified steels
The specifications of the materials and products used, including type or grade if applicable, shall be identified by
(a) mill test certificates or manufacturer's certificates satisfactorily correlated to the materials or products to which they pertain; or
(b) legible markings on the material or product made by the manufacturer in accordance with the applicable material or product standard.

Otherwise, Clause 10.4.11.2 shall apply.

10.4.11.2 Unidentified steels
Structural steels not identified as specified in Clause 10.4.11.1 shall not be used unless tested by an Approved testing laboratory in accordance with CSA G40.20. The results of such testing, taking into account both mechanical and chemical properties, shall form the basis for classifying the steels as to specification. Once classified, the specified minimum values for steel at the applicable specification grade shall be used for design.

10.4.12 Coefficient of thermal expansion
The coefficient of linear thermal expansion for steel shall be taken as $12 \times 10^{-6}/°C$.

10.5 Design theory and assumptions

10.5.1 General
Structural members and components shall be proportioned to satisfy the requirements for the ultimate, serviceability, and fatigue limit states.

10.5.2 Ultimate limit states
The factored resistances specified in this Section shall be equal to or greater than the effect of factored loads specified in Section 3 for all relevant ULS considerations, including strength, rupture, bending, buckling, lateral-torsional bucking, sliding, overturning, and uplift.

10.5.3 Serviceability limit states

10.5.3.1 General
The SLS considerations shall be those of deflection, yielding, slipping of bolted joints, and vibration.

10.5.3.2 Deflection
The requirements of Clause 10.16.4 shall apply.

10.5.3.3 Yielding
Members of all classes of sections shall be proportioned so that general yielding does not occur. Localized limited yielding shall be permitted.

10.5.3.4 Slipping of bolted joints
The requirements of Clause 10.18 shall apply.

10.5.3.5 Vibration
The requirements of Section 3 shall apply.

10.5.4 Fatigue limit state
The requirements of Clause 10.17 shall apply.

10.5.5 Fracture control
The requirements of Clause 10.23 shall apply.

10.5.6 Seismic requirements
The requirements of Clause 4.8 shall apply.

10.5.7 Resistance factors
Resistance factors shall be taken as follows:
(a) flexure: $\phi_s = 0.95$;
(b) shear: $\phi_s = 0.95$;
(c) compression: $\phi_s = 0.90$;
(d) tension: $\phi_s = 0.95$;
(e) torsion: $\phi_s = 0.90$;
(f) tension in cables: $\phi_{tc} = 0.55$;
(g) reinforcing steel in composite construction: $\phi_r = 0.90$;
(h) concrete in composite construction: ϕ_c as specified in Section 8;
(i) bolts: $\phi_b = 0.80$;
(j) load bearing in bolted connections: $\phi_{br} = 0.67$;
(k) welds: $\phi_w = 0.67$;
(l) shear connectors: $\phi_{sc} = 0.85$;
(m) beam web bearing, interior: $\phi_{bi} = 0.80$; and
(n) beam web bearing, end: $\phi_{be} = 0.75$.

10.5.8 Analysis
Unless other methods are Approved, the methods of analysis used shall be as specified in this Section and Section 5. The design of supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder, or truss is connected or supported.

10.5.9 Design lengths of members

10.5.9.1 Span lengths
Span lengths shall be taken as the distance between centres of bearings or other points of support.

10.5.9.2 Compression members

10.5.9.2.1 General
The design of a compression member shall be based on its effective length, *KL.*

The unbraced length, *L*, shall be taken as the length of the compression member measured centre-to-centre of restraints.

The unbraced length may differ for different cross-sectional axes of a member. For the bottom level of a multi-level bent or for a single-level bent, *L* shall be measured from the top of the base plate.

The effective length factor, *K*, shall be as specified in Clauses 10.5.9.2.2, 10.5.9.2.3, or 10.5.9.2.4, depending on the potential failure modes and whether failure is by buckling or in-plane bending.

10.5.9.2.2 Failure modes involving in-plane bending
The effective length shall be taken as the actual unbraced length, i.e., $K = 1.0$, for beam-columns that would fail by in-plane bending, but only if, when applicable, the sway effects have been included in the analysis of the structure to determine the end moments and forces acting on the beam-columns.

10.5.9.2.3 Failure modes involving buckling
The effective length for axially loaded columns that would fail by buckling and for beam-columns that would fail by out-of-plane lateral torsional buckling shall be based on the rotational and translational restraint afforded at the ends of the unbraced length.

10.5.9.2.4 Compression members in trusses
The effective length for members that would fail by in-plane bending shall be taken as the actual unbraced length, i.e., $K = 1.0$. The effective length for members that would fail by buckling shall be based on the rotational and translational restraint afforded at the ends of the unbraced length. For half-through or pony-truss spans, the critical buckling load of the compression chord shall be determined in accordance with Clause 10.14.3.6.

10.6 Durability

10.6.1 General
The requirements of Clauses 10.6.2 to 10.6.7 shall apply unless superseded by the requirements of the Regulatory Authority.

10.6.2 Corrosion as a deterioration mechanism
The deterioration mechanisms considered for steel components shall include corrosion.

10.6.3 Corrosion protection
Corrosion protection shall be provided by alloying elements in the steel, protective coatings, or other Approved means. The type and degree of corrosion protection to be provided shall be shown on the Plans.

10.6.4 Superstructure components

10.6.4.1 General
The minimum corrosion protection shall be as specified in Table 10.1 for the applicable superstructure component and environmental exposure condition.

10.6.4.2 Structural steel

Structural steel, including diaphragms and bracing but excluding surfaces in contact with concrete and the contact surfaces of bolted joints, shall be coated with an Approved coating system for a minimum distance of 3000 mm from expansion and fixed joints.

Surfaces of girders that are subject to water runoff from the deck shall either be coated with an Approved coating system or the cross-sections shall be increased to account for the estimated loss of section over the design life of the structure.

10.6.4.3 Cables, ropes, and strands

All wires in the cables of suspension bridges and the stay cables of cable-stayed bridges shall be hot-dip galvanized. Suspension bridge and arch bridge hangers and other ropes or strands shall be hot-dip galvanized.

The completed main cables of suspension bridges shall also be treated with zinc dust paste and wrapped with soft-annealed galvanized wire.

The stay cables of cable-stayed bridges shall also be encased in a tube or sheath filled with an Approved grease or wax.

10.6.4.4 High-strength bars

When not sheathed and grouted, high-strength bars shall be hot-dip galvanized.

10.6.4.5 Steel decks

In marine environments and in areas where roadways are likely to be salted for winter maintenance, steel decks, except for open grid decks, shall be waterproofed and provided with a skid-resistant wearing surface.

10.6.5 Other components

The minimum protective measures for steel components not covered by Clauses 10.6.4.2 to 10.6.4.5 other than superstructure components shall be as specified in Table 10.2 for the applicable environmental exposure condition.

Stainless steel inserts in submerged members shall be electrically connected to the reinforcement.

Table 10.1
Corrosion protection for superstructure components
(See Clause 10.6.4.1.)

Component	Environmental exposure condition									
	No direct chlorides			Air-borne chlorides or light industrial atmosphere			Heavy industrial atmosphere			
	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry	Marine
All superstructures (minimum)	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat	Investigate	Investigate	Coat
Structure with clearance of less than 3 m over stagnant water or less than 1.5 m over fresh water	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat
Structure over depressed roadways with tunnel effect	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat
Open grid decks	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize
Structure supporting open grid decks	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat
Faying surfaces of joints	—	—	—	—	—	—	—	—	—	—
Cables, ropes, and strands (see also Clause 10.6.4.3)	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize

Table 10.2
Corrosion protection for other components
(See Clause 10.6.5.)

Component	Environmental exposure condition											
	No direct chlorides			Air-borne chlorides or light industrial atmosphere			Heavy industrial atmosphere			Marine or de-icing runoff	In fresh water	In ground-water
	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry			
Substructures	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat	Uncoated weathering steel	Uncoated weathering steel	Coated	Investigate site conditions	Investigate site conditions	Coated	Uncoated	Uncoated
Sheet piling	Coat or increase section thickness	Uncoated	Coat or increase section thickness	Coat or increase section thickness	Uncoated	Coat or increase section thickness	Coat or increase section thickness	Coat or increase section thickness	Coat or increase section thickness	Coat or increase section thickness	Uncoated	Uncoated
Light poles, luminaires, and sign support structures	Galvanize	Uncoated weathering steel	Galvanize	Galvanize	Galvanize	Galvanize	Investigate site conditions	Investigate site conditions	Investigate site conditions	Investigate site conditions	—	—
Deck drains	Galvanize	Uncoated weathering steel	Uncoated weathering steel	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	—	—
Expansion joints	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	—	—
Bearings (excluding stainless steel and faying surfaces)	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	—	—
Faying surfaces of bearing assemblies (excluding stainless steel and Teflon®)	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	—	—
Moving components or rockers, roller bearings, and pins	Grease	Grease	Grease	Grease	Grease	Grease	Grease	Grease	Grease	Grease	—	—
Railings	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	—	—
Utility supports and hardware	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	—	—
Components of mechanically stabilized earth structures, bin walls, and gabions	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize

10.6.6 Areas inaccessible after erection
Areas inaccessible after erection shall be marked in the Plans and shall be given an Approved protective coating before erection.

10.6.7 Detailing for durability

10.6.7.1 Drip bars
Drip bars shall be secured to the bottom flanges of plate girders near expansion joints.

10.6.7.2 Interior bracing
Interior bracing shall be detailed to allow access for inspection and maintenance over the full length of the bridge.

10.6.7.3 Angles and tees
Angles and tees exposed to the environment shall be placed with their vertical legs or webs extending downward wherever practical.

10.6.7.4 End floor beams and end diaphragms
End floor beams and end diaphragms under expansion joints shall be arranged to permit coating and future maintenance of surfaces that are exposed to surface runoff. The end diaphragms of box girders shall be detailed to prevent ingress of water into the boxes.

10.6.7.5 Overpasses
Girder sections of overpasses over expressways and over urban streets with traffic speed limits greater than 70 km/h shall be detailed to minimize the detrimental effects of salt spray.

10.6.7.6 Pockets and depressions
Pockets and depressions that could retain water shall be avoided, provided with effective drainage, or filled with water-repellent material.

10.7 Design detail

10.7.1 General
Members and connections shall be detailed to minimize their susceptibility to fatigue, brittle fracture, and lamellar tearing.

10.7.2 Minimum thickness of steel
The minimum thickness of steel shall be as follows:
(a) gusset plates for main members and all material in end floor beams and end diaphragms and their connections: 10 mm;
(b) closed sections, e.g., tubular members or closed ribs in orthotropic decks that are sealed against entry of moisture: 6 mm;
(c) webs of rolled shapes: 6 mm;
(d) webs of plate girders and box girders: 10 mm; and
(e) other structural steel except for fillers, railings, and components not intended to resist loads: 8 mm.

10.7.3 Floor beams and diaphragms at piers and abutments
Floor beams and diaphragms at piers and abutments shall be designed to facilitate jacking of the superstructure unless the main longitudinal members are designed to be jacked directly.

10.7.4 Camber

10.7.4.1 Design

Girders with spans 25 m long or longer shall be cambered to compensate at least for dead load deflections and to suit the highway profile grade. For composite beams, an allowance shall also be made for the effects of creep and shrinkage of the concrete.

The Plans shall show

(a) the deflection of the girders due to the dead load of the steel alone; and

(b) the deflection due to the full dead load, including that of the steel, slab, barriers, sidewalks, and wearing surface.

For spans shorter than 25 m, the deflections and the profile of the concrete deck slab over the beams may be accommodated by increasing the slab thickness over the beams in lieu of providing a camber, if specified on the Plans.

10.7.4.2 Fabrication

Shop drawings shall show the total camber diagram to be used as a web cutting profile. The camber diagram shall include compensation for the deflection due to full dead load, an allowance for fabrication and welding distortion, and an allowance (if applicable) for the vertical alignment of the highway.

10.7.4.3 Horizontally heat-curved rolled or welded beams

For rolled beams and welded I-section plate girders that are heat curved to obtain a horizontal curvature in order to compensate for the non-recoverable vertical deflection that occurs during construction and in service, the following additional camber at mid-length shall be provided:

$$\Delta_r = \frac{0.02L^2F_y}{E_sY_o}\left[\frac{305\ 000 - R}{260\ 000}\right] \geq 0$$

where

Δ_r = additional camber, mm

L = span length for simple spans, mm

= distance between the points of dead load contraflexure for continuous spans, mm

F_y = specified minimum yield stress, MPa

E_s = modulus of elasticity of steel, MPa

Y_o = distance from the neutral axis to the extreme outer fibre, mm

R = horizontal radius of curvature, mm

The additional camber shall be made proportionate to the dead load camber.

10.7.5 Welded attachments

All attachments to primary tension and fracture-critical members, including transverse and longitudinal stiffeners, shall be connected by continuous welds. Longitudinal stiffeners shall be spliced by complete joint penetration groove welds.

10.8 Tension members

10.8.1 General

10.8.1.1 Proportioning
Tension members shall be proportioned on the basis of their gross and effective net cross-sectional areas and an examination of block tearout of the material. In cases where not all portions of a cross-section are directly connected to the adjoining elements, an effective net area shall be calculated as an allowance for shear lag.

10.8.1.2 Slenderness
The slenderness ratio of a tension member shall not exceed 200 unless otherwise Approved.

10.8.1.3 Cross-sectional areas

10.8.1.3.1 General
The gross and effective net cross-sectional areas to be used in calculating the resistance of a tension member shall be as follows:

(a) The gross cross-sectional area shall be the sum of the products of the thickness times the gross width of each element in the cross-section, measured perpendicular to the longitudinal axis of the member.

(b) The effective net cross-sectional area, A_{ne}, shall be determined by summing the critical net areas, A_n, of each segment along a potential path of minimum resistance. Such potential paths of minimum resistance can extend from one side of the member to the other or can define a block of material within the member that tears out, i.e., block tearout. The net areas shall be calculated as
   (i) $A_n = w_n t$ for any segment normal to the force (i.e., in direct tension);
   (ii) $A_n = 0.6L_n t$ for any segment parallel to the force (i.e., in shear); and
   (iii) $A_n = w_n t + s^2 t/4g$ for any segment inclined to the force

   where

   w_n = net width
   = gross width – sum of hole diameters in the gross width

   L_n = net length
   = gross length – sum of hole diameters in the gross length

Deductions for fastener holes shall be made using a hole diameter 2 mm greater than the specified hole diameter.

10.8.1.3.2 Reduced effective net area accounting for shear lag effects
In general, each portion of the cross-section of a tension member shall be connected at its ends with sufficient fasteners (bolts or welds) to transmit the load attributable to the portion being connected. Where this is not practicable, an effective net area shall be calculated as

$$A'_{ne} = A_{ne}\left(1 - \bar{x}L\right)$$

where

$\bar{x}$ = distance perpendicular to axis of member from the fastener plane to the centroid of the portion of the area of the cross-section under consideration

In the absence of a more precise method, the reduced effective net area for shear lag shall be established as follows:

(a) For bolted connections where the calculated critical effective net area includes the net area of unconnected elements:
  (i) for WWF, W, M, or S shapes with flange widths at least two-thirds the depth and for structural tees cut from those shapes, when only the flanges are connected and there are three or more transverse lines of fasteners: $A'_{ne} = 0.90A_{ne}$;
  (ii) for WWF, W, M, or S shapes not specified in Item (i) and for channels, connected in each case with three or more transverse lines of fasteners: $A'_{ne} = 0.85A_{ne}$;
  (iii) for any structural shape specified in Item (ii) but connected with two transverse lines of fasteners: $A'_{ne} = 0.75A_{ne}$;
  (iv) for angles connected by one leg with four or more transverse lines of fasteners: $A'_{ne} = 0.80A_{ne}$; and
  (v) for angles connected by one leg with fewer than four transverse lines of fasteners: $A'_{ne} = 0.60A_{ne}$.

(b) For welded connections where the calculated cross-sectional area includes unconnected elements, the requirements for bolted connections specified in Item (a) may be used on the basis that the length of weld provided is the same as that of an equivalent bolted connection.

(c) When transverse welds transmit load to some, but not all, portions of the cross-section, the reduced effective net area shall be taken as the area of the connected elements.

(d) When only longitudinal welds are used to transmit load to single plates, the reduced effective net area of a plate shall be taken as follows:
  (i) $L \geq 2w$: $A'_{ne} = 1.00A_g$;
  (ii) $2w > L \geq 1.5w$: $A'_{ne} = 0.87A_g$; and
  (iii) $1.5w > L \geq w$: $A'_{ne} = 0.75A_g$.

10.8.1.4 Pin-connected members in tension

In pin-connected members in tension, the net area, A_n, across the pin hole and normal to the axis of the member shall be at least 1.33 times the cross-sectional area of the body of the member. The net area beyond the pin hole of any section on either side of the axis of the member, measured at an angle of 45° or less to the axis of the member, shall be not less than 0.9 times the cross-sectional area of the member.

The distance from the edge of the pin hole to the edge of the member, measured transverse to the axis of the member, shall not exceed four times the thickness of the material at the pin hole.

The diameter of a pin hole shall be not more than 1 mm larger than the diameter of the pin.

10.8.2 Axial tensile resistance

The factored tensile resistance, T_r, shall be taken as the least of

(a) $\phi_s A_g F_y$;

(b) $0.85\phi_s A_{ne} F_u$; and

(c) $0.85\phi_s A'_{ne} F_u$.

10.8.3 Axial tension and bending

Members subjected to bending moments and axial tensile forces shall satisfy the following relationship:

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1.0$$

where

M_r = $\phi_s M_p$ for Class 1 and 2 sections

= $\phi_s M_y$ for Class 3 sections

$$\frac{M_f}{M_r} - \frac{T_f Z}{M_r A} \leq 1.0 \text{ for Class 1 and 2 sections}$$

$$\frac{M_f}{M_r} - \frac{T_f S}{M_r A} \leq 1.0 \text{ for Class 3 sections}$$

Note: *Section classes are specified in Clause 10.9.2.1. M_r is specified in Clause 10.10.2 for Class 1 and 2 sections and in Clause 10.10.3 for Class 3 sections.*

10.8.4 Tensile resistance of cables

The factored axial tensile resistance, T_r, shall be taken as

$$T_r = \phi_{tc} T_u$$

10.9 Compression members

10.9.1 General

10.9.1.1 Cross-sectional area

Compression members shall be proportioned based on the gross area of the cross-section calculated by summing the products of the thickness and gross width of each element taken normal to the axis of the member.

10.9.1.2 Method of calculation

Provided that the requirements of Table 10.3 are met, the expressions for compressive resistance in Clause 10.9.3 shall apply. Flexural buckling with respect to the principal axes of the cross-section and torsional or flexural-torsional buckling shall be considered. Methods for calculating the compressive resistance of members, other than those specified in Clause 10.9.3, shall require Approval.

10.9.1.3 Slenderness

The slenderness ratio shall not exceed 120 for main compression members or 160 for secondary and bracing members.

10.9.2 Width-to-thickness ratio of elements in compression

10.9.2.1 General

Structural sections shall be designated as Class 1, 2, 3, or 4 depending on the width-to-thickness ratio of the elements that make up the cross-section and on the conditions of loading. A Class 1 section is one that will attain the plastic moment capacity, adjusted for the presence of axial force if necessary, and permit subsequent redistribution of bending moment. A Class 2 section is one that will attain the plastic moment capacity, adjusted for the presence of axial force if necessary, but not necessarily permit subsequent moment redistribution. A Class 3 section is one that will attain the yield moment capacity, adjusted for the presence of axial force if necessary. A Class 4 section is one in which the slenderness of the elements making up the cross-section exceeds the limits of Class 3. The capacity of a Class 4 section shall be treated on a case-by-case basis in accordance with this Code.

The width-to-thickness ratios of elements subject to compression shall not exceed the limits specified in Table 10.3.

Table 10.3
Width-to-thickness ratio of elements in compression
(See Clauses 10.9.1.2, 10.9.2.1, 10.10.2.1, and 10.10.3.1.)

Description of element	Class 1	Class 2	Class 3
Legs of angles and elements supported along one edge, except as covered elsewhere in this Table	—	—	$b/t \le 200/(\sqrt{F_y})$
Angles in continuous contact with other elements; plate girder stiffeners	—	—	$b/t \le 200/(\sqrt{F_y})$
Stems of T-sections	$b/t \le 145/(\sqrt{F_y})$*	$b/t \le 170/(\sqrt{F_y})$*	$b/t \le 340/(\sqrt{F_y})$
Flanges of I- or T-sections; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact†	$b/t \le 145/(\sqrt{F_y})$	$b/t \le 170/(\sqrt{F_y})$	$b/t \le 200/(\sqrt{F_y})$
Flanges of channels	—	—	$b/t \le 200/(\sqrt{F_y})$
Flanges of rectangular hollow structural shapes	$b/t \le 420/(\sqrt{F_y})$	$b/t \le 525/(\sqrt{F_y})$	$b/t \le 670/(\sqrt{F_y})$
Flanges of box girder sections; flange cover plates and diaphragm plates between lines of fasteners or welds	$b/t \le 525/(\sqrt{F_y})$	$b/t \le 525/(\sqrt{F_y})$	$b/t \le 670/(\sqrt{F_y})$
Perforated cover plates	—	—	$b/t \le 840/(\sqrt{F_y})$
Webs in axial compression	$h/w \le 670/(\sqrt{F_y})$	$h/w \le 670/(\sqrt{F_y})$	$h/w \le 670/(\sqrt{F_y})$
Webs in flexural compression	$h/w \le 1100/(\sqrt{F_y})$	$h/w \le 1700/(\sqrt{F_y})$	$h/w \le 1900/(\sqrt{F_y})$
Webs in combined flexural and axial compression	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left[1-0.39\frac{C_f}{C_y}\right]$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left[1-0.61\frac{C_f}{C_y}\right]$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left[1-0.65\frac{C_f}{C_y}\right]$
Circular and multi-sided hollow sections in axial compression	—	—	$\frac{D}{t} \le \frac{23\,000}{F_y}$
Circular and multi-sided hollow sections in flexural compression	$\frac{D}{t} \le \frac{13\,000}{F_y}$	$\frac{D}{t} \le \frac{18\,000}{F_y}$	$\frac{D}{t} \le \frac{66\,000}{F_y}$

**Class 1 and 2 sections subjected to flexure having an axis of symmetry in the plane of loading unless the effects of asymmetry of the section have been included in the analysis.*

†Can be considered a Class 1 or 2 section, as applicable, only if angles are continuously connected by adequate mechanical fasteners or welds and there is an axis of symmetry in the plane of loading.

10.9.2.2 Elements supported along one edge

For elements supported along only one edge that is parallel to the direction of the compressive force, the width, *b*, shall be taken as follows:

(a) for plates: the distance from the free edge to the line of bolts or welds;
(b) for legs of angles, flanges of channels and zees, or stems of tees: the full nominal dimension; and
(c) for flanges of I-shapes and tees: one-half of the flange width.

10.9.2.3 Elements supported along two edges

For elements supported along two edges that are parallel to the direction of the compressive force, the width, *b* or *h*, as applicable, shall be taken as follows:

(a) for flange or diaphragm plates in built-up sections, *b* shall be taken as the distance between adjacent lines of bolts or lines of welds;
(b) for the sides of rectangular hollow structural sections, *b* or *h* shall be taken as the clear distance between edge-supporting elements less two wall thicknesses;
(c) for webs of built-up sections, *h* shall be taken as the distance between the nearest lines of bolts connecting either edge of the web or as the clear distance between flanges when welds are used; and
(d) for webs of rolled sections, *h* shall be taken as the clear distance between flanges.

10.9.2.4 Thickness

In all cases, the thickness of elements shall be taken as the nominal thickness. For tapered flanges, the thickness shall be taken as that at the midpoint of the element.

10.9.2.5 Multi-sided hollow sections

For multi-sided hollow sections that approximate a circle, *D* shall be taken as the diameter of the circle that inscribes the outside of the midpoint of the flats of the section.

10.9.3 Axial compressive resistance

10.9.3.1 Flexural buckling

The factored axial compressive resistance, C_r, of a member conforming to the limitations specified in Clauses 10.9.1 and 10.9.2 shall be taken as

$$C_r = \phi_s A F_y (1 + \lambda^{2n})^{-1/n}$$

where

$\lambda = \dfrac{KL}{r}\sqrt{\dfrac{F_y}{\pi^2 E_s}}$

n = 1.34 for hot-rolled W-shapes, fabricated shapes, and hollow structural sections manufactured in accordance with CSA G40.20, Class C, i.e., cold-formed non-stress-relieved sections

= 2.24 for welded H-shapes with flame-cut flange edges and hollow structural sections manufactured in accordance with CSA G40.20, Class H, i.e., hot-formed or cold-formed stress-relieved sections

10.9.3.2 Torsional or flexural-torsional buckling

The torsional or flexural-torsional buckling resistance of asymmetric, singly symmetric, and cruciform sections shall be calculated by using $n = 1.34$ and replacing λ in Clause 10.9.3.1 by λ_e, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

where

(a) for cruciform sections, the critical torsional elastic buckling stress, F_e, is

$$F_e = \left[\frac{\pi^2 E_s C_w}{(K_z L)^2} + G_s J\right]\frac{1}{(I_x + I_y)}$$

(b) for sections singly symmetric about the y-axis, the critical flexural-torsional elastic buckling stress, F_e, is

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right]$$

(c) for asymmetric sections, the critical flexural-torsional elastic buckling stress, F_e, is the lowest root of

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})(x_0/r_0)^2 - F_e^2(F_e - F_{ex})(y_0/r_0)^2 = 0$$

and F_{ex} and F_{ey} are calculated with respect to the principal axes

where

K_z = the effective length factor for torsional buckling, taken as 1.0 unless a lesser value is established by rigorous analysis

F_{ey} = $\dfrac{\pi^2 E_s}{(K_y L / r_y)^2}$

F_{ez} = $\left[\dfrac{\pi^2 E_s C_w}{(K_z L)^2} + G_s J\right] / A r_0^2$

H = $1 - y_0^2 / r_0^2$ for sections singly symmetric about the y-axis

F_{ex} = $\dfrac{\pi^2 E_s}{(K_x L / r_x)^2}$

x_0, y_0 = the coordinates of the shear centre of the section with respect to the centroid

r_0^2 = $y_0^2 + r_x^2 + r_y^2$ for sections singly symmetric about the y-axis

= $x_0^2 + y_0^2 + r_x^2 + r_y^2$ for asymmetric sections

10.9.4 Axial compression and bending

10.9.4.1 Cross-sectional and member strengths — All classes of sections except Class 1 sections of I-shaped members

Members subject to coincident bending and axial compressive force shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{M_{ry}} \le 1.0$$

where

U_{1x} = the value as specified in Clause 10.9.4.2, but not less than 1.0

U_{1y} = the value as specified in Clause 10.9.4.2

The resistance of the member shall be determined by taking the following into consideration:

(a) Cross-sectional strength, for which $C_r = \phi_s A F_y$; M_{rx} and M_{ry} are defined by M_r in Clause 10.10.2.2 for Class 2 sections and Clause 10.10.3.2 for Class 3 sections, with respect to the x-axis and y-axis, respectively; and U_{1x} and U_{1y} are taken as 1.0.

(b) Overall member strength, for which C_r is as specified in Clause 10.9.3.1 and is based on the maximum slenderness ratio for biaxial bending. For uniaxial strong axis bending, $C_r = C_{rx}$. M_{rx} and M_{ry} are as specified in Item (a) and U_{1x} and U_{1y} are as specified in Clause 10.9.4.2.

(c) Lateral-torsional buckling strength, for which C_r is as specified in Clause 10.9.3 and is based on weak axis, torsional, or flexural-torsional buckling, as appropriate. M_{rx} is as specified in Clause 10.10.2.3 for Class 2 sections and Clause 10.10.3.3 for Class 3 sections. M_{ry} is as specified in Clause 10.10.2.2 for Class 2 sections and Clause 10.10.3.2 for Class 3 sections.

Note: *Item (c) does not apply to members with a closed cross-section because these members do not generally fail by lateral-torsional buckling.*

10.9.4.2 Values of U_1

In lieu of a more detailed analysis, the value of U_1 accounting for the second-order effects due to the deformation of a member between its ends shall be taken as

(a) $U_{1x} = \dfrac{\omega_{1x}}{1 - \dfrac{C_f}{C_{ex}}}$

(b) $U_{1y} = \dfrac{\omega_{1y}}{1 - \dfrac{C_f}{C_{ey}}}$

where

ω_1 = the value specified in Clause 10.9.4.3

C_e = the Euler buckling load

10.9.4.3 Values of ω_1

Unless otherwise determined by analysis, the following values shall be used for ω_1:

(a) for members not subject to transverse loads between supports: $0.6 - 0.4\kappa \geq 0.4$;

(b) for members subject to distributed loads or a series of point loads between supports: 1.0; and

(c) for members subject to a concentrated load or moment between supports: 0.85.

10.9.4.4 Member strength and stability — Class 1 sections of I-shaped members

Members required to resist coincident bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{0.60U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

where all of the terms in this expression are as specified in Clauses 10.9.4.1, 10.9.4.2, 10.10.2.2, and 10.10.2.3.

The resistance of the member shall be calculated for

(a) cross-sectional strength;

(b) overall member strength; and

(c) lateral torsional buckling strength.

In addition, the member shall meet the following requirement:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$$

where M_{rx} and M_{ry} are as specified in Clause 10.10.2.2 or 10.10.2.3, as applicable.

10.9.5 Composite columns

10.9.5.1 General
The requirements of Clause 10.9.5 shall apply to composite columns consisting of steel hollow structural sections completely filled with concrete. The type of concrete, its strength, and its other properties shall comply with Section 8.

10.9.5.2 Application
Hollow structural sections designated as Class 1, 2, or 3 sections shall be assumed to carry compressive load as composite columns. Class 4 hollow structural sections that are completely filled with concrete and are designed as composite columns shall have, for walls of rectangular sections, width-to-thickness ratios that do not exceed $1350/\sqrt{F_y}$, and for circular sections, outside diameter-to-thickness ratios of circular sections that do not exceed $28\,000/\sqrt{F_y}$.

10.9.5.3 Axial load on concrete
The axial load assumed to be carried by the concrete at the top level of a column shall be only that portion applied by direct bearing on the concrete. A base plate or similar means shall be provided for load transfer at the bottom.

10.9.5.4 Compressive resistance
The factored compressive resistance of a composite column, C_{rc}, shall be taken as

$$C_{rc} = \tau C_r + \tau' C_r'$$

where

C_r = the value specified in Clause 10.9.3.1

C_r' = $0.85\phi_c f_c' A_c \lambda_c^{-2}\left[\sqrt{1+0.25\lambda_c^{-4}} - 0.5\lambda_c^{-2}\right]$

where

λ_c = $\dfrac{KL}{r_c}\sqrt{\dfrac{f_c'}{\pi^2 E_c}}$

E_c = initial elastic modulus for concrete, taking into consideration the effects of long-term loading for normal weight concrete, with f_c' expressed in megapascals

= $2500(1+S/T)\sqrt{f_c'}$

where

S = short-term load

T = total load on the column

For all rectangular hollow structural sections, and for circular hollow structural sections with a height-to-diameter ratio of 25 or greater, the following shall apply:

(a) $\tau = \tau' = 1.0$; or

$$\tau = \frac{1}{\sqrt{1+\rho+\rho^2}}$$

$$\tau' = 1+\left[\frac{25\rho^2 t}{D/t}\right]\left[\frac{F_y}{0.85 f_c'}\right]$$

where

ρ = $0.02\,(25 - L/D)$

10.9.5.5 Bending resistance

The factored bending resistance, M_{rc}, of a composite concrete-filled hollow structural section shall be taken as

$$M_{rc} = C_r e + C_r' e'$$

where

(a) for a rectangular hollow structural section:

$$C_r = \frac{\phi_s A_s F_y - C_r'}{2}$$

$$C_r' = \phi_c a(b - 2t) f_c'$$

$$C_r + C_r' = T_r = \phi_s A_{st} F_y$$

Note: *The concrete in compression is taken to have a rectangular stress block of intensity f_c' over a depth of $a = 0.85c$, where c is the depth of concrete in compression.*

(b) for a circular hollow structural section:

$$C_r = \phi_s F_y \beta \frac{Dt}{2}$$

$$e = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta} \right]$$

$$C_r' = \phi_c f_c' \left[\frac{\beta D^2}{8} - \frac{b_c}{2} \left[\frac{D}{2} - a \right] \right]$$

$$e' = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)} \right]$$

where

β = value in radians derived from the following recursive equation:

$$\beta = \frac{\phi_s A_s F_y + 0.25\phi_c D^2 f_c' \left[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4) \right]}{0.125\phi_c D^2 f_c' + \phi_s D t F_y}$$

b_c = $D \sin(\beta/2)$

a = $b_c/2 \tan(\beta/4)$

Conservatively, M_{rc} may be taken as

$$M_{rc} = \left(Z - 2th_n^2 \right)\phi_s F_y + \left[2/3(0.5D - t)^3 - (0.5D - t)h_n^2 \right]\phi_c f_c'$$

where

$$h_n = \frac{\phi_c A_c f_c'}{2D\phi_c f_c' + 4t\left(2\phi_c F_y - \phi_c f_c'\right)}$$

Z = plastic modulus of the steel section alone

10.9.5.6 Axial compression and bending resistance

Members required to resist both bending moments and axial compression shall be proportioned analogously with Clause 10.9.4 so that

$$\frac{C_f}{C_{rc}} + \frac{B\omega_1 M_f}{M_{rc}\left[1 - \frac{C_f}{C_{ec}}\right]} \leq 1.0$$

$$\frac{M_f}{M_{rc}} \leq 1.0$$

$$B = \frac{C_{rco} - C_{rcm}}{C_{rco}}$$

where

C_{rco} = factored compressive resistance with $\lambda = 0$

C_{rcm} = factored compressive resistance that can coexist with M_{rc}

M_{rc} = the value specified in Clause 10.9.5.5

Conservatively, B may be taken as equal to 1.0.

10.10 Beams and girders

10.10.1 General

10.10.1.1 Cross-sectional area

Beams and girders shall be proportioned on the basis of the geometric properties of the gross section, except that a deduction shall be made for the area of the bolt holes exceeding 15% of the gross flange area.

10.10.1.2 Flange cover plate restrictions

Flanges of welded beams or girders shall consist of single plates or a series of plates, joined end-to-end by complete penetration groove welds. The use of welded partial-length cover plates shall require Approval.

10.10.1.3 Lateral support

Lateral support of compression flanges shall be provided by adequate connection to the deck or by bracing capable of restraining lateral displacement and twisting of the beams and girders unless it can be demonstrated that such restraint is developed between the steel beam and the concrete slab. Wood decks shall not be considered to provide lateral support unless the deck and fastenings are designed for this purpose.

10.10.1.4 Flange-to-web connections

Welds connecting flanges to webs shall be proportioned to resist interface shear due to bending combined with any loads that could be transmitted from the flange to the web other than by direct bearing.

10.10.2 Class 1 and 2 sections

10.10.2.1 Width-to-thickness ratios

Class 1 and 2 sections subject to flexure and having an axis of symmetry in the plane of loading shall meet the requirements of Clause 10.9.2. For calculating the limiting width-to-thickness ratios of the web of monosymmetric sections, h in Table 10.3 shall be replaced by $2d_c$.

10.10.2.2 Laterally supported members

When continuous lateral support is provided to the compression flange of a member subjected to bending about its major axis, the factored moment resistance, M_r, shall be calculated as

$$M_r = \phi_s Z_x F_y = \phi_s M_{px}$$

10.10.2.3 Laterally unbraced members

For a section subjected to bending about its major axis and laterally unbraced over a length, L, the factored moment resistance, M_r, shall be calculated as

(a) $M_r = 1.15\phi_s M_p \left[1 - \frac{0.28 M_p}{M_u}\right] \le \phi_s M_p$, when $M_u > 0.67M_p$; or

(b) $M_r = \phi_s M_u$, when $M_u \le 0.67M_p$

The critical elastic moment, M_u, of a monosymmetric section shall be taken as

$$M_u = \frac{\omega_2 \pi}{L}\left[\sqrt{E_s I_y G_s J}\left[B_1 + \sqrt{1 + B_2 + B_1^2}\right]\right]$$

where

ω_2 = $1.75 + 1.05\kappa + 0.3\kappa^2 \le 2.5$ for unbraced lengths subjected to end moment

= 1.0 when the bending moment at any point within the unbraced length is larger than the larger end moment or when there is no effective lateral support for the compression flange at one of the ends of the unsupported length, i.e., the free end of a cantilever

B_1 = $\pi \frac{\beta_x}{2L}\sqrt{\frac{E_s I_y}{G_s J}}$

where

β_x = coefficient of monosymmetry

B_2 = $\frac{\pi^2 E_s C_w}{L^2 G_s J}$

For doubly symmetric sections,

$\beta_x = 0.0$

$B_1 = 0.0$

so that

$$M_u = \frac{\omega_2 \pi}{L}\left[\sqrt{E_s I_y G_s J}\left[\sqrt{1 + B_2}\right]\right]$$

$$= \frac{\omega_2 \pi}{L}\sqrt{E_s I_y G_s J + \left[\frac{\pi E_s}{L}\right]^2 I_y C_w}$$

The general expression for the critical elastic moment and formulas for β_x, J, and C_w for I-girder and open-top box girders as specified in Clause C10.10.2.3 of CSA S6.1 may be used for guidance. A more rigorous analysis, taking into account both elastic and inelastic behaviour, may also be used.

10.10.2.4 Bending about the minor axis

For a section subjected to bending about its minor axis, whether laterally braced or unbraced, the factored moment resistance, M_r, shall be calculated as

$$M_r = \phi_s Z_y F_y = \phi_s M_{py}$$

10.10.3 Class 3 sections

10.10.3.1 Width-to-thickness ratios
Class 3 sections subject to flexure and having an axis of symmetry in the plane of loading shall meet the requirements of Clause 10.9.2. For calculating the limiting width-to-thickness ratios of the web of monosymmetric sections, h in Table 10.3 shall be replaced by $2d_c$.

10.10.3.2 Laterally supported members
When continuous lateral support is provided to the compression flange of a member subject to bending about its major axis, the factored moment resistance, M_r, shall be calculated as

$$M_r = \phi_s S_x F_y = \phi_s M_y$$

10.10.3.3 Laterally unbraced members
For a section subjected to bending about its major axis and laterally unbraced over a length, L, the factored moment resistance , M_r, shall be calculated as

$$M_r = 1.15\phi_s M_y \left[1 - \frac{0.28M_y}{M_u}\right] \le \phi_s M_y \text{, when } M_u > 0.67M_y$$

$$= \phi_s M_u \text{, when } M_u \le 0.67M_y$$

where

M_u = the value specified in Clause 10.10.2.3 for doubly symmetric and monosymmetric sections

10.10.3.4 Class 4 sections
For beams and girders with continuous lateral support provided to the compression flange, with webs that meet the requirements of Class 3, and whose flanges exceed the slenderness limits of Class 3, the factored moment resistances shall be computed as for a Class 3 section, except that the elastic section modulus, S, shall be replaced by an effective section modulus, S_e, determined using

(a) an effective flange width of $670t/\sqrt{F_y}$ for flanges supported along two edges; and

(b) an effective projecting flange width of $200t/\sqrt{F_y}$ for flanges supported along one edge. However, the projecting flange width shall not exceed $30t$.

Sections with flanges that meet the requirements of Class 3 may have unstiffened Class 4 webs provided that $h/w \le 150$ and the factored moment resistance is reduced by the factor specified in Clause 10.10.4.3. Sections having webs with $h/w > 150$ shall be designed as stiffened plate girders in accordance with Clause 10.10.4.

10.10.3.5 Bending about the minor axis
For a section subjected to bending about its minor axis, whether laterally braced or unbraced, the factored resistance, M_r, shall be calculated as

$$M_r = \phi_s S_y F_y = \phi_s M_{ry}$$

10.10.4 Stiffened plate girders

10.10.4.1 Width-to-thickness ratio of flanges
Stiffened plate girders shall have Class 1, 2, or 3 flanges.

10.10.4.2 Width-to-thickness ratios of webs

The width-to-thickness ratios of transversely stiffened webs without longitudinal stiffeners shall meet the requirements of Clause 10.17.2.5. When a longitudinal stiffener is provided in accordance with Clause 10.10.7, the width-to-thickness ratio, h/w, shall not exceed $6000/\sqrt{F_y}$.

10.10.4.3 Moment resistance

The factored moment resistance shall be determined in accordance with Clause 10.10.3.2 or 10.10.3.3, as applicable. For girders without longitudinal stiffeners where $2d_c/w > 1900/\sqrt{F_y}$, the moment resistance, calculated for the compression flange, shall be reduced by the following factor:

$$1-\frac{1}{300+\dfrac{1200A_{cf}}{A_w}}\left[\frac{2d_c}{w}-\frac{1900}{\sqrt{M_f/\phi_s S}}\right]$$

10.10.5 Shear resistance

10.10.5.1 Factored shear resistance

The factored shear resistance of the web of a flexural member, V_r, shall be taken as

$$V_r = \phi_s A_w F_s$$

where A_w, the shear area, is calculated using d for rolled shapes and h for fabricated or manufactured girders, and F_s, the ultimate shear stress, is equal to $F_{cr} + F_t$, where F_{cr} and F_t shall be taken as follows:

(a) when $\dfrac{h}{w} \le 502\sqrt{\dfrac{k_v}{F_y}}$:

$$F_{cr} = 0.577F_y$$

$$F_t = 0$$

(b) when $502\sqrt{\dfrac{k_v}{F_y}} < \dfrac{h}{w} \le 621\sqrt{\dfrac{k_v}{F_y}}$:

$$F_{cr} = \frac{290\sqrt{F_y k_v}}{h/w}$$

$$F_t = \left[0.5F_y - 0.866F_{cr}\right]\left[\frac{1}{\sqrt{1+(a/h)^2}}\right]$$

(c) when $\dfrac{h}{w} > 621\sqrt{\dfrac{k_v}{F_y}}$:

$$F_{cr} = \frac{180\,000 k_v}{(h/w)^2}$$

$$F_t = \left[0.5F_y - 0.866F_{cr}\right]\left[\frac{1}{\sqrt{1+(a/h)^2}}\right]$$

where

$k_v = 4 + \dfrac{5.34}{(a/h)^2}$ when $a/h < 1$

$\quad = 5.34 + \dfrac{4}{(a/h)^2}$ when $a/h \geq 1$

For unstiffened webs, a/h shall be considered infinite, so that $k_v = 5.34$.

When used, intermediate transverse stiffeners shall be spaced to suit the shear resistance determined in this Clause, except that at girder end panels and adjacent to large openings in the web, the resistance shall be calculated using $F_t = 0$ unless means are provided to anchor the tension field.

10.10.5.2 Combined shear and moment

When subject to the simultaneous action of shear and moment, transversely stiffened webs that depend on tension field action to carry shear, i.e., with $h/w > 502\sqrt{k_v/F_y}$, shall be proportioned so that

(a) $\dfrac{V_f}{V_r} \leq 1.0$;

(b) $\dfrac{M_f}{M_r} \leq 1.0$; and

(c) $0.727\dfrac{M_f}{M_r} + 0.455\dfrac{V_f}{V_r} < 1.0$

where V_r is determined in accordance with Clause 10.10.5.1 and M_r is determined in accordance with Clause 10.10.2, 10.10.3, or 10.10.4, as applicable.

10.10.6 Intermediate transverse stiffeners

10.10.6.1 General

Clause 10.10.6 shall apply to girders with intermediate transverse web stiffeners. For webs that are stiffened both transversely and longitudinally, Clause 10.10.7 shall apply.

Web stiffeners are not required when the unstiffened shear resistance, calculated in accordance with Clause 10.10.5.1, exceeds the factored shear and $h/w \leq 150$.

Transverse stiffeners, when required, shall be provided at a spacing, a, in order to develop the shear capacity. The distance between stiffeners, a, shall not exceed $67\,500h/(h/w)^2$ when h/w is greater than 150 and shall not exceed $3h$ when h/w is less than or equal to 150.

10.10.6.2 Proportioning transverse stiffeners

Intermediate transverse stiffeners provided on one or both sides of the web shall be proportioned so that

(a) $I \geq aw^3 j$ mm^4

where

$j = 2.5(h/a)^2 - 2$ but is not less than 0.5

I shall be taken about an axis at the mid-plane of the web for stiffener pairs or at the near face of the web for single stiffeners.

(b) $$A_s = \left[\frac{aw}{2}\left[1 - \frac{a/h}{\sqrt{1+(a/h)^2}}\right]\frac{V_f}{V_r}CD - 18w^2\right]Y \geq 0$$

where

V_f / V_r = the larger ratio of the two panels adjacent to the stiffener

C = $1 - \dfrac{310\ 000k_v}{F_y\,(h/w)^2}$ but is not less than 0.10

D = 1.0 for stiffeners provided in pairs

= 1.8 for single-angle stiffeners with the attached leg parallel to the web

= 2.4 for single-plate stiffeners

= 3.0 for single-angle stiffeners with the attached leg perpendicular to the web

The width of a plate used as a stiffener shall not be less than 50 mm plus $h/30$ and shall not be less than one-quarter of the full width of the flange.

The width-to-thickness ratio of intermediate transverse stiffeners shall not exceed $200/\sqrt{F_y}$ unless the section properties of the stiffeners are deemed to be based on an effective width of $200/\sqrt{F_y}$.

The projecting stiffener width shall not exceed $30t$.

10.10.6.3 Connection to web

The connection between the web and the stiffener or stiffeners shall be designed for a shear force of $0.0001hF_y^{1.5}$ N per mm of web depth, h. When the largest computed ULS shear, V_f, in adjacent panels is less than V_r as calculated in accordance with Clause 10.10.5.1, this requirement may be reduced in the proportion of V_f/V_r, but shall never be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener.

10.10.6.4 Stiffener details at flanges

The distance between the end of the stiffener weld and the near edge of the web-to-flange fillet weld shall not be less than $4w$ or more than $6w$. Transverse stiffeners need not have a snug fit with the tension flange. However, stiffeners provided on one side of the web shall have at least a snug fit against the compression flange and preferably be attached to it. Stiffeners used as connecting plates for diaphragms, cross-frames, or floor beams shall be connected by welding or bolting to both flanges. The requirements of Clause 10.17.3.2 shall also apply.

10.10.7 Longitudinal web stiffeners

10.10.7.1 General

Clause 10.10.7 shall apply to girders with both longitudinal and transverse stiffeners.

The spacing, a, of transverse stiffeners of longitudinally stiffened webs shall not exceed $1.5h_p$, where h_p is the maximum subpanel depth. The total web depth, h, shall be used in determining the shear capacity, V_r, of longitudinally stiffened girders as specified in Clause 10.10.5.1.

10.10.7.2 Proportioning

When one longitudinal stiffener is used, it shall be placed at a distance $0.2h$ from the inner surface of the compression flange for doubly symmetric sections and at $0.4d_c$ from the inner surface of the compression flange for monosymmetric sections. If more than one longitudinal stiffener is used, the design shall be based on Approved methods of analysis.

Longitudinal stiffeners shall be placed on the side of the girder web opposite to the transverse stiffeners, unless otherwise Approved.

Longitudinal stiffeners shall be proportioned so that

(a) the stiffener width-to-thickness ratio does not exceed $200/\sqrt{F_y}$ unless the section properties are deemed to be based on an effective width of $200/\sqrt{F_y}$;

(b) the projecting stiffener width is less than or equal to $30t$;

(c) $I \geq hw^3(2.4(a/h)^2 - 0.13)$; and

(d) $r \geq a\dfrac{\sqrt{F_y}}{1900}$

where I and r are calculated about a centroidal axis parallel to the web for a section comprising the stiffener or stiffeners and a strip of web $10w$ wide on each side.

10.10.7.3 Transverse stiffener requirements for longitudinally stiffened webs

Transverse stiffeners for girder panels with longitudinal stiffeners shall meet the requirements of Clause 10.10.6. In addition, the section modulus of the transverse stiffener, where I is calculated with respect to the base of the stiffener, shall not be less than $S_t = hS_h / 3a$.

When j is calculated for the purpose of calculating the required moment of inertia, I, of the transverse stiffener in accordance with Clause 10.10.6.2(a), the depth of subpanel, h_p, shall be used for calculating h/a.

When Clause 10.10.6.2(b) is applied to longitudinally stiffened girders, the depth of subpanel, h_p, shall be used for calculating a/h in the equation for A_s; the full depth of the web, h, shall be used for calculating C and V_r.

10.10.8 Bearing stiffeners

10.10.8.1 Web crippling and yielding

Bearing stiffeners shall be provided where the factored concentrated loads or reactions at the ULS exceed the factored compressive resistance of the webs of beams or girders. The factored compressive resistance of the web, B_r, shall be calculated as follows:

(a) for concentrated loads applied at a distance from the member end greater than the member depth, the lesser of

(i) $B_r = \phi_{bi} w(N + 10t)F_y$

(ii) $B_r = 1.45\phi_{bi} w^2 \sqrt{F_y E_s}$

where

ϕ_{bi} = 0.80

N = length of bearing

t = flange thickness

(b) for end reactions, the lesser of

(i) $B_r = \phi_{be} w(N + 4t)F_y$

(ii) $B_r = 0.60\phi_{be} w^2 \sqrt{F_y E_s}$

where

ϕ_{be} = 0.75

N = length of bearing

t = flange thickness

10.10.8.2 Bearing resistance and details

Bearing stiffeners shall extend the full depth of the web and shall be fitted to bear against the flange through which the loads are transmitted or be connected to the flange by welds. Stiffeners shall preferably be symmetrical about the web and extend as close to the edge of the flanges as practicable.

The width-to-thickness ratio of bearing stiffeners shall not exceed $200/\sqrt{F_y}$.

The factored bearing resistance of the bearing stiffeners, B_r, shall be calculated as

$$B_r = 1.50\phi_s A_s F_y$$

where

A_s = area of stiffener in contact with the flange

F_y = yield stress of the stiffener or flange, whichever is less

10.10.8.3 Compressive resistance

Bearing stiffeners shall be designed as compression members in accordance with Clause 10.9, assuming a column section comprising all of the projecting stiffener element plus a strip of web extending not more than $12w$ on both sides of each stiffener element. The effective column length shall be taken as not less than 0.75 times the depth of the girder. Connections shall be designed for the interface force transmitted from the web to the stiffeners.

10.10.9 Lateral bracing, cross-frames, and diaphragms

10.10.9.1 Intermediate cross-frames or diaphragms

The spacing of intermediate cross-frames or diaphragms shall be determined from an investigation of the lateral torsional buckling resistance of the longitudinal girders, the need to transfer lateral wind forces, and the need to provide torsional restraint to the girders for any anticipated applied torsional loading.

Cross-frames and diaphragms shall be designed for the lateral loads they are required to resist plus a lateral load equivalent to 1% of the compression flange force in the beam or girder at the location under consideration.

If the intermediate cross-frames or diaphragms are included in the structural model used to determine the forces in the girders, they shall be designed for the forces that they attract.

Intermediate cross-frames shall be placed normal to the main members when the supports are skewed more than 20° and shall be designed for the forces they attract.

Where girders support deck slabs proportioned in accordance with the empirical design method of Clause 8.18.4, the spacing of intermediate cross-frames, ties, or diaphragms shall satisfy the requirements of Clause 8.18.5.

10.10.9.2 Lateral Bracing

If lateral loads are not resisted by the girders alone, a lateral bracing system shall be provided at or close to either the top or bottom flanges. Bracing systems shall be designed to resist a lateral load equivalent to at least 1% of the compression flange force in the beam or girder at the location under consideration, in addition to other applied forces for the limit state under consideration. The bracing system shall have sufficient stiffness to maintain the stability of the braced flange when the system has displaced, at the location under consideration, through the distance required to develop the bracing resistance. A steel or concrete deck used for this function shall be rigidly connected to the compression flange. Timber floors shall not be considered to provide adequate lateral support unless the floor and fastenings are designed for this purpose.

Unless otherwise justified by analysis, girder spans longer than 50 m shall have a system of lateral bracing at or close to the bottom flange.

As required by Clause A5.1.6, lateral bracing systems shall be designed for the forces they attract in maintaining the compatibility of deformations of girders under vertical loading.

10.10.9.3 Pier and abutment cross-frames or diaphragms
Beam and girder bridges shall have cross-frames or diaphragms at piers and abutments, which shall be proportioned to transmit all lateral forces to the bearings. Cross-frames and diaphragms shall be as deep as practicable. Diaphragms, where practicable, shall support the end of the deck slab.

10.11 Composite beams and girders

10.11.1 General
Clause 10.11 shall apply to structures consisting of steel beams or girders and a concrete slab in which resistance to shear at the interface between the beams or girders and the slab is provided by mechanical shear connectors (including bridges that are unshored during placement of the slab). It shall apply to steel beams and girders that are both symmetric and asymmetric about the major axis. Where the beams are shored during casting of the deck, the design methods used shall be subject to Approval.

10.11.2 Proportioning
The steel section alone shall be proportioned to support all factored loads applied before the concrete strength reaches $0.75f_c'$. The lateral restraint conditions existing when the different loads are applied shall be taken into account.

The web of the steel section shall be designed to carry the total vertical shear and shall meet the requirements of Clauses 10.10.5 to 10.10.8.

The type of concrete, its strength and other properties, and provisions for control of cracking shall comply with Section 8.

The effective slab width shall be determined in accordance with Clause 5.8.2.1.

10.11.3 Effects of creep and shrinkage
To account for the effect of creep due to that portion of dead load that is applied after the concrete strength has reached $0.75f_c'$, and in lieu of more detailed calculations, a modular ratio of $3n$ shall be used in calculating the section properties.

For the SLS, a differential shrinkage strain corresponding to the difference between the restrained and the free shrinkage of the concrete shall be considered in the design.

10.11.4 Control of permanent deflections
For composite beams and girders, the normal stress in either flange of the steel section due to serviceability dead and live loads shall not exceed $0.90\ F_y$. The following requirements shall also be satisfied:

(a) in positive moment regions:

$$\frac{M_d}{S} + \frac{M_{sd}}{S_{3n}} + \frac{M_L}{S_n} \le 0.90F_y$$

(b) in negative moment regions:

$$\frac{M_d}{S} + \frac{M_{sd} + M_L}{S'} \le 0.90F_y$$

10.11.5 Class 1 and Class 2 sections

10.11.5.1 General
The portions of the steel section in compression shall comply with Clause 10.9.2.

10.11.5.2 Positive moment regions

10.11.5.2.1 Stress distribution
The factored moment resistance of the section in bending shall be calculated using a fully plastic stress distribution, as shown in Figure 10.1.

10.11.5.2.2 Compressive resistance of concrete
The factored compressive resistance of the slab used to calculate the factored resistance of the section shall be the smaller of C_1 and C_2, calculated as follows:

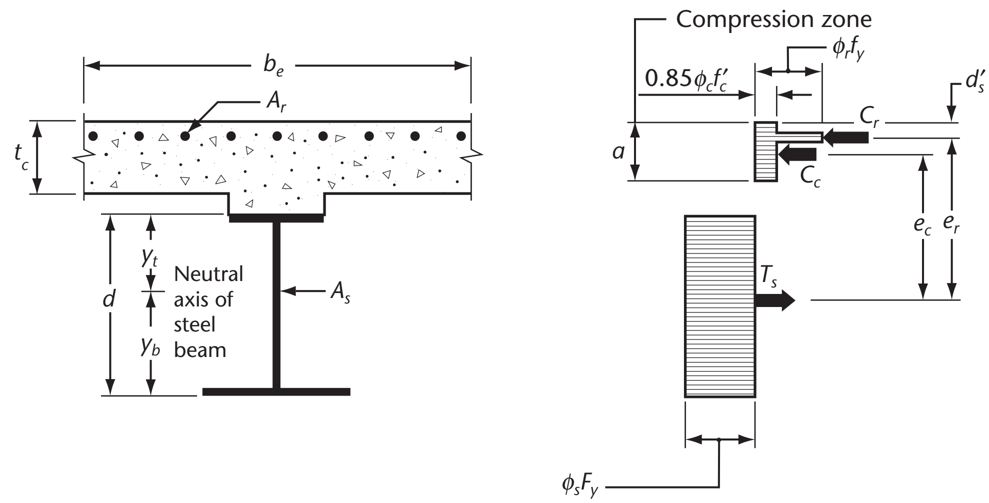
$$C_1 = C_c + C_r$$

$$C_2 = \phi_s A_s F_y$$

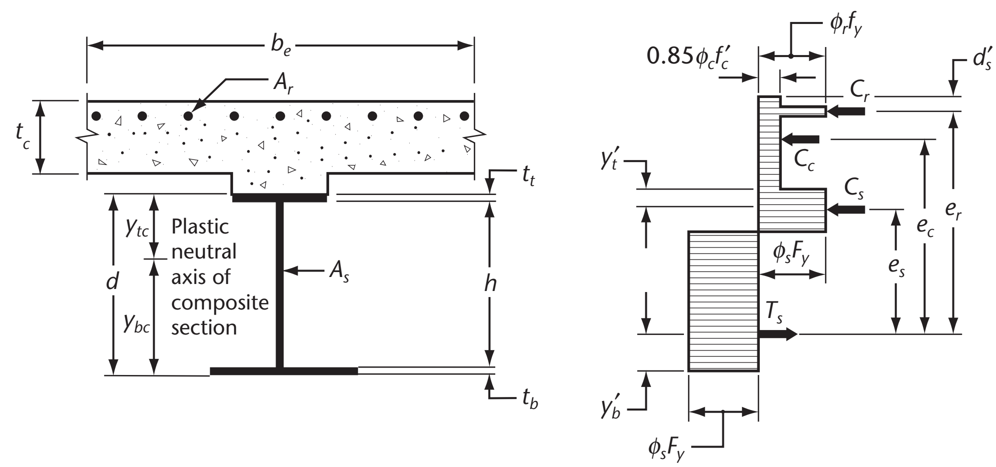
where

C_c = $0.85\phi_c b_e t_c f_c'$

C_r = $\phi_r A_r f_y$



(a) Plastic neutral axis in the concrete slab



(b) Plastic neutral axis in the steel section

Figure 10.1
Class 1 and 2 sections in positive moment regions
(See Clauses 10.11.5.2.1, 10.11.5.2.3, and 10.11.5.2.4.)

10.11.5.2.3 Plastic neutral axis in concrete

When C_1 is greater than C_2, the plastic neutral axis is in the concrete slab as shown in Figure 10.1(a), and the depth of the compressive stress block, a, shall be calculated as

$$a = \frac{C_2 - \phi_r A_r f_y}{0.85\phi_c b_e f_c'}$$

The factored moment resistance, M_r, of the section shall be calculated as

$$M_r = C_c e_c + C_r e_r$$

where

C_c = $0.85\phi_c b_e a f_c'$

10.11.5.2.4 Plastic neutral axis in steel

When C_1 is less than C_2, the plastic neutral axis is in the web as shown in Figure 10.1(b), and the depth of the compressive stress block, a, shall be taken as equal to t_c. The factored moment resistance, M_r, shall be calculated as

$$M_r = C_c e_c + C_r e_r + C_s e_s$$

where

C_c = $0.85\phi_c b_e t_c f_c'$

C_s = $0.5(\phi_s A_s F_y - C_1)$

10.11.5.3 Negative moment regions

10.11.5.3.1 Moment resistance of composite section

When

(a) shear connectors are provided; and

(b) slab reinforcement is continuous over interior supports or the slab is prestressed longitudinally,

the factored moment resistance, M_r, of the section shall be calculated on the basis of a fully plastic stress distribution in the structural steel, reinforcement, and prestressing strands, as shown in Figure 10.2, as follows:

$$M_r = T_r e_r + T_s e_s$$

where

T_r = $\phi_r A_r f_y$

T_s = $0.5(\phi_s A_s F_y - T_r)$

The requirements of Clause 8.5.3 shall also be satisfied.

When shear connectors are not provided in the negative moment regions, the factored moment resistance shall be taken as that of the steel section alone, calculated in accordance with Clause 10.10.2.

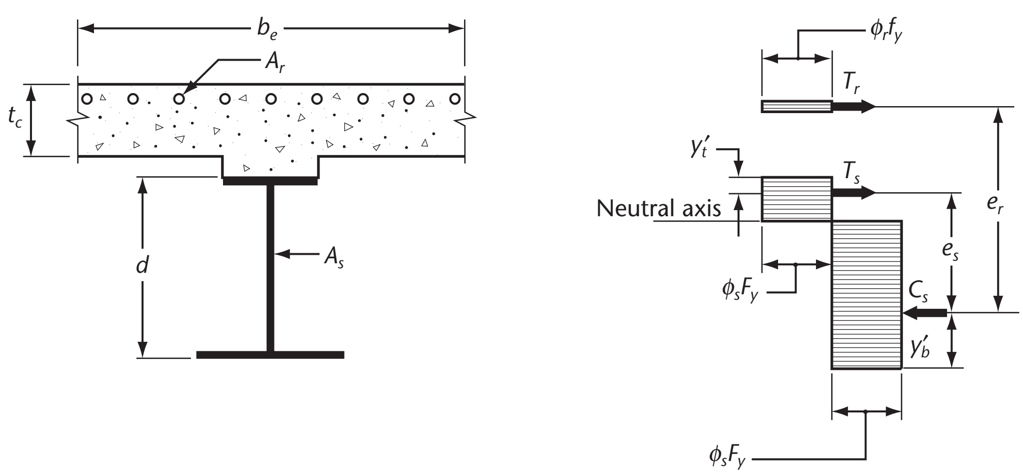


Figure 10.2
Class 1 and 2 sections in negative moment regions
(See Clause 10.11.5.3.1.)

10.11.5.3.2 Longitudinal reinforcement in non-prestressed slabs

The longitudinal reinforcement in non-prestressed slabs, including longitudinal distribution reinforcement, shall not be less than 1% of the cross-sectional area of the slab. At least two-thirds of this reinforcement shall be placed in the top layer of the slab reinforcement and within the effective width of the slab.

10.11.5.3.3 Negative moment regions without shear connectors

When shear connectors are not provided in the negative moment region, the longitudinal reinforcement shall be extended into the positive moment regions in accordance with Clause 8.15, and additional shear connectors shall be provided in accordance with Clause 10.17.2.7.

10.11.6 Class 3 sections

10.11.6.1 Width-to-thickness ratios

The portions of the steel section in compression shall comply with Clause 10.9.2.

10.11.6.2 Positive moment regions

10.11.6.2.1 Moment resistance

For composite sections in which the depth of the compression portion of the web of the steel section, calculated on the basis of a fully plastic stress distribution, equals or is less than $850w/\sqrt{F_y}$, the factored moment resistance shall be determined in accordance with Clause 10.11.5.2.

10.11.6.2.2 Moment resistance of slender members

When the depth of the compression portion of the web of the steel section, calculated in accordance with Clause 10.11.6.2.1, exceeds $850w/\sqrt{F_y}$, the factored moment resistance, M_r, of the composite section shall be calculated on the basis of fully plastic stress blocks, as shown in Figure 10.3, as follows:

$$M_r = C_c e_c + C_r e_r + C_s e_s$$

where

$C_c = 0.85\phi_c b_e t_c f_c'$

$C_r = \phi_r A_r f_y$

$C_s = \phi_s A_{sc}' F_y$

The area of the steel section in compression, A_{sc}', shall include the top flange and a web area of $(850w^2)/\sqrt{F_y}$, and the area of the steel section in tension, A_{st}', shall be calculated as follows:

$$A_{st}' = \frac{C_c + C_r + C_s}{\phi_s F_y}$$

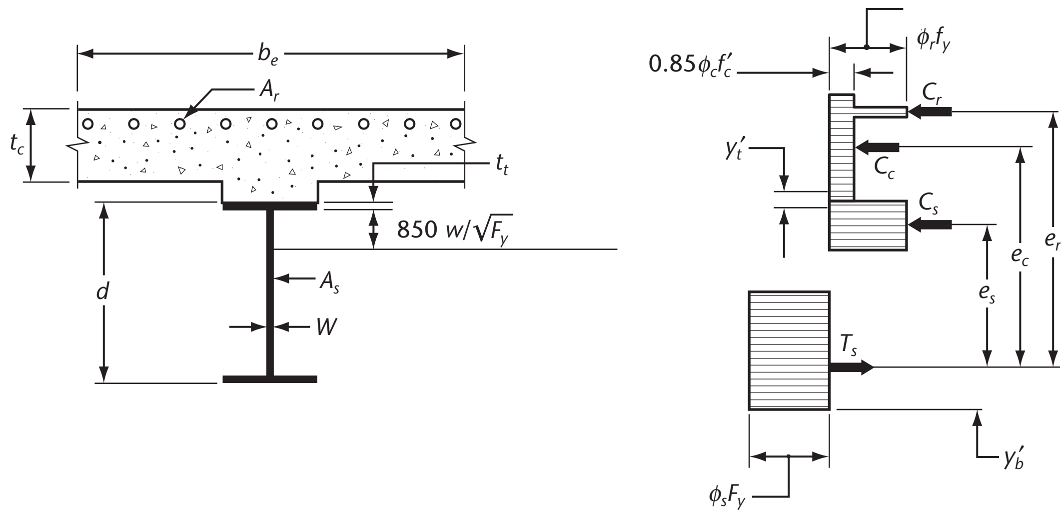


Figure 10.3
Class 3 Sections in positive moment regions
(See Clause 10.11.6.2.2.)

10.11.6.3 Negative moment regions

10.11.6.3.1 Composite sections

10.11.6.3.1.1

When
(a) shear connectors are provided; and
(b) slab reinforcement is continuous over interior supports or the slab is prestressed longitudinally,
the factored moment resistance of the composite section shall be taken as the resultant moment based on the linear stress distribution at first yielding or buckling, as shown in Figure 10.4, and the requirements specified in Clause 10.11.6.3.1.2 shall be satisfied.

10.11.6.3.1.2

The following requirements shall be satisfied:
(a) $M_{fd}/S + (M_{fsd} + M_{fl})/S' \leq \phi_s F_{cr}$
where S and S' are the elastic section moduli (see Clause 10.3.1) with respect to the bottom fibre, $F_{cr} = M_r/\phi_s S$, and M_r is determined in accordance with Clause 10.10.3.3, based on the steel section.
(b) $M_{fd}/S + (M_{fsd} + M_{fl})/S' \leq \phi_s F_y$
where S and S' are the elastic section moduli with respect to the top fibre of the steel section.
(c) $(M_{fsd} + M_{fl})/S' \leq \phi_r f_y$
where S' is the elastic section modulus with respect to the centroid of the top layer of longitudinal slab reinforcement.

The applicable requirements of Clauses 8.5.3 and 10.11.5.3.2 shall also be satisfied.

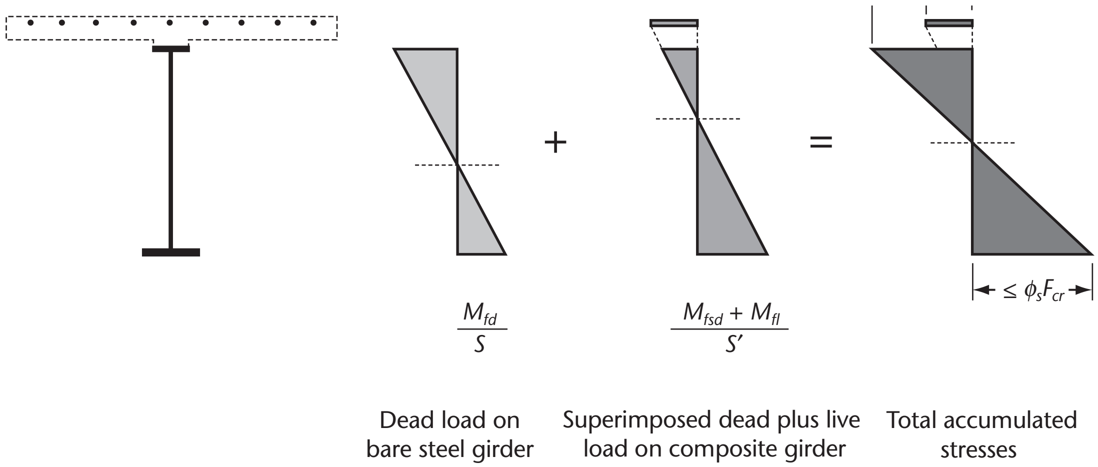


Figure 10.4
Class 3 Sections in negative moment regions
(See Clause 10.11.6.3.1.)

10.11.6.3.2 Non-composite sections

When shear connectors are not provided in the negative moment regions, the factored moment resistance shall be taken as that of the steel section alone, calculated in accordance with Clause 10.10.3. The requirements of Clause 10.11.5.3.3 shall also be satisfied.

10.11.7 Stiffened plate girders

10.11.7.1 Width-to-thickness ratios

Stiffened plate girders shall meet the requirements of Clauses 10.10.4.1 and 10.10.4.2.

10.11.7.2 Positive moment regions

10.11.7.2.1 Moment resistance

For composite sections in which the depth of the compression portion of the web of the steel section, calculated on the basis of a fully plastic stress distribution, does not exceed $850w/\sqrt{F_y}$, the factored moment resistance shall be determined in accordance with Clause 10.11.5.2.

10.11.7.2.2 Moment resistance of slender webs

When the depth of the compression portion of the web of the steel section calculated in accordance with Clause 10.11.7.2.1 exceeds $850w/\sqrt{F_y}$, whether or not longitudinal stiffeners are provided, the factored moment resistance of the composite section shall be calculated on the basis of fully plastic stress blocks, as in Clause 10.11.6.2.2.

10.11.7.3 Negative moment regions

10.11.7.3.1 Composite sections

When
(a) shear connectors are provided; and
(b) slab reinforcement is continuous over interior supports or the slab is prestressed longitudinally,

the factored moment resistance of the section shall be calculated in accordance with Clause 10.11.6.3.1.

If longitudinal stiffeners are not provided and $2d_c/w > 1900\sqrt{F_y}$, the factored moment resistance shall be reduced by the factor specified in Clause 10.10.4.3.

10.11.7.3.2 Non-composite sections

When shear connectors are not provided in the negative moment region, the factored moment resistance shall be taken as that of the steel section alone (see Clause 10.10.4). The requirements of Clause 10.11.5.3.3 shall also apply.

10.11.8 Shear connectors

10.11.8.1 General

Shear connectors shall comply with the applicable materials specification of Clause 10.4 and shall be capable of resisting both horizontal and vertical movements between the concrete slab and the steel beam or girder.

The fatigue resistance of the base metal at the connection weld of shear connectors shall meet the requirements of Clause 10.17.2.3. The fatigue resistance of stud shear connectors shall meet the requirements of Clause 10.17.2.7.

10.11.8.2 Placement, spacing, cover, and edge distances

Shear connectors shall be spaced uniformly or at intervals that vary according to the variation in the interface shear. The spacing of shear connectors shall not exceed 600 mm.

The clear depth of concrete cover over the tops of shear connectors shall be meet the requirements of Clause 8.11.2.2. Shear connectors shall extend into the concrete deck so that the clear distance from the underside of the head of the shear connector to the top of the bottom transverse reinforcement or, when the slab is haunched, to the top of the transverse reinforcement in the slab haunch, is at least 25 mm. The clear distance between the edge of a girder flange and a shear connector shank shall be at least 25 mm.

10.11.8.3 Shear connector resistance

The factored shear resistance, q_r, of an end-welded stud shear connector, headed or hooked with $h/d \geq 4$, shall be taken as

$$q_r = 0.5\phi_{sc} A_{sc} \sqrt{f_c' E_c} \leq \phi_{sc} F_u A_{sc}$$

where

F_u = minimum tensile strength of the stud steel (410 MPa for commonly available studs)

A_{sc} = cross-sectional area of one stud shear connector, mm^2

In solid slabs of normal-density concrete, the factored shear resistance for channel shear connectors shall be taken as

$$45\phi_{sc}(t + 0.5w)L_c\sqrt{f_c'}$$

where

t = average thickness of channel shear connector flange

w = thickness of channel shear connector web

L_c = length of channel shear connector

The number of shear connectors between points of maximum and zero moment at the ULS shall be calculated as follows:

$$N = P/q_r$$

where P is determined as follows:
(a) for positive moment:
   (i) when the plastic neutral axis is in the concrete slab: $P = \phi_s A_s F_y$; and
   (ii) when the plastic neutral axis is in the steel section: $P = 0.85 \phi_c f_c' b_e t_c + \phi_r A_r f_y$; and
(b) for negative moment: $P = \phi_r A_r f_y$.

10.11.8.4 Longitudinal shear

The longitudinal factored shear resistance along any potential shear planes shall be greater than the factored longitudinal shear.

The factored longitudinal shear in the slab of a composite beam, V_u, shall be taken as

$$V_u = \Sigma q_r - 0.85\phi_c A_c f_c' - \phi_r A_{rL} f_y$$

For normal-weight concrete, the factored shear resistance along any potential shear surface in the concrete slab shall be calculated as

$$V_r = 0.80\phi_r A_{rt} f_y + 2.76\phi_c A_{cv} \leq 0.50\phi_c A_{cv} f_c'$$

For lightweight concrete, the constant 2.76 with units of MPa shall be replaced by 1.38.

10.11.9 Lateral bracing, cross-frames, and diaphragms

The requirements of Clause 10.10.9 shall be met.

10.12 Composite box girders

10.12.1 General

Clause 10.12.1 applies to the design of simple and continuous composite box girder bridges of spans up to 110 m, consisting of one or more straight steel single-cell box girders, acting compositely with a concrete deck, and symmetrical about a vertical axis. For longer spans, other requirements may apply.

The top of the box may be open with twin steel flanges or closed with a steel flange plate.

Exterior access holes with hinged and locked doors shall be provided. Openings in box sections shall be screened to exclude animals.

The requirements of Clauses 10.11.2 to 10.11.4 shall also apply.

10.12.2 Effective width of tension flanges

The effective width of bottom flange plates in tension shall be taken as not more than one-fifth of the span for simply supported structures and not more than one-fifth of the distance between points of contraflexure under dead load for continuous structures.

10.12.3 Web plates

Webs shall be proportioned in accordance with Clause 10.10 and, for single box girders, in accordance with Clause 10.12.8.5. The shear force to be considered on each web shall be $V_f / \cos\theta$, where V_f is one-half of the total vertical shear force at the ULS on one box girder and θ is the angle of inclination of the web plate to the vertical.

The inclination of the web plates shall not exceed 1 horizontal to 4 vertical.

10.12.4 Flange-to-web welds

The total effective throat of the flange-to-web welds shall not be less than the thickness of the web unless internal diaphragms or cross-frames are spaced in accordance with Clause 10.12.6.1 and a minimum of two intermediate diaphragms per span are used inside the box. If fillet welds are used, they shall be placed on both sides of the connecting flange or web plate.

10.12.5 Moment resistance

10.12.5.1 Composite and non-composite sections

10.12.5.1.1

The factored moment resistance of the steel section acting alone before the attainment of composite action shall be determined in accordance with

(a) Clauses 10.10, 10.12.2, and 10.12.8.4 for regions of positive moment; and

(b) Clauses 10.10 and 10.12.5.2 to 10.12.5.4 for regions of negative moment, using the elastic section modulus of the steel section alone.

10.12.5.1.2

The factored moment resistance of the composite section shall be determined in accordance with

(a) Clauses 10.11, 10.12.2, and 10.12.8.4 for regions of positive moment; and

(b) Clause 10.11 for regions of negative moment, except that in applying Clause 10.11.6.3.1.1 or 10.11.6.3.1.2, F_{cr} shall be determined in accordance with the applicable requirements of Clauses 10.12.5.2 to 10.12.5.4.

10.12.5.2 Unstiffened compression flanges

The factored moment resistance with respect to the compression flange shall be calculated as follows:

(a) when $b/t \le 510/\sqrt{F_y}$:

$$M_r = \phi_s F_y S'$$

(b) when $510/\sqrt{F_y} < b/t \le 1100/\sqrt{F_y}$:

$$M_r = \phi_s F_{cr} S'$$

where

$$F_{cr} = 0.592 F_y \left[1 + 0.687 \sin \frac{\pi C}{2}\right]$$

where

$$C = \frac{1100 - (b/t)\sqrt{F_y}}{590}$$

(c) when $b/t > 1100/\sqrt{F_y}$:

$$M_r = \phi_s F_{cr} S'$$

where

$$F_{cr} = \frac{7.24 \times 10^5}{(b/t)^2}$$

10.12.5.3 Compression flanges stiffened longitudinally

10.12.5.3.1

The factored moment resistance with respect to the compression flange shall be calculated as follows:

(a) when $b_s/t \le 255\sqrt{k_1/F_y}$:

$$M_r = \phi_s F_y S'$$

(b) when $255\sqrt{k_1/F_y} < b_s/t \le 550\sqrt{k_1/F_y}$:

$$M_r = \phi_s F_{cr} S'$$

where

$F_{cr} = 0.592 F_y \left[1 + 0.687 \sin \dfrac{\pi C_s}{2}\right]$

where

$C_s = \dfrac{550\sqrt{k_1} - (b_s/t)\sqrt{F_y}}{295\sqrt{k_1}}$

(c) when $b_s/t > 550\sqrt{k_1/F_y}$:

$M_r = \phi_s F_{cr} S'$

where

$F_{cr} = \dfrac{18 k_1 \times 10^4}{(b_s/t)^2}$

10.12.5.3.2

The buckling coefficient, k_1, in Clause 10.12.5.3.1 shall be determined as follows:

(a) For $n = 1$:

$$k_1 = \left[\frac{8 I_s}{b_s t^3}\right]^{1/3} \le 4.0$$

(b) For $n > 1$:

$$k_1 = \left[\frac{14.3 I_s}{b_s t^3 n^4}\right]^{1/3} \le 4.0$$

where

n = number of longitudinal stiffeners

I_s = moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener

10.12.5.3.3

The longitudinal stiffeners shall be equally spaced across the flange width. A transverse stiffener shall be placed near the point of contraflexure under dead load and shall be equal in size to a longitudinal stiffener.

10.12.5.4 Compression flanges stiffened longitudinally and transversely

The factored moment resistance with respect to the compression flange shall be calculated as follows:

(a) when $b_s/t > 550\sqrt{k_1/F_y}$:

$M_r = \phi_s F_y S'$

(b) when $255\sqrt{k_2/F_y} < b_s/t \le 550\sqrt{k_2/F_y}$:

$M_r = \phi_s F_{cr} S'$

where

$F_{cr} = 0.592 F_y \left[1 + 0.687 \sin \dfrac{\pi C_s}{2}\right]$

where

$$C_s = \frac{550\sqrt{k_2} - (b_s/t)\sqrt{F_y}}{295\sqrt{k_2}}$$

(c) when $b_s/t > 550\sqrt{k_2/F_y}$:

$M_r = \phi_s F_{cr} S'$

where

$$F_{cr} = \frac{18k_2 \times 10^4}{(b_s/t)^2}$$

$$k_2 = \frac{[1+(a/b)^2]^2 + 87.3}{(n+1)^2 (a/b)^2 [1+0.1(n+1)]}$$

The longitudinal stiffeners shall be equally spaced across the flange width and shall be proportioned so that the moment of inertia of each stiffener, I_s, about a transverse axis at the base of the stiffener is at least equal to

$$I_s = 8t^3 b_s$$

The transverse stiffeners shall be proportioned so that the moment of inertia of each stiffener, I_t, about a longitudinal axis through its centroid is at least equal to

$$I_t = 0.55(n+1)^3 b_s^3 \frac{F_{cr} A_f}{E_s a}$$

The ratio a/b shall not exceed 3.0. The maximum value of the buckling coefficient k_2 shall be 4.0. When k_2 has its maximum value, the transverse stiffeners shall have a spacing, a, equal to or less than $4b_s$. The transverse stiffeners need not be connected to the flange plate but shall be connected to the webs of the box and to each longitudinal stiffener. The connection to the web shall be proportioned for a vertical force, R_w, of

$$R_w = \frac{\phi_s F_y S_t}{2b}$$

The connection to each longitudinal stiffener shall be proportioned for a vertical force, R_s, of

$$R_s = \frac{\phi_s F_y S_t}{nb}$$

10.12.6 Diaphragms, cross-frames, and lateral bracing

10.12.6.1 Diaphragms and cross-frames within girders

Internal diaphragms, cross-frames, or other means shall be provided at each support to resist transverse rotation, displacement, and distortion and to transfer vertical, transverse, and torsional loads to the bearings. The effect of access holes shall be considered and adequate reinforcement provided if necessary.

Intermediate internal diaphragms or cross-frames shall be used to control deformation, torsional warping, and distortion of open box girders during fabrication, transportation, erection, and placement of the deck. Cross-frames, diaphragms, or cross-ties between top flanges shall be used in open trapezoidal box girders to resist the transverse resultant induced by the sloping web force opposing the vertical loads acting on the top flanges during construction.

Vertical stiffeners used as connecting plates for diaphragms or cross-frames shall be connected to both flanges.

For single box girder bridges, internal intermediate diaphragms or cross-frames shall be placed at intervals not greater than 8 m unless it can be shown that the degree of cross-sectional distortion is not critical.

10.12.6.2 Diaphragms and cross-frames between girders

When diaphragms and cross-frames are included in the structural model used to determine the forces in the girders, they shall be designed for the forces that they attract.

Where girders support deck slabs proportioned in accordance with the empirical design method of Clause 8.18.4, the spacing of intermediate cross-frames or diaphragms shall satisfy the requirements of Clause 8.18.5.

10.12.6.3 Lateral Bracing

The need for lateral bracing shall be assessed for all stages of construction as well as for the service condition. The bracing shall be designed for the forces it attracts.

For multiple open-box girders, the need for lateral bracing between the flanges of individual boxes shall be investigated to ensure that deformations and stability of the box sections are adequately controlled during fabrication, erection, and deck construction. Top bracing shall be placed as close to the plane of the top flanges as possible, except that the requirements of Clause 10.17.3.2.2 shall also be met. If the bracing is attached to the webs below the plane of the top flanges, a means shall be provided to transfer horizontal forces from the bracing to the top flanges.

Bridges consisting of a single trough-type open-box section shall have top lateral bracing between the flanges. The bracing shall be designed to resist the shear flow in the section prior to the curing of the concrete deck. Forces in the bracing due to flexural bending shall also be considered.

The structural section assumed to resist the portion of factored horizontal wind or seismic loading in the plane of the bottom flange shall consist of the bottom flange acting as a web and 12 times the thickness of the webs acting as flanges.

10.12.7 Multiple box girders

10.12.7.1 General

The distance centre-to-centre of flanges of each box shall be the same. The distance centre-to-centre of flanges of adjacent boxes at mid-span shall be within the range of 0.80 to 1.20 of the distance centre-to-centre of the flanges of each adjacent box. When the boxes are not parallel, the distance centre-to-centre of adjacent flanges at supports shall be within the range of 0.65 to 1.35 of the distance centre-to-centre of the flanges of each adjacent box.

The cantilever overhang of the deck slab, including curb and parapet, shall not exceed 0.60 of the average distance between the centres of the top steel flanges of the exterior box section or 1800 mm, unless special precautions are taken during design and construction.

10.12.7.2 Relative deflection of boxes of multiple box girders

Control of cracking in the deck slab due to relative deflection of box girders shall be considered, taking into account the requirements of Clauses 5.7.1.2 and 10.12.6.2.

10.12.8 Single box girders

10.12.8.1 General

Single box girder sections shall be symmetric about a vertical axis and the line of action of the dead load shall be as close to the shear centre of the box as practicable.

Structural steel in tension under dead load shall be considered fracture critical unless analysis shows that the full dead and live load can be supported after a notional complete fracture of the tension steel occurs at any cross-section.

Thermal forces shall be considered in the design. Uplift at the bearings shall be considered for ULS load combinations.

Sufficient internal cross-frames shall be provided to maintain the shape of the cross-section under the action of eccentric loads and to limit distortional bending and warping. Longitudinal warping normal stresses shall be taken into account for fatigue, but need not be considered for the ULS.

10.12.8.2 Analysis

The analytical model used shall permit the assessment of both torsional and flexural effects for all load conditions.

The transverse positions of the bearings shall be modelled so that the reactions can be calculated directly for all load conditions, including eccentric live loads.

Live loads shall be positioned so as to cause the maximum flexural/torsional effect on the girder component being investigated. Load effects from multiple traffic lanes shall be investigated.

10.12.8.3 Bearings

Bearings for single box girders shall be located to ensure stability of the bridge against overturning under all conditions of loading. If single bearings are used, the remaining double bearings shall be sufficient to prevent overturning under all conditions of loading.

Single bearings shall be located vertically below the shear centre of the box girder to the extent practicable.

10.12.8.4 Moment resistances

The factored moment resistance of single box girders shall be determined in accordance with Clause 10.12.5 using a reduced normal stress, $R_v F_y$, for the tensile resistance of the bottom flange in place of F_y, with R_v as follows:

$$R_v = \sqrt{1 - 3\left[\frac{f_s}{F_y}\right]^2}$$

10.12.8.5 Combined shear and torsion

Both the web plates and the shear connectors shall be proportioned for the sum of the factored shears due to bending and torsion.

10.13 Horizontally curved girders

10.13.1 General

Clauses 10.13.2 to 10.13.8 shall apply to all simple and continuous bridges that are curved in plan, are up to 60 m in span, and employ either rolled or fabricated sections. For longer spans, other considerations may apply.

10.13.2 Special considerations

Note: *The design of curved girders necessitates special consideration of super-elevation and centrifugal forces, thermal forces, and uplift.*

10.13.2.1 Dynamic load allowance

The dynamic load allowance shall be as specified in Clause 3.8.4.5 unless a dynamic analysis is used.

10.13.2.2 Super-elevation and centrifugal forces
The super-elevation of the deck shall be considered when the wheel loads due to the combined effects of the centrifugal forces and the vertical live loads are being determined.

10.13.2.3 Thermal forces
Tangential and radial movements, potential uplift at the bearings, and induced restraining forces due to the temperature changes and gradients shall be taken into account.

10.13.2.4 Uplift
The structure shall be assessed for uplift at the supports. The assessment for dead loads shall take into account the intended sequence of construction.

10.13.3 Design theory

10.13.3.1 General
The whole structure shall be modelled in the analysis, including the transverse members. The analysis shall include, in addition to the torsional shear stresses, an evaluation of the longitudinal stresses due to restrained warping of members with non-uniform torsion.

10.13.3.2 Limiting curvature
Provided that the conditions of Clause A5.1.3.2 are met, the bridge shall be considered straight for the purposes of structural analysis.

10.13.4 Bearings
Bearings shall be designed to resist the vertical loads (including uplift) and the horizontal loads to which they could be subjected (including centrifugal force effects) and be designed and oriented to permit thermal movement consistent with the design assumptions.

10.13.5 Diaphragms, cross-frames, and lateral bracing
Unless otherwise Approved, longitudinal girders shall be connected at each support by diaphragms designed to prevent twisting of the girders.

Diaphragms or cross-frames shall be provided between I-girders, i.e., at intervals between supports, to further facilitate resistance to twisting of the girders. Each line of diaphragms or cross-frames shall extend continuously across the full width of the bridge.

Diaphragms or cross-frames shall be provided between box girders where needed to augment the resistance of the girders to torsion. Diaphragms or cross-frames shall be provided inside the girders in line with those provided between the girders.

Diaphragms and cross-frames shall be treated as main structural members. They shall be approximately as deep as the girders they connect and shall be connected to the girders to transfer all of the loads that they attract.

In addition to the diaphragms or cross-frames used to control torsion, other cross-frames shall be provided in box girders, if necessary, to resist the distortional effects of eccentric loads on the cross-section.

The need for lateral bracing between the top flanges of curved box girders and between I-girders to ensure stability and resist the effects of wind shall be assessed for all stages of construction as well as for service conditions.

Where girders support deck slabs proportioned in accordance with the empirical design method of Clause 8.18.4, the lateral spacing of intermediate cross-frames or diaphragms shall satisfy the requirements of Clause 8.18.5.

10.13.6 Steel I-girders

10.13.6.1 Non-composite girder design

10.13.6.1.1 Limits of applicability

The following requirements shall apply:

(a) The absolute value of the ratio of the torsional warping normal stress to the normal flexural stress shall, as far as possible, not exceed 0.5 at any point in the girder.

(b) The unbraced length between cross-frames shall not exceed 25 times the width of the flange or 0.1 times the mean radius of the girder.

(c) Flanges shall be Class 3 or better.

10.13.6.1.2 Flanges

Flanges shall be proportioned to satisfy the following requirements:

(a) Strength of either flange:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fw}}{M_{ry}} < 1$$

(b) Stability of compression flange:

$$M_{fx} \leq M'_{rx}$$

where

M_{fx} = factored bending moment due to flexure

M_{rx} = $\phi_s F_y S_x$

where

S_x = elastic section modulus of the girder about its major axis

M_{fw} = factored bending moment in the flange due to torsional warping

M_{ry} = $\phi_s F_y S_y$

where

S_y = elastic section modulus of the flanges only about an axis in the plane tangent to the web of the girder

M'_{rx} = $\phi_s F_y S_x (1 - 3\lambda^2)\, \rho_b \rho_w$

where

$$\lambda = \frac{L}{2b}\sqrt{\frac{F_y}{\pi^2 E_s}}$$

$$\rho_b = \frac{1}{1 + \left[\frac{L}{R}\right]\left[\frac{L}{2b}\right]}$$

ρ_w = smaller of ρ_{w1} and ρ_{w2} when f_w / f_b is positive

= ρ_{w1} when f_w / f_b is negative

where

$$\rho_{w1} = \frac{1}{1-\frac{f_w}{f_b}\left[1-\frac{L}{150b}\right]}$$

$$\rho_{w2} = \frac{0.95+\dfrac{\frac{L}{2b}}{30+8000\left[0.1-\frac{L}{R}\right]^2}}{1+0.6\left[\frac{f_w}{f_b}\right]}$$

f_w = coexisting warping normal stress

f_b = flexural stress due to the larger of the two moments at either end of the braced segment

f_w / f_b is positive when f_w is compressive on the outside edge of the curved flange.

10.13.6.1.3 Webs

The factored shear resistance shall be calculated in accordance with Clause 10.10.5.1(a) (neglecting tension field action). The following requirements shall also apply:

(a) Webs without stiffeners: transverse stiffeners shall not be required if $h/w \le 150$.

(b) Webs with transverse stiffeners only: when

$$150 < h/w < \frac{3150}{\sqrt{F_y}}\left[1-8.6\left[\frac{a}{R}\right]+34\left[\frac{a}{R}\right]^2\right]$$

transverse stiffeners shall be provided at a spacing $a < h$.

(c) Webs with transverse and longitudinal stiffeners: when

$$\frac{3150}{\sqrt{F_y}}\left[1-8.6\left[\frac{a}{R}\right]+34\left[\frac{a}{R}\right]^2\right] < h/w < \frac{6000}{\sqrt{F_y}}\left[1-2.9\left[\frac{a}{R}\right]^{0.5}+2.2\left[\frac{a}{R}\right]\right]$$

the longitudinal stiffeners shall be provided at a distance $0.2h$ from the compression flange. When same-size longitudinal stiffeners are located $0.2h$ from both the compression flange and the tension flange, the web slenderness ratio shall not exceed $6000/\sqrt{F_y}$.

(d) Proportioning of transverse web stiffeners: in addition to meeting the b/t requirements of Clause 10.9.2, stiffeners shall be proportioned so that

$$I \ge a w^3 j$$

where

$$j = \left[2.5\left[\frac{h}{a}\right]^2 - 2\right] X \ge 0.5$$

where

$$X = 1.0 \text{ when } \frac{a}{h} \le 0.78$$

$$= 1.0 + \frac{\left[\frac{a}{h}-0.78\right]}{1775} Z^4 \text{ when } 0.78 < \frac{a}{h} < 1.0,\ 0 \le Z \le 10, \text{ and } Z = 0.95\frac{a^2}{Rw}$$

I shall be taken about an axis at the mid-plane of the web for stiffener pairs, or at the near face of the web for single stiffeners.

(e) Longitudinal stiffeners shall meet the requirements of Clause 10.10.7.2.
(f) Monosymmetric sections: the slenderness ratio of the compression portion of webs of monosymmetric sections with an axis of symmetry in the plane of loading shall not exceed one-half of the applicable value specified in Item (a), (b), or (c).
Longitudinal stiffeners, if required, shall be placed $0.4d_c$ from the inner surface of the compression flange.

10.13.6.2 Composite I-girders

10.13.6.2.1 General

Clauses 10.13.6.2.2 to 10.13.6.2.4 shall apply to simple and continuous span curved I-girders, fastened throughout their full length to a concrete deck by shear connectors.

10.13.6.2.2 Webs

The web of the steel girder shall be designed to carry the entire vertical shear in accordance with Clause 10.13.6.1.3.

10.13.6.2.3 Flanges

Flanges shall be Class 3 or better and meet the strength and stability requirements of Clause 10.13.6.1.2.

10.13.6.2.4 Shear connectors

Shear connectors shall meet the requirements of Clauses 10.11.8 and 10.17.2.7.

10.13.7 Composite box girders

10.13.7.1 General

Clause 10.13.7.1 shall apply to simple or continuous span curved box girders fastened throughout their length to a concrete deck by shear connectors.

The behaviour of a girder before the concrete deck has cured may be analyzed as a quasi-closed section by replacing the lateral bracing with an equivalent plate. The top flanges shall meet the requirements of Clause 10.13.6.2.3 and the bottom flange shall meet the requirements of Clause 10.13.7.4.

10.13.7.2 Webs

The inclination of the webs from the normal to the bottom flange shall not exceed 1 in 4. The webs shall meet the requirements of Clause 10.13.6.1.3 for combined flexural and torsional shear.

10.13.7.3 Top flanges

The top flanges of open-box girders shall meet the requirements of Clause 10.13.6.1.2 and the top flanges of closed-box girders shall meet the requirements of Clause 10.13.7.4. In both cases, lateral bending and buckling of the steel flange shall be checked for strength and stability during placement of concrete, but may be assumed to be restrained by the concrete deck when the deck has cured.

10.13.7.4 Bottom flanges

10.13.7.4.1 Tension flanges

The factored moment resistance with respect to the tension flange shall be determined using an effective width in accordance with Clause 10.12.5.1.2(a) and using a reduced normal stress, $R_v F_y$, in place of F_y, with R_v as follows:

$$R_v = \sqrt{1 - 3\left[\frac{f_s}{F_y}\right]^2}$$

where

f_s = coexisting shear stress due to torsion

10.13.7.4.2 Stiffened and unstiffened compression flanges

10.13.7.4.2.1 Stiffened compression flanges

The following requirements shall apply to stiffened compression flanges:

(a) When the torsional shear stress $f_s \le 0.75F_y/\sqrt{3}$, the factored moment resistance, M_r, shall be taken as

(i) $M_r = \phi_s R_v F_y S'$ for $\dfrac{b_s}{t} \le \dfrac{R_1}{\sqrt{F_y}}$

(ii) $M_r = \phi_s F_y\left[(R_v - 0.4) + 0.4\sin\left[\dfrac{C_s\pi}{2}\right]\right]S'$ for $\dfrac{R_1}{\sqrt{F_y}} < \dfrac{b_s}{t} \le \dfrac{R_2}{\sqrt{F_y}}$

(iii) $M_r = \phi_s F_{cr} S'$ for $\dfrac{b_s}{t} > \dfrac{R_2}{\sqrt{F_y}}$

(b) When $0.75F_y/\sqrt{3} < f_s \le F_y/\sqrt{3}$ and $\dfrac{b_s}{t} \le \dfrac{R_1}{\sqrt{F_y}}$, the factored moment resistance, M_r, shall be taken as

$M_r = \phi_s R_v F_y S'$

where

$$R_1 = \frac{255\sqrt{k_1}}{\sqrt{\frac{1}{2}\left[R_v + \left[R_v^2 + 4\left[\frac{f_s}{F_y}\right]^2\left[\frac{k_1}{k_s}\right]^2\right]^{0.5}\right]}}$$

$$R_2 = \frac{550\sqrt{k_1}}{\sqrt{\frac{1}{1.2}\left[(R_v - 0.4) + \left[(R_v - 0.4)^2 + 4\left[\frac{f_s}{F_y}\right]^2\left[\frac{k_1}{k_s}\right]^2\right]^{0.5}\right]}}$$

$$C_s = \frac{R_2 - \left[\frac{b_s}{t}\right]\sqrt{F_y}}{R_2 - R_1}$$

$$F_{cr} = \frac{18k_1 \times 10^4}{\left[\frac{b_s}{t}\right]^2} - \frac{f_s^2 k_1\left[\frac{b_s}{t}\right]^2}{18k_s^2 \times 10^4}$$

where

k_1 = the buckling coefficient, which shall not exceed 4.0

and, when at least one longitudinal stiffener is provided,

$$k_s = \frac{5.34 + 2.84\left[\dfrac{I_s}{b_s t^3}\right]^{1/3}}{(n+1)^2} \leq 5.34$$

where

I_s = moment of inertia of stiffener, designed and detailed in accordance with Clause 10.12.5.3

10.13.7.4.2.2 Unstiffened compression flanges

The requirements of Clause 10.13.7.4.2.1 shall apply to unstiffened compression flanges, except that the following values shall apply:

(a) $k_1 = 4$;
(b) $k_s = 5.34$; and
(c) $b_s = b$ = width of flange between webs.

10.13.8 Camber

Girders shall be cambered for dead load deflections, including twisting effects. When heat-curved girders are used, they shall be provided with additional camber in accordance with Clause 10.7.4.3.

10.14 Trusses

10.14.1 General

Main truss members shall be symmetrical about the centroidal longitudinal vertical plane of the truss. When the centroidal axes of axially loaded members joined at their ends do not intersect at a common point, the effect of connection eccentricity shall be taken into account.

The fabricated length of members shall be such that the resulting camber of the truss is in accordance with Clause 10.7.4. The design of gusset plates shall be in accordance with Clause 10.18.5.2.

10.14.2 Built-up members

10.14.2.1 General

Unless otherwise Approved, the components shall be connected by solid plates. Batten plates shall be ignored in calculating the radius of gyration of the section.

Diaphragms or stiffeners shall be provided in trusses at the end connections of floor beams.

10.14.2.2 Tie plates

The separate components of tension members composed of shapes shall be connected by tie plates or other Approved means. The length of end tie plates shall not be less than 1.25 times the distance between the inner lines of the fasteners or welds connecting them to the flanges. Intermediate tie plates shall have a length at least 0.5 times the distance between the inner lines of the fasteners or welds connecting them to the flanges.

Tie plates shall have a thickness not less than 0.02 times the distance between the fasteners or welds connecting them to the flanges.

A diaphragm between gusset plates engaging main members shall be provided if the end tie plate is 1200 mm or more from the point of intersection of the members.

10.14.2.3 Perforated cover plates

The thickness of perforated cover plates shall be not less than the unsupported width multiplied by $\sqrt{F_y}/840$.

Perforated cover plates shall be proportioned so that

(a) the transverse distance from the edge of a perforation to the nearest line of connecting fasteners or welds does not exceed 12 times the thickness of the plate;
(b) the length of perforations in the longitudinal direction does not exceed twice the width;
(c) the clear distance between perforations in the longitudinal direction is not less than the clear distance between the inner lines of connecting fasteners or welds;
(d) the clear distance between the end perforation and the end of the plate, or the end of half perforation, is not less than 1.25 times the clear distance between the inner lines of connecting fasteners or welds; and
(e) no part of a perforation has a radius smaller than 25 mm.

10.14.2.4 Battens for compression members

The use of battened compression members shall be limited to members not subjected to bending in the plane of the battens.

The spacing of battens shall meet the following requirements:

(a) if the slenderness ratio of the member about the axis perpendicular to the battens is equal to or less than 0.8 times the slenderness ratio of the member about the axis parallel to the battens, the spacing of battens centre-to-centre of end fasteners or the clear distance between welds shall be such that the slenderness ratio of either main component over that distance shall not exceed 50 or 0.7 times the slenderness ratio of the member about the axis parallel to the battens; and
(b) if the slenderness ratio of the member about the axis perpendicular to the battens exceeds 0.8 times the slenderness ratio of the member about the axis parallel to the battens, the spacing of battens centre-to-centre of end fasteners or the clear distance between welds shall be such that the slenderness ratio of either main component over that distance shall not exceed 40 or 0.6 times the slenderness ratio of the member as a whole about its weaker axis.

Battens such as plates, channels, or beam sections shall be bolted or welded to the main components so as to resist simultaneously a longitudinal shear force of

$$V_f = \frac{0.025 C_f d}{na}$$

and a moment of

$$M_f = \frac{0.025 C_f d}{2n}$$

The effective length of a batten shall be taken as the longitudinal distance between end bolts or end welds, or as the length of continuous welds. Battens shall have an effective length not less than the distance between the innermost connecting bolts or welds, or less than twice the width of one main component in the plane of the batten.

Except for batten plates with stiffened edges or rolled shapes with flanges perpendicular to the main components, the thickness of batten plates shall be not less than 0.02 times the minimum distance between the innermost lines of connecting bolts or welds.

10.14.3 Bracing

10.14.3.1 Top and bottom bracing

Through-truss spans, deck-truss spans, and spandrel-braced-arch spans shall have top and bottom lateral bracing systems.

10.14.3.2 Chord bracing
The use of lateral bracing shallower than the chords shall require Approval. Bracing shall be connected effectively to both flanges of the chords.

10.14.3.3 Through-truss spans
Through-truss spans shall have portal bracing rigidly connected to the end post and top chord flanges. Portal bracing shall be proportioned to take the full reaction of the top chord lateral system and the end posts shall be proportioned for the reaction. Sway bracing shall be located at the necessary panel points.

10.14.3.4 Deck-truss spans
Deck-truss spans shall have sway bracing in the plane of the end posts. Unless an analysis performed in accordance with Section 5 indicates that sway bracing is unnecessary, sway bracing shall be provided at all intermediate panel points and shall extend the full depth of the trusses below the floor system. The end sway bracing shall be proportioned to carry all of the upper lateral forces to the supports through the end posts of the truss.

10.14.3.5 Minimum force
Bracing systems between straight compression members or straight flanges shall be designed to carry the shear forces from external loads plus 1% of the compression forces in the supported members or flanges.

10.14.3.6 Half-through trusses and pony trusses
The top chord of a half-through or pony truss shall be designed as a column with elastic lateral supports at each panel point. The factored compressive resistance of the column shall be at least equal to the maximum force in any panel of the top chord resulting from loads at the ULS.

The vertical truss members, floor beams, and connections between them shall be proportioned to resist at the ULS a lateral force of at least 8 kN/m applied at the top chord panel points.

10.15 Arches

10.15.1 General
The design of solid web arch ribs at the ULS shall be based on an amplified first-order analysis or a second-order analysis in accordance with Section 5 and take into account the deformations that occur at the ULS load levels.

10.15.2 Width-to-thickness ratios
The width-to-thickness ratios of flanges and web stiffeners of arch ribs shall meet the requirements for Class 1 or 2 sections specified in Clause 10.9.2.

The width-to-thickness ratio, h/w, of webs of arch ribs shall not exceed $560/\sqrt{F_y}$, $840/\sqrt{F_y}$, or $1120/\sqrt{F_y}$ for a web with no, one, or two longitudinal stiffeners, respectively.

10.15.3 Longitudinal web stiffeners
The moment of inertia, I_s, of a longitudinal web stiffener about an axis at its base shall not be less than $0.75ht^3$ when one stiffener is provided or $2.2ht^3$ when two stiffeners are provided.

10.15.4 Axial compression and bending
Arch ribs required to resist bending moments in addition to an axial compressive force shall be proportioned to meet the requirements of Clause 10.9.2 for Class 2 sections.

10.15.5 Arch ties

Arch ties shall be considered fracture-critical members unless constructed of several components in such a manner that a fracture of one component does not propagate into another.

10.16 Orthotropic decks

10.16.1 General

Clause 10.16 shall apply to the design of orthotropic steel decks comprising a deck plate stiffened and supported by longitudinal ribs and transverse floor beams. Connections between the deck and other structural members shall be designed to ensure full interaction.

The effects of distortion of the cross-sectional shape due to torsion shall be taken into account in the analysis of orthotropic box girder bridges.

10.16.2 Effective width of deck

10.16.2.1 Ribs

The effective width of deck plate acting as the top flange of a longitudinal rib shall be calculated in accordance with Clause 5.8.2.2.1 or by another Approved method.

10.16.2.2 Girders and transverse beams

The effective width of a deck acting as the top flange of a longitudinal superstructure component or transverse beam shall be calculated in accordance with Clause 5.8.2.2.2 or by another Approved method.

10.16.3 Superposition of local and global effects

10.16.3.1 General

In calculating extreme force effects in the deck, the global or overall effects induced by flexure and axial forces in the main longitudinal girders and the local effects for the same configuration and position of live load shall be superimposed.

10.16.3.2 Decks in longitudinal tension

Decks subject to global tension and local flexure shall be proportioned so that

$$\frac{T_f}{T_r} + \frac{M_{fr}}{M_{rr}} \leq 1.33$$

where T_f, the factored tensile force induced in the deck by flexure and axial tension in the main longitudinal girders, increased for simultaneous global shear, is $A_{de}\,(f_g^2 + 3\,f_{vg}^2)^{0.5}$.

10.16.3.3 Decks in longitudinal compression

Unless it can be shown by rigorous analysis that overall buckling of the deck will not occur as a result of the global compressive force in the main longitudinal girders combined with local flexural compressive force in the longitudinal ribs, the longitudinal ribs, including the effective width of deck plate, shall be designed as independent beam columns assumed to be simply supported at each transverse beam.

10.16.3.4 Transverse flexure

Transverse beams shall be proportioned so that

$$\frac{M_{fb}}{M_{rb}} \leq 1.00$$

where

M_{fb} = factored bending moment in the transverse beam at ULS

M_{rb} = factored moment resistance of the transverse beam

10.16.4 Deflection

At the SLS, the deflection due to live load plus the dynamic load allowance shall not exceed the following:

(a) for the deck plate, 0.0033 times the spacing of rib webs; and

(b) for longitudinal ribs and transverse beams, 0.001 times their respective spans.

In addition, the extreme relative deflection between adjacent ribs shall not exceed 2.5 mm.

10.16.5 Girder diaphragms

Diaphragms or cross-frames shall be provided at each support and shall be of sufficient stiffness and strength to transmit lateral forces to the bearings and to resist transverse rotation, displacement, and distortion.

Intermediate diaphragms or cross-frames shall be provided at locations consistent with the analysis of the girders.

10.16.6 Design detail requirements

10.16.6.1 Minimum plate thickness

The deck plate thickness, t, shall not be less than 14 mm or less than 0.04 times the larger spacing of rib webs.

10.16.6.2 Closed ribs

The thickness of closed ribs shall not be less than 6 mm.

The parameter $C = \dfrac{t_r\,(a')^3}{t_{de}^3 h'}$ shall not exceed 400

where

t_{de} = effective thickness of the deck plate, taking into consideration the stiffening effect of the surfacing as specified in Clause 10.16.7

Closed ribs shall be sealed against entrance of moisture by continuous welds at the rib-to-deck plate interface and by welded diaphragms at their ends.

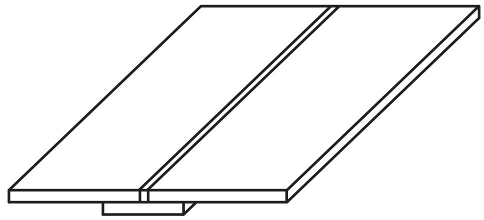
Partial penetration groove welds between the webs of closed ribs and the deck plate shall not be less than 80%.

10.16.6.3 Deck and rib details

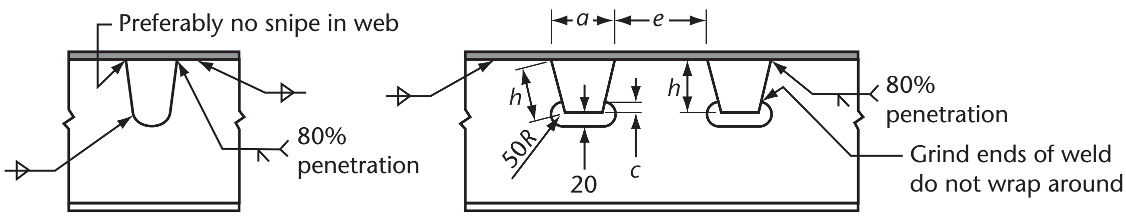
Deck and rib splices shall be welded or mechanically fastened with high-strength bolts, using details consistent with Figure 10.5. The fatigue requirements of Clause 10.17 shall also be satisfied.

Ribs shall extend continuously through cutouts in the webs of transverse floor beams, as shown in Figure 10.5.

Welding of attachments, utility supports, lifting lugs, or shear connectors to the deck plates or the ribs shall require Approval.

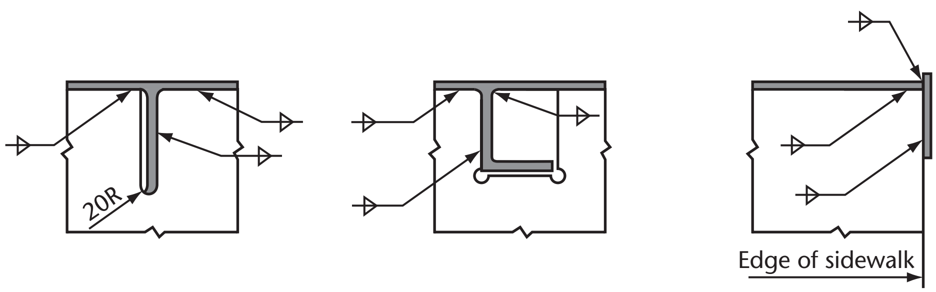


(a) Welded longitudinal field splice of deck plate



Note: $c \geq h/3$ (75 mm *minimum*).

(b) Intersection of closed ribs with floor beams



(c) Open ribs at floor beams

Figure 10.5
Detailing requirements for orthotropic decks
(See Clause 10.16.6.3.)

10.16.7 Wearing surface

The wearing surface shall be considered an integral part of the orthotropic deck and shall be bonded to the top of the deck plate.

The contribution of a wearing surface to the stiffness of the members of an orthotropic deck shall not be considered unless the structural and bonding properties are satisfactorily demonstrated over the design temperature range. If the contribution is considered, the required engineering properties of the wearing surface shall be specified on the Plans.

For the purpose of design, the following requirements shall apply:
(a) the long-term composite action between the deck plate and the wearing surface shall be demonstrated by both static and dynamic cyclic load tests;
(b) the determination of force effects in the wearing surface and at the interface with the steel deck plate shall take into account the engineering properties of the wearing surface at anticipated extreme service temperatures; and
(c) the wearing surface shall be assumed to act compositely with the deck plate whether or not the deck plate is designed on that basis.

10.17 Structural fatigue

10.17.1 General

The FLS considered shall include direct live load effects, i.e., live load-induced fatigue, and the effects of local distortion within the structure, i.e., distortion-induced fatigue.

10.17.2 Live-load-induced fatigue

10.17.2.1 Calculation of stress range

The stress range for load-induced fatigue shall be calculated using ordinary elastic analysis and the principles of mechanics of materials. A more sophisticated analysis shall be required only in cases not covered in Tables 10.7 and 10.8, such as major access holes and cutouts. Because the stress range shall be the algebraic difference between the maximum stress and minimum stress at a given location, only the stresses due to live load shall be considered.

At locations where the stresses resulting from the permanent loads are compressive, load-induced fatigue shall be disregarded when the compressive stress is at least twice the maximum tensile live load stress.

10.17.2.2 Design criteria

For load-induced fatigue, except in bridge decks, each detail shall satisfy the requirement that

$$0.52 f_{sr} < F_{sr}$$

where

f_{sr} = calculated fatigue stress range at the detail due to passage of the CL-W Truck, as specified in Clause 3.8.3.2

For load-induced fatigue in bridge decks, each detail shall satisfy the requirement that

$$0.62 f_{sr} \leq F_{sr}$$

where

f_{sr} = calculated fatigue stress range at the detail due to passage of a tandem set of 125 kN axles spaced 1.2 m apart and with a transverse wheel spacing of 1.8 m

10.17.2.3 Fatigue stress range resistance

10.17.2.3.1 Fatigue stress range resistance of a member or detail

The fatigue stress range resistance of a member or a detail, F_{sr} , other than for shear studs or cables, shall be calculated as follows:

$$F_{sr} = (\gamma / N_c)^{1/3} \geq F_{srt} / 2$$

where

γ = fatigue life constant pertaining to the detail category established in accordance with Clause 10.17.2.4 and specified in Table 10.4

N_c = $365 y N_d (ADTT_f)$

where

y = design life (equal to 75 years unless otherwise specified by the Owner or Engineer)

N_d = number of design stress cycles experienced for each passage of the design truck, as specified in Table 10.5

$ADTT_f$ = single-lane average daily truck traffic, as obtained from site-specific traffic forecasts. In lieu of such data, $ADTT_f$ shall be estimated as $p\ (ADTT)$, where p is 1.0, 0.85, or 0.80 for the cases of one, two, or three or more lanes available to trucks, respectively, and $ADTT$ is as specified in Table 10.6

10.17.2.3.2 Fatigue stress range resistance of fillet welds transversely loaded

The fatigue stress range resistance, F_{sr}, of fillet welds transversely loaded shall be calculated as a function of the weld size and plate thickness, as follows:

$$F_{sr} = F_{sr}^{c}\,[(0.06 + 0.79D/t)/(0.64t^{1/6})]$$

where

F_{sr}^{c} = fatigue stress range resistance for Category C, as determined in accordance with Clause 10.17.2.3.1, based on no penetration of the weld root

D = weld leg size

t = plate thickness

Table 10.4
Fatigue life constants and constant amplitude threshold stress ranges
(See Clauses 10.17.2.3.1 and 13.8.17.7.3.)

Detail category	Fatigue life constant, γ	Constant amplitude threshold stress range, F_{srt}, MPa
A	8190×10^9	165
B	3930×10^9	110
B1	2000×10^9	83
C	1440×10^9	69
C1	1440×10^9	83
D	721×10^9	48
E	361×10^9	31
E1	128×10^9	18
M164	561×10^9	214
M253	1030×10^9	262

Table 10.5
Values of N_d
(See Clause 10.17.2.3.1.)

Longitudinal members	Span length, L, ≥ 12 m	Span length, L, < 12 m
Simple-span girders	1.0	2.0
Continuous girders		
Near interior support (within 0.1L on either side)	1.5	2.0
All other locations	1.0	2.0
Cantilever girders	5.0	5.0
Trusses	1.0	1.0
Transverse members	**Spacing ≥ 6 m**	**Spacing < 6 m**
All cases	1	2

Table 10.6
Average daily truck traffic
(See Clause 10.17.2.3.1.)

Class of highway	*ADTT*
A	4000
B	1000
C	250
D	50

10.17.2.4 Detail categories

The detail categories shall be as specified in Tables 10.7 and 10.8.

The following details shall be prohibited for use when cyclic loading is present:

(a) partial penetration groove welds loaded transversely; and

(b) cover plates attached to girder flanges using only fillet welds that are oriented transversely with respect to the direction of stress in the member.

10.17.2.5 Width-to-thickness ratios of transversely stiffened webs

The width-to-thickness ratios of transversely stiffened webs, h/w, shall not exceed $3150/\sqrt{F_y}$ unless a longitudinal stiffener is provided in accordance with Clause 10.10.7.

In determining a width-to-thickness ratio, F_y may be replaced by the maximum compressive stress due to the factored ULS loads if the maximum shear at the FLS does not exceed V_r calculated in accordance with Clause 10.10.5.1, taking $F_t = 0$ and $\phi_s = 1.0$.

Table 10.7
Detail categories for load-induced fatigue

(See Clauses 10.17.2.1 and 10.17.2.4 and Table 10.8.)

General condition	Situation	Detail category	Illustrative example (see Figure 10.6)*
Plain members	Base metal		1, 2
	With rolled or cleaned surfaces. Flame-cut edges with a surface roughness not exceeding 1000 (25 μm) as specified in CSA B95.	A	
	Of unpainted weathering steel	B	
	At net section of eyebar heads and pin plates	E	
Built-up members	Base metal and weld metal in components, without attachments, connected by one of the following:		3, 4, 5, 7
	Continuous full-penetration groove welds with backing bars removed	B	
	Continuous fillet welds parallel to the direction of applied stress	B	
	Continuous full-penetration groove welds with backing bars in place	B1	
	Continuous partial-penetration groove welds parallel to the direction of applied stress	B1	
	Base metal at ends of partial-length cover plates		
	With bolted slip-critical end connections	B	22
	Narrower than the flange (with or without end welds) or wider than the flange (with end welds)		
	Flange thickness ≤ 20 mm	E	7
	Flange thickness > 20 mm	E1	7
	Wider than the flange (without end welds)	E1	7
Groove-welded splice connections with weld soundness established by non-destructive testing and all required grinding in the direction of the applied stresses	Base metal and weld metal at full-penetration groove-welded splices		
	Of plates of similar cross-sections with welds ground flush	B	8, 9
	With 600 mm radius transitions in width (with welds ground flush)	B	11
	With transitions in width or thickness (with welds ground to provide slopes not steeper than 1.0 to 2.5)		10, 10A
	CSA G40.21, 700Q or 700QT	B1	
	Other base metal grades	B	
	With or without transitions with slopes not greater than 1.0 to 2.5, when weld reinforcement is not removed	C	8, 9, 10, 10A

(Continued)

Table 10.7 (Continued)

General condition	Situation	Detail category	Illustrative example (see Figure 10.6)
Longitudinally loaded groove-welded attachments	Base metal at details attached by full- or partial-penetration groove welds:		
	When the detail length in the direction of applied stress is		
	Less than 50 mm	C	6, 18
	Between 50 mm and 12 times the detail thickness, but less than 100 mm	D	18
	Greater than either 12 times the detail thickness or 100 mm		
	Detail thickness < 25 mm	E	18
	Detail thickness ≥ 25 mm	E1	18
	With a transition radius, *R*, with the ends of welds ground smooth, regardless of detail length		12
	$R \geq 600$ mm	B	
	600 mm $> R \geq 150$ mm	C	
	150 mm $> R \geq 50$ mm	D	
	$R < 50$ mm	E	
	With a transition radius, *R*, with ends of welds not ground smooth	E	12
Transversely loaded groove-welded attachments with weld soundness established by non-destructive testing and all required grinding transverse to the direction of stress	Base metal at detail attached by full-penetration groove welds with a transition radius, *R*, as follows:		12
	To flange, with equal plate thickness and weld reinforcement removed		
	$R \geq 600$ mm	B	
	600 mm $> R \geq 150$ mm	C	
	150 mm $> R \geq 50$ mm	D	
	$R < 50$ mm	E	
	To flange, with equal plate thickness and weld reinforcement not removed or to web		
	$R \geq 150$ mm	C	
	150 mm $> R \geq 50$ mm	D	
	$R < 50$ mm	E	
	To flange, with unequal plate thickness and weld reinforcement removed		
	$R \geq 50$ mm	D	
	$R < 50$ mm	E	
	To flange, for any transition radius with unequal plate thickness and weld reinforcement not removed	E	
Fillet-welded connections with welds normal to the direction of stress	Base metal		
	At details other than transverse stiffener to flange or transverse stiffener to web connections	C	19
	At the toe of transverse stiffener to flange and transverse stiffener to web welds	C1	6
Fillet-welded connections with welds normal and/or parallel to the direction of stress	Shear stress on the weld throat	E	16

(Continued)

Table 10.7 (Concluded)

General condition	Situation	Detail category	Illustrative example (see Figure 10.6)
Longitudinally loaded fillet-welded attachments	Base metal at details attached by fillet welds		
	When the detail length in the direction of applied stress is		
	Less than 50 mm (or stud-type shear connectors are used)	C	13, 15, 18, 20
	Between 50 mm and 12 times the detail thickness, but less than 100 mm	D	18, 20
	Greater than either 12 times the detail thickness or 100 mm		7, 16, 18, 20
	Detail thickness < 25 mm	E	
	Detail thickness ≥ 25 mm	E1	
	With a transition radius, R, with the ends of welds ground smooth, regardless of detail length		12
	$R \geq 50$ mm	D	
	$R < 50$ mm	E	
	With a transition radius, R, with ends of welds not ground smooth	E	12
Transversely loaded fillet-welded attachments with welds parallel to the direction of primary stress	Base metal at details attached by fillet welds		12
	With a transition radius, R, with end of welds ground smooth		
	$R \geq 50$ mm	D	
	$R < 50$ mm	E	
	With a transition radius, R, with ends of welds not ground smooth	E	
Mechanically fastened connections	Base metal		17
	At gross section of high-strength bolted slip-critical connections, except axially loaded joints in which out-of-plane bending is induced in connected materials	B	
	At net section of high-strength bolted non-slip-critical connections	B	
	At net section of riveted connections	D	
Anchor bolts and threaded parts	Tensile stress on the tensile stress area of the threaded part, including effects of bending	E	—
Hollow structural sections fillet-welded to base	Shear stress on fillet weld	E1	21
ASTM A 325 and ASTM A 325M bolts in axial tension	Tensile stress on area A_b	M164	—
ASTM A 490 and ASTM A 490M bolts in axial tension	Tensile stress on area A_b	M253	—

**The numbering of the diagrams in Figure 10.6 parallels the numbering used in Figure 2 of CAN/CSA-S16.*

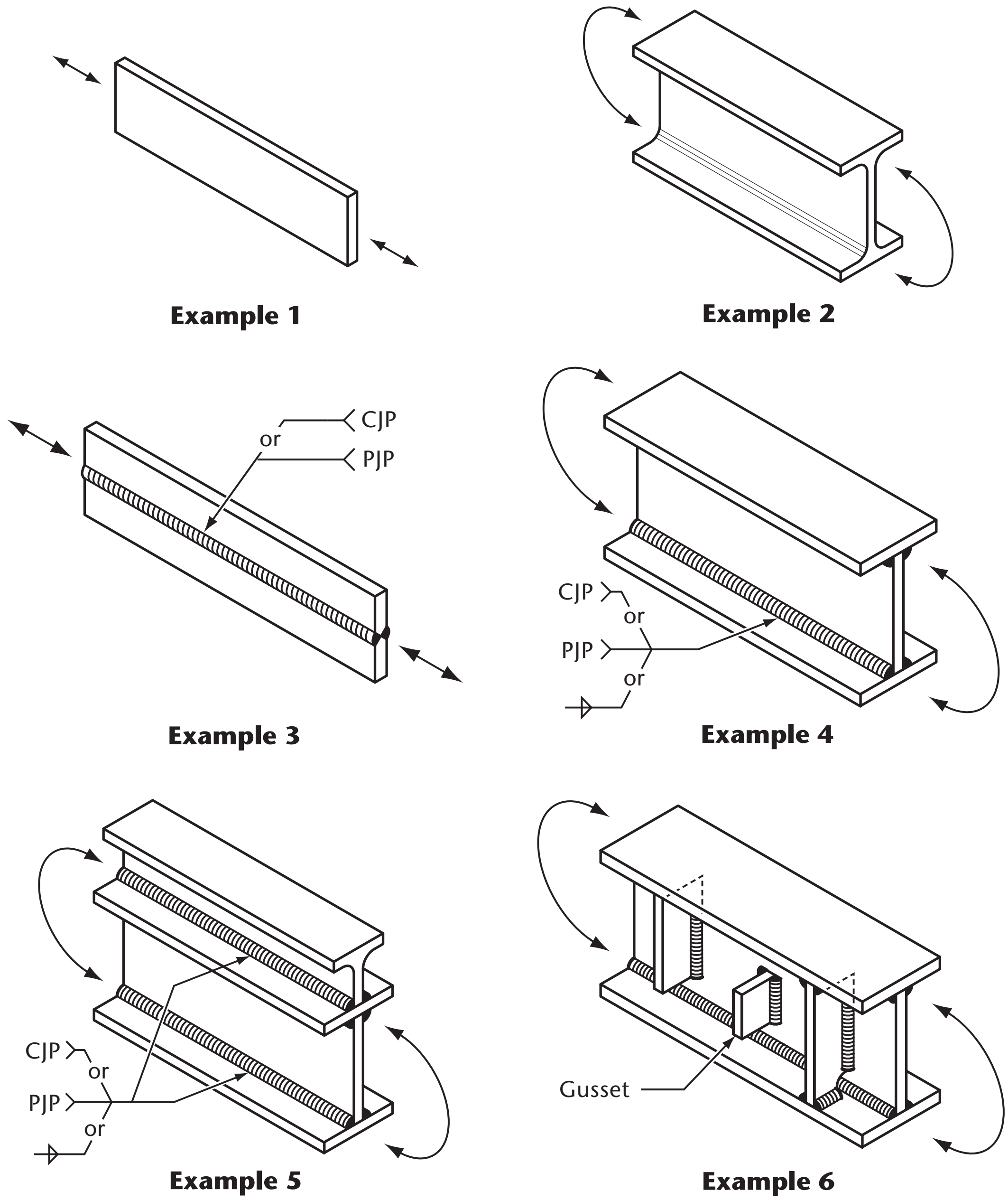


Figure 10.6
Detail categories for load-induced fatigue
(See Table 10.7.)

(Continued)

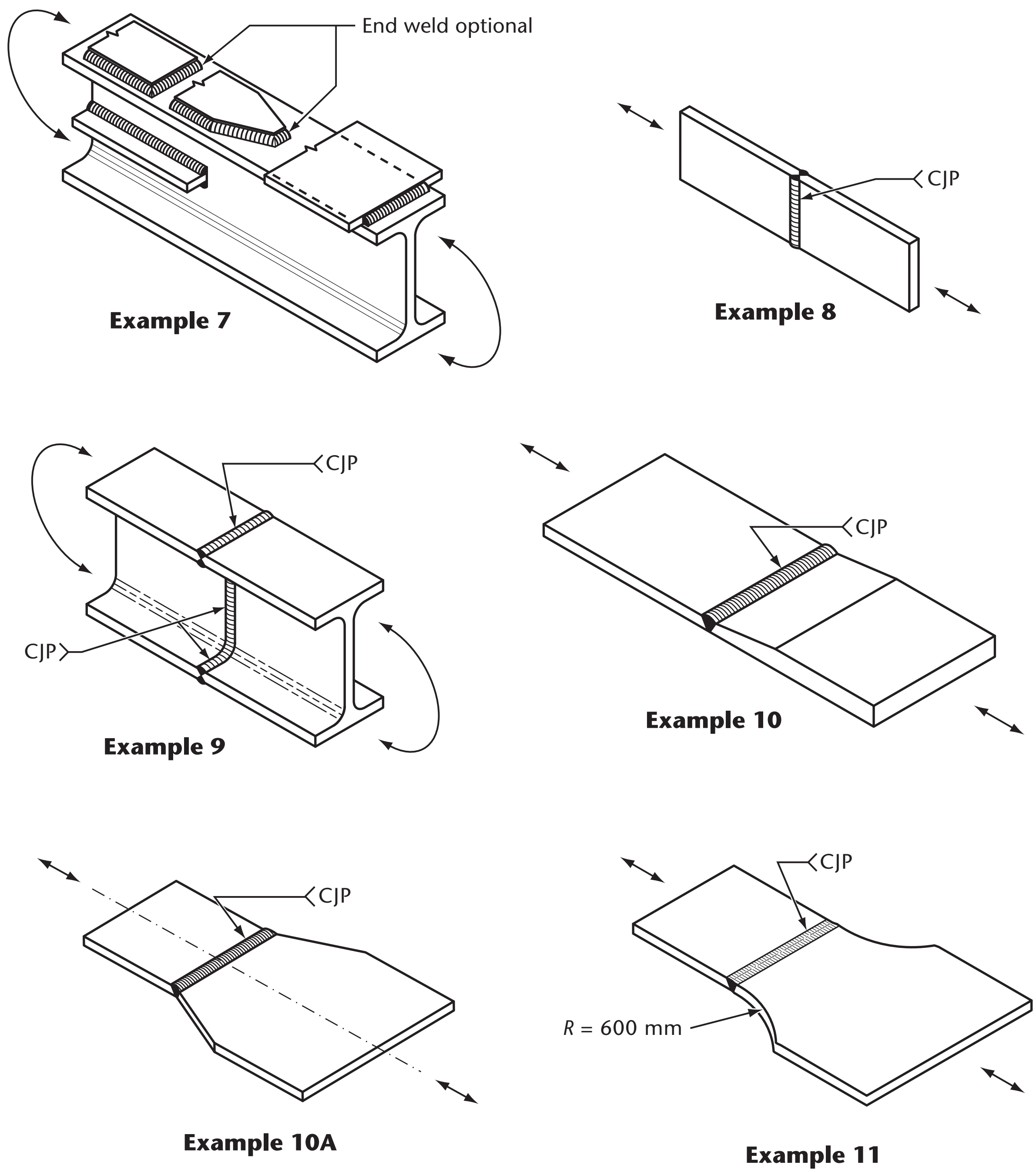
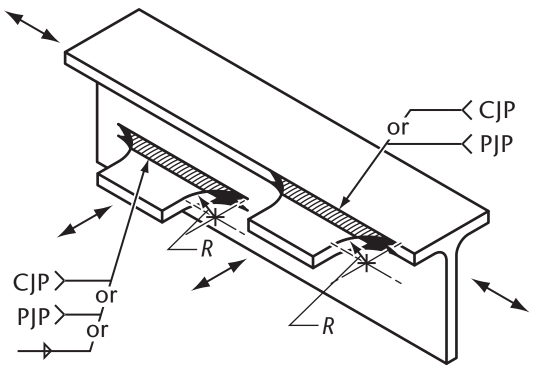
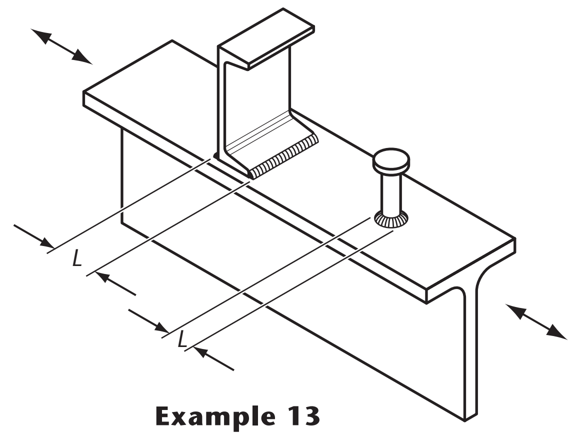


Figure 10.6 (Continued)

(Continued)

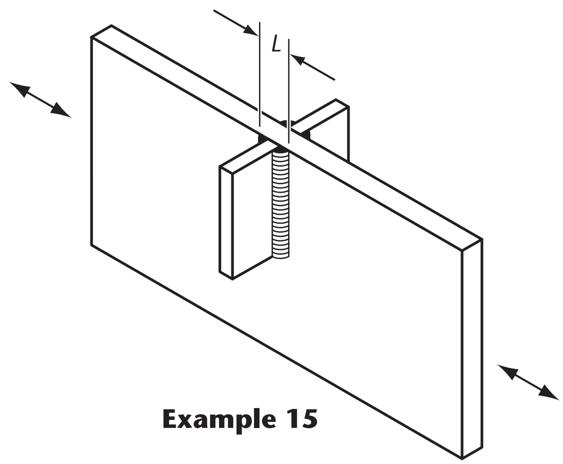


Example 12

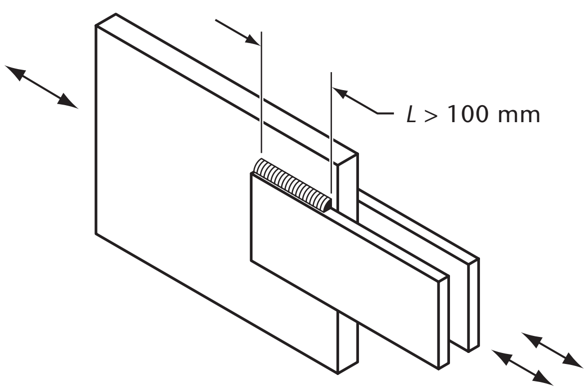


Example 13

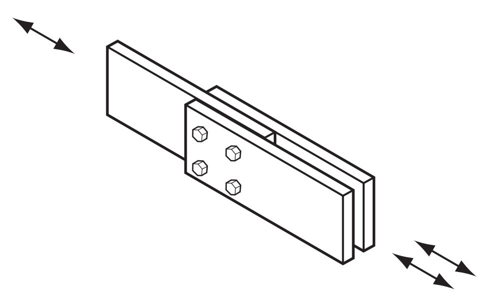
Example 14 not used for bridges



Example 15



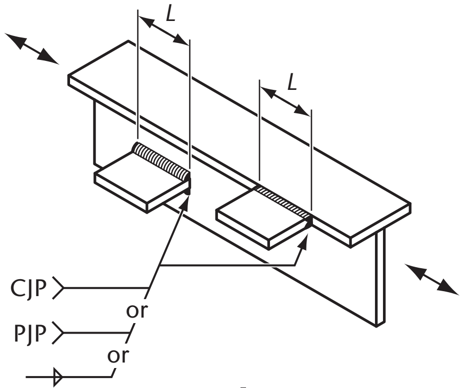
Example 16



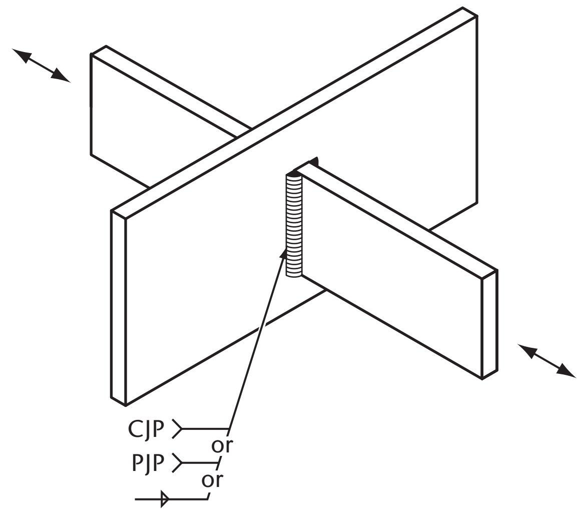
Example 17

Figure 10.6 (Continued)

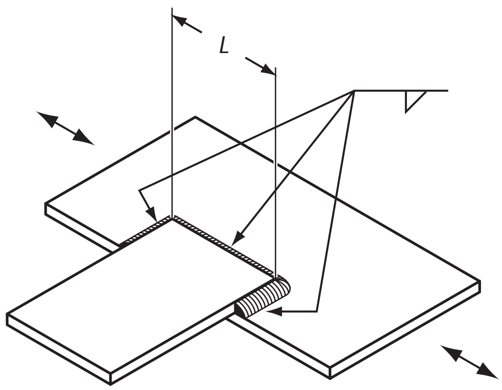
(Continued)



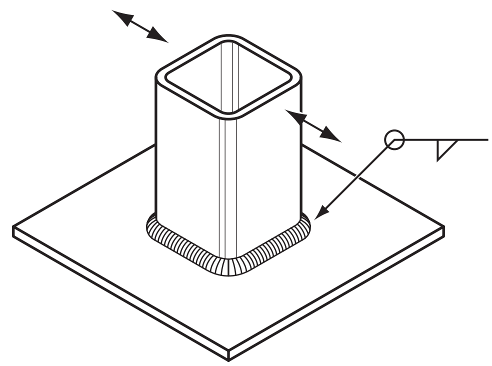
Example 18



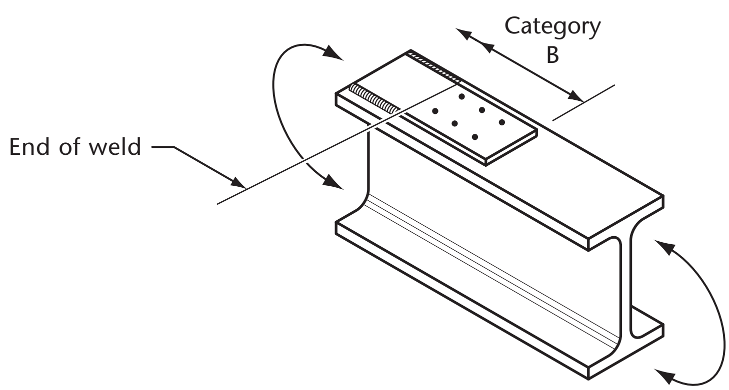
Example 19



Example 20



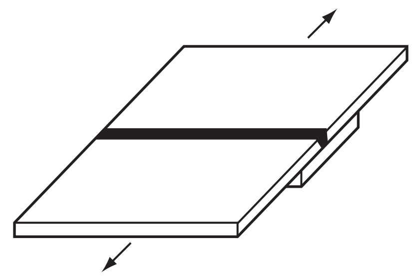
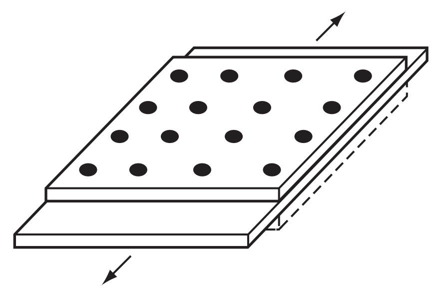
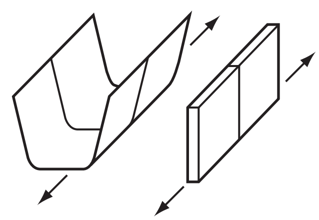
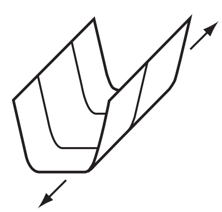
Example 21



Example 22

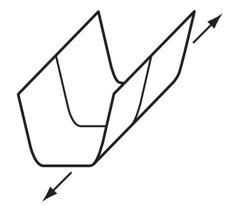
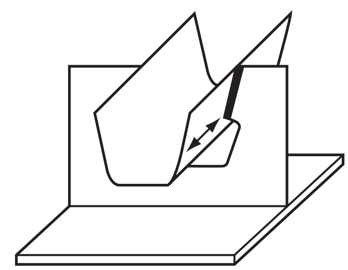
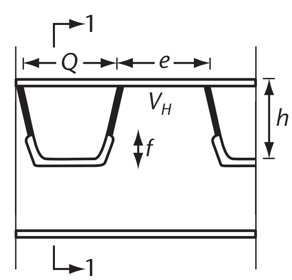
Figure 10.6 (Concluded)

Table 10.8
Detail categories for load-induced fatigue of orthotropic decks
(See Clauses 10.17.2.1 and 10.17.2.4.)

General condition	Situation	Detail category	Illustrative example
Welded transverse deck plate splice	Single-groove butt weld on permanent backing bar. Backing bar fillet welds shall be continuous.	D	
Bolted transverse deck plate splice	In unsymmetrical splices, effects of eccentricity shall be considered in calculating stress. See also "Mechanically fastened connections" in Table 10.7.	B	
Welded rib splices	Double-groove welds. The height of weld convexity shall not exceed 20% of weld width. Weld runoff tabs shall be used and subsequently removed. Plate edges shall be ground flush in the direction of stress.	C	
Welded rib splice with backing bar	Single-groove butt weld with permanent backing bar. Backing bars at fillet welds shall be continuous.	E	

(Continued)

Table 10.8 (Concluded)

General condition	Situation	Detail category	Illustrative example
Welded rib splice without backing bar	Single-groove butt weld without backing bar	E1	
Rib intersection with floor beam	Axial stress in rib wall at the lower end of rib to floor beam weld	D	
Deck plate to floor beam connection	Deck plate stress parallel to floor beam at deck to floor beam junction	D	
Floor beam web at cutout	Vertical stress in floor beam web at floor beam cutout at the bottom of rib. f = stress in floor beam web due to bending moment, $V_H h$, where $V_H = V_{LL+I}\,(a + e)\,Q/I$ and Q and I are properties of the floor beam cross-section at Section 1-1.	D	

10.17.2.6 Fatigue resistance of high-strength bolts loaded in tension

High-strength bolts subjected to tensile cyclic loading shall be pretensioned to the minimum preload specified in Clause 10.24.6.3. Connected parts shall be arranged so that prying forces are minimized. The calculated prying force shall not exceed 30% of the externally applied load.

10.17.2.7 Fatigue resistance of stud shear connectors

The permissible range of interface shear (*N*) of an individual stud-type shear connector, Z_{sr}, shall be taken as

$$Z_{sr} = (238 - 29.5 \log N_c)\, d^2 \geq 19 d^2$$

where

N_c = number of cycles, as specified in Clause 10.17.2.3

d = stud diameter, mm

The shear connectors shall be designed for the following range of interface shear, q_{sr}:

$$q_{sr} = 0.52\frac{V_{sr}Q}{I_t}\frac{s}{n}$$

When shear connectors are not provided in negative moment regions, additional connectors, N_a in number, shall be provided at each location of contraflexure, where

$$N_a = 0.52\frac{A_r f_{sr}}{Z_{sr}}$$

These additional connectors shall be placed within a distance equal to one-third of the effective slab width on each side of the point of dead load contraflexure.

10.17.2.8 Fatigue resistance of cables

10.17.2.8.1 Suspension cables and hangers

Cables and hangers used in suspension bridge construction need not be designed for fatigue, unless, in the judgment of the Engineer, special fatigue provisions are required.

10.17.2.8.2 Cable-stays and cable-stayed bridge tie-downs

10.17.2.8.2.1 Inspectable stays

The fatigue stress range for cable-stays and tie-downs that are replaceable without significant loss of function of the bridge, and in which wire breaks can be detected in service, shall not exceed the fatigue stress resistance established by test. An acceptable test is a test of cable and sockets in which the stress range is applied for 2 000 000 cycles and, at the end of which, the test stay has at least 0.95 of its specified breaking strength. The lowest stress range of three successful tests shall be taken as the fatigue stress range resistance.

For the purpose of this Clause, secondary (bending) stresses shall be calculated, but only secondary stresses exceeding 50 MPa shall be added to the primary (tension) stress to derive the test fatigue stress range.

10.17.2.8.2.2 Non-inspectable or non-replaceable stays

The fatigue stress range resistances for cable stays and tie-downs in which wire breaks cannot be detected while they are in service, or for cable stays and tie-downs that cannot be readily replaced, shall not exceed 0.75 of the fatigue stress range resistance established by test.

10.17.3 Distortion-induced fatigue

10.17.3.1 General

When members designed in accordance with Clause 10.17.2 for load-induced fatigue are provided with interconnection components such as diaphragms, cross-bracing, and lateral bracing, both the members and the interconnection components shall be examined for distortion-induced fatigue. Wherever practicable, elements of the primary member shall be fastened to the interconnection member unless otherwise Approved. The requirements for controlling web buckling and flexing of girder webs specified in Clause 10.17.3.2.2 shall apply.

10.17.3.2 Connection of diaphragms, cross-frames, lateral bracing, and floor beams

10.17.3.2.1 Connection to transverse elements
Unless otherwise Approved, the connections of diaphragms, including internal diaphragms, cross-frames, lateral bracing, floor beams, etc., to main members shall be made using transverse connection plates that are welded or bolted to both the tension and compression flanges of the main member. If transverse stiffeners of the main members form part of the connection, they shall be similarly connected.

In straight non-skewed bridges, the connections shall be designed to resist a factored horizontal force of 90 kN unless a more exact value is determined by analysis.

10.17.3.2.2 Connection to lateral elements
If connections of diaphragms, including internal diaphragms, cross-frames, lateral bracing, floor beams, etc., are to be made to elements that are parallel to the longitudinal axis of the main member, the lateral connection plates shall be attached to both the tension and compression flanges of the main member. Where this is not practicable, then lateral connection plates shall be located as follows:

(a) Transversely stiffened girders: where lateral connection plates are fastened to a transversely stiffened girder, the attachment shall be located at a vertical distance not less than one-half the flange width from the flange. If located within the depth of the web, the lateral connection plate shall be centred with respect to the transverse stiffener, whether or not the stiffener and the connection plate are on opposite sides of the web. If the lateral connection plate and the transverse stiffener are located on the same side of the web, the plate shall be attached to the stiffener. Transverse stiffeners at locations where lateral connection plates are attached shall be continuous between the flanges and shall be fastened to them. Bracing members attached to the lateral connection plates shall be located so that their ends are at least 100 mm from the face of the girder web and the transverse stiffener.

(b) Transversely unstiffened girders: lateral connection plates may be fastened to a transversely unstiffened girder, provided that the attachment is located a vertical distance not less than one-half the flange width or 150 mm from the flange. Bracing members attached to the lateral connection plates shall be located so that their ends are at least 100 mm from the face of the girder web.

10.17.4 Orthotropic decks
Distortion-induced fatigue shall be minimized through appropriate detailing in accordance with Clause 10.16.6. The stress ranges for live-load-induced fatigue shall be as specified in Clause 10.17.2.

10.18 Splices and connections

10.18.1 General

10.18.1.1 General design considerations
Splices and connections shall be designed at the ULS for the larger of

(a) the calculated forces at the splice or connection; or

(b) 75% of the factored resistance of the member, such resistance to be based on the condition of tension, compression, bending, or shear that governed selection of the member.

Except for handrails and non-load-carrying components, connections shall contain at least two 16 mm diameter high-strength bolts or equivalent welds.

10.18.1.2 Alignment of axially loaded members
When the centroidal axes of axially loaded members meeting at a joint do not intersect at a common point, the effect of joint eccentricity shall be considered.

10.18.1.3 Proportioning of splices and connections
Splices and connections shall be designed for all of the forces, including axial, bending, and shear forces, that can occur in the connected components (allowing for any eccentricity of loading). Where the fatigue requirements of Clause 10.17 govern the design, the connections shall be designed to the same requirements.

10.18.1.4 Compression members finished to bear
At the ends of compression members that are finished to bear, splice material and connecting bolts or welds shall be arranged to hold all of the components in place and shall be proportioned to resist not less than 50% of the force effects at the ULS.

10.18.1.5 Beam and girder connections
End connections for beams and girders that are proportioned to resist vertical reactions only shall be detailed to minimize the flexural end restraint, except that inelastic action in the connection at the SLS shall be permitted in order to accommodate the end rotations of unrestrained simple beams.

The connections of beams and girders subject to both reaction shear and end moment due to rigid, continuous, or cantilever construction shall be proportioned for the loads at the ULS. Axial forces, if present, shall also be considered.

10.18.2 Bolted connections

10.18.2.1 General
All high-strength bolts shall be pretensioned in accordance with Clause 10.24.6.3.

10.18.2.2 Bolts in tension

10.18.2.2.1 Tensile resistance at the ultimate limit states
The factored tensile resistance, T_r, developed by the bolts in a bolted joint subject to tension, T_f, shall be taken as

$$T_r = 0.75\phi_b nA_b F_u$$

where

F_u = specified ultimate tensile strength of the bolt material

Bolts in tension shall be proportioned to resist the factored tensile force, T_f, taken as the sum of the factored external load and any additional tension resulting from prying action produced by the deformations of the connected parts, but neglecting bolt pretension.

10.18.2.2.2 Tensile resistance at the fatigue limit state
High-strength bolts subjected to tensile cyclic loading shall meet the requirements of Clause 10.17.2.6.

10.18.2.3 Bolted joints in shear

10.18.2.3.1 General
Bolted joints required to resist shear between the connected parts shall be designed as slip-critical connections unless otherwise Approved.

10.18.2.3.2 Slip resistance at the serviceability limit states
The slip resistance, V_s, of a bolted joint in a slip-critical connection subjected to shear, V, shall be taken as

$$V_s = 0.53c_1 k_s mnA_b F_u$$

where

k_s = mean slip coefficient determined in accordance with Table 10.9 or by Approved tests

c_1 = coefficient that relates the specified initial tension and mean slip to a 5% probability of slip for bolts installed by turn-of-nut procedures, as specified in Table 10.9

For installation using other procedures, different values of c_1 apply. In long-slotted holes, the shear resistance shall be taken as $0.75V_s$.

A slip-critical connection shall also satisfy the shear and bearing criteria at the ULS.

Table 10.9
Values of k_s and c_1
(See Clauses 10.18.2.3.2 and 10.24.6.2.)

Contact surface of bolted parts				
Class	**Description**	k_s	c_1 **(ASTM A 325)**	c_1 **(ASTM A 490)**
A	Clean mill-scale or blast cleaned with Class A coatings	0.33	0.82	0.78
B	Blast-cleaned surfaces or blast-cleaned surface with Class B coatings	0.50	0.90	0.85
C	Hot-dip galvanized with hand wire-brushed surfaces	0.40	0.90	0.85

Note: *Class A and B coatings are those coatings that provide a mean slip-coefficient of not less than 0.33 and 0.50, respectively. Values of c_1 for a 5% probability of slip for values of k_s other than those specified in this Table shall be determined by an Approved means.*

10.18.2.3.3 Shear resistance at the ultimate limit states

The factored shear resistance of a bolted joint subject to a shear force, V_f, shall be taken as the lesser of

(a) the bearing resistance, B_r, of the plate adjacent to the bolts, as follows:

$$B_r = 3\phi_{br} ntdF_u$$

where

F_u = ultimate strength of the plate

ϕ_{br} = 0.67

(b) the shear resistance, V_r, of the bolts, as follows:

$$V_r = 0.60\phi_b \, nmA_b \, F_u$$

where

F_u = ultimate strength of the bolt material

If any bolt threads are intercepted by a shear plane, the factored shear resistance of the joint shall be taken as $0.7V_r$. In an axially loaded splice, with $L > 15d$, where d is the bolt diameter and L is the shear transfer length, the shear resistance of the bolts shall be taken as $1.075 - 0.005L/d$, but not less than 0.75 times the applicable value specified in this Clause.

10.18.2.4 Bolts in shear and tension

10.18.2.4.1 Resistance at the serviceability limit states

Bolts in a connection subjected to loads that cause shear, V, and tension, T, shall satisfy the following relationship:

$$\frac{V}{V_s} + \frac{1.9T}{nA_bF_u} \leq 1.0$$

The requirements of Clause 10.18.2.3.2 shall also be met.

10.18.2.4.2 Resistance at the ultimate limit states

A bolt that is required to resist a tensile force and a shear force at the ULS shall satisfy the following relationship:

$$\left[\frac{V_f}{V_r}\right]^2 + \left[\frac{T_f}{T_r}\right]^2 \leq 1.0$$

10.18.3 Welds

10.18.3.1 General

Welding design shall comply with CSA W59, except as otherwise specified in Clause 10.18.3. The matching electrode classifications for CSA G40.21 and CSA W59 steels shall be as specified in Table 10.10.

Table 10.10
Matching electrode classifications for CSA G40.21 and CSA W59 steels
(See Clause 10.18.3.1.)

Matching electrode*	CSA G40.21 grade						
	260	300	350	380	400	480	700
430	X	X†					
490	X	X	X‡	X			
550					X‡		
620						X	
820							X

**The number indicates the tensile strength of the weld metal in megapascals, as indicated in the electrode classification number.*
†For hollow structural steel sections only.
‡For uncoated applications using "A" or "AT" steels, where the deposited weld metal is to have atmospheric corrosion resistance and/or corrosion characteristics similar to those of the base metal, the requirements of Clauses 5.2.1.4 and 5.2.1.5 of CSA W59 shall apply.

10.18.3.2 Shear

10.18.3.2.1 Complete and partial joint penetration groove welds

The factored shear resistance, V_r, shall be taken as the lesser of

(a) for base metal:

$V_r = 0.67\phi_w A_m F_u$

(b) for weld metal:

$V_r = 0.67\phi_w A_w X_u$

10.18.3.2.2 Fillet welds

The factored resistance for tension- or compression-induced shear, V_r, shall be taken as the lesser of

(a) for base metal:

$$V_r = 0.67\phi_w A_m F_u$$

(b) for weld metal:

$$V_r = 0.67\phi_w A_w X_u (1.00 + 0.50 \sin^{1.5}\theta)$$

where

θ = angle between the axis of the weld and the line of action of the force (0° for a longitudinal weld and 90° for a transverse weld)

Note: *It is a conservative simplification to take the bracketed quantity in Item (b) equal to 1.0.*

10.18.3.3 Tension normal to the weld axis

Complete joint penetration groove welds shall be made with matching electrodes. The factored tensile resistance shall be taken as that of the base metal.

10.18.3.4 Compression normal to the weld axis

Complete and partial joint penetration groove welds shall be made with matching electrodes.

The factored compressive resistance shall be taken as that of the effective area of the base metal in the joint. For partial penetration groove welds, the effective area in compression shall be taken as the nominal area of the fusion face normal to the compression plus the area of the base metal fitted to bear.

10.18.3.5 Hollow structural sections

The provisions of Appendix L of CSA W59 should be applied to hollow structural sections.

Note: *Use of these provisions is strongly recommended.*

10.18.4 Detailing of bolted connections

10.18.4.1 Contact of bolted parts

Bolted parts shall fit together solidly when assembled and shall not be separated by gaskets or any other interposed compressible material.

10.18.4.2 Hole size

The nominal diameter of a hole shall not be more than 2 mm greater than the nominal bolt size, except that where shown on the Plans, oversize or slotted holes may be used with high-strength ASTM A 325 or ASTM A 490 bolts 5/8 in or larger in diameter, or with ASTM A 325M or ASTM A 490M bolts 16 mm or larger in diameter.

Joints with oversize or slotted holes shall meet the following requirements:

(a) Oversize holes shall not be more than 4 mm larger than bolts 22 mm or less in diameter, not more than 6 mm larger than bolts 24 mm in diameter, and not more than 8 mm larger than bolts 27 mm or more in diameter. Oversize holes used in any plies of connections shall be provided with hardened washers under heads or nuts adjacent to plies containing oversize holes.

(b) Short slotted holes shall not be more than 2 mm wider than the bolt diameter and shall not have a length that exceeds the oversize diameter requirements of Item (a) by more than 2 mm. When used in any plies of connections, hardened washers shall be provided under heads or nuts adjacent to plies containing slotted holes.

(c) Long slotted holes shall not be more than 2 mm wider than the bolt diameter and shall not be greater than 2.5 times the bolt diameter. They shall also comply with the following requirements:
   (i) when the slotted hole is normal to the direction of the load, the shear resistance shall be as specified in Clause 10.18.2.3.2;
   (ii) they shall be used in only one of the connected parts at a given faying surface; and

(iii) structural plate washers or a continuous bar not less than 8 mm thick shall cover long slots that are in the outer plies of joints after installation.

(d) When ASTM A 490 or ASTM A 490M bolts larger than 26 mm in diameter are used in oversize or slotted holes in outer plies, the hardened washers shall be at least 8 mm thick and comply with ASTM F 436.

(e) The requirements for the nominal diameter of a hole shall not preclude the use of the following bolt diameters and hole combinations:
   (i) a 3/4 in bolt or an M20 bolt in a 22 mm diameter hole;
   (ii) a 7/8 in bolt or an M22 bolt in a 24 mm diameter hole; and
   (iii) a 1 in bolt or an M24 bolt in a 27 mm diameter hole.

10.18.4.3 Coatings

The faying surfaces of slip-critical connections shall be shown on the Plans as coated or uncoated.

Where faying surfaces are to be coated, one of the following processes shall be used:

(a) hot-dip galvanizing, provided that the faying surfaces are hand wire-brushed after galvanizing and before assembly;

(b) sprayed-zinc coatings, applied in accordance with CSA G189; or

(c) other Approved materials and methods, provided that these have been tested in accordance with the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* issued by the Research Council on Structural Connections.

10.18.4.4 Bolt spacing

The minimum distance between centres of bolt holes shall not be less than 3 bolt diameters wherever practicable and never less than 2.7 diameters. The maximum bolt spacing shall be governed by the requirements for sealing or stitching specified in Clauses 10.18.4.5 to 10.18.4.7.

10.18.4.5 Sealing bolts

For sealing bolts, the pitch, p, between bolts on a single line adjacent to a free edge of an outside plate or shape shall be equal to or less than

$$(100 + 4t) \leq 180$$

When a second line of fasteners is uniformly staggered with those in the line adjacent to the free edge, at a gauge less than $40 + 4t$ therefrom, the staggered pitch, p, in two such lines considered together shall be equal to or less than

$$100 + 4t - \left[\frac{3g}{d}\right] \leq 180$$

or one-half the requirement for a single line, whichever is greater.

10.18.4.6 Stitch bolts

Unless closer spacing is required for transfer of load or for sealing inaccessible surfaces, the longitudinal spacing in-line between intermediate bolts in built-up compression members shall not exceed $12t$. The gauge, g, between adjacent lines of bolts shall not exceed $24t$. The staggered pitch between two adjacent lines of staggered holes shall not exceed

$$p \leq 15t - \left[\frac{3g}{8}\right] \leq 12t$$

The pitch for tension members shall not exceed twice that specified for compression members. The gauge for tension members shall not exceed $24t$.

10.18.4.7 Stitch bolts at the ends of compression members

All component parts that are in contact with one another at the ends of built-up compression members shall be connected by bolts spaced longitudinally not more than four diameters apart for a distance of 1.5 times the width of the member from the end.

10.18.4.8 Minimum edge distance

The minimum edge distance from the centre of a bolt hole to any edge shall be that specified in Table 10.11.

Table 10.11
Minimum edge distance for bolt holes
(See Clauses 10.18.4.8 and 10.18.4.9.)

Bolt designation and diameter, in	Minimum edge distance at sheared edge, mm	Minimum edge distance at rolled or gas-cut edge, mm*
M16 (5/8)	28	22
M20 (3/4)	32	25
M22 (7/8)	38†	28
M24 (1)	44†	32
M27 (1-1/8)	51†	38
M30 (1-1/4)	57	41
M36 (over 1-1/4)	1.75 × diameter	1.25 × diameter

**Edge distances in this column may be decreased by* 3 mm *when the hole is at a point where calculated stress under factored loads is not greater than 0.3 times the yield stress.*
†*At the ends of beam framing angles, the minimum distance shall be* 32 mm.

10.18.4.9 Minimum end distance

When tension member connections have more than two bolts in line parallel to the direction of load, the minimum end distance measured from the centre of the end fastener to the nearest end of the connected part shall not be less than the applicable edge distance value specified in Table 10.11.

In connections with one or two bolts in the line of the load, the end distance shall not be less than 1.5 bolt diameters.

10.18.4.10 Maximum edge or end distance

The maximum distance from the centre of a bolt to the nearest edge of connected components shall be the lesser of eight times the thickness of the outside connected component and 125 mm.

10.18.4.11 Sloping surfaces

Bevelled washers shall be used under the head or nut in accordance with Clause 10.24.6.5 when the two bearing surfaces are not parallel.

10.18.4.12 Fillers

When load-carrying fasteners pass through fillers with a total thickness greater than 6 mm, the fillers shall be extended beyond the splice material and the filler extension shall be secured by sufficient fasteners to distribute the total force in the member at the ULS uniformly over the combined section of the member and filler. Alternatively, an equivalent number of fasteners shall be included in the connection without extending the filler. Fillers shall not consist of more than two plates unless a greater number of plates is Approved.

10.18.5 Connection reinforcement and stiffening

10.18.5.1 General
Connections shall be made by suitably designed pins, by direct welding of one member to another, or by bolts or welds with gusset plates.

10.18.5.2 Gusset plates
The tensile resistance (including block tearout) and the compressive resistance of gusset plates shall be assessed as appropriate.

The factored shear resistance, V_r, of the gusset plate shall be taken as

$V_r = 0.50\phi_s A_g F_y$ on the gross section

$\quad = 0.50\phi_s A_n F_u$ on the net section

where

A_n = minimum cross-sectional area subjected to shear, allowing for holes if present

Re-entrant cuts, except curves made for appearance, shall be avoided as far as practicable. The unsupported edge of a gusset plate shall be stiffened if its length exceeds $930/\sqrt{F_y}$ times its thickness.

10.18.5.3 Moment connections

10.18.5.3.1 General
For beams rigidly framed to the flange of an H-shaped member, stiffeners shall be provided on the web of the H-shaped member in accordance with Clauses 10.18.5.3.2 to 10.18.5.3.5.

10.18.5.3.2 Stiffeners opposite compression flanges
Stiffeners shall be provided opposite the compression flange of the beam when one of the following factored bearing resistances, B_r, is exceeded:

(a) $$B_r = 300\phi_s w_c^2\left[1+3\left[\frac{t_b}{h_c}\right]\left[\frac{w_c}{t_c}\right]^{1.5}\right]\sqrt{\frac{F_{yc}t_c}{w_c}} < \frac{M_f}{d}$$

(b) $$B_r = \phi_s w_c\left(t_b + 5k\right)F_{yc} < \frac{M_f}{d}$$

(c) $$B_r = \phi_s\left[\frac{640\ 000}{\left(h_c/w_c\right)^2}\right]w_c\left(t_b + 5k\right) < \frac{M_f}{d}$$ (for H-shaped members with Class 3 webs)

10.18.5.3.3 Stiffeners opposite tension flanges
Stiffeners shall be provided opposite the tension flange of the beam when the following factored tensile resistance, T_r, is exceeded:

$$T_r = 7\phi_s F_{yc} t_c^2 < \frac{M_f}{d}$$

10.18.5.3.4 Stiffener force
The stiffener or pair of stiffeners opposite either beam flange shall be proportioned for a factored force at the ULS of

$$F_{st} = \frac{M_f}{d} - B_r$$

10.18.5.3.5 Stiffener connection and stiffener length
Stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one side of a column only, the stiffeners need not be longer than one-half the depth of the column.

10.19 Anchors

10.19.1 General
Anchors provided to connect the superstructure to the substructure shall be proportioned to withstand the effect of uplift forces, bending moments, and shear at the ULS determined in accordance with Sections 3 and 5.

Anchor bolts for bearing assemblies shall have a minimum diameter of 30 mm and a minimum embedment length of 300 mm.

The compression resistance of the concrete, the anchorage of bolts, the shear resistance between the baseplate and substructure, and the moment resistance of anchorage systems shall be determined in accordance with Clause 8.16.7.

10.19.2 Anchor bolt resistance

10.19.2.1 Tension
The factored tensile resistance, T_r, of an anchor bolt shall be taken as

$$T_r = \phi_b A_s F_u$$

where

A_s = tensile stress area

$= \frac{\pi}{4}(d - 0.938p)^2$

where

p = pitch of threads, mm

10.19.2.2 Shear
The factored shear resistance, V_r, of an anchor bolt shall be taken as

$$V_r = 0.60\phi_b\, n\, A_b\, F_u$$

but not be greater than the lateral bearing resistance specified in Clause 8.16.7.3. When bolt threads are intercepted by the shear plane, the factored resistance shall be taken as $0.70V_r$.

10.19.2.3 Combined tension and shear
An anchor bolt required to develop resistance to both tension and shear shall be proportioned so that

$$\left[\frac{V_f}{V_r}\right]^2 + \left[\frac{T_f}{T_r}\right]^2 \le 1.0$$

where

V_r = portion of the total shear per bolt transmitted by bearing of the anchor bolts on the concrete, as required by Section 8

10.19.2.4 Combined tension and bending

Anchor bolts required to develop resistance to both tension and bending shall be proportioned to meet the requirements of Clause 10.8.3. The tensile and moment resistances, T_r and M_r, respectively, shall be based on the properties of the cross-section at the critical section; M_r shall be taken as $\phi_b S F_y$.

10.20 Pins, rollers, and rockers

10.20.1 Bearing resistance

The factored bearing resistance, B_r, developed by a component or portion of a component subjected to bearing shall be calculated as follows:

(a) on the contact area of machined, accurately sawn, or fitted parts and on the bearing area of pins:

$B_r = 1.50\phi_s F_y A$

where the bearing area of pins is taken as the pin diameter multiplied by the thickness of the connected parts; and

(b) on expansion rollers or rockers:

$$B_r = 0.00026\phi_s \left[\frac{R_1}{1 - \frac{R_1}{R_2}} \right] L F_y^2$$

where

F_y = specified minimum yield point of the weaker part in contact

10.20.2 Pins

10.20.2.1 Bending resistance

The factored bending resistance of a pin shall be taken as $M_r = \phi_s S F_y$.

10.20.2.2 Shear resistance

The factored shear resistance of a pin shall be taken as $V_r = 0.60\phi_s A F_y$.

10.20.2.3 Combined bending and shear

Sections of pins subject to both bending and shear shall be proportioned so that

$$\frac{M_f}{M_r} + \left[\frac{V_f}{V_r} \right]^3 \leq 1.0$$

10.20.2.4 Pin Connection Details

Pins shall be of sufficient length to ensure full bearing of all parts connected to the turned body of the pin. They shall be secured in position by hexagonal recessed nuts or by hexagonal solid nuts with washers or, if the pins are bored, by throughrods with recessed cap washers. Pin nuts shall be malleable steel castings and shall be secured by cotter pins in the screw ends.

Components shall be held against lateral movement on pins.

The location of pins with respect to the centroidal axes of components shall be such as to minimize stresses due to bending.

Pin plates shall have a width commensurate with the dimension of the member. Their length, measured from pin centre to end, shall be at least equal to their width. Pin plates shall contain sufficient fasteners to distribute their due portion of the pin load to the full cross-section of the component. Only fasteners located in front of two lines drawn from the centre of the pin toward the body of the components and inclined at 45° on either side of the axis of the component shall be considered effective for this purpose. For welded H-shapes, pin plates shall be provided on both flanges and shear lag effects shall be considered.

10.21 Torsion

10.21.1 General

Members and their connections subjected to torsion shall have sufficient strength and rigidity to resist the torsional moments and forces in addition to other moments and forces. The torsional deformations at the SLS shall be within acceptable limits.

10.21.2 Members of closed cross-section

10.21.2.1 Torsional resistance

The factored torsional resistance, Q_r, taking the warping constant, C_w, to be zero, shall be calculated as

$$Q_r = \frac{2}{\sqrt{3}}\phi_s F_y A't$$

where

t = minimum thickness of material, provided that the width-to-thickness ratios of elements are

(a) for flat plate elements:

$b/t \le 1100/\sqrt{F_y}$

(b) for circular hollow sections (or multi-sided hollow sections that approximate a circle):

$D/t \le 18\,000/F_y$

When a width-to-thickness ratio exceeds a ratio specified in Item (a) or (b), as applicable, the torsional resistance shall be calculated using an elastic analysis.

10.21.2.2 Combined axial compression, flexure, and torsion

Members of closed cross-section subject to combined axial compression, flexure, and torsion shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{M_f}{M_r\left[1-\frac{C_f}{C_e}\right]} + \left[\frac{Q_f}{Q_r}\right]^2 \le 1.0$$

where C_r is as specified in Clause 10.9.3, M_r is as specified in Clause 10.10, and Q_r is as specified in Clause 10.21.2.1.

10.21.2.3 Reinforcement of cut-outs

Members with cut-outs whose torsional design is based on the cross-sectional properties of the closed cross-section shall be detailed as follows:

(a) cut-outs shall have semicircular ends;

(b) the width of a cut-out shall not exceed 0.17 times the circumference of the member;

(c) a stiffener shall be provided around the perimeter of the cutout and welded to develop the full cross-section of the wall. The stiffener shall have a cross-sectional area, A, of

$A = L\left(t/\sqrt{3}\right)$

where

L = length of the cut-out measured parallel to the longitudinal axis of the member

t = thickness of the wall of the member

(d) the width-to-thickness ratio of the outstanding portions of the stiffener shall not exceed $170/\sqrt{F_y}$.

10.21.3 Members of open cross-section

10.21.3.1 St. Venant torsional constant

The St. Venant torsional constant shall be calculated as

$$J = \Sigma \frac{bt^3}{3}$$

where

b = length of the plate element

t = thickness of the plate element

The width-to-thickness ratios of elements shall meet the requirements of Clause 10.9.2.

10.21.3.2 Warping constant

The warping constant, C_w, shall be calculated as follows:

(a) for a doubly symmetric I-shaped section:

$$C_w = \frac{I_y h^2}{4}$$

(b) for a monosymmetric I-shaped section:

$$C_w = \frac{h^2 I_{y1} I_{y2}}{\left(I_{y1} + I_{y2}\right)}$$

where

I_y = moment of inertia about the minor axis

I_{y1}, I_{y2} = moment of inertia of the upper and lower flanges, respectively, about the y-axis of symmetry

(c) for a closed rectangular section:

$$C_w = 2I_f \left[\frac{d}{2}\right]^2 + 2I_w \left[\frac{b}{2}\right]^2$$

where

$$I_f = \frac{b^3 t}{12}$$

$$I_w = \frac{d^3 w}{12}$$

10.21.3.3 Torsional resistance

The factored torsional resistance of members of open cross-section shall be calculated based on accepted principles of elastic torsional analysis, taking into account the St. Venant and warping torsional resistance as a function of the loading and restraint conditions.

10.21.3.4 Combined bending and torsion

For I-shaped members subject to torsion or combined bending and torsion, the maximum combined normal stress due to warping torsion and bending at SLS loads, as determined by an elastic analysis, shall not exceed F_y.

10.22 Piles

10.22.1 Steel piles

The design of steel piles shall be in accordance with this Section and Section 6.

10.22.2 Effective length

The effective length factor, K, of an axially loaded pile shall be determined on the basis of the restraint provided by the soil, the pile cap, the superstructure, and the substructure as specified in Section 6.

10.22.3 Splices

Splices shall be proportioned to develop the full cross-sectional strength of the pile.

10.22.4 Composite tube piles

For composite tube piles, the applicable requirements of Clause 10.9.5 shall be met.

10.23 Fracture control

10.23.1 General

Fracture control shall be considered throughout material selection and structural design. Consideration shall be given to
(a) designation of fracture-critical and primary tension members;
(b) the level of quality control, inspection, and monitoring during fabrication;
(c) the likelihood of crack initiation and crack growth;
(d) selection of steel and welding consumables with appropriate toughness; and
(e) controlling stress concentrations and improper alignment.

10.23.2 Identification

The components identified as fracture-critical members and primary tension members shall be clearly identified in the Plans. Shop drawings shall identify the extent of fracture-critical and primary tension members. Attachments longer than 100 mm in the direction of tension and welded to the tension zone of a fracture-critical or primary tension member shall be treated as part of that member.

For each component of a fracture-critical or primary tension member, records shall be kept to identify the heat number of the material and its corresponding mill test certificate.

The fracture-toughness and welding requirements of Clauses 10.23.3 to 10.23.5 shall apply only to members designated as fracture-critical and primary tension members.

10.23.3 Fracture toughness

10.23.3.1 General

The Charpy V-notch requirements specified in Clause 10.23.3 shall apply only to standard full-size specimens. For plates from 8 to 11 mm thick, subsize specimens with adjusted energy levels may be used, as permitted by CSA G40.20. The requirements of Clause 10.23.3 shall apply to both bolted and welded construction.

10.23.3.2 Fracture-critical members
For fracture-critical members, Charpy V-notch tests shall be specified on a per plate frequency, as defined in CSA G40.20/G40.21. The steel shall meet the impact energy requirements specified in Table 10.12.

10.23.3.3 Primary tension members
For primary tension members, Charpy V-notch tests shall be specified on a per heat frequency, as defined in CSA G40.20/G40.21. The steel shall meet the impact energy requirements specified in Table 10.13.

10.23.3.4 Service temperature
The applicable minimum service temperature, T_s, shall be the minimum daily mean temperature in Figure A3.1.2.

10.23.3.5 Weld metal toughness
For fracture-critical and primary tension members, the weld metal shall meet the impact energy requirements of Clause 10.23.4 and Table 10.14.

10.23.3.6 Steel for permanent backing bars
Steel for permanent backing bars shall meet the requirements of Clause 5.5.1.1 of CSA W59 and shall meet the Charpy impact energy requirements of Table 10.12 or 10.13, as applicable.

Table 10.12
Impact test temperatures and Charpy impact energy requirements for fracture-critical members
(See Clauses 10.23.3.2, 10.23.3.6, and 10.23.4.4.)

CSA G40.21 grade	Minimum average energy, J	Test temperature, T_t, °C, for minimum service temperature, T_s, °C		
		$T_s \geq -30$	$-30 > T_s \geq -60$	$T_s < -60$
Commonly used steels				
260 WT	20	–10	–30	–50
300 WT	20	–10	–30	–50
350 WT and AT	27	–10	–30	–50
400 WT and AT	27	–10	–30	–50
Steels requiring Approval				
480 WT and AT	40	–10	–40	–60
700 QT	50	–20	–40	–60

Table 10.13
Impact test temperatures and Charpy impact energy requirements for primary tension members

(See Clauses 10.23.3.3, 10.23.3.6, and 10.23.4.4.)

CSA G40.21 grade	Minimum average energy, J	Test temperature, T_t, °C, for minimum service temperature, T_s, °C		
		$T_s \geq -30$	$-30 > T_s \geq -60$	$T_s < -60$
Commonly used steels				
260 WT	20	0	–20	–30
300 WT	20	0	–20	–30
350 WT and AT	27	0	–20	–30
400 WT and AT	27	0	–20	–30
Steels requiring Approval				
480 WT and AT	27	–10	–30	–40
700 QT	34	–20	–40	–50

Table 10.14
Impact test temperatures and Charpy impact energy requirements for weld metal

(See Clauses 10.23.3.5 and 10.23.4.2–10.23.4.5.)

Base metal CSA G40.21 grade	Minimum average energy, J	Test temperature, T_t, °C, for minimum service temperature, T_s, °C	
		$T_s \geq -40$	$T_s < -40$
260 WT	20	–30	–40
300 WT	20	–30	–40
350 WT and AT	27	–30	–40
400 WT and AT	27	–30	–40
480 WT and AT	27	–45	–45
700 QT	40	–45	–45

10.23.4 Welding of fracture-critical and primary tension members

10.23.4.1 General

The requirements of Clauses 10.23.4.2 to 10.23.4.6 and 10.24 shall apply to the welding of fracture-critical and primary tension members.

10.23.4.2 Welding consumables

Except as permitted by Clause 10.23.4.5, only welding consumables with Charpy V-notch toughness requirements in compliance with Table 10.14 and certified by the Canadian Welding Bureau to CAN/CSA-W48 shall be used. In the absence of an applicable CAN/CSA-W48 requirement, the applicable Standard(s) in the American Welding Society A5 series of Standards shall be used.

In groove welds connecting two different grades of steel, the classification of consumables used, including Charpy V-notch impact requirements, shall be that applicable to the grade with the lower ultimate tensile strength.

10.23.4.3 Approval and verification of consumables
For groove welds in fracture-critical and primary tension members using certified consumables where the Charpy V-notch test temperature required by Table 10.14 is lower than the test temperature required by CAN/CSA-W48 or the applicable Standard(s) in the American Welding Society A5 series of Standards, or where these Standards are not applicable, welding consumables shall be Approved by the Canadian Welding Bureau and qualified using a verification test assembly to establish the impact properties of the weld metal. The test procedures shall be those specified in CAN/CSA-W48 or the applicable American Welding Society Standard, except that only Charpy V-notch tests shall be required and welding shall be carried out using the preheat and the maximum heat input to be used in practice. The Charpy V-notch results shall meet the requirements of Table 10.14. Qualification shall be required for each electrode diameter used and for the consumables supplied by each manufacturer. The qualification shall be valid for consumables for all groove weld procedures that use a heat input the same as or lower than that used in the qualification test.

10.23.4.4 Welding 700Q and 700QT steels
For groove weld procedures involving fracture-critical and primary tension members made of 700Q and 700QT steels, consumables shall be qualified by welding procedure tests and Approved by the Canadian Welding Bureau. The tests shall be conducted in accordance with CSA W47.1 using 700Q or 700QT steels for the base plate and shall include weld metal and heat-affected zone (HAZ) Charpy V-notch impact tests in accordance with Appendix E of CSA W47.1. Weld metal impact tests shall meet the requirements of Table 10.14 and HAZ impact tests shall meet the requirements of Table 10.1 or 10.2 for the base plate, as applicable. Only manufacturers of qualified consumables shall supply consumables for fabrication. The qualification shall be valid for all groove weld procedures that use a heat input the same as or lower than that used in the qualification test.

10.23.4.5 Qualification of consumables
When the welding consumables have not previously been certified by the Canadian Welding Bureau, they shall be qualified by welding procedure tests in accordance with Clause 11.8.2.1(b) of CSA W47.1 and shall include Charpy V-notch impact tests of the weld metal. For steels other than 700Q and 700QT Charpy V-notch tests in the HAZ shall not be required. Weld metal Charpy V-notch properties shall be established by qualification tests in accordance with CSA W47.1 (including Appendix E) and shall meet the requirements of Table 10.14. Only manufacturers of qualified consumables shall supply consumables for fabrication. Qualification testing shall be performed for each lot or batch of consumables. The qualification shall be valid for all weld procedures that use a heat input the same as or lower than that used in the qualification test. Consumables for 700Q and 700QT steels shall be qualified in accordance with Clause 10.23.4.4.

10.23.4.6 Tack welds and temporary welds
Tack welds shall not be used on fracture-critical or primary tension members unless they are incorporated into the final weld. Temporary welds shall not be used on fracture-critical or primary tension members, or on flange material in compression, unless Approved.

10.23.5 Welding corrections and repairs to fracture-critical members

10.23.5.1 General
Except as specified in Clause 10.23.5.4(c), repairs to base metal and to welded joints shall be documented. The documentation shall include all of the details specified in Clauses 10.23.5.6 and 10.23.5.7. Welding repair procedures shall be Approved by the Engineer in accordance with Clauses 10.23.5.4 and 10.23.5.5, as applicable.

10.23.5.2 Repair of base metal
Repair of base metal by welding at the producing mill shall not be permitted.

10.23.5.3 Approval and tests for repairs
Repair welding may be performed using any appropriate welding procedure Approved by the CWB for the fabrication of fracture-critical members and primary tension members. All repair welding shall be subject to non-destructive tests as specified in Clauses 10.23.5.7(n) and 10.23.6.

10.23.5.4 Approval for non-critical repairs
The constructor shall prepare written repair procedures for non-critical repairs as specified in this Clause and submit them to the Engineer for prior Approval. These procedures shall apply to shop repair of discontinuities identified during fabrication. Such Approved repair procedures shall be employed after the Engineer or the Engineer's agent has verified that the discontinuity to be repaired is as described in the Approved procedures. Repairs that may receive prior Approval include the following:
(a) Repairs of welds because of rollover, undercut, or insufficient throat that does not require excavation.
(b) Repairs of welds requiring excavation of defects (including porosity, slag, and lack of fusion), repair of arc strikes, and removal of tack welds not incorporated into a final weld.
(c) Visually detected planar and laminar discontinuities as specified in Table 5.2 of CSA W59, but not deeper than 25 mm or one-half the thickness of the edge of the cut plate, whichever is less. Such discontinuities shall not be within 300 mm of a tension groove weld. There shall also be no visible planar or laminar discontinuity on any prepared face of a tension groove joint prior to welding.
(d) Occasional gouges exceeding 5 mm, but not more than 10 mm deep on edges not to be welded, which may be repaired by welding. The procedures specified in Clause 5.3.4 of CSA W59 shall be followed.

Gouges not more than 5 mm deep on otherwise satisfactory cut or rolled surfaces that can be repaired by machining or grinding without welding shall not require prior Approval. The procedures specified in Clause 5.3.4 of CSA W59 shall be followed.

10.23.5.5 Approval for critical repairs
Repair procedures beyond those described in Clause 10.23.5.4 shall be considered critical and shall be Approved individually before repair welding can begin.

Note: *Critical repairs include the following:*
(a) repairs of lamellar tears, laminations, and cracks, except those meeting the requirements of Clause 10.23.5.4(c);
(b) repairs of surface or internal defects in rolled products, except those meeting the requirements of Clause 10.23.5.4(c);
(c) dimensional corrections requiring weld removal and rewelding; and
(d) any correction by welding to compensate for a fabrication error, e.g., improper cutting or punching or incorrect assembly (other than tack-welded or temporary assemblies).

10.23.5.6 Descriptions of deficiencies and repairs
Repair procedures in accordance with Clauses 10.23.5.4 and 10.23.5.5 shall include sketches or full-size drawings, as necessary, to adequately describe the deficiencies and the proposed method of repair. Critical repair procedures in accordance with Clause 10.23.5.5 shall include the location of the discontinuity.

10.23.5.7 Minimum steps for repair
Repair procedures, except in cases meeting the requirements of Clause 10.23.5.4(a), shall include at least the following steps, which shall be performed in the following order:
(a) Surfaces shall be cleaned, ground, or both, as necessary, to aid visual and non-destructive tests to enable the constructor and Engineer to identify and quantify the discontinuities.
(b) The discontinuities shall be drawn as they appear from visual inspection and non-destructive testing.
(c) Arc-air gouging, when necessary, shall be part of the Approved welding procedure.
(d) Magnetic particle inspection, or another inspection method approved by the Engineer, shall be used to determine whether the discontinuities were removed as planned.
(e) All air-carbon-arc gouged and oxygen-cut surfaces that form a boundary for a repair weld shall be ground to form a smooth, bright surface. Oxygen gouging shall not be used.
(f) All required runoff tabs and backup bars shall be shown in detail.
(g) Preheat and interpass temperatures shall be in accordance with Table 10.15. Preheat and interpass temperatures shall be maintained without interruption until the repair is completed.

(h) The repair procedures shall refer to the applicable welding procedure specification and the related data sheet. If both of these were Approved by the Canadian Welding Bureau before fabrication, they need not be qualified by test for the specific method of repair unless a change in essential variables has been made or unless otherwise required by the Engineer.
(i) If the geometry of the repair joint or of the excavation is similar to the geometry of a prequalified joint preparation as specified in CSA W59 and permits good access to all portions of such joints or excavations during the proposed sequence of welding, it shall not require qualification by test unless required by the Engineer.
(j) Peening, when required, shall be completely described and shall be Approved. Peening equipment shall not contaminate the joint.
(k) Post-heat shall be employed and shall continue without interruption from the completion of repair welding to the end of the minimum specified post-heat period. Post-heat of the repair area shall be between 200 and 260 °C and shall continue for at least 1 h for each 25 mm of weld thickness, or for 2 h, whichever is less.
(l) Faces of repairs shall be ground flush with the plate or blended to the same contour and throat dimension as the remaining sound weld.
(m) If stress-relief heat treatment is required, it shall be completely described. Final acceptance non-destructive testing shall be performed after stress relief is complete.
(n) Repairs of groove welds in fracture-critical members shall be examined by ultrasonic testing and radiographic testing. Fillet weld repairs shall be examined by magnetic particle testing. Radiographic testing shall comply with Clause 7.4.2 of CSA W59 and may be performed as soon as the weldment has cooled to ambient temperature. Ultrasonic testing and magnetic particle testing shall comply with Clause 7.4.3 and 7.4.4, respectively, of CSA W59. Final acceptance testing by magnetic particle and ultrasonic methods shall not be performed until the steel weldments have been at ambient temperature for at least the elapsed time specified in Table 10.16.

Table 10.15
Preheat and interpass temperature for steel grades
(See Clause 10.23.5.7.)

	CSA G40.21 grade	
Plate thickness, *t*, mm	**260WT, 300WT, 350WT, 400WT, 480WT, 350AT 400AT, and 480AT**	**700QT***
$t \leq 25$	65 °C	65 °C
$25 < t \leq 40$	120 °C	120 °C
$t > 40$	175 °C	175 °C

**The maximum preheat and interpass temperatures shall not exceed the recommendations of the steel manufacturer.*

Table 10.16
Minimum elapsed time for acceptance testing
(See Clause 10.23.5.7.)

Plate thickness, t, mm	Magnetic particle for fillet welds	Ultrasonic examination of groove welds
$t \leq 50$	24 h	24 h
$t > 50$	24 h	48 h

10.23.5.8 Compliance with Approved procedures
All repair welding and non-destructive testing shall be performed as described in the Approved repair procedure.

10.23.5.9 Records
Approved critical repair procedures shall be retained as part of the project records.

10.23.6 Non-destructive testing of fracture-critical members
The use of Cobalt 60 as a radiographic source in quality control shall be permitted only when the steel being tested is more than 75 mm thick.

The Constructor shall maintain documentation of all visual and non-destructive testing for review and confirmation by the Engineer. The documentation shall be submitted to the Engineer on completion of the project.

10.24 Construction requirements for structural steel

10.24.1 General
Clauses 10.24.2 to 10.24.10 specify requirements for the construction of structural steel for highway bridges and applies unless otherwise specified by the authority having jurisdiction. The requirements specified in these Clauses are provided to ensure compliance with the design philosophy of this Section.

10.24.2 Submissions

10.24.2.1 General
Erection diagrams, shop details, welding procedures, and erection procedure drawings and calculations shall be submitted to the Owner. This requirement shall be stipulated on the Plans.

10.24.2.2 Erection diagrams
Erection diagrams are general-arrangement drawings showing or indicating the principal dimensions of the bridge, piece marks, the sizes of all members, field welding requirements, the sizes and types of bolts, and bolt installation requirements.

10.24.2.3 Shop details
Shop details shall provide
(a) full detail dimensions and sizes of all component parts of the structure. These dimensions shall make allowance for changes in shape due to weld shrinkage, camber, and any other effects that cause finished dimensions to differ from initial dimensions;
(b) all necessary specifications for the materials to be used;
(c) identification of areas requiring special surface treatment;
(d) identification of fracture-critical and primary tension members and component parts;

(e) bolt installation requirements; and
(f) details of all welds.

10.24.2.4 Welding procedures
Welding procedures shall comply with CSA W47.1.

10.24.2.5 Erection procedure drawings and calculations
The erection procedure drawings and calculations shall fully indicate the proposed method of erection, including the sequence of erection, the weights and lifting points of the members, and the location and lifting capacities of the cranes used to lift them. Details of temporary bracing and bents to be used during construction shall be shown. Calculations shall be provided to show that members and supports are not overloaded during erection.

10.24.2.6 Symbols for welding and non-destructive testing
The symbols for welding and non-destructive testing on shop drawings shall be in accordance with CSA W59.

10.24.3 Materials

10.24.3.1 Steel
Substitution of steel members or components for size and grade shall not be permitted unless Approved. All steel shall be new. Acceptance of any material by an inspector shall not preclude subsequent rejection of the material if it is found defective.

10.24.3.2 High-strength bolts, nuts, and washers
Nuts and bolts shall be shipped together as an assembly.

ASTM A 325/A 325M or ASTM A 490/A 490M high-strength bolts for use with unpainted corrosion-resistant steel shall be Type 3 unless corrosion protection is provided by an Approved protection system.

ASTM A 490 bolts shall not be galvanized or plated.

The nuts of galvanized fasteners shall be overtapped by the minimum amount required for assembly and shall be lubricated with a lubricant containing a visible dye. The use of a mechanically deposited zinc coating shall require Approval.

10.24.3.3 Electrodes
The selection, supply, and storage of electrodes and fluxes shall be in accordance with Clause 5 of CSA W59. Only controlled hydrogen (CH) designation electrodes shall be used for the flux-cored welding process.

The weld metal in fracture-critical and primary tension members shall meet the Charpy V-notch energy requirements specified in Clause 10.23.3.5.

As required by CSA W59, weld metal used with corrosion-resistant steels shall have corrosion resistance and be of a colour similar to that of the base metal.

10.24.3.4 Shear connectors
Shear connectors shall be of a headed stud type in accordance with Appendix H of CSA W59.

10.24.4 Fabrication

10.24.4.1 Quality of work
The standards for quality of work and finish shall comply with the best modern practices for steel bridge fabrication (with particular attention to the appearance of parts exposed to view).

10.24.4.2 Storage of materials

Plain or fabricated structural steel shall be stored above the ground on skids or other supports and kept free from dirt and other foreign matter. Long members shall be adequately supported to prevent excessive deflection.

10.24.4.3 Plates

10.24.4.3.1 Direction of rolling

Unless otherwise shown on the Plans, steel plates for main members (and their splice plates) shall be cut so that the primary direction of rolling is parallel to the direction of tensile or compressive stress.

10.24.4.3.2 Plate edges

Sheared edges of plates more than 16 mm thick and carrying calculated tension shall be planed, milled, or ground to a minimum depth of 3 mm.

Oxygen cutting of structural steel shall be done by machine, except that hand-guided cutting shall be allowed for copes, blocks, and similar cuts where machine cutting is impracticable. Re-entrant corners shall be free from notches and shall have a fillet of the largest practical radius, but not less than 25 mm.

The quality and repair of the cut edges shall comply with Clause 5 of CSA W59. All cut edges that are not to be welded shall have a surface roughness not greater than 1000, as specified by CSA B95.

10.24.4.3.3 Camber in web plates

Webs shall be cut to the prescribed camber, with allowance for shrinkage due to cutting and subsequent welding. The requirements of Clauses 10.7.4.2 and 10.7.4.3 shall also apply.

10.24.4.3.4 Bent plates

The following requirements shall apply to bent plates:

(a) Load-carrying, rolled steel plates to be bent shall
   (i) be cut from the stock plates so that the bend line is at right angles to the direction of rolling, except as otherwise Approved for orthotropic decks; and
   (ii) have their corners lightly chamfered by grinding in the region of the bend before bending.

(b) Cold bending shall be carried out so that no cracking or tearing of the plate occurs. Minimum bend radii, measured to the concave face of the metal, shall be as shown in Table 10.17.

(c) Hot bending at a plate temperature not greater than 600 °C shall be used to form radii less than those specified for cold bending. Accelerated cooling using compressed air or water shall be used for a hot bent component only when its temperature is below 300 °C.

Table 10.17
Minimum bend radii for bent plates
(See Clause 10.24.4.3.4.)

t, mm	Minimum radius
12 or less	$2t$
Over 12 to 25	$2.5t$
Over 25 to 38	$3t$
Over 38 to 65	$3.5t$
Over 65 to 100	$4t$

10.24.4.4 Straightening material

All steel shall be flat and straight before being worked. Steel with sharp kinks or bends may be rejected. Attempts to straighten sharp kinks or bends shall require Approval.

Rolled plates, sections, and built-up members may be straightened using mechanical means or by the application of a controlled heating procedure in accordance with Clause 5.10.5 of CSA W59. After straightening of a bend or buckle, the surface of the steel shall be examined for evidence of fracture or other damage and corrective action taken if necessary.

10.24.4.5 Bolt holes

10.24.4.5.1 General

All holes shall be drilled or reamed to the finished diameter, except that punched holes shall be allowed in material up to 16 mm thick.

When shown on the Plans, oversize or slotted holes in accordance with Clause 10.18.4.2 are permitted.

10.24.4.5.2 Punched holes

The diameter of a punched hole shall be not more than 2 mm larger than the nominal diameter of the bolt unless oversize holes are specified.

The diameter of the die shall not exceed the diameter of the punch by more than 2 mm. Holes shall be clean cut and without ragged or torn edges, but the lightly conical hole that results from clean cutting shall be acceptable. Holes may be reamed to admit fasteners.

10.24.4.5.3 Reamed holes

Holes that are to be reamed to final diameter shall first be subdrilled or subpunched to 4 mm smaller than the nominal bolt diameter of the bolt. With the connecting parts assembled and securely held, the holes shall then be reamed to 2 mm larger than the nominal diameter of the bolts. The parts shall be match-marked before disassembly.

10.24.4.5.4 Drilled holes

Holes that are drilled full-size shall be 2 mm larger than the nominal diameter of the bolt unless oversize holes have been specified. They shall be accurately located by using suitable numerically controlled drilling equipment, or by using a steel template carefully positioned and clamped to the steel. The accuracy of the holes prepared in this manner, and their locations, shall be such that like parts are identical and require no match-marking.

The holes for any connection may be drilled full-size when the connecting parts are assembled and clamped in position, in which case the parts shall be match-marked before disassembly.

10.24.4.6 Pins and rollers

Pins and rollers shall be accurately turned to the dimensions and finish shown on the drawings and shall be straight and free from flaws. Pins and rollers more than 175 mm in diameter shall be forged and annealed. Pins and rollers 175 mm or less in diameter may be either forged and annealed or of cold-finished carbon-steel shafting.

Holes for pins shall be bored to the specified diameter and finish at right angles to the axis of the member. The diameter of the pin hole shall not exceed that of the pin by more than 0.5 mm for pins 125 mm or less in diameter or more than 0.75 mm for larger pins. Pin holes shall be bored on completion of the assembly of built-up members.

10.24.4.7 Curved girders

10.24.4.7.1 General

Flanges of curved, welded I-girders may be cut to the radius. However, they may be curved by applying heat if the radius, R, is greater than 45 000 mm and also exceeds

$$\frac{37 b_f h}{\sqrt{F_y \psi w}} \text{ and } \frac{51\,700 b_f}{F_y \psi}$$

where

F_y = specified minimum yield stress of the web

10.24.4.7.2 Heat curving of rolled beams and welded girders

Steel beams and girders with a specified minimum yield point greater than 350 MPa shall not be heat curved. In heat curving using the continuous or V-type heating pattern, the temperature of the steel shall not exceed 600 °C, as measured by temperature-indicating crayons.

A detailed procedure for the heat-curving operation shall be submitted for review. The procedure shall describe the type of heating to be employed, the extent of the heating patterns, the sequence of operations, and the method of support of the girder, including an assessment of any dead-load stresses present during the operation.

Transverse web stiffeners may be welded in place either before or after the heat-curving operation. However, unless allowance is made for the longitudinal shrinkage, bracing connection plates and bearing stiffeners shall be located and welded after curving.

Girders shall be cambered before heat curving. Rolled sections may be heat cambered using an Approved procedure. Plate girders shall have the required camber cut into the web, with suitable allowance for camber loss due to cutting, welding, and heat curving.

10.24.4.8 Identification marking

Each member shall carry an erection mark for identification.

10.24.5 Welded construction

10.24.5.1 General

All welding procedures, including those related to quality of work, techniques, repairs, and qualifications, shall comply with CSA W59, except where modified by Clauses 10.24.5.2 to 10.24.5.7.

10.24.5.2 Processes with limited application

The electroslag and electrogas welding processes specified in Clause 5 of CSA W59 shall not be used for welding quenched and tempered steels or for welding components of members subject to tension stress or stress reversal.

10.24.5.3 Primary tension and fracture-critical members

Members and components of members designated primary-tension or fracture-critical shall meet the requirements of Clause 10.23 in addition to the requirements of CSA W59. The use of heat to alter the sweep or camber of fracture-critical girders shall require Approval.

10.24.5.4 Submissions

Welding procedure specifications, data sheets, and repair procedures for prequalification shall be submitted to the Owner in compliance with the Plans.

10.24.5.5 Certification of fabrication companies

Any company undertaking welded fabrication in accordance with this Section shall be certified to Division 1 or 2 of CSA W47.1.

10.24.5.6 Complete joint penetration groove welds
Complete joint penetration groove welds shall meet the requirements of Clauses 10 and 12.4 of CSA W59. Unless produced with the aid of a steel backing, they shall have the root of the initial weld gouged, chipped, or otherwise removed to sound metal before welding of the other side is started.

Runoff tabs or extension bars shall be provided so that groove welds terminate on the tab. The welds that attach the tabs to the piece being welded shall be placed inside the joint so that they are incorporated into the final weld.

10.24.5.7 Web to flange fillet welds
Where practicable, web to flange fillet welds shall be made continuously by machine or automatic welding. Welds may be repaired using either a semi-automatic or manual process, but the repaired weld shall blend smoothly with the adjacent welds.

10.24.6 Bolted construction

10.24.6.1 General
Clauses 10.24.6.2 to 10.24.6.11 specify requirements for bolted steel construction using ASTM A 325/A 325M or ASTM A 490/A 490M high-strength bolts.

10.24.6.2 Assembly
When assembled, all joint surfaces, including those adjacent to bolt heads, nuts, and washers, shall be free from loose scale, burrs, dirt, and foreign material that would prevent the solid seating of the parts.

The faying surfaces of connections designed in accordance with Clause 10.18.2.3.2 shall be prepared as follows:

(a) For clean mill scale, the surfaces shall be free from oil, paint, lacquer, or any other coating in all areas within the bolt pattern and for a distance beyond the edge of the bolt hole that is the greater of 25 mm or the bolt diameter.

(b) For Classes A and B (see Table 10.9), the surfaces shall have the same blast cleaning and coating application as was used in the tests to determine the mean slip coefficient. Coated joints shall not be assembled before the coating has cured for the minimum time used in the tests to determine the mean slip coefficient.

(c) For Class C (see Table 10.9), the surfaces shall be hot-dip galvanized in accordance with CAN/CSA-G164 and subsequently roughened by hand wire-brushing. Power wire-brushing shall not be used.

10.24.6.3 Installation of bolts
Pretensioned bolts shall be tightened in accordance with Clause 10.24.6.6 to at least 70% of the minimum tensile strength specified in the applicable ASTM Standard.

10.24.6.4 Hardened washers
The following requirements shall apply to hardened washers:

(a) Hardened washers shall be provided as follows under the element turned (head or nut) during installation:
  (i) as required by Clause 10.24.6.7; and
  (ii) for ASTM A 490/A 490M bolts.

(b) Hardened washers shall also be required
  (i) for oversize or slotted holes that meet the requirements of Clause 10.18.4.2;
  (ii) under the head and nut of ASTM A 490/A 490M bolts when used with steel with a specified minimum yield strength of less than 280 MPa; and
  (iii) when ASTM A 490/A 490M bolts of greater than 26 mm diameter are used in oversize or slotted holes. The washers in this case shall have a minimum thickness of 8 mm.

10.24.6.5 Bevelled washers
Bevelled washers shall be used to compensate for lack of parallelism where, in the case of ASTM A 325/A 325M bolts, an outer face of bolted parts has more than a 5% slope with respect to a plane normal to the bolt axis. In the case of ASTM A 490/A 490M bolts, bevelled washers shall be used to compensate for any lack of parallelism due to the slope of outer faces.

10.24.6.6 Turn-of-nut tightening
After the holes in a joint are aligned, a sufficient number of bolts shall be placed and brought to a snug-tight condition to ensure that the parts of the joint are brought into full contact with each other.

Following the initial snugging operation, bolts shall be placed in any remaining open holes and brought to snug-tightness. Resnugging can be necessary in large joints.

When all bolts are snug-tight, each bolt in the joint shall be further tightened by the applicable amount of relative rotation specified in Table 10.18, with tightening progressing systematically from the most rigid part of the joint to its free edges. During this operation, there shall be no rotation of the part not turned by the wrench unless the bolt and nut are match-marked to enable the amount of relative rotation to be determined.

Table 10.18
Nut rotation from snug-tight condition*
(See Clauses 10.24.6.6 and 10.24.6.7.)

Disposition of outer faces of bolted parts	Bolt length†	Turn from snug
Both faces normal to the bolt axis or one face normal to the axis and the other sloped 1:20 (bevelled washers not used)‡	Up to and including four diameters	1/3
	Over four diameters and not exceeding eight diameters or 200 mm	1/2
	Exceeding eight diameters or 200 mm	2/3
Both faces sloped 1:20 from normal to the bolt axis (bevelled washers not used)‡	All lengths	3/4

**Nut rotation is rotation relative to a bolt regardless of whether the nut or bolt is turned. The tolerance on rotation is 30° over or under. This Table applies to coarse-thread, heavy-hex structural bolts of all sizes and lengths used with heavy-hex semi-finished nuts.*
†Bolt length is measured from the underside of the head to the extreme end point.
‡Bevelled washers are necessary when ASTM A 490/A 490M bolts are used.

10.24.6.7 Inspection
An inspector shall determine whether the requirements of Clauses 10.24.3.2 and 10.24.6.2 to 10.24.6.6 have been met. Installation of bolts shall be observed to ascertain that a proper tightening procedure is employed. The turned element of all bolts shall be visually examined for evidence that they have been tightened.

When properly installed, the tip of the bolt shall be flush with or outside the face of the nut.

Tensions in bolts installed by the turn-of-nut method exceeding those specified in Clause 10.24.6.3 shall not be cause for rejection.

When there is disagreement concerning the results of an inspection of bolt tension, the following arbitration procedure shall be used unless a different procedure has been specified:
(a) The inspector shall use an inspection wrench that is a manual or power torque wrench capable of indicating a selected torque value.
(b) Three bolts of the same grade and diameter as those under inspection and representative of the lengths and conditions of those in the bridge shall be placed individually in a calibration device capable of measuring bolt tension. There shall be a washer under the part turned if washers are so used in the bridge or, if no washer is used, the material abutting the part turned shall be of the same specification as that in the bridge.

(c) When the inspection wrench is a manual wrench, each bolt specified in Item (b) shall be tightened in the calibration device by any convenient means to an initial tension of approximately 15% of the required fastener tension, and then to the minimum tension specified for its size in Clause 10.24.6.3. Tightening beyond the initial condition shall not produce greater nut rotation beyond that permitted by Table 10.18. The inspection wrench shall then be applied to the tightened bolt and the average torque necessary to turn the nut or head 5° in the tightening direction shall be determined. The average torque measured in these tests of three bolts shall be taken as the job inspection torque to be used in the manner specified in Item (e). The job inspection torque shall be established at least once each working day.

(d) When the inspection wrench is a power wrench, it shall first be applied to produce an initial tension of approximately 15% of the required fastener tension and then adjusted so that it will tighten each bolt specified in Item (b) to a tension of at least 5% but not more than 10% greater than the minimum bolt tension specified for its size in Clause 10.24.6.3. This setting of the wrench shall be taken as the job inspection torque to be used in the manner specified in Item (e). Tightening beyond the initial condition shall not produce greater nut rotation than that permitted by Table 10.18. The job inspection torque shall be established at least once each working day.

(e) Bolts represented by the sample specified in Item (b) that have been tightened in the bridge shall be inspected by applying, in the tightening direction, the inspection wrench and its job inspection torque to 10% of the bolts (but not fewer than two bolts) selected at random in each connection. If no nut or bolt head is turned by this application of the job inspection torque, the connection shall be accepted as being properly tightened. If any nut or bolt head is turned by the application of the job inspection torque, this torque shall be applied to all of the bolts in the connection, and all of the bolts whose nut or head is turned by the job inspection torque shall be retightened and reinspected. Alternatively, the fabricator or erector, at his or her option, may retighten all of the bolts in the connection and then resubmit the connection for inspection.

10.24.6.8 Reuse of bolts

ASTM A 490/A 490M and galvanized ASTM A 325/A 325M bolts shall not be reused once they have been fully tightened. Other ASTM A 325/A 325M bolts may be reused up to two times, provided that proper control on the number of reuses can be established. Touch-up of pretensioned bolts in a multi-bolt joint shall not constitute a reuse unless a bolt becomes substantially unloaded as other parts of the joint are bolted.

10.24.6.9 Shop trial assembly

Girders and other main components shall be preassembled in the shop in order to prepare or verify the field-splices. Components shall be supported in a manner consistent with the finished geometry of the bridge, as specified on the Plans, with allowance for any camber required to offset the effects of dead load deflection.

Holes in the webs and flanges of main components shall be reamed or drilled to final size while in assembly. The components shall be pinned and firmly drawn together by bolts before reaming or drilling. Drifting done during assembly shall be sufficient only to align the holes and not to distort the steel. If necessary, reaming shall be used to enlarge holes.

When a number of sequential assemblies are necessary because of the length of the bridge, the second and subsequent assemblies shall include at least one section from the preceding assembly to provide continuity of alignment.

Trial assemblies shall be required whether the field-splices are bolted or welded. Each assembly shall be checked for camber, alignment, accuracy of holes, and fit-up of welded joints and milled surfaces. Corrective work, if necessary, shall be carried out at no cost to the Owner.

10.24.6.10 Holes drilled using numerically controlled machines

As an alternative to the trial assembly specified in Clause 10.24.6.9 when the bolt holes have been prepared by numerically controlled drilling or using a suitable template, the accuracy of the drilling may be demonstrated by a check assembly consisting of the first components of each type to be made. If the check assembly is satisfactory, further assemblies of like components shall not be required.

If the check assembly is unsatisfactory for any reason, the work shall be redone or repaired in a manner acceptable to the Owner. Further check assemblies shall be required, as specified by the Owner, to demonstrate that the required accuracy of fit-up has been achieved.

10.24.6.11 Match-marking

Connecting parts that are assembled in the shop for reaming or drilling holes shall be match-marked. A drawing shall be prepared to show how the marked pieces should be assembled in the field to replicate the shop assembly.

10.24.7 Tolerances

10.24.7.1 Structural members

Structural members consisting of a single rolled shape shall meet the straightness tolerances of CSA G40.20, except that columns shall not deviate from straight by more than 1/1000 of the length between points of lateral support.

A variation of 1 mm from the detailed length shall be permissible in the length of members that have both ends finished for contact bearing. Other members without finished ends may have a variation from the detailed length of not more than 2 mm for members 10 m or less in length, and not more than 4 mm for members over 10 m in length.

10.24.7.2 Abutting joints

When compression members are butted together to transmit loads in bearing, the contact faces shall be milled or saw-cut. The completed joint shall have at least 75% of the entire contact area in full bearing, defined as not more than 0.5 mm separation, and the separation of the remainder shall not exceed 1 mm.

At joints where loads are not transferred in bearing, the nominal dimension of the gap between main members shall not exceed 10 mm.

10.24.7.3 Facing of bearing surfaces

The surface finish of bearing surfaces that are in contact with each other or with concrete shall meet the roughness requirements specified in CSA B95 and Table 10.19.

Surfaces of flanges that are in contact with bearing sole plates shall be flat within 0.5 mm over an area equal to the projected area of the bearing stiffeners and web. Outside this area, a 2 mm deviation from flat shall be acceptable. The bearing surface shall be perpendicular to the web and bearing stiffeners.

Table 10.19
Facing of bearing surfaces roughness requirements
(See Clause 10.24.7.3.)

	Surface roughness	
Contact surfaces	**Micro-inches**	**Microns**
Steel slabs or plates in contact with concrete	2000	50
Plates in contact as part of bearing assemblies	1000	25
Milled ends of compression members	500	13
Milled or ground ends of stiffeners	500	13
Bridge rollers or rockers	250	6
Pins and pin holes	125	3
Sliding bearings — Steel/copper alloy or steel/stainless steel	125	3

10.24.7.4 Bearing plates

Bearing plates shall meet the following requirements:

(a) rolled steel bearing plates 50 mm or less in thickness may be used without planing if a satisfactory contact bearing is obtained;

(b) rolled steel bearing plates more than 50 mm thick but not more than 100 mm thick may be straightened by pressing or by planing on all bearing surfaces to obtain a satisfactory contact bearing; and

(c) rolled steel bearing plates more than 100 mm thick shall be planed on all bearing surfaces, except for those surfaces that are in contact with concrete foundations and are grouted to ensure full bearing.

10.24.7.5 Fabricated components

The tolerances for welded components shall comply with Clause 5.4 of CSA W59. The dimensional tolerances of welded structural members shall be those specified in Clauses 5.8 and 12.5.3 of CSA W59.

Built-up, bolted structural members shall comply with the straightness tolerances specified in CSA G40.20 for rolled wide-flanged shapes.

Bearing stiffeners fitted to bear shall have a minimum bearing contact area of 75% and a maximum separation of 1 mm over the remaining area. Fitted intermediate stiffeners shall have a minimum bearing contact area of 25% and a maximum separation of 2 mm.

10.24.8 Quality control

10.24.8.1 Qualification of inspectors

Welding inspectors shall be qualified by the CWB to the requirements of CSA W178.2.

10.24.8.2 Non-destructive testing of welds

At least the following non-destructive testing of welds shall be performed:

(a) visual inspection of all welds;

(b) radiographic inspection of groove welds in flanges and webs of built-up girders, as follows:
   (i) flange splices in tension or stress reversal zones: 100%;
   (ii) flange splices in compression zones: 25%; and
   (iii) web splices: 100% for one-half of the depth from the tension flange and 25% for the remainder of the web;

(c) magnetic particle inspection of web-to-flange fillet welds, as follows:
   (i) submerged-arc welds: 25%;
   (ii) semi-automatic welds: 50%; and
   (iii) manual welds: 100%; and

(d) magnetic particle inspection of fillet welds, as follows, for connection plates and stiffeners to which cross-bracing or diaphragms are attached:
   (i) for one-half of the depth from the tension flange: 100%; and
   (ii) transverse welds on tension flanges: 100%.

Radiographic and ultrasonic testing shall be performed before assembly of the flanges to the webs.

10.24.8.3 Acceptance standards for weld defects

The acceptance standards for dynamically loaded structures specified in Clause 12.5.4 of CSA W59 shall apply to weld defects.

10.24.8.4 Repair of welds

Welds that do not meet the acceptance standards specified in Clause 10.24.8.3 shall be removed, rewelded, and retested. Repairs and non-destructive testing of fracture-critical and primary-tension members shall be performed in accordance with Clause 10.23.

10.24.8.5 Identification of structural steel

In the fabricator's plant, the specification and grade of steel used for main components shall be identified by use of suitable markings or recognized colour coding. Cut pieces that are identified by piece mark and contract number need not continue to carry specification identification markings when it has been established that such pieces conform to the required material specifications.

Records shall be kept to identify the heat number of the material and the corresponding mill test report for each component of a fracture-critical or primary tension member.

10.24.9 Transportation and delivery

Structural steel shall be loaded for shipping, transported, unloaded, and stored clear of the ground at its destination without being excessively stressed, deformed, or otherwise damaged. Plate girders shall be transported with their webs in the vertical plane.

10.24.10 Erection

10.24.10.1 Erection conditions

Components shall be lifted and placed using appropriate lifting equipment, temporary bracing, guys, or stiffening devices so that they are not overloaded or unstable. Additional permanent material may be provided, if Approved, to ensure that the member capacities are not exceeded during erection.

10.24.10.2 Falsework

All falsework, including necessary foundations, required for the safe construction of a bridge shall be designed, furnished, maintained, and removed by the contractor. The contractor shall not use any of the material intended for use in the finished bridge for temporary purposes during erection, unless such use is Approved.

10.24.10.3 Removal of temporary bracing or guys

Temporary bracing or guys shall be removed when no longer required for the stability of the bridge, unless otherwise Approved.

10.24.10.4 Maintaining alignment and camber

The bridge shall be erected to the proper alignment on plan and in elevation, taking into account the specified dead load camber.

10.24.10.5 Field assembly

Parts shall be assembled following the piece marks shown on the erection drawings and match-marks.

Main girder splices and field connections shall have half their holes filled with fitting-up bolts and drift-pins (half bolts and half pins) before the installing and tightening of the balance of the connection bolts. The fitting-up bolts may be the same high-strength bolts used in the installation. The pins shall be 1 mm larger in diameter than the bolts. Excessive drifting that distorts the metal and enlarges the holes shall not be allowed, although reaming up to 2 mm over the nominal hole diameter shall be permitted, except for oversize or slotted holes.

10.24.10.6 Cantilever erection

When cantilever erection is used, splices that support the cantilevering member shall be fully bolted before the cantilever is further extended or loaded.

10.24.10.7 Repairs to erected material

With the exception of splices of main material, the correction of minor misfits involving minor amounts of reaming, cutting, and shimming shall be permitted. The correction of other shop fabrication, or any deformation resulting from handling or transportation that prevents the proper assembly and fitting of the parts, shall require Approval.

10.24.10.8 Field welding

Any company undertaking field welding in accordance with this Section shall be certified to Division 1 or 2 of CSA W47.1.

10.24.10.9 Attachments

Tack welds intended to be used for attachments or for any other purpose shall not be used unless they subsequently become a part of the welds shown on the Plans. Tack welds that are not part of the welds shown on the Plans shall not be used on any portion of the girders.

10.24.10.10 Protection of the substructure against staining

The substructure shall be protected against rust staining by water runoff from the bridge, as specified on the Plans.

Section 11 — Joints and bearings

Section 11
Joints and bearings

11.1 Scope

This Section specifies minimum requirements for the design, selection, and detailing of joints and bearings.

11.2 Definitions

The following definitions apply in this Section:

Armour — an edging to the deck joint comprising a steel angle or a steel plate permanently attached to the concrete dam corners.

Bearing — a structural device that transmits loads while allowing translation, rotation, or both.

Bridging plate — a structurally integral cantilever plate, e.g., a finger plate, that is rigidly fastened to one side of a joint and permits free movement of the joint.

Concrete dam — the area adjacent to the joint that anchors the joint assembly or mechanism. It also provides protection against dynamic impact effects resulting from direct wheel traffic loading.

Cover plate — a plate that is not necessarily structurally integral with the joint but covers the joint to provide a safe riding surface.

Deck joint or **expansion joint** — a structural discontinuity between two elements, at least one of which is a deck element, that is designed to permit relative translation or rotation, or both, of abutting structural elements.

Disc bearing — a bearing consisting of a restrained single moulded disc of unreinforced elastomer confined by upper and lower metal-bearing plates and prevented from moving horizontally by a shear-restricting mechanism.

Effective elastomer thickness — the sum of the thicknesses of all of the elastomeric layers in a bearing, excluding the outer layers.

Elastomer — a compound containing
(a) virgin natural polyisoprene (natural rubber) (when used in pot bearings and plain or laminated elastomeric bearings);
(b) virgin polychloroprene (neoprene) (when used in plain or laminated elastomeric bearings); or
(c) polyether-urethane polymer (when used in disc bearings).

Elastomeric concrete — a viscous mixture of elastomer, chemical additives, and aggregates that, after being placed as an end expansion-joint dam and cured, retains the joint assembly while providing a resilient transition in the riding surface.

Fixed bearing — a bearing that prevents differential translation while permitting rotation of abutting structural elements.

Integral abutment bridge — a bridge whose superstructure and abutments are connected monolithically.

Joint anchorage — each side of the deck joint assembly anchored permanently to the structure in order to transfer all static and dynamic loads from the joint assembly to the structure.

Joint seal — a poured or preformed elastomeric component designed to prevent moisture and debris from penetrating joints.

Laminated bearing — a bearing made from alternate laminates of elastomer and reinforcing material, fully bonded together during vulcanization.

Longitudinal joint — a joint provided to separate a deck into two independent longitudinal structural systems.

Metal rocker — a bearing that carries vertical load by direct contact between two metal surfaces and accommodates movement by rolling of one surface with respect to the other.

Modular joint — a prefabricated deck joint consisting of multiple joint openings filled with seals.

Open joint — a structural discontinuity that permits the passage of water and debris.

Plain elastomeric pad — a pad made only of elastomer.

Pot bearing — a bearing consisting of a metal piston supported by a single moulded disc of unreinforced elastomer confined within a hollow metal cylinder.

Sealed joint — a structural discontinuity that does not permit the passage of water and debris through the joint.

Shape factor — the ratio of the area of the loaded face of a bearing and the area of an elastomeric layer around the perimeter of the bearing that is free to bulge.

Sliding bearing — a bearing that accommodates differential translation.

Spherical bearing — a bearing comprising two spherical metal surfaces in contact with and sliding on matching curved surfaces.

Translation — horizontal movement of a bridge in the longitudinal or transverse direction.

Volume control joint — a joint assembly that comprises an elastoplastic material that seals and controls the deck joint opening by its ability to vary its shape at constant volume.

Zero movement point — a stationary point to which movements resulting from volumetric changes in the structure are related.

11.3 Abbreviations and symbols

11.3.1 Abbreviations

The following abbreviations apply in this Section:

FLS — fatigue limit state

PTFE — polytetra fluoroethylene polymer

SLS — serviceability limit state

ULS — ultimate limit state

11.3.2 Symbols

The following symbols apply in this Section:

D = diameter of the loaded contact surface of a spherical bearing projected on the horizontal plane, mm
D_d = diameter of elastomeric element in a disc bearing, mm
D_p = internal diameter of pot in a pot bearing, mm
D_1 = diameter of the curved surface of a rocker or roller unit, mm
D_2 = diameter of the curved surface of a mating unit, mm
E_s = modulus of elasticity of steel, MPa
e = induced eccentricity of the loading on a bearing, mm
F_y = yield strength of steel, MPa
H_u = horizontal load on a bearing or restraint at ULS, N
h_e = total effective elastomer thickness, mm
L = smaller dimension of a rectangular bearing in plan, mm; length of contact of a cylindrical surface, mm
P_s = total load at SLS, N
P_{ud} = minimum dead load at ULS, N
p_u = average pressure on the elastomer in a pot bearing at ULS, MPa
R = radius of a curved bearing contact surface, mm; radius of a circular bearing, mm
S = shape factor of the thickest layer of elastomer or of the thickness of a plain bearing pad
t_w = thickness of the pot wall in a pot bearing, mm
w = height of the piston rim in a pot bearing, mm
β = effective friction angle, degrees
θ_u = relative rotation of the top and bottom surfaces of a bearing at ULS, degrees
μ = coefficient of friction
ϕ = resistance factor

11.4 Common requirements

11.4.1 General

Deck joints and bearings shall be designed to resist loads and accommodate movements at SLS and ULS. The movements and loads shall be in accordance with the requirements of Section 3.

The selection and layout of the joints and bearings shall be consistent with the designed articulation of the structure. The articulation shall accommodate all anticipated deformations induced by loads, restraints, and volumetric changes.

No damage due to joint or bearing movement shall be permitted at SLS and no irreparable damage shall occur at ULS. Joint or bearing movements and loads assumed in the design shall be clearly identified on the Plans.

All exposed steel components of joints and bearings shall be protected against corrosion. The details and specifications of the corrosion protection system shall be Approved.

In the designing and detailing of deck joints and bearings, the following shall be considered:

(a) the properties of the materials in the structure, including the coefficient of thermal expansion, the modulus of elasticity, Poisson's ratio, elastic shortening, creep, and shrinkage;
(b) the effective temperature range of the structure;
(c) the sizes of the structural members in contact with the bearings;
(d) the method and sequence of construction;
(e) the anticipated tilt, settlement, and movement of supports;

(f) the construction tolerances;
(g) the static and dynamic response of the structure;
(h) the interaction of the force effects to which the structure could be subjected, including those due to dead and live loads, wind, earthquake, and earth pressures;
(i) the structural restraints; and
(j) inspection and maintenance requirements.

In all cases, both short-term and long-term effects shall be considered.

11.4.2 Design requirements

Thermal movements calculated from the extreme temperatures specified in Section 3 and the estimated setting temperature shall be accounted for in the design of the joints and bearings.

The setting of deck joints and bearings shall be based on the effective bridge temperature at the time of installation, which may be assumed to be the mean shade air temperature taken over the previous 48 h for concrete structures and the previous 24 h for steel structures.

The resistance factor, ϕ, applied to the capacity of a joint or a bearing assembly shall be in accordance with the applicable Section of this Code.

11.5 Deck joints

11.5.1 General requirements

11.5.1.1 Functional requirements

Deck joints shall be designed and detailed to accommodate the translation and rotation of the structure at the joint.

Deck joints shall be designed to provide for the unhindered passage of traffic across the joints without impairing the riding characteristics of the roadway or damaging vehicles.

The type of joint and size of surface gap shall accommodate the safe passage of motorcycles, bicycles, and pedestrians, as necessary. In particular, where bicycle paths and pedestrian walkways are designed as part of the roadway, the gap opening shall be controlled by cover plates or bridging plates so that the maximum opening does not exceed 25 mm.

Joint armour, armour connections, and anchors shall be detailed to avoid damage from snowplows. Sealing elements shall be located at least 10 mm below the riding surface.

The deck joint components in the vertical faces of curbs, parapet walls, or barrier walls exposed to the action of snowplows or other maintenance equipment shall be recessed at least 20 mm.

Where cover plates are used over the sidewalk and curb areas, they shall be installed with the free end pointing in the direction of the adjacent traffic. Protection against snowplow action shall be considered for cover plate installations in driving lanes over roadway areas.

Deck joints shall be detailed to prevent damage to components of the structure (e.g., the deck, bearings, piers, and abutments) from water, de-icing chemicals, and roadway debris.

Longitudinal deck joints shall be provided, but only where necessary, to accommodate the effects of differential movements between adjacent longitudinal segments of the bridge.

Sealed joints shall remain watertight at SLS.

11.5.1.2 Design loads

A joint shall be designed to withstand combinations of wheel and horizontal loads with appropriate load factors and dynamic load allowance.

A single wheel load, in accordance with the requirements of Section 3, shall be used to calculate the maximum force effects in the various components of the joint. Any portion of the wheel load over the joint gap shall be applied at only one edge of the gap. Load dispersion at an angle not exceeding 45° shall be permitted within the joint components where justified by the continuity and rigidity of the joint.

A horizontal load of 60 kN per metre length of the joint shall be applied at the roadway surface, in combination with forces that result from movement of the joint, to produce maximum force effects.

11.5.1.3 Structural requirements

Deck joints shall satisfy the requirements of SLS, FLS, and ULS. The joints and their supports shall be designed to withstand factored load effects over the range of movements, as specified in Section 3.

A joint shall be detailed in such a way that any damage to the joint occurring at ULS is repairable while the bridge remains in service.

In calculating the movement at a joint in a bridge superstructure, the length taken as affecting the movement shall be the distance between the reference point and the zero movement point. For curved superstructures, this length shall be taken along the chord. In calculating the location of the zero movement point, the stiffnesses of the supporting systems of the bridge shall be taken into account.

All joints, including those in curbs, parapets, and barrier walls, shall be positioned and oriented to accommodate total movement with reference to the zero movement point.

The moving components of the joint shall be designed to work in concert with the bearings to avoid binding of the joints and the resulting adverse force effects on the bearings and structural elements.

11.5.1.4 Materials

The surface of the joint exposed to pedestrian traffic shall be skid resistant. All materials in the joint shall be durable and resistant to abrasion, corrosion, and damage from traffic and snowplows.

Materials directly in contact with each other shall be electrically, thermally, and chemically compatible; where incompatibility exists, materials should be insulated from each other.

All fasteners for joints exposed to de-icing chemicals shall be fully protected against corrosion.

11.5.1.5 Maintenance

Deck joints shall be designed to operate with a minimum of maintenance. They shall be replaceable (except for elements permanently attached to the structure) and accessible for inspection and maintenance.

Sufficient space for access to the joints from below the deck shall be provided by proper detailing of adjacent components. For the deck joints of large bridges not directly accessible from the ground, access, e.g., inspection hatches, ladders, platforms, and catwalks, shall be provided where practicable.

Joint armour, armour connections, and anchors shall be detailed to avoid damage from snowplows.

The top surface of piers and abutments under deck joints shall be sloped to prevent the accumulation of water and debris.

11.5.2 Selection

11.5.2.1 Number of joints

The number of deck joints in a structure shall be kept to a minimum. Preference shall be given to continuous floor systems and superstructures. To permit expansion when required, a joint shall be provided on the approach slabs of integral abutment bridges.

The deck and supporting structural system shall be designed to minimize and withstand the forces generated by restraint to movements, unless deck joints and bearings are provided to facilitate the movements.

11.5.2.2 Placement

The longitudinal movement of deck joint elements shall be consistent with that provided by the bearings at that location.

11.5.2.3 Types of deck joints

A sealed deck joint shall be provided where the joint is located directly above structural members and bearings that would be adversely affected by water and debris accumulation, and where de-icing chemicals are used. It shall seal the surface of the deck, including curbs, sidewalks, medians, and, where necessary, parapet or barrier walls. The joint shall prevent the accumulation of water and debris that could restrict its operation.

An open deck joint shall be used only if drainage away from the bearings can be ensured year round. Where de-icing chemicals are used, the drainage system shall be adequately protected against corrosion.

11.5.3 Design

11.5.3.1 Bridge deck movements

11.5.3.1.1 Sealed deck joint

The width of a roadway surface gap in a transverse deck joint, measured normal to the joint at SLS movement, shall not exceed 100 mm for a joint with a single opening and 80 mm for any gap in a joint with multiple openings.

Gaps in a deck joint with multiple openings shall remain equal and parallel to each other.

When the skew angle of the deck joint exceeds 20°, only those deck joints whose movement capacity has been demonstrated by the manufacturer shall be permitted.

11.5.3.1.2 Open deck joint

The width of the roadway surface gap in an open transverse deck joint shall be not less than 25 mm or greater than 60 mm at SLS movements. Openings exceeding 60 mm shall be used only if Approved.

11.5.3.2 Components

11.5.3.2.1 Bridging plates

Joint bridging plates shall be designed as cantilevers capable of supporting wheel loads and accommodating bridge articulation.

The possible differential settlement between the two sides of a joint bridging plate should be accommodated in the design and detailing of the bridging plates.

11.5.3.2.2 Armour

The armour shall be detailed to eliminate the formation of air voids during placing of adjacent concrete.

The armour shall be provided with studs with a minimum diameter of 20 mm or snowplow plates with a minimum thickness of 10 mm. The length of the studs or plates shall be not less than 200 mm. The spacing shall be not more than 200 mm for studs and not more than 300 mm for plates.

11.5.3.2.3 Joint anchorage

The joint anchorage shall be connected directly to the structural steel supports or engaged with the reinforced concrete or the elastomeric concrete substrate through bonding. Joint anchorage within elastomeric concrete shall require Approval.

Joint anchorage on each side of the deck joint assembly shall satisfy the following minimum requirements:

(a) the factored resistance of the joint anchorage shall not be less than 600 kN/m in any direction;
(b) the spacing of the armour anchors shall not exceed 250 mm; and
(c) where the deck joint assembly is attached by reinforcing bars, studs, or bolts cast into concrete, the total cross-sectional area of the steel anchors shall be not less than 1600 mm^2/m.

11.5.3.2.4 Bolts

All anchor bolts for bridging plates, joint seals, and joint anchors shall be high-strength bolts fully torqued in accordance with the applicable ASTM Standard. Cast-in-place anchors shall be used only in new concrete. Expansion anchors and countersunk anchor bolts shall not be permitted on any joint connection.

11.5.4 Fabrication

Deck joint components shall be of sufficient thickness to stiffen the assembly and prevent distortion due to welding and galvanizing.

To ensure proper fit and function, joint components shall be fully assembled in the shop. If possible, the joint and seal shall be shipped to the job site fully assembled; otherwise, permanent seals shall not be placed before joint armouring and anchorage installation have been completed.

11.5.5 Installation

The Plans shall include, in tabular form, the installation gap openings throughout the designated installation temperature range.

Construction joints and blockouts shall be used where practicable to permit the placement and adjustment of the joint after the backfill and major components have been placed.

Where staged construction is used, joint design shall include details for transverse field splices. Splices shall be designed to provide satisfactory fatigue life. Where practicable, splices should be located away from the wheel paths and the gutter areas.

Seals shall be installed in one continuous piece.

11.5.6 Joint seals

All seals for joints shall accommodate required movements at SLS and ULS and be designed to remain watertight and prevent the accumulation of water and debris that could restrict the operation of the joints.

Elastomeric glands or membranes shall be placed in such a way that they remain below the roadway surface at the minimum gap opening in accordance with Clause 11.5.1.1.

11.5.7 Sealed joint drainage

Where practicable, drainage accumulated in the sealed joint shall not be discharged on any portion of the structure.

11.5.8 Open joint drainage

In the design of open joints, the discharge of water and debris shall be diverted from the bearing areas and structural elements by a suitable system, e.g., a trough-collector-downspout system. Troughs shall have a minimum of 10% slope to facilitate drainage.

11.5.9 Volume control joint

A volume control joint shall be designed to transfer all static and dynamic wheel loads to the structure.

A volume control joint shall be used only when the maximum joint gap below the seal is less than 20 mm.

The width of the joint binder shall be at least ten times the maximum gap of the joint below the seal. The sealant shall have sufficient bond strength with all surfaces with which it is in contact.

The use of proprietary volume control joints shall require Approval.

11.6 Bridge bearings

11.6.1 General

11.6.1.1

Bearings shall support and transfer all loads while accommodating translations and rotations in the structure.

Uplift-restraint devices shall not restrict the function of a bearing.

The bearing seats of the structure shall be detailed to ensure complete contact with the bearing under all load combinations.

The following maximum and minimum loads and movements corresponding to the critical combinations at SLS and ULS shall be shown on the Plans:

(a) dead load;
(b) total load;

(c) lateral loads;
(d) rotations; and
(e) translations.

Any other requirements that need to be satisfied shall be shown on the Plans.

For bearings other than elastomeric bearings, the design-bearing rotation, θ_u, shall be taken as the sum of the rotations due to ULS loads and tolerances in fabrication and installation, plus 1°.

Bearings shall be designed to operate with minimal maintenance. They shall be accessible for inspection and maintenance and replaceable without damage to the structure or removal of anchorages permanently attached to the structure. To facilitate their placement, bearings shall be detailed so that they can be removed by jacking the superstructure by an amount not exceeding the vertical relaxation recovery of the elastomeric material within the bearing plus 5 mm.

For bearings with sliding elements, the Plans shall include a table of the required settings throughout the probable temperature range at the time of installation.

Bearings shall be plant assembled so that their assembly remains intact during transportation and installation. The temporary connections shall be removed only after the bearings have been installed with permanent connections. The bearings shall be set to the specified plane within a tolerance of ± 0.2° in any direction. The top of a bearing shall be set at the specified elevation within Approved tolerances.

Grout bedding for bearings used for surface levelling shall meet the requirements of the Regulatory Authority. The grout shall be inert and free from shrinkage and staining. Grout bedding shall not be used with elastomeric bearings unless steel masonry plates are also used.

The bearing design shall take account of induced moments and the horizontal forces induced by sliding friction, rolling friction, or deformation of a flexible element in the bearing.

11.6.1.2

Fixed and guided bearings shall be capable of resisting lateral loads in the restrained direction as required by the design, but not less than the following:

(a) 10% of the vertical load capacity for bearings with a total vertical load capacity of up to 5000 kN at SLS; and
(b) 500 kN, plus 5% of the vertical load exceeding 5000 kN, for bearings with a total vertical load capacity exceeding 5000 kN at SLS.

11.6.2 Metal back, roller, and spherical bearings

11.6.2.1 General design considerations

The rotation axis of rocker and roller bearings shall be aligned with the axis of the largest expected rotation of the supported member. Steps shall be taken to ensure that the bearing alignment does not change during the life of the bridge. Multiple roller bearings shall be connected by gearing to ensure that individual rollers remain parallel to each other and at their original spacing.

11.6.2.2 Materials

Rocker, roller, and spherical bearings shall be made of carbon steel that complies with CSA G40.20/G40.21, stainless steel that complies with ASTM A 240/A 240M, or other Approved materials.

11.6.2.3 Geometric requirements

A bearing with two curved surfaces shall be symmetric about a line joining their centres of curvature. The bearing shall be designed to be stable. If the bearing consists of a roller unit with two cylindrical faces, each of which bears on a flat plate, stability shall be achieved by making the distance between the two contact surfaces not greater than the sum of the radii of the two cylindrical surfaces.

The dimensions of a bearing shall be chosen to account for both the contact pressure and its movement due to rolling.

11.6.2.4 Contact pressure

The contact pressure shall be maintained at a safe level by ensuring that the SLS load, P_s, applied across metal-to-metal contact surfaces satisfies the following:

(a) for cylindrical surfaces:

$$P_s \leq 8\left[\frac{LD_1}{1-\frac{D_1}{D_2}}\right]\frac{F_y^2}{E_s}$$

(b) for spherical surfaces:

$$P_s \leq 40\left[\frac{D_1}{1-\frac{D_1}{D_2}}\right]^2\frac{F_y^3}{E_s^2}$$

The diameter, D_2, shall be taken as positive if the curvatures have the same sign, infinite if the mating surface is flat, and negative if the two surfaces have curvatures of opposite sign.

11.6.3 Sliding surfaces

11.6.3.1 General

PTFE is used to provide sliding surfaces for bridge bearings to accommodate translation or rotation. All sliding surfaces, other than guides, shall satisfy the requirements of this Section.

11.6.3.2 PTFE layer

The PTFE layer shall be made from pure virgin PTFE resin satisfying the requirements of ASTM D 4894. It shall be fabricated as unfilled sheet or filled sheet reinforced with random or woven fibres.

Unfilled sheets shall be made from PTFE resin alone. Filled sheets shall be made from PTFE resin uniformly blended with glass fibres, carbon fibres, or other chemically inert fibres. The maximum filler content shall be 15% for glass fibres and 25% for carbon fibres.

Sheet PTFE may contain dimples to act as reservoirs for a lubricant. The dimple diameter shall not exceed 8 mm at the surface of the PTFE, and their depth shall be not less than 2 mm and not more than half the thickness of the PTFE. The reservoirs shall be uniformly distributed over the surface area and shall cover more than 20% but less than 30% of it. The lubricant shall be silicone grease, effective to –40 °C, and comply with U.S. Department of Defense MIL-S-8660C.

11.6.3.3 Mating surface

The PTFE shall be used with a mating surface large enough to cover the PTFE at all times. For plane surfaces, the mating surface shall be stainless steel positioned above the PTFE element. For spherical surfaces, the mating surface shall be stainless steel or anodized aluminum alloy positioned above or below the PTFE element.

Stainless steel shall comply with ASTM A 240/A 240 M. The roughness of the contact surface, measured in accordance with CSA B95, shall not be greater than 0.25 mm arithmetic average for plane surfaces and 0.50 mm arithmetic average for curved surfaces. The roughness of anodized aluminum machined metallic surfaces shall not exceed 0.40 mm.

11.6.3.4 Attachment

11.6.3.4.1 PTFE layer

Sheet PTFE may be confined or unconfined. Confined sheet PTFE shall be set in a recess in a rigid metal backing plate to a depth specified in Table 11.1. Unconfined sheet PTFE shall be bonded by an Approved method to a metal surface or an elastomeric layer with a Shore A durometer hardness of at least 70. Woven PTFE on a metallic substrate shall be attached to the metallic substrate by mechanical interlocking that can resist a shear force at least 0.10 times the applied compressive force.

Table 11.1
Dimensions for confined sheet PTFE
(See Clauses 11.6.3.4.1 and 11.6.3.5.1.)

Maximum dimension of PTFE (diameter or diagonal), mm	Minimum thickness, mm	Depth of recess, mm
≤ 1200	5.0	2.5
> 1200	5.5	3.0

11.6.3.4.2 Mating surface

The mating surface for flat sliding surfaces shall be attached to a backing plate by welding in such a way that it remains flat and in full contact with the backing plate throughout its service life. The weld shall form an effective moisture seal around the entire perimeter of the mating surface so that interface corrosion cannot occur. The attachment shall be capable of resisting the maximum friction force that can be developed by the bearing. The welds used for the attachment shall be kept clear of the contact and sliding area of the PTFE surface.

11.6.3.5 Minimum thickness

11.6.3.5.1 PTFE layer

For all applications, the thickness of the PTFE layer shall be at least 2 mm after compression. The minimum thickness of a confined PTFE layer shall be as specified in Table 11.1 with respect to its maximum dimension in plan.

11.6.3.5.2 Stainless steel mating surfaces

The thickness of the stainless steel sheet shall be related to the dimensional difference between the stainless steel and the PTFE in the direction of movement in accordance with Table 11.2.

Table 11.2
Dimensions for stainless steel
(See Clause 11.6.3.5.2.)

Dimensional difference between PTFE and stainless steel, mm	Minimum thickness of stainless steel, mm
≤ 300	1.5
> 300 and ≤ 500	2.0
> 500 and ≤ 1500	3.0

11.6.3.6 Contact pressure

The average contact pressure between the PTFE and the mating surface shall be calculated by dividing the load by the projection of contact area onto a plane perpendicular to the direction of the load and shall not exceed the relevant maximum specified in Table 11.3.

The contact area of dimpled lubricated PTFE shall be taken as the gross area of the PTFE, without deduction for the area occupied by the lubrication reservoirs.

The contact pressure at the edge of the PTFE layer at SLS shall be calculated by taking into account the maximum moment transferred by the bearing, assuming a linear distribution of pressure across the PTFE layer, and shall not exceed 1.2 times the relevant maximum specified in Table 11.3.

Table 11.3
Maximum average contact pressure for PTFE, MPa
(See Clauses 11.6.3.6, 11.6.7.4, and 11.6.8.6.)

	SLS		ULS	
Material	**Permanent load**	**All loads**	**Permanent load**	**All loads**
Unconfined PTFE				
Unfilled sheet	15	20	20	30
Filled sheet*	30	45	45	65
Confined sheet PTFE	30	45	45	65
Woven PTFE fibre over a metallic substrate	30	45	45	65

**These figures are for maximum filler content. Contact pressure for intermediate filler contents shall be obtained by linear interpolation.*

11.6.3.7 Coefficient of friction

The design coefficient of friction of the PTFE sliding surface shall be in accordance with Table 11.4 (using linear interpolation for any average bearing pressure at the relevant SLS that lies between the pressures specified in the Table). Where friction is required to resist applied loads, the design coefficient of friction under dynamic loading shall be taken as not more than 10% of the applicable value specified in Table 11.4.

Table 11.4
Design coefficient of friction
(See Clause 11.6.3.7.)

Material	Average bearing pressure at the SLS, MPa			
	3	7	14	> 21
Unfilled PTFE				
Unlubricated flat sheet	0.16	0.14	0.12	0.08
Lubricated flat sheet	0.10	0.09	0.08	0.06
Lubricated dimpled sheet	0.08	0.07	0.06	0.04
Filled PTFE (sheet or woven)	0.20	0.18	0.15	0.10
Woven fabric from PTFE resin	0.10	0.09	0.08	0.06
Woven fabric from PTFE fibre and metallic substrate	0.08	0.07	0.05	0.04

11.6.4 Spherical bearings

11.6.4.1 General

Spherical bearings shall consist of two metal parts with matching curved surfaces and a low-friction sliding interface. The material properties, characteristics, and frictional properties of the sliding interface shall meet the requirements of Clause 11.6.3.

11.6.4.2 Geometric requirements

The radius of the curved surface shall be large enough to ensure that the maximum pressure on the bearing surface satisfies the pressure limitations specified in Clause 11.6.3.6.

The induced eccentricity, e, resulting from shifting in the axial load from the centre of the bearing rotation shall be calculated from $e = \mu R$. At SLS, the shift in the axial load from the centre of the bearing shall not exceed 10% of the diameter in plan of the curved sliding interface.

11.6.4.3 Lateral load capacity

In bearings that are required to resist horizontal loads, an external restraint system shall be used or the radius of the curved bearing surface, R, shall satisfy

$$R \leq \frac{D}{2\sin(\beta + \theta_u)}$$

where the effective friction angle, β, is given by

$$\beta = \arctan\left[\frac{H_u}{P_{ud}}\right]$$

The external restraint system shall permit transmission of vertical and horizontal force components without significantly restraining the rotation of the bearing.

11.6.5 Pot bearings

11.6.5.1 General

Pot bearings shall consist of a hollow metal cylinder, a confined one-piece moulded unreinforced elastomer, sealing rings, and a piston. They shall permit transmission of vertical and horizontal force components without significant restraint of rotation between the top and bottom loaded areas of the bearing.

For the purpose of establishing the forces and deformations imposed on a pot bearing, the axis of rotation shall be taken as lying in the horizontal plane at the interface between the elastomer and piston.

11.6.5.2 Materials

The elastomer disc shall be made from a compound based on virgin polyisoprene or polychloroprene. Its nominal hardness shall lie between 50 and 60 on the Shore A scale. Preference shall be given to polyisoprene for use in low-temperature regions.

The pot and piston shall be made from carbon steel that complies with CSA G40.20/G40.21, Grade 260W, 300W, or 350A; stainless steel that complies with ASTM A 240/A 240M; or other Approved materials. The piston shall not be made from a steel with a higher yield strength than that of the pot.

Sealing rings shall be made from brass that complies with ASTM B 36/B 36M, half-hard (for rings of rectangular cross-section) and ASTM B 121, Composition 2 (for rings of circular cross-section).

11.6.5.3 Geometric requirements

The pot shall be deep enough for the seal and piston rim to remain in full contact with the vertical face of the pot wall.

Provision for rotation about any horizontal axis shall be by deformation of the elastomer. The rotation of the elastomer about a horizontal axis shall be limited so that the vertical strain induced at the perimeter of the elastomer at SLS shall not exceed 15% of the elastomer thickness. A pot bearing shall be loaded with at least 25% of the SLS load in order to provide satisfactory rotational operation.

The induced eccentricity, e, as a result of shifting of the axial load from the centre of the bearing under the maximum rotation at SLS shall not exceed 4% of the diameter of the elastomer.

11.6.5.4 Elastomeric disc

The average pressure on the elastomer at SLS shall not exceed 40 MPa. All surfaces of the elastomer shall be treated with a lubricant that is not detrimental to the elastomer.

11.6.5.5 Sealing rings

11.6.5.5.1 General

A seal shall be used between the pot and the piston. At SLS, the seal shall be designed to prevent escape of elastomer under compressive load and simultaneously applied cyclic rotations. At ULS, it shall also be sufficient to prevent escape of elastomer under the compressive load and simultaneously applied static rotation. These requirements shall be deemed satisfied if the sealing rings meet the requirements of Clause 11.6.5.5.2 or 11.6.5.5.3.

The Engineer may approve other sealing systems on the basis of experimental evidence.

11.6.5.5.2 Rings with rectangular cross-section

When the cross-section of the rings is rectangular, three rings shall be used. Each ring shall be circular in plan and shall be cut at one point around its circumference. The faces of the cut shall be bevelled at 45° to the vertical. The rings shall be oriented so that the three cuts are equally spaced around the circumference of the pot.

The width of each ring shall be equal to or greater than the larger of $0.02D_p$ and 6 mm, but shall not exceed 20 mm. The depth of each ring shall be equal to or greater than the larger of 0.2 times the width and 1 mm.

11.6.5.5.3 Rings with circular cross-section
When the cross-section of the rings is circular, one circular closed ring with an outside diameter of D_p shall be used. It shall have a cross-sectional diameter equal to or greater than the larger of $0.0175D_p$ and 4 mm.

11.6.5.6 Pot
The pot shall consist of a wall and a base. All of the components of the pot shall be designed to act structurally as a single unit.

The thickness of the base shall be equal to or greater than the larger of $0.06D_p$ and 20 mm when bearing directly on concrete or grout, and equal to or greater than the larger of $0.04D_p$ and 15 mm when bearing directly on steel girders or load distribution plates.

At ULS, the pot wall shall be thick enough to resist all induced forces. In lieu of rigorous analysis, this requirement may be satisfied for unguided sliding pot bearings by using a wall thickness, t_w, as follows:

$$t_w \geq \frac{D_p}{2\phi F_y} p_u$$

The wall thickness of guided or fixed pot bearings shall be determined by rigorous analysis.

11.6.5.7 Piston
The piston shall have the same plan shape as the inside of the pot. The piston shall be thick enough to resist the loads imposed on it, but not less than $0.06D_p$ thick.

The perimeter of the piston shall have a rim through which horizontal loads can be transmitted. The diameter of the piston rim shall be smaller than D_p by 0.5 to 1.25 mm. The piston perimeter above the rim shall be set back or tapered to prevent binding. The height, w, of the piston rim shall be large enough to transmit the horizontal forces between the pot and the piston, assuming a contact area of $0.33wD_p$ and a maximum bearing pressure of ϕF_y. w shall not be less than the smaller of $0.03D_p$ and 6 mm.

11.6.6 Elastomeric bearings

11.6.6.1 General
Elastomeric bearings may be plain bearings consisting entirely of elastomer or laminated bearings with embedded laminae consisting of alternating layers of elastomer and lamina.

11.6.6.2 Materials

11.6.6.2.1 Laminae
Laminae shall be made of rolled mild steel with a minimum yield strength of 230 MPa or another Approved material.

11.6.6.2.2 Elastomers
Elastomers shall meet the following requirements:
(a) virgin natural polyisoprene and virgin polychloroprene shall be the only raw polymers allowed;
(b) the physical properties of vulcanized elastomer shall be determined using test specimens taken from sample bearings; and
(c) the physical properties of any polyisoprene and polychloroprene shall be in accordance with the requirements specified in Table 11.5.

Table 11.5
Physical properties of polyisoprene and polychloroprene
(See Clause 11.6.6.2.2.)

Property	Test	Requirement	
		Polyisoprene	**Polychloroprene**
Hardness, °Shore A	ASTM D 2240	55 ± 5	55 ± 5
Tensile strength, MPa	ASTM D 412	Minimum 17.0	Minimum 17.0
Ultimate elongation, %	ASTM D 412	Minimum 400	Minimum 400
Heat resistance	ASTM D 573	70 h at 70 °C	70 h at 100 °C
Change in hardness, °Shore A		Maximum +10	Maximum +15
Change in tensile strength, %		Maximum –25	Maximum –15
Change in ultimate elongation, %		Maximum –25	Maximum –40
Compression set, %	ASTM D 395, Method B	22 h at 70 °C, maximum 25	22 h at 100 °C, maximum 35
Ozone resistance	ASTM D 1149, Mounting Procedure A, 20% strain, 40 ± 2 °C	25 pphm, 48 h, no cracks	100 pphm, 100 h, no cracks
Bond between steel and elastomer laminae, $N{\bullet}mm^{-1}$	ASTM D 429, Method B	Minimum 7.0	Minimum 7.0
Brittleness at –40 °C	ASTM D 746, Procedure B	No failure	No failure
Low temperature crystallization increase in hardness, °Shore A	ASTM D 2240	168 h at –25 °C, maximum +15	168 h at –10 °C, maximum +15

11.6.6.3 Geometric requirements

Bearings shall have the following proportions to ensure stability:
(a) for plain bearings: $L \geq 5h_e$ and $R \geq 3h_e$, with 10 mm < h_e < 30 mm; and
(b) for laminated bearings: $L \geq 3h_e$ and $R \geq 2h_e$.

An elastomeric bearing pad in form of a single continuous strip may be used only under precast slabs placed side by side or under a cast-in-place slab, provided that the bearing pressure meets the requirements of Clause 11.6.6.7.

11.6.6.4 Deformation and rotation

Translation shall be accommodated by the shear deformation of the elastomer. Rotation shall be accommodated by the vertical deformation of the elastomer.

The average compressive deformation of the effective elastomer thickness shall not exceed $0.07h_e$ at SLS. Where rotation occurs, the bearing shall be proportioned so that there is no uplift corresponding to a maximum edge deformation of $0.14h_e$ at the edge of the bearing at SLS.

The shear deformation in any direction shall not exceed $0.5h_e$ at SLS.

11.6.6.5 Fabrication

11.6.6.5.1 Plain bearings

Plain bearing pads shall be moulded individually, cut from previously moulded strips or slabs of the required thickness, or extruded and cut to length.

11.6.6.5.2 Laminated bearings

Laminated bearings shall be moulded as a single unit under pressure and heat in moulds that produce a smooth surface finish.

Steel laminae shall meet the following requirements:

(a) all laminae and elastomer layers shall be of uniform thickness;
(b) internal steel plates or laminae shall be free from sharp edges;
(c) laminae shall be completely bonded on all surfaces to the elastomeric material during moulding; and
(d) where pintles are specified, pintle holes shall be of such a depth as to fully engage only one lamina. Cover over pintle holes shall not be required.

The elastomeric cover on the side surfaces shall be at least 5 mm thick. The elastomeric cover of the outer layers, top and bottom, shall not be thicker than 70% of the thickness of an individual internal elastomeric layer.

11.6.6.6 Positive attachment

To prevent displacement of the bearing, positive attachment shall be provided if either of the following conditions exists:

(a) the shear force generated by the bearing exceeds the frictional resistance between the structure and the loaded faces of the bearing; or
(b) the minimum average pressure on the bearing is less than 1.5 MPa under SLS.

A continuous strip of elastomeric bearing pad meeting the requirements of Clause 11.6.6.3 shall not require positive attachment.

11.6.6.7 Bearing pressure

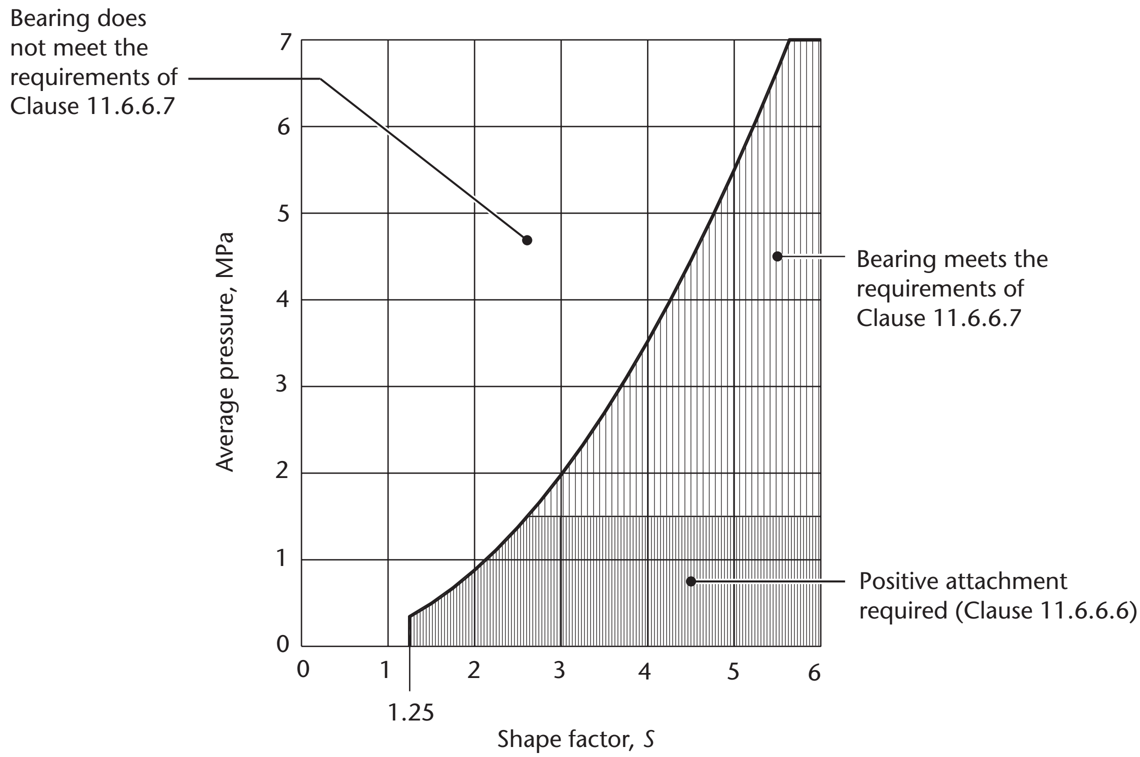
At SLS under permanent loads, the average pressure on a laminated bearing and the average pressure on a layer of elastomer shall not exceed 4.5 MPa.

At SLS under all loading combinations, the average pressure on a laminated bearing shall not exceed 7.0 MPa.

At SLS, the average pressure on a layer of elastomer, assuming no rotation, shall not exceed the permitted pressure indicated in Figure 11.1 with respect to the shape factor of the layer. The shape factor shall be based on the thickest layer within the laminated bearing.

At ULS under permanent loads, the average pressure on a laminated bearing shall not exceed 7.0 MPa.

At ULS under all loading combinations, the average pressure on a laminated bearing shall not exceed 10.0 MPa.



Note: *Average pressure = $0.22S^2$.*

Figure 11.1
Maximum average pressure on a layer of elastomeric bearing at SLS without rotation
(See Clause 11.6.6.7.)

11.6.7 Disc bearings

11.6.7.1 General

Disc bearings shall consist of a restrained single moulded disc of unreinforced elastomer, upper and lower metal bearing plates, and a shear-restriction mechanism and shall permit transmission of vertical and horizontal force components without significant restraint of rotation between the top and bottom loaded areas of the bearing.

For the purpose of establishing the forces and deformations imposed on a disc bearing, the axis of rotation may be taken as lying in the horizontal plane at mid-height of the disc. The disc shall be held in place by a positive location device.

11.6.7.2 Materials

The elastomeric disc shall be made from a compound based on polyether urethane, using only virgin materials. The hardness shall lie between 45 and 65 on the Shore D scale.

The metal components of the bearing shall be made from carbon steel meeting the requirements of CSA G40.20/G40.21, stainless steel meeting the requirements of ASTM A 240/A 240M, or other Approved materials.

11.6.7.3 Geometric requirements

The induced eccentricity as a result of the shift in the axial load from the centre of the bearing under the maximum bearing rotation at SLS shall not exceed 10% of the diameter of the elastomeric disc.

The horizontal clearance between the upper and lower components of the shear-restricting mechanism shall not exceed the value for guide bars specified in Clause 11.6.8.3.

11.6.7.4 Elastomeric disc

The disc shall be designed so that under SLS

(a) its instantaneous deflection under total load does not exceed 10% of the unstressed disc thickness and the additional deflection due to creep does not exceed 8% of the unstressed disc thickness;
(b) the components of the bearing always remain in contact;
(c) the average compressive pressure on the disc does not exceed 35 MPa; and
(d) the pressure on the PTFE sliding surface (if such a surface exists) does not exceed the allowable values for pressures specified in Table 11.3.

In addition, the effect of moment induced by deformation of the disc shall be included in the stress analysis. This moment is the result of the induced eccentricity, e, due to shift in the axial load.

11.6.7.5 Steel plates

The thickness of both the upper and the lower steel plate shall not be less than $0.045D_d$ if they are in direct contact with a steel girder or distribution plate or $0.06D_d$ if they bear directly on grout or concrete.

11.6.8 Guides for lateral restraints

11.6.8.1 General

Guides shall be used to restrict movement of the structure in one direction and shall have a low-friction material at their sliding contact surfaces. The seismic design considerations specified in Section 4 shall be applied as necessary.

11.6.8.2 Materials

Guides shall conform to the material requirements of the bearing specified in Clauses 11.6.5.2, 11.6.2.2, and 11.6.6.2, as applicable.

11.6.8.3 Geometric requirements

Guide bars shall be parallel, long enough to accommodate the full design movement of the structure in the sliding direction, and have a clearance of 1.5 mm in the restrained direction.

11.6.8.4 Design loads

Guides shall be designed for the lateral loads specified in Clause 11.6.1.

11.6.8.5 Load location

The horizontal load acting on a guide shall be assumed to act at the centroid of the low-friction sliding interface material. The design of the connection between the guide and the body of the bearing system shall take into account shear and the induced overturning moments.

11.6.8.6 Contact pressure

The contact pressure on the low-friction material shall not exceed that recommended by the manufacturer. For PTFE, the pressure shall not exceed the applicable value specified in Table 11.3.

11.6.8.7 Attachment of low-friction material

Low-friction material shall be attached using at least two of the following methods:

(a) mechanical fastening;
(b) bonding;
(c) mechanical interlocking with a metal substrate; and
(d) recessing.

11.6.9 Other bearing systems
Bearing systems made from components not covered by Clauses 11.6.2 to 11.6.8 may also be used, subject to Approval. Such bearings shall meet the requirements of Clause 11.6.1.

11.6.10 Load plates and attachment for bearings

11.6.10.1 Plates for load distribution
The bearing, together with any additional plates, shall be designed so that
(a) the combined system is stiff enough to prevent distortions of the bearing that would impair its proper functioning; and
(b) the bearing resistance of the concrete satisfies the requirements of Section 8.

In lieu of a more precise analysis, the loads from the bearing may be assumed to disperse at a slope of 1.5:1, horizontal to vertical, from the edge of the smallest element of the bearing that carries the compressive load.

11.6.10.2 Tapered plates
Where necessary, a tapered plate shall be used to provide a level load surface on a bearing.

11.6.10.3 Attachment
All load distribution plates shall be positively secured to the superstructure or the substructure by bolting, welding, or anchoring. Connections shall be designed in accordance with the applicable Sections of this Code.

Section 12 — Barriers and highway accessory supports

Section 12
Barriers and highway accessory supports

12.1 Scope

This Section specifies requirements for the design of permanent bridge barriers and highway accessory supports.

12.2 Definitions

The following definitions apply in this Section:

Anchorage — a bolt, stud, reinforcing bar, or assembly that is installed in concrete to anchor a structure or a component.

Average annual daily traffic (AADT) — the total volume of traffic during a year divided by the number of days in the year.

Barrier clearance — the clearance between the outside edge of the traffic lanes and the roadway face of a barrier.

Barrier exposure index — an index that reflects traffic volumes and bridge site characteristics and is used for determining barrier performance levels.

Barrier joint — a discontinuity in a barrier that permits relative rotation or translation between barrier components on opposite sides of the discontinuity.

Bikeway — part of a highway designated for the movement of bicycles.

Breakaway support — a support designed to fail in such a way that, when struck by a vehicle, damage to the vehicle and injury to its occupants does not exceed a specified level.

Cantilevered support — a support that cantilevers out over a roadway.

Crash cushion — a barrier used for protecting vehicles from a roadside hazard and designed to fail in such a way that, when struck by a vehicle, damage to the vehicle and injury to its occupants does not exceed a specified level.

Crash test — a test of a barrier or highway accessory support carried out by crashing a vehicle into it and monitoring the vehicle-barrier or vehicle–highway accessory support interaction.

Design speed — the speed for which a highway at a bridge site is designed.

Highway accessory — a component required for the operation of a highway, e.g., a sign, luminaire, traffic signal, surveillance installation, noise barrier, or privacy barrier.

Highway accessory support — a structure (including supporting brackets, maintenance walkways, and mechanical devices, where present) that is designed to support highway accessories.

Luminaire — a complete lighting fixture (including the light source, reflector, refractor, housing, and ballast, where present, but excluding support members).

Overhead support — a support that has a member on each side of a roadway supporting a horizontal member that spans over the roadway.

Performance level — the specified level to which a traffic barrier is to perform in reducing the consequences of a vehicle leaving the roadway, as required by the applicable crash test requirements (see Clauses 12.4.3.2 and 12.4.3.4).

> **Performance Level 1 (PL-1)** — the performance level for traffic barriers on bridges where the expected frequency and consequences of vehicles leaving the roadway are similar to those expected on low-traffic-volume roads. For PL-1, the AASHTO *Guide Specifications for Bridge Railings* (see Clause 12.4.3.4.2) require crash testing with a small automobile and a pickup truck.
>
> **Performance Level 2 (PL-2)** — the performance level for traffic barriers on bridges where the expected frequency and consequences of vehicles leaving the roadway are similar to those expected on high-to-moderate-traffic-volume highways. For PL-2, the AASHTO *Guide Specifications for Bridge Railings* (see Clause 12.4.3.4.2) require crash testing with a small automobile, a pickup truck, and a single-unit truck.
>
> **Performance Level 3 (PL-3)** — the performance level for traffic barriers on bridges where the expected frequency and consequences of vehicles leaving the roadway are similar to those expected on high-traffic-volume highways with high percentages of trucks. For PL-3, the AASHTO *Guide Specifications for Bridge Railings* (see Clause 12.4.3.4.2) require crash testing with a small automobile, a pickup truck, and a tractor-trailer truck.

Post and railing barrier — an open barrier consisting of railings that follow the profile of a bridge and posts that support the railings at discrete locations.

Roadside support — a support adjacent to a roadway, with no part of the support or its accessory extending over the roadway.

Sign — a panel for displaying messages.

Traffic barrier termination — the start or end point of a longitudinal run of traffic barrier.

Traffic barrier transition — the portion of an approach roadway traffic barrier that is adjacent to a bridge traffic barrier and provides a transition between the two barrier types.

Traffic signal — a complete signal device consisting of traffic lights and housing.

12.3 Abbreviations and symbols

12.3.1 Abbreviations

The following abbreviations apply in this Section:

PL-1 — Performance Level 1

PL-2 — Performance Level 2

PL-3 — Performance Level 3

12.3.2 Symbols

The following symbols apply in this Section:

$AADT_1$ = average annual daily traffic for the first year after construction

B_e = barrier exposure index

H = height of barrier, m

K_c = highway curvature factor

K_g = highway grade factor
K_h = highway type factor
K_s = superstructure height factor
L = span of overhead and cantilevered support members, m
P_ℓ = longitudinal traffic load on barrier, kN
P_t = transverse traffic load on barrier, kN
P_v = vertical traffic load on barrier, kN
W_p = pedestrian or bicycle load on barrier, kN or kN/m

12.4 Barriers

12.4.1 General

Barriers shall be classified as traffic, pedestrian, bicycle, or combination barriers according to their function.

In addition to the requirements of Clauses 12.4.2 to 12.4.6, the following factors shall be considered in the appraisal of a barrier:
(a) durability;
(b) ease of repair;
(c) snow accumulation on and snow removal from deck;
(d) visibility through or over barrier;
(e) deck drainage;
(f) future wearing surfaces; and
(g) aesthetics.

12.4.2 Barrier joints

Barrier joints shall be detailed to allow for the movements specified in Section 3.

12.4.3 Traffic barriers

12.4.3.1 General

Traffic barriers shall be provided on both sides of highway bridges to delineate the superstructure edge and to reduce the consequences of vehicles leaving the roadway. Barrier adequacy in reducing the consequences of vehicles leaving the roadway shall be determined from crash tests, except that the adequacy of a barrier that has the same details as those of an existing traffic barrier may be determined from an evaluation of the existing barrier's performance when struck by vehicles.

12.4.3.2 Performance level

12.4.3.2.1 General

The performance level used for a bridge site shall be Performance Level 1, 2, or 3, determined in accordance with Clauses 12.4.3.2.3 and 12.4.3.2.4, unless alternative performance levels are Approved in accordance with Clause 12.4.3.2.2.

12.4.3.2.2 Alternative performance levels

Performance levels other than Performance Levels 1, 2, and 3 shall be Approved by the Regulatory Authority for the bridge and shall be defined by specifying their crash test requirements. These alternative performance levels shall be considered along with Performance Levels 1, 2, and 3 when the optimum performance level for a bridge site is being determined. The optimum performance level shall be taken to be the performance level with the least costs, where the costs for each performance level include the costs of supplying and maintaining an appropriate traffic barrier as well as the costs of all accidents expected with the use of that barrier.

12.4.3.2.3 Determination of barrier exposure index

The barrier exposure index used for determining the performance level shall be based on the estimated average annual daily traffic for the first year after construction, $AADT_1$, which shall be limited to a maximum value of 10 000 vehicles per day per traffic lane for vehicle speeds of 80 km/h or greater. $AADT_1$ shall be multiplied by highway type, highway curvature, highway grade, and superstructure height factors to calculate the barrier exposure index, as follows:

$$B_e = \frac{(AADT_1)K_h K_c K_g K_s}{1000}$$

The highway type, highway curvature, highway grade, and superstructure height factors shall be as specified in Tables 12.1 to 12.4.

Table 12.1
Highway type factors, K_h
(See Clause 12.4.3.2.3.)

Highway type	Design speed, km/h	K_h
One-way*	50–110	2.00
Two-way divided†	50–110	1.00
Two-way undivided, with five or more lanes†‡	50–110	1.00
Two-way undivided, with four or fewer lanes†‡§	50	1.20
	60	1.30
	80	1.45
	100	1.60
	110	1.65

*$AADT_1$ *is based on one-way traffic.*
†$AADT_1$ *is based on two-way traffic.*
‡*Number of lanes refers to total number of lanes on bridge.*
§*Interpolate highway type factors for design speeds not given.*

Table 12.2
Highway curvature factors, K_c
(See Clause 12.4.3.2.3.)

Radius of curve, m*	Barrier on outside of curve, K_c	Barrier on inside of curve, K_c
≤ 300	4.00	2.00
350	3.00	1.65
400	2.40	1.45
450	1.90	1.30
500	1.50	1.15
550	1.20	1.05
≥ 600	1.00	1.00

**Interpolate highway curvature factors for radii of curves not given.*

Table 12.3
Highway grade factors, K_g
(See Clause 12.4.3.2.3.)

Grade, %*†	K_g
≥ –2	1.00
–3	1.25
–4	1.50
–5	1.75
≤ –6	2.00

**Positive grade increases in the direction that traffic is travelling.*
†Interpolate highway grade factors for grades not given.

Table 12.4
Superstructure height factors, K_s
(See Clause 12.4.3.2.3.)

Superstructure height above ground or water surface, m*	K_s	
	High-occupancy land use† or deep water‡ beneath bridge	Low-occupancy land use or shallow water beneath bridge
≤ 5	0.70	0.70
6	0.80	0.70
7	0.90	0.70
8	1.00	0.70
9	1.15	0.80
10	1.25	0.95
11	1.35	1.05
12	1.50	1.20
13	1.60	1.30
14	1.70	1.45
15	1.85	1.55
16	1.95	1.70
17	2.05	1.80
18	2.20	1.95
19	2.30	2.05
20	2.40	2.20
≥ 24	2.85	2.70

**Interpolate superstructure height factors for superstructure heights not given.*
†Includes highways or railways beneath bridge.
‡Water deeper than 3 m.

12.4.3.2.4 Determination of performance level

Except when alternative performance levels are Approved in accordance with Clause 12.4.3.2.2, the optimum performance level to be used for a traffic barrier shall be determined from Tables 12.5 to 12.7. When alternative performance levels are Approved, the optimum performance level shall be determined in accordance with Clause 12.4.3.2.2.

Consideration shall be given to the use of an increased design speed whenever the design speed at a bridge site is not limited by highway alignment or roadway surface.

Table 12.5
Optimum performance levels — Barrier clearance less than or equal to 2.25 m
(See Clause 12.4.3.2.4.)

Design speed, km/h	Trucks, %	Barrier exposure index PL-1	PL-2	PL-3
50	0	< 224.8	≥ 224.8	—
	5	< 75.2	≥ 75.2	—
	10	< 32.0	32.0–222.5	> 222.5
	15	< 20.5	20.5–126.3	> 126.3
	20	< 15.1	15.1–88.3	> 88.3
	25	< 12.0	12.0–67.7	> 67.7
	40	< 7.4	7.4–40.0	> 40.0
60	0	< 53.2	≥ 53.2	—
	5	< 27.4	≥ 27.4	—
	10	< 16.5	16.5–111.3	> 111.3
	15	< 12.0	12.0–63.8	> 63.8
	20	< 9.6	9.6–44.8	> 44.8
	25	< 7.8	7.8–34.4	> 34.4
	40	< 5.2	5.2–20.4	> 20.4
80	0	< 7.2	≥ 7.2	—
	5	< 6.3	6.3–188.6	> 188.6
	10	< 5.4	5.4–61.4	> 61.4
	15	< 4.8	4.8–36.7	> 36.7
	20	< 4.3	4.3–26.1	> 26.1
	25	< 3.9	3.9–20.3	> 20.3
	40	< 3.0	3.0–12.2	> 12.2
100	0	< 3.1	≥ 3.1	—
	5	< 2.9	2.9–113.2	> 113.2
	10	< 2.8	2.8–44.8	> 44.8
	15	< 2.6	2.6–28.0	> 28.0
	20	< 2.5	2.5–20.3	> 20.3
	25	< 2.4	2.4–15.9	> 15.9
	40	< 2.2	2.2–9.7	> 9.7
110	0	< 2.4	≥ 2.4	—
	5	< 2.3	2.3–84.9	> 84.9
	10	< 2.3	2.3–39.4	> 39.4
	15	< 2.2	2.2–25.6	> 25.6
	20	< 2.1	2.1–19.0	> 19.0
	25	< 2.0	2.0–15.1	> 15.1
	40	< 1.9	1.9–9.4	> 9.4

Table 12.6
Optimum performance levels — Barrier clearance greater than 2.25 m and less than or equal to 3.75 m
(See Clause 12.4.3.2.4.)

Design speed, km/h	Trucks, %	Barrier exposure index PL-1	PL-2	PL-3
50	0	—	—	—
	5	< 121.5	≥ 121.5	—
	10	< 48.2	48.2–350.1	> 350.1
	15	< 30.3	30.3–198.8	> 198.8
	20	< 22.2	22.2–138.8	> 138.8
	25	< 17.5	17.5–106.6	> 106.6
	40	< 10.7	10.7–62.9	> 62.9
60	0	< 76.6	≥ 76.6	—
	5	< 39.5	≥ 39.5	—
	10	< 22.6	22.6–171.3	> 171.3
	15	< 16.3	16.3–99.7	> 99.7
	20	< 12.7	12.7–70.3	> 70.3
	25	< 10.6	10.6–54.2	> 54.2
	40	< 6.9	6.9–32.3	> 32.3
80	0	< 9.9	≥ 9.9	—
	5	< 8.4	8.4–247.3	> 247.3
	10	< 7.2	7.2–70.6	> 70.6
	15	< 6.3	6.3–41.2	> 41.2
	20	< 5.6	5.6–29.1	> 29.1
	25	< 5.0	5.0–22.5	> 22.5
	40	< 3.8	3.8–13.4	> 13.4
100	0	< 3.6	≥ 3.6	—
	5	< 3.5	3.5–140.4	> 140.4
	10	< 3.4	3.4–49.8	> 49.8
	15	< 3.3	3.3–30.3	> 30.3
	20	< 3.2	3.2–21.8	> 21.8
	25	< 3.0	3.0–16.9	> 16.9
	40	< 2.7	2.7–10.2	> 10.2
110	0	< 2.8	≥ 2.8	—
	5	< 2.7	2.7–102.7	> 102.7
	10	< 2.7	2.7–43.2	> 43.2
	15	< 2.6	2.6–27.4	> 27.4
	20	< 2.6	2.6–20.1	> 20.1
	25	< 2.5	2.5–15.8	> 15.8
	40	< 2.4	2.4–9.6	> 9.6

Table 12.7
Optimum performance levels — Barrier clearance greater than 3.75 m
(See Clause 12.4.3.2.4.)

Design speed, km/h	Trucks, %	Barrier exposure index		
		PL-1	PL-2	PL-3
50	0	—	—	—
	5	< 255.1	≥ 255.1	—
	10	< 85.5	≥ 85.5	—
	15	< 51.9	51.9–394.1	> 394.1
	20	< 37.2	37.2–274.9	> 274.9
	25	< 29.1	29.1–211.0	> 211.0
	40	< 17.5	17.5–124.4	> 124.4
60	0	< 139.4	≥ 139.4	—
	5	< 71.8	≥ 71.8	—
	10	< 36.3	36.3–260.5	> 260.5
	15	< 25.1	25.1–151.6	> 151.6
	20	< 19.3	19.3–106.0	> 106.0
	25	< 15.7	15.7–81.5	> 81.5
	40	< 10.1	10.1–48.1	> 48.1
80	0	< 13.0	≥ 13.0	—
	5	< 11.2	11.2–314.7	> 314.7
	10	< 9.6	9.6–88.5	> 88.5
	15	< 8.4	8.4–51.5	> 51.5
	20	< 7.5	7.5–36.3	> 36.3
	25	< 6.7	6.7–28.1	> 28.1
	40	< 5.2	5.2–16.7	> 16.7
100	0	< 4.4	≥ 4.4	—
	5	< 4.1	4.1–181.5	> 181.5
	10	< 4.0	4.0–63.4	> 63.4
	15	< 3.9	3.9–38.4	> 38.4
	20	< 3.7	3.7–27.5	> 27.5
	25	< 3.6	3.6–21.5	> 21.5
	40	< 3.2	3.2–12.9	> 12.9
110	0	< 3.2	≥ 3.2	—
	5	< 3.1	3.1–135.2	> 135.2
	10	< 3.0	3.0–54.5	> 54.5
	15	< 3.0	3.0–34.2	> 34.2
	20	< 3.0	3.0–24.8	> 24.8
	25	< 2.9	2.9–19.5	> 19.5
	40	< 2.8	2.8–11.9	> 11.9

Table 12.8
Minimum barrier heights, *H**
(See Clauses 12.4.3.3, 12.4.4.2, and 12.4.5.2.)

Type of barrier	*H*, m
Traffic	
PL-1	0.68
PL-2	0.80
PL-3	1.05†
Combination (pedestrian)	1.05
Combination (bicycle)	1.37
Pedestrian	1.05
Bicycle	1.37

**The height of the barrier is the vertical distance from the top to the bottom of the roadway, sidewalk, or bikeway face of the barrier, as applicable. For combination barriers, the height of the barrier is measured on the sidewalk or bikeway face of the barrier.*
†*For freeways and high-speed rural arterial highways, consideration shall be given to increasing the barrier height to* 1.37 m.

12.4.3.3 Geometry and end treatment details
The roadway face of a traffic barrier shall have a smooth continuous alignment and a smooth transition into the roadway face of the approach roadway traffic barrier (where one is present). Where no approach roadway traffic barrier is present, traffic barrier termination details shall be consistent with the roadside safety standards of the approach roadway.

Traffic barriers shall comply with the minimum height requirements specified in Table 12.8.

Where a traffic barrier is located between the roadway and a sidewalk or bikeway, the sidewalk or bikeway face of the barrier shall have a smooth surface without snag points and a minimum height of 0.60 m measured from the surface of the sidewalk or bikeway.

12.4.3.4 Crash test requirements

12.4.3.4.1 General
The traffic barrier crash test requirements specified in Clause 12.4.3.4.2 shall be satisfied along the entire length of a traffic barrier, including at the locations of any changes in barrier type, shape, alignment, or strength that could affect barrier performance.

When a traffic barrier is to be placed on a bridge curb or sidewalk, the traffic barrier crash test requirements shall be satisfied with the barrier placed on a similar curb or sidewalk.

12.4.3.4.2 Crash test requirements for traffic barriers
Except as specified in Clauses 12.4.3.4.4 and 12.4.3.4.5, traffic barriers shall meet the crash test requirements of the optimum performance level determined in accordance with Clause 12.4.3.2, or of a more severe performance level if considered desirable.

The crash test requirements for traffic barriers for Performance Levels 1, 2, and 3 shall be the crash test requirements specified in the AASHTO *Guide Specifications for Bridge Railings*.

The crash test requirements for performance levels other than Performance Levels 1, 2, and 3 shall be Approved in accordance with Clause 12.4.3.2.2.

12.4.3.4.3 Crash test requirements for traffic barrier transitions

Except as specified in Clauses 12.4.3.4.4 and 12.4.3.4.5, traffic barrier transitions shall meet the crash test requirements used for appraising the approach roadway traffic barrier.

12.4.3.4.4 Alternative crash test requirements

A traffic barrier or traffic barrier transition shall be assumed to have met the requirements of Clauses 12.4.3.4.2 and 12.4.3.4.3, respectively, if it has been crash tested to requirements that test its geometry, strength, and behaviour to an equivalent or more severe level than the requirements of Clauses 12.4.3.4.2 and 12.4.3.4.3, respectively.

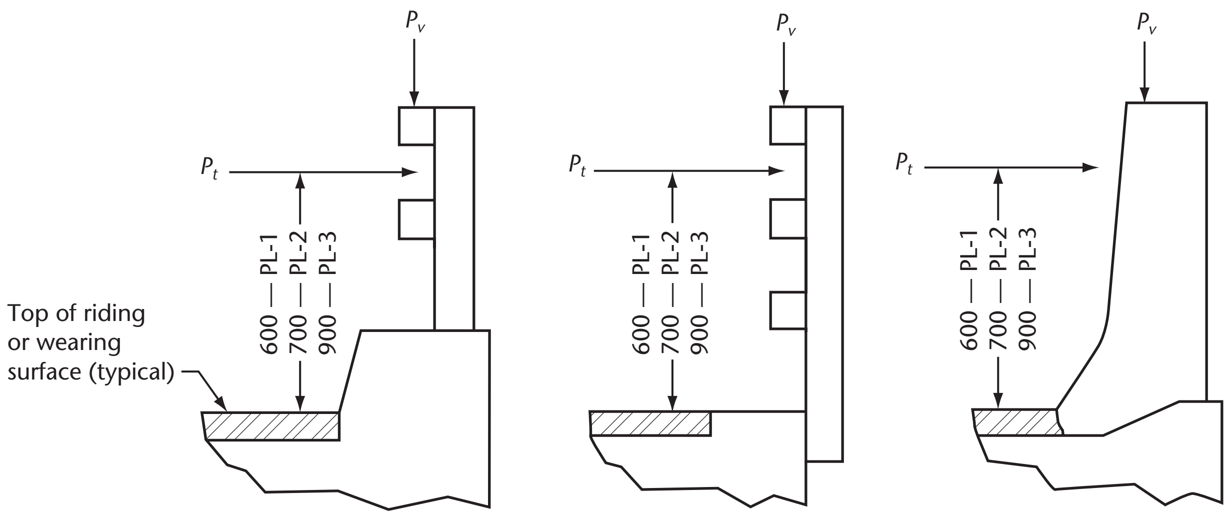
The crash test requirements for longitudinal barrier Test Levels 2, 4, and 5 of NCHRP Report 350 shall be taken as meeting the crash test requirements for Performance Levels 1, 2, and 3, respectively.

12.4.3.4.5 Changes to crash-tested traffic barriers and traffic barrier transitions

Changes to the details of a traffic barrier or traffic barrier transition that meets the requirements of Clauses 12.4.3.4.2 to 12.4.3.4.4 may be made, provided that any changes affecting the geometry, strength, or behaviour of the traffic barrier or traffic barrier transition can be demonstrated to not adversely affect vehicle-barrier interaction.

12.4.3.5 Anchorages

The suitability of a traffic barrier anchorage shall be based on its performance during crash testing of the traffic barrier. For an anchorage to be considered acceptable, significant damage shall not occur in the anchorage or deck during crash testing. If crash testing results for the anchorage are not available, the anchorage and deck shall be designed to resist the maximum bending, shear, and punching loads that can be transmitted to them by the traffic barrier, except that these loads need not be taken as greater than those resulting from the loads specified in Clause 3.8.8 and applied as shown in Figure 12.1.



Notes:

(1) *Traffic barrier types are illustrative only and other types may be used.*

(2) *Transverse load P_t shall be applied over a barrier length of* 1200 mm *for PL-1 barriers,* 1050 mm *for PL-2 barriers, and* 2400 mm *for PL-3 barriers.*

(3) *Longitudinal load P_ℓ shall be applied at the same locations and over the same barrier lengths as P_t. For post and railing barriers, the longitudinal load shall not be distributed to more than three posts.*

(4) *Vertical load P_v shall be applied over a barrier length of* 5500 mm *for PL-1 and PL-2 barriers and* 12 000 mm *for PL-3 barriers.*

(5) *These loads shall be used for the design of traffic barrier anchorages and decks only.*

Figure 12.1
Application of traffic design loads to traffic barriers
(See Clause 12.4.3.5.)

12.4.4 Pedestrian barriers

12.4.4.1 General

Pedestrian barriers shall be provided on both sides of pedestrian bridges and on the outside edges of highway bridge sidewalks separated from the roadway by a traffic barrier.

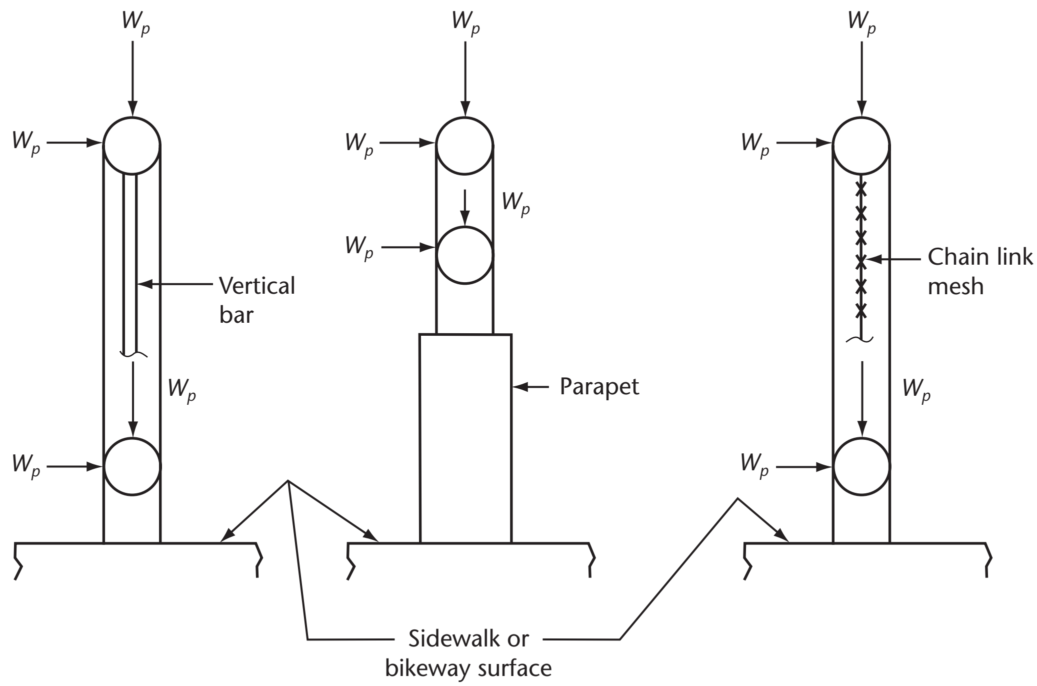
12.4.4.2 Geometry

Pedestrian barriers shall comply with the minimum height requirements specified in Table 12.8.

Openings in pedestrian barriers shall not exceed 150 mm in the least direction or shall be covered with chain link mesh. Openings in chain link mesh shall not be larger than 50 × 50 mm. The wires making up the mesh shall have a minimum diameter of 3.5 mm.

12.4.4.3 Design loading

The design loading for pedestrian barriers shall be as specified in Clause 3.8.8 and the loads applied shall be as shown in Figure 12.2. Only one railing shall be loaded at a time when posts of post-and-railing barriers are being designed.



Note: *Traffic barrier types are illustrative only and other types may be used.*

Figure 12.2
Application of pedestrian and bicycle design loads to barriers
(See Clauses 12.4.4.3 and 12.4.5.3.)

12.4.5 Bicycle barriers

12.4.5.1 General
Bicycle barriers shall be provided on both sides of bicycle bridges and on the outside edges of highway bridge bikeways where the bikeway is separated from the roadway by a traffic barrier.

12.4.5.2 Geometry
Bicycle barriers shall comply with the minimum height requirements specified in Table 12.8.

Openings in bicycle barriers for the lower 1050 mm of barrier shall not exceed 150 mm in the least direction or shall be covered with chain link mesh. Openings in chain link mesh shall not be larger than 50 × 50 mm. The wires making up the mesh shall have a minimum diameter of 3.5 mm.

12.4.5.3 Design loading
The design loading for bicycle barriers shall be as specified in Clause 3.8.8 and the loads applied shall be as shown in Figure 12.2. Only one railing shall be loaded at a time when posts of post-and-railing barriers are being designed.

12.4.6 Combination barriers

12.4.6.1 General

Combination barriers shall be provided on the outside edges of bridge sidewalks and bikeways not separated from the traffic lanes by a traffic barrier. They shall meet the requirements of Clause 12.4.3 as well as the requirements of Clauses 12.4.4 and 12.4.5, as applicable to the type of barrier, except as specified in Clause 12.4.6.2.

12.4.6.2 Geometry

Openings in combination barriers shall be less than or equal to 150 mm in the least direction for the lower 600 mm of barrier and 380 mm in the least direction above the lower 600 mm of barrier.

12.5 Highway accessory supports

12.5.1 General

When required by roadside safety standards, highway accessory supports shall be designed as breakaway supports or protected from traffic by a barrier or crash cushion. Breakaway supports shall not be used in situations where they are likely to fall across the roadway after being struck by a vehicle.

12.5.2 Vertical clearances

Vertical clearances over roadways shall comply with Clause 1.5.2.2.

12.5.3 Maintenance

Suitable access for maintaining and repairing highway accessories and their supports with minimal disruption to traffic shall be provided.

12.5.4 Aesthetics

The aesthetics of highway accessories and their supports shall be considered, with due regard for the surrounding environment.

12.5.5 Design

12.5.5.1 General

Wind loads on highway accessory supports shall be in accordance with Annex A3.2.

12.5.5.2 Ultimate limit states

12.5.5.2.1 General

The factored resistances of concrete, wood, and steel components and connections shall be determined in accordance with Sections 8, 9, and 10, respectively.

The factored resistances of aluminum components and connections shall be determined in accordance with CSA S157, except as specified in Clauses 12.5.5.2.2 and 12.5.5.2.3.

12.5.5.2.2 Heat treatment of aluminum

The yield strengths of heat affected-zones for aluminum alloy 6063 sections up to 9.5 mm thick that are welded in the T4 temper with filler alloy 4043 and then artificially aged by precipitation heat treatment to the T6 temper after welding shall be taken as 85% of the yield strengths of the non-welded alloy 6063-T6.

The yield strengths of heat-affected zones for aluminum alloy 6005 sections up to 6.4 mm thick that are welded in the T1 temper with filler alloy 4043 and then artificially aged by precipitation heat treatment to the T5 temper after welding shall be taken as 85% of the yield strengths of the non-welded alloy 6005-T5.

The extent of heat-affected zones shall be determined in accordance with CSA S157.

12.5.5.2.3 Aluminum castings
The factored resistances of aluminum castings shall be based on the resistance factors for aluminum specified in CSA S157 and the aluminum alloy strengths specified in Table 12.9, or on strength testing of the castings. Factored resistances determined from tests shall have a 98% probability of exceedance.

Table 12.9
Strengths for aluminum castings
(See Clause 12.5.5.2.3.)

	Strength, base metal, MPa		Strength, heat-affected zone, MPa
Product and alloy	**Tensile yield**	**Compressive yield**	**Tensile yield**
Permanent mold castings			
A440.0-T4	60	60	50
356.0-T6	80	80	50
356.0-T7	70	70	50
A356.0-T61	90	90	50
Sand castings			
356.0-T6	60	60	50
356.0-T7	60	60	50

12.5.5.2.4 Anchorages
Highway accessory support anchorages shall satisfy the requirements of Clause 8.16.7 and shall fully develop the strength of the support.

12.5.5.3 Serviceability limit states
Highway accessory support components and connections shall be proportioned to satisfy the applicable serviceability limit state requirements of this Code and CSA S157.

Support deformations shall be calculated for the load combination specified in Table A3.2.1 and shall be acceptable for the intended use of the support.

12.5.5.4 Fatigue limit state

12.5.5.4.1 General
Highway accessory support components and connections shall be proportioned so that their fatigue capacities are equal to or greater than the fatigue effects of the load combination specified in Table A3.2.1. Fatigue capacities shall be determined in accordance with Sections 8 and 10 and CSA S157, as applicable.

Vortex shedding excitation arising from across-wind loads at the fatigue limit state shall be considered both with and without highway accessories installed. The use of damping or energy-absorbing devices shall be considered for highway accessory supports that are subject to significant vortex shedding excitation.

12.5.5.4.2 Anchor bolts
In determining the stress range in an anchor bolt at the fatigue limit state, the effects of bending and of preloading of the bolt shall be considered.

12.5.6 Breakaway supports

12.5.6.1 General
Breakaway supports shall satisfy the requirements of Clause 12.5.5. In addition, they shall satisfy the crash test requirements of Clause 12.5.6 or have a satisfactory record of performing safely in actual service when struck by vehicles. Breakaway support crash test requirements shall be satisfied with highway shoulder and ditch geometry adjacent to the support that is similar to the geometry that will be adjacent to the support in service.

12.5.6.2 Crash test requirements
Breakaway supports shall be crash tested in accordance with Tests 62 and 63 of NCHRP Report 230, except that the impact point for Test 63 may be the centre of bumper of the impacting vehicle. In addition, the maximum change in vehicle velocity during impact shall not exceed 5 m/s (a maximum of 3 m/s is preferred).

All breakaway support columns in multiple-support roadside sign structures shall be considered as acting together to cause a change in vehicle velocity during crash testing unless
(a) each support column is designed to release independently from the sign panel;
(b) the sign panel has sufficient torsional strength to ensure this release; and
(c) the clear distance between support columns is 2100 mm or greater.

12.5.6.3 Alternative crash test requirements
A breakaway support may be assumed to have met the requirements of Clause 12.5.6.2 if it has been crash tested to requirements that test its breakaway behaviour to an equivalent or more severe level than the requirements of Clause 12.5.6.2.

The crash test requirements for breakaway utility poles Test Level 3 of NCHRP Report 350 shall be taken as meeting the crash test requirements of Clause 12.5.6.2.

12.5.6.4 Changes to crash-tested highway accessory supports
Changes to the details of the breakaway support columns of multiple-support roadside sign structures that meet the requirements of Clauses 12.5.6.2 and 12.5.6.3 may be made if all of the support columns within 2100 mm of each other have a total mass per unit length of less than 65 kg/m and a total mass of less than 270 kg between their breakaway bases and their release points from the sign panel.

12.5.6.5 Geometry
No substantial remains of a breakaway support, after it is broken away, shall project more than 100 mm above ground level.

The release point of a breakaway support column shall be at least 2100 mm above ground level.

12.5.7 Foundations

12.5.7.1 General
Foundations for highway accessory supports shall comply with Section 6, except as specified in Clause 12.5.7.2.

The foundation design shall be based on the lowest ground elevation expected to occur during the life of the support, including ground elevations occurring during construction.

12.5.7.2 Foundation investigation
The foundation investigation for standard highway accessory support foundations that are designed for a wide range of soil conditions may be based on geotechnical information obtained from investigations at neighbouring sites, soil borings for highway design, or other appropriate sources provided that the soil conditions anticipated at the site fall within the range of soil conditions used to design the foundation.

12.5.8 Corrosion protection

12.5.8.1 Steel

Corrosion protection of steel components shall be provided in accordance with Section 10, except that
(a) lapped joints of tubular steel supports shall be hot-dip galvanized; and
(b) components of breakaway supports directly involved in the breakaway function, and components of anchorages cast into concrete foundations, shall be stainless steel or hot-dip galvanized steel.

Stainless steel shall be ASTM A 167 Type 316 stainless steel.

12.5.8.2 Aluminum

Corrosion protection of aluminum components shall be provided in accordance with Section 2 and CSA S157.

12.5.8.3 Drainage and air circulation

The top surface of a support foundation shall have a minimum wash slope of 2%, and with the exception of breakaway support foundations shall not be less than 75 mm above ground level. The ground adjacent to the support foundation shall be graded to prevent the ponding of water around the foundation.

Support components shall be detailed to allow for inspection and maintenance, to prevent the accumulation of debris, and to allow for the free drainage of water and the free circulation of air both within and between components.

12.5.9 Minimum thicknesses

12.5.9.1 Steel

The minimum thicknesses of steel members shall meet the requirements of Section 10, except that
(a) the minimum thicknesses of steel truss members shall be 4.5 mm for chords and 3.0 mm for diagonals and bracing; and
(b) the minimum thicknesses of steel pole supports of closed cross-section shall be 3.0 mm.

12.5.9.2 Aluminum

The minimum thicknesses of aluminum truss members shall be 4.5 mm for chords and 3.0 mm for diagonals and bracing.

The minimum thicknesses of aluminum pole supports of closed cross-section shall be 4.5 mm.

12.5.10 Camber

Horizontal highway accessory support members shall be cambered to compensate for deflection due to unfactored dead loads. In addition, camber not less than $L/1000$ shall be provided for horizontal members of overhead and cantilevered supports.

12.5.11 Connections

12.5.11.1 Bolts

Bolts for structural connections in aluminum members shall be stainless steel or hot-dip galvanized steel.

12.5.11.2 Circumferential welds

Circumferential welds in pole-support members shall be complete penetration welds, except that the connections of steel pole-support members to base plates for luminaire and traffic signal supports not greater than 16 m in height may be socket-type connections with a continuous fillet weld on the inside of the base plate at the end of the shaft and another continuous fillet weld on the outside at the top of the base plate.

12.5.11.3 Longitudinal welds

Longitudinal seam welds in steel pole-support members within 150 mm of a complete penetration circumferential weld or within 150 mm of a lapped joint shall be complete penetration welds ground flush after welding.

12.5.11.4 Lapped joints

Lapped joints in tubular members shall be of sufficient length to develop the full strength of the lapped members. The ends of the plates in the joint shall not be chamfered over more than 50% of their thickness.

Section 13 — Movable bridges

Section 13
Movable bridges

13.1 Scope

This Section specifies requirements for the design, inspection, maintenance, construction, and rehabilitation of conventional movable highway bridges, i.e., bascule (including rolling lift), swing, and vertical lift bridges, and deals primarily with the components involved in the operation of such bridges.

13.2 Definitions

The following definitions apply in this Section:

Bascule bridge — a movable bridge that rotates about a horizontal axis.

Bridge closed or **in closed position** or **in seated position** or **in fixed position** — the bridge is in a position that permits highway traffic to use it.

Bridge open or **in open position** — the bridge is in a position that allows navigation to proceed.

Operating mode — any position during opening or closing when neither navigation nor highway traffic can proceed.

Operator — a person or persons who control a movable bridge system.

Primary power — the power required to operate a prime mover (electric power in the case of electric motors).

Prime mover — the normal means provided for driving machinery, e.g., human effort, compressed air, hydraulics, an electric motor, or an internal combustion engine.

Rolling lift bridge — a type of bascule bridge that rotates in the vertical plane and translates horizontally at the same time.

Swing bridge — a movable bridge that rotates about a vertical axis.

Vertical lift bridge — a movable bridge that raises and lowers vertically, guided by a tower or other means at each end.

13.3 Symbols

The following symbols apply in this Section:

B = angle of helical strand with axis of rope, radians (degrees)

D = rolling diameter of segment, mm (in); pitch diameter of sheave or drum, mm (in)

D_o = dead load (bridge open in any position or closed with ends just touching), kN (lb)

D_t = dead load (bridge closed; counterweight supported for repairs), kN (lb)

d = diameter of journal or step bearing, mm (in); mean diameter of collar or screw, mm (in); diameter of roller or rocker, mm (in); diameter of segmental girder, mm (in); diameter of shaft, mm (in)

d_r = rope diameter, mm (in)

d_w = diameter of largest individual wire, mm (in)
E = modulus of elasticity of wire, taken as 197 000 MPa (28 500 000 psi)
f = extreme fibre stress, MPa (psi)
H_o = horizontal force, taken as 5% of the total moving load carried by the steel towers and/or special parts of the structure that support the counterweight assembly, with the bridge open in any position, kN (lb)
I = initial tension in a rope, kN (lb)
I_o = operating impact, taken as 20%
K = impact factor
L = angle of helical wire with axis of strand, radians (degrees); length of shaft between bearings, mm (in)
L_c = live load (including dynamic load allowance), with the bridge closed and the ends just touching, and the bridge considered as a continuous structure (reactions at both ends to be positive), kN (lb)
L_s = live load (including dynamic load allowance) on one arm as a simple span, with the bridge closed and the ends just touching, kN (lb)
L_t = live load (including dynamic load allowance), with the bridge closed and the counterweight supported for repairs, kN (lb)
M = actual bending moment, N•mm (ft•lb)
M_o = maximum forces on structural parts caused by the operation of machinery, increased 100% for impact, N•mm (ft•lb)
N = number of threads of lead of worm
n = revolutions per minute; revolutions per minute of rotating part
P = minimum tension in the slack rope, kN (lb)
p = circular pitch of teeth on wheel; the least of the values of the yield strength of the material in the roller, rocker, roller bed, or track, MPa (psi); the lesser of the values of the yield strength of the steel in the segmental girder tread or track, MPa (psi)
R = radius of worm, mm (in)
r = radius of roller, mm (in); radius of gyration, mm (in)
T = twisting moment or torque, N•mm (ft•lb)
T_i = maximum operating tension in a rope (including unbalance, if any), kN (lb)
W_o = wind load, with the bridge open in any position or closed with the ends just touching, kN (lb)
α_D = dead load factor (see Table 13.6)

13.4 Materials

13.4.1 General

The material and product standards for machinery shall comply with the applicable requirements of Clause 13.8 or be subject to Approval.

13.4.2 Structural steel

Structural steel materials and products shall be in accordance with Clause 10.4.

13.4.3 Concrete

Concrete materials and products shall be in accordance with Clause 8.4.

13.4.4 Timber
Timber materials and fasteners shall be in accordance with Section 9.

13.4.5 Carbon steel
Hot-rolled carbon steel bars shall comply with ASTM A 675/A 675M.

13.4.6 Forged steel
Forged carbon steel and forged alloy steel shall comply with ASTM A 668/A 668M.

13.4.7 Cast steel or iron
Cast steel shall comply with ASTM A 27/A 27M and ASTM A 148/A 148M. Cast iron shall comply with ASTM A 48/A 48M.

13.4.8 Bronze
Bronze shall comply with ASTM B 22.

13.4.9 Bolts
Carbon steel bolts and studs shall comply with ASTM A 307 or ASTM F 568M. Quenched and tempered steel bolts and studs shall comply with ASTM A 449. High-strength structural bolts shall comply with ASTM A 325/A 325M or ASTM A 490/A 490M.

13.4.10 Wire rope
Wire rope shall comply with CSA G4, subject to the other requirements of this Section.

13.5 General design requirements

13.5.1 General
Any movable bridge in a closed position shall be designed as a fixed bridge in accordance with this Section and other applicable Sections of this Code.

13.5.2 Type of deck
Consideration shall be given to the use of a solid deck of lightweight construction to improve rideability, reduce noise, and protect the systems under the deck.

13.5.3 Piers and abutments
The drawings shall contain information on the magnitude, direction, and points of application of all loads and forces that components of the movable bridge could exert on the piers and abutments for all load combinations.

13.5.4 Navigation requirements
The location of the movable span relative to the waterway and the vertical and horizontal clearances for the bridge, in both the open and closed positions, shall meet the requirements of the *Navigable Waters Protection Act*.

The type, quantity, and location of lights, signs, and beacons for navigation and aircraft protection shall meet the requirements of Transport Canada and any other authority having jurisdiction.

13.5.5 Vessel collision
The vessel collision requirements of Section 3 shall apply to movable bridges. Any part of a superstructure that is exposed to vessel collision in the open position shall also be protected.

13.5.6 Protection of traffic

Traffic warning signs, lights, bells, or gates, or other safety devices, shall be provided for the protection of pedestrian and vehicular traffic. They shall be designed to be operative prior to the opening of the movable span and remain operative until the span has been completely closed. They shall also comply with the requirements of the Transportation Association of Canada's *Manual of Uniform Traffic Control Devices for Canada*. Where, after a bridge is closed to vehicular traffic, a vehicle continuing in its path could fall into the waterway, the use of movable barriers shall be considered.

13.5.7 Fire protection

Provision shall be made for effective smoke and fire detection and for the protection of those components of movable bridges that contain combustible material, e.g., timber decks and operator's houses, in accordance with the requirements of the *National Building Code of Canada* and the *National Fire Code of Canada*.

13.5.8 Time of operation

Under normal operation, the operating machinery shall drive the span from the seated to the fully open position, or vice versa, in not more than about 2 min.

13.5.9 Aligning and locking

Movable bridges shall be equipped with suitable mechanisms to level and align the fixed and movable roadway elements and to fasten the movable span securely in position so that it cannot be displaced either horizontally or vertically under all loading conditions. Effective end-lifting devices shall be used for swing bridges and span locks shall be used for bascule bridges. Span locks shall also be provided for vertical lift bridges when specified by the Owner.

Span locks on movable bridges shall be designed so that they cannot be driven unless the movable parts are within 15 mm of their proper positions.

13.5.10 Houses for machinery, electrical equipment, and operators

A suitable house or houses shall be provided for the machinery, electrical equipment, and operator. Houses shall be large enough to permit easy access to all equipment. They shall be fireproof, weatherproof, and climate controlled and shall comply with the requirements of the *National Building Code of Canada*, Class D or F (as determined by the authority having jurisdiction), as well as with all applicable health and safety regulations.

The operator's house shall be located so as to afford a clear view of vehicular, pedestrian, and water-borne traffic.

13.5.11 New devices

The use of state-of-the-art or recently developed mechanical or electrical devices, materials, or techniques that might be suitable for use in movable bridges and are not covered in this Section may be considered. If any such devices, materials, or techniques are used, they shall be in accordance with good commercial practice, have a history of successful application for similar uses, and be subject to the approval of the Engineer.

13.5.12 Access for routine maintenance

Non-combustible stairways, platforms, and walkways protected by metal railings shall be provided to give safe access to the operator's house, machinery, trunnions, counterweights, lights, bridge seats, and all other points requiring maintenance, inspection, and servicing. Ladders may be installed only where stairways are not feasible and shall be provided with safety devices when required by applicable codes. In vertical lift bridges, ladders and walkways shall be installed to give access to the moving span in any position from either tower. The requirements of the authority having jurisdiction shall also apply.

Machinery platforms and access walkways shall be strong enough to support components of machinery parts during dismantling for minor repairs or inspection, in addition to the mass of the workers.

In tower-drive vertical lift bridges, an electrically driven elevator should be provided in each tower unless the lift is short.

Machinery assemblies shall be designed so that all parts that can require maintenance, adjustment, or replacement are readily accessible. Ample clearance shall be provided for easy removal or replacement of such parts.

13.5.13 Durability

The durability requirements for structural materials and details shall be in accordance with Section 2.

Durability of the operational aspects of the structure shall be achieved through conservative design, proper allowance for wear, adjustment in alignment, and ease of replacement.

For mechanical, electrical, and hydraulic systems, design for durability shall take into account the operational environment, frequency of operation, and need for reliability.

A manual shall be prepared specifying proper maintenance and inspection procedures.

13.6 Movable bridge components

13.6.1 General features

13.6.1.1 Counterweights

13.6.1.1.1 General

The counterweights for bascule, vertical lift, and rolling lift bridges shall be designed to balance the moving span and all of its attached parts in any operating position, except that there shall be a small positive reaction on the span shoes when the span is closed.

Counterweights for swing spans shall be used to counteract unsymmetrical dead loads on the span.

13.6.1.1.2 Centres of gravity

Final calculations for the total mass of the moving span, including all attached parts, and the counterweight, including its supporting framework or box, shall be based on the mass calculated from the shop drawings. Final calculations for the positions of the centre of gravity of the moving span and counterweight shall also be based on this calculation of mass.

The total mass and location of the centre of gravity of the moving span and of the counterweight shall be separately shown on the assembly drawings or erection drawings.

All final calculations shall be submitted to the Engineer.

13.6.1.1.3 Unsymmetrical counterweights

When a movable bridge is unsymmetrical in transverse section, the counterweight shall be designed so that its centre of gravity will lie in the same vertical plane as that of the moving span.

13.6.1.1.4 Design

Counterweights should be supported by an embedded structural steel frame designed to carry the full mass of the counterweight. Alternatively, a counterweight may consist of a structural steel plate box suitably braced and filled with concrete.

Care shall be taken to prevent corrosion of the structural steel at all points where the structural steel enters or is adjacent to the concrete. Structural steel entering concrete shall be protected 300 mm into the concrete and be sealed at the steel/concrete interface.

Reinforcing bars shall be designed to provide for all conditions of internal stresses, with the counterweight in any position, and to distribute adequately the mass of concrete to the panel points of the supporting steel or to the connections between the counterweight and the bridge members.

13.6.1.1.5 Connections

The connections between the counterweight and its supporting bridge members shall be designed for fatigue. For design purposes, the bond between the concrete of the counterweight and the surfaces of the structural steel shapes or plates of the support frames or bridge members shall be ignored.

13.6.1.1.6 Contact surfaces

All surfaces of structural steel that come into contact with the counterweight concrete shall remain uncoated, except as specified in Clause 13.6.1.1.4.

13.6.1.1.7 Concrete

Counterweights shall be of non-air-entrained concrete and designed in accordance with Section 8. The concrete strength shall be 30 MPa (minimum) at 28 days unless otherwise specified.

For design purposes, concrete for counterweights shall be assumed to have a density of 2355 kg/m^3 unless special aggregates are specified and used.

At the start of a contract, the contractor shall determine the mass of the concrete experimentally, using aggregates typical of those to be used at the time the counterweight is to be constructed. At least three samples made from separate batches, each with a volume of at least 0.1 m^3, shall be used for mass determinations.

The test samples shall be moist cured for 28 days and air cured thereafter. The mass shall be determined at 28 days and thereafter at regular intervals until the mass is stabilized.

The design of the counterweight shall be modified if the experimental density differs from the density used in the design.

Concrete in counterweights that rotate about a horizontal axis during the operation of the moving span shall be placed in one continuous pour whenever practicable.

13.6.1.1.8 Counterweight adjustment

Counterweights shall be arranged so that adjustments in mass can be made to allow for variation in the mass of the moving span and to provide for minor discrepancies between the calculated and actual mass of the moving span and counterweight.

Adjustments in mass shall normally be made by adding or removing concrete balance blocks. In special cases, cast iron balance blocks may be used. Balance blocks shall be of a size that can be lifted by one person (in accordance with applicable labour codes) and shall have lifting handles or lifting lugs. For design purposes, it shall be assumed, in order to establish the dimensions of the fixed portion of each counterweight, that 3.5% of each counterweight will be in the form of balance blocks.

The total mass of balance blocks to be supplied by the contractor shall be equal to that required to balance the span plus spare blocks equal to 0.5% of the mass of the counterweight.

13.6.1.1.9 Pockets

The balance blocks shall preferably be placed in pockets or galleries in the counterweight and means shall be provided, where necessary, to hold them in position. The centre of gravity shall not be displaced.

Space for balance blocks shall be provided in each counterweight so that adjustments in mass amounting to 3.5% under and 5.0% over the calculated mass of the counterweight can be effected.

13.6.1.1.10 Drain holes

Pockets shall be provided with drainage holes that have a minimum diameter of 40 mm.

13.6.1.1.11 Covers

Removable but secured weather protection covers shall be provided for the balance block pockets.

13.6.1.1.12 Unequal arm swing bridges

Counterweights shall be used to balance unequal arm swing bridges about the centre of rotation.

13.6.1.1.13 Vertical lift bridge overtravel
Counterweights for vertical lift bridges shall clear the fixed structure by at least 1 m when the span is raised to its open position. In determining this clearance, the stretch in the counterweight ropes due to initial loading plus any lengthening during service shall be assumed to be 1.0% of the calculated length of the rope.

13.6.1.1.14 Counterweight temporary support
For vertical lift bridges, provision shall be made for the independent support of counterweights during construction and for rope replacements.

13.6.1.2 Buffers
Movable bridges may be equipped with buffers or hydraulic shock absorbers designed to absorb energy when the span is being seated.

13.6.1.3 Bridge stops
Bridge stops shall be provided in order to limit the travel of the moving span in the open position. They may be made of wood or another material suitable for cushioning or may take the form of buffers.

13.6.1.4 Span aligning and locking
To prevent both horizontal and vertical displacement under the action of traffic, wind, or any other cause of displacement, moving spans shall have centring and seating devices that accurately align and securely lock the spans into position.

For swing bridges, the aligning mechanism may be an automatically closing latch or other suitable aligning and locking device operated by the end-lift mechanism, or the end-lifts may themselves be designed to align the bridge.

Locks at the junction of double-leaf bascule bridges shall be designed to transmit live load shear when there is live load on one leaf only.

Where the ends of bascule bridge decks are located behind the centre of rotation and calculations indicate that the toe could be lifted from the toe rest under the passage of live load, tail locks shall be provided in order to resist the maximum reactions from live load.

13.6.1.5 Equalizing devices
For power-operated swing bridges with two or more main pinions, the shafts of the pinions shall be connected by a device that will equalize the turning forces at the pinions.

For power-operated bascule and rolling lift bridges in which two or more racks per leaf are used, a device shall be provided to equalize the load on the main pinions.

Separate drives for each main pinion, with common control to provide equalization, may be used in lieu of mechanical equalization.

On span-drive vertical lift bridges, take-ups shall be provided at the anchored end of each operating rope for adjusting and equalizing the loads in them. These take-ups shall be self-locking and accessible for maintenance and inspection.

On tower-drive vertical lift bridges, warping devices shall be provided to level the span in the transverse direction.

13.6.1.6 Traffic signals
Traffic lights should be installed at least 25 m from each end of the moving part of the structure.

When required by safety regulations, or if specified by the Owner, a sound-producing device shall be installed at the bridge to warn that the bridge is about to open. The sound shall be clearly audible at a distance of 450 m in still air.

13.6.1.7 Traffic warning gates

Traffic warning gates shall be provided for movable bridges in order to prevent pedestrians and vehicular traffic from getting onto the movable span during the operating cycle. These warning gates shall be power operated and controlled from the operator's house unless otherwise specified by the authority having jurisdiction. Provision shall be made for hand operation in case of power failure.

The warning gates should not be closer than 15 m from the ends of the movable span. They shall be painted so as to attract attention and be readily visible both day and night. They shall have red lights, reflectors, and, if required by the Owner, danger or stop signs.

Electric bells controlled from the operator's house shall be provided to warn that the gates are about to close.

Warning gates and safety equipment should be units of standard commercial manufacture for which replacements and spare parts are readily obtainable. The use of custom-built units shall be avoided as far as possible.

13.6.1.8 Movable barriers

Consideration shall be given to including energy-absorbing movable barriers in addition to warning gates for vertical lift and swing bridges and for the toe ends of single-leaf bascule or rolling lift bridges.

When movable barriers are specified, they shall be installed in the area between the warning gates and the movable span.

The minimum distance from the ends of the movable span to the movable barriers shall be determined by the anticipated deflection of the barriers under traffic impact.

13.6.1.9 Interlocking

The bridge-operating machinery shall be interlocked in such a manner that it can operate only in a predetermined and specified sequence for both opening and closing the bridge.

The controls for the operating machinery shall be interlocked with locks and/or wedges and with traffic signals, traffic gates, and/or traffic barriers in such a manner that the machinery for opening the bridge cannot be started until the locks and/or wedges are drawn and the traffic signals, gates, and/or barriers are set at the stop position. Similarly, it shall be impossible for signals, gates, and/or barriers to be set at the go position until all of the operations for closing the bridge have been performed.

Lockable bypass switches may be added to allow manual operation of individual devices and the span drive in the event of failure of any device or any part of the interlocking controls.

13.6.1.10 Position indicator

An indicator that shows the position of the moving span at all times shall be installed on the control desk and adjacent to any emergency span-operating station.

13.6.1.11 Houses for machinery, electrical equipment, and operator

Suitable houses or housings shall be provided for the protection of the machinery, electrical equipment, and operator. They shall be weatherproof, of fireproof construction, and equipped with fire extinguishers.

Houses shall be large enough for easy access to equipment. Where practicable, a hand-operated chain block and trolley-beam system shall be provided in order to facilitate the replacement of motors and other machinery parts.

In houses in which control or other electrical equipment is installed, adequate means of temperature control shall be provided.

An opening or openings in the form of doors, removable windows, or panels that are large enough to accommodate the largest piece of equipment shall be provided.

Floors shall have non-skid surfaces.

Control desks and electrical equipment may be located in the machinery enclosure or in a separate room or house.

The location and construction of the operator's house shall be such that the operator has an adequate view of all vehicular, pedestrian, and water-borne traffic.

The operator's house shall be adequately insulated and climate controlled during the operating season in accordance with health and safety regulations and to suit local climatic conditions.

Sanitary conveniences shall be provided for the use of the operator.

Adequate facilities shall be provided for routine maintenance and emergency repairs.

13.6.1.12 Provision for jacking

For swing bridges, provision shall be made for jacking needed in the repair or replacement of components such as the centre pivot, circular roller track assembly, and wedges. For bascule bridges, provision shall be made for the inspection, repair, or replacement of all main trunnions and trunnion bearings, unless otherwise directed by the Owner. The structure shall be designed to withstand the reactions from the jacks.

13.6.2 Swing bridge components

13.6.2.1 Centre bearing

13.6.2.1.1 Arrangement at centre

Centre-bearing swing bridges shall be designed so that, when the bridge is rotating, the entire mass of the moving span will be carried on a centre pivot. When the bridge is closed, the girders or trusses should be supported for live load at their centres on wedges.

13.6.2.1.2 Bearings

Centre-bearing bridges shall rotate on spherical disc thrust bearings.

Disc bearings shall consist of a bronze disc and a hardened steel disc designed so that sliding will occur entirely between the bronze and hardened steel surfaces. The discs shall be turned, accurately ground to a highly polished finish, and positively locked against rotation. Straight oil grooves shall be cut into the bronze discs as necessary for proper lubrication.

The span shall be effectively held laterally to resist the specified wind force on the bridge while swinging, and provision shall be made for removal of the bearings without jacking up the structure more than is necessary to take the load off the centre pivot, and without interfering with the operation of traffic over the bridge.

13.6.2.1.3 Pivot support

The centre pedestal supporting the pivot shall be made of cast or welded steel. It shall be proportioned for both strength and rigidity and shall be securely anchored to the support.

13.6.2.1.4 Balance wheels

No fewer than eight wheels running on a circular track shall be provided to limit the tilting of the bridge and transmit wind loads and any other unbalanced effects to the track while the bridge is rotating. Balance wheels shall be adjustable for height and shall be machined conical or crowned on treads except where rails are used for the track.

13.6.2.1.5 Hub length

Where the axles for balance wheels are fixed and the whole wheel bearing rotates about the axle, the wheel hub shall be of such a length that any line normal to the wheel tread shall lie well within the outside edge of the wheel bearing.

13.6.2.2 Rim bearing

13.6.2.2.1 Arrangement at centre
Rim-bearing swing bridges shall be designed so that most or all of the mass of the moving span is carried on a circular conical roller track assembly when the bridge is rotating. The roller assembly shall be proportioned for the combined effect of the specified dead load and wind load when the bridge is rotating, and for the combined dead load, live load (including dynamic effects), and wind load when the bridge is closed.

13.6.2.2.2 Load distribution
The load on the rim girder of a rim-bearing or combined rim and centre-bearing swing bridge shall, if practicable, be distributed equally among the bearing points. The bearing points shall as nearly as is practicable be spaced equally around the rim girder.

13.6.2.2.3 Struts
Rigid struts shall connect the rim girder to a centre pivot firmly anchored to the pier. A strut shall be attached to the rim girder at each bearing point and at intermediate points if necessary.

13.6.2.2.4 Rim girders
Rim girders and upper treads shall be designed so that the load will be properly distributed over the rollers. For calculating stresses in the girder, the load shall be assumed to be distributed equally among all rollers. The span lengths shall be taken as the developed length of the girder between adjacent bearing points and analyzed as a continuous girder.

13.6.2.2.5 Rollers
Rollers shall be machined conical on the treads. Rollers shall be adjustable axially to provide equal load sharing among all rollers.

13.6.2.3 Main pinions

13.6.2.3.1 General
At least two main pinions shall be used. Where two pinions are used, they shall be placed diametrically opposite one another. Where four pinions are used, they shall be placed in two diametrically opposite pairs.

13.6.2.3.2 Pinion-bearing supports
The brackets and connections that support the main pinion bearings shall be designed for at least twice the maximum design torque in the pinion.

13.6.2.3.3 Pinion bearings
Pinion-bearing housings and bearing caps shall be secured with turned bolts. The bearings shall be designed with enough shims to provide for overrun or under run in the diameter of the rack.

13.6.2.4 Racks
Rack segments should be made from cast steel.

Racks should be bolted to the supporting steelwork to facilitate adjustment and replacement. They shall be machined at connections to supports and at their joints. Where racks are mounted on tracks, the joints shall be staggered.

Separate rack and track segments should be used.

Racks mounted on the substructure that are not attached to the track shall be anchored to the foundation by an ample number of anchor bolts; the tractive force developed when turning the bridge shall be taken by at least one lug on each rack segment, extended from the bottom of the rack downward into the support and set in cement mortar, concrete, or grout.

13.6.2.5 Track and treads

The lower treads of rim-bearing swing bridges and the tracks for centre-bearing swing bridges shall be made sufficiently strong and stiff to properly distribute the maximum roller or balance-wheel load to the substructure.

The rolling surfaces of treads and tracks shall be machined conical to provide pure rolling motion with the rollers as the bridge rotates.

The track segments shall be made of cast steel or steel weldments. For small spans, where balance wheel loads are light, steel rails may be used for track. These rails shall be connected to structural steel plates to secure adequate anchorage.

The treads attached to rim girders shall be rolled steel slabs or steel castings and shall not be considered part of the girder flange material. The treads shall be considered pedestals that distribute the line-bearing pressures from the roller to the girders. That part of the outstanding leg of a girder flange angle that is beyond the outside face of the vertical leg shall not be considered the bearing area.

The surface of the bottom flange of rim girders that bears on the tread shall be machined. The centreline shall be inscribed on the surface of the treads.

13.6.2.6 End-lifts

13.6.2.6.1 Type

The end-lift mechanism to be used shall be simple and positive in action. The actuating mechanism shall be non-reversible under the action of the live load.

13.6.2.6.2 Capacity

The end-lifting machinery shall be designed to exert an upward force equal to at least 1.5 times the maximum negative end reaction of the live load (including dynamic load allowance) plus the reaction caused by the deflection due to temperature differential.

13.6.2.6.3 Height of lift

The end-lifting machinery shall be proportioned to lift the ends of the span an amount that will ensure a positive reaction under all conditions of live load and to remove deflection due to temperature differential.

The vertical height of lift shall be the sum of the following:

(a) the deflection due to 1.5 times the maximum live load negative reaction;
(b) the deflection due to the temperature differential between the top and bottom chords;
(c) the height to which the end of the bridge can tilt until limited by the balance wheels;
(d) adequate clearance for swinging; and
(e) additional temperature deformations in the longitudinal and transverse directions (in the case of solid decks).

13.6.2.7 Wedges

Wedges shall be designed so that they cannot be displaced by action of the moving load or by power failure. They shall be capable of adjustment.

The centre wedges and supports shall be proportioned for the reaction from live load (including dynamic effects) and shall have suitable means to achieve an equal bearing without the span being lifted.

End wedges shall be proportioned to provide the required vertical height of lift and to support all forces from the wedging action and applied loads.

13.6.3 Bascule bridge components

13.6.3.1 Centring devices
Transverse centring of the toe ends of single- and double-leaf bascule spans shall be provided by devices located on or near the centreline of the bridge. The lateral clearance in the centring device shall not exceed 2 mm.

13.6.3.2 Locking devices
Single-leaf bascule spans shall have, at a minimum, a locking device at the toe end of each outside girder or truss to hold the leaf down against its seat.

Double-leaf bascule spans shall have, at a minimum, shear locks at the toe ends of each outside pair of girders or trusses to align the leaves vertically and maintain alignment with live load on one leaf only.

Tail locks shall be provided when the deck of the bascule leaves extends behind the centreline of trunnions to resist the maximum reaction from live loads.

13.6.3.3 Trunnions and trunnion hubs
Trunnions for bascule spans shall be steel forgings and shall be interference fitted into cast steel hubs. Hubs and trunnions shall be tightly dowelled or keyed against rotation.

Trunnion hubs shall be interference fitted into bascule spans and shall also be secured by a sufficient number of bolts to transmit the entire applied load and frictional torque. The design of the trunnion shall minimize stress concentrations.

13.6.4 Rolling lift bridge components

13.6.4.1 Segmental and track girders

13.6.4.1.1 General
The flanges of segmental and track girders for rolling lift bridges shall be symmetrical about their web or webs. The width of contact between the web of the segmental girder and the back of the tread plate shall be equal to the corresponding width of contact in the track girder.

13.6.4.1.2 Machining
The face of the flange shall be machined for full bearing on the tread or track plate.

13.6.4.1.3 Design
The unit-bearing pressure of the web plate on the tread shall not exceed one-half of the yield stress of the material in tension. In calculating the unit-bearing pressure, the force shall be considered as distributed over a rectangular area whose width is the thickness of the web and whose length is 1.6 times the least thickness of the tread.

13.6.4.2 Treads and track
The treads attached to the segmental girders and the track girders shall be rolled steel plates, forgings, or castings. They shall not be considered part of the girder flange material. The thickness of treads shall be at least 75 mm plus $0.004D$, where D is the rolling diameter of the segment in millimetres.

The top and bottom surfaces of the treads and track shall be machined. Their ends shall be machined to bear and they shall be designed to be replaceable.

The treads and track shall be continuous (without any joint) if practicable. They shall be connected to the segmental and track girders so that, as far as possible, they act monolithically with them to prevent any working at the contact surfaces.

Where treads and tracks are made in segments, the number of joints shall be kept to a minimum. The faces of the joints between the segments shall be in planes at right angles to the rolling surface and the girders shall be fully stiffened at these joints.

13.6.5 Vertical lift bridge components

13.6.5.1 Auxiliary counterweights

Auxiliary counterweights shall be used to balance the mass of the main counterweight ropes as the lift span opens and closes. Any unbalanced rope mass not compensated by auxiliary counterweights shall be included in the lift span power calculations.

13.6.5.2 Span guides

13.6.5.2.1 Lower guides

The span guides for all vertical lift bridges shall be attached at the level of the bottom lateral bracing system. At one end of the moving span, the two guides shall control the movement of the span in the longitudinal and transverse directions. At the other end, the two guides shall control the movement of the span in the transverse direction only.

13.6.5.2.2 Upper guides

For through-truss bridges, two upper guides shall be provided at each end of the moving span at the level of the top chord lateral system. They shall control the movement of the span in the transverse direction only.

13.6.5.2.3 Clearances

The normal running clearance between the mating surfaces of the span guides and the tower guide track shall be 15 mm in the transverse and longitudinal directions. All span guides shall be adjustable to allow for accurate alignment between the lift span and tower structures.

13.6.5.2.4 Guide material

Span guides shall be made of steel plate, steel weldments, or steel castings. Sliding guides shall be fitted with bronze liners arranged for easy replacement when necessary. Guide rollers shall be made of steel.

13.6.5.2.5 Tower guide track

The tower guide track shall be designed to transfer the loads from the span guides to the tower structure. All guide track running surfaces shall be machined.

The tower guide track may be flared at the bottom to reduce the normal running clearances of the lower guides from 15 to 3 mm as the lift span approaches its closed position, thereby centring the span before it seats. Alternatively, centring devices may be incorporated into end floor beams.

13.6.5.3 Counterweight guides

13.6.5.3.1 Guide shoes

A minimum of four guide shoes shall be provided for each counterweight, two on each side, spaced as far apart as practicable in the vertical direction.

13.6.5.3.2 Clearances

The normal running clearance between the mating surfaces of the counterweight guide shoes and the guide track shall be 20 mm in the transverse direction and 15 mm in the longitudinal direction. All counterweight guide shoes shall be mounted on shims for transverse and longitudinal adjustments.

13.6.5.3.3 Shoe material

Counterweight guide shoes shall be made of steel weldments, steel castings, or bronze castings and shall be adjustable and replaceable. Steel shoes shall be fitted with bronze liners arranged for easy replacement.

13.6.5.3.4 Counterweight guide track
The running surfaces of counterweight guide track that are subject to transverse loads shall be machined. Joints shall be machined and provision shall be made for lateral adjustment.

13.6.5.4 Counterweight sheaves

13.6.5.4.1 General
Counterweight sheaves shall be cast or welded and shall have interference fits on the trunnion shafts. Hubs shall be secured to trunnion shafts with driving-fit dowels set in holes drilled after the sheave is shrunk onto the trunnion shaft. Rope grooves shall be machined to suit the diameter of the rope. The space between ropes shall be at least 6 mm. All of the grooves of all of the sheaves shall have a uniform pitch diameter and the variation from the specified diameter shall not exceed ± 0.25 mm.

13.6.5.4.2 Welded counterweight sheaves
Counterweight sheaves fabricated by welding shall have structural steel plate rims and webs and forged carbon steel hubs with controlled chemical content to ensure weldability. Welded sheaves shall be stress relieved prior to machining. Details shall be suitable for the anticipated load cycles on the sheaves. The allowable stress ranges specified in Clause 10.17 shall be used for design. A dynamic load allowance of at least 20% shall be included in the design loading.

13.6.5.4.3 Operating rope drums and deflector sheaves
Rope grooves shall be machined to suit the diameter of the rope. The clear space between ropes shall be at least 3 mm.

Deflector sheaves shall generally have the same diameter as the drums. Intermediate deflector sheaves shall be provided, as necessary, to prevent rubbing of the ropes on fixed parts of the lift span and to avoid excessive unsupported lengths of rope.

Lightly loaded intermediate deflector sheaves shall be supported on anti-friction bearings and shall be as light as practicable to ensure easy turning.

All deflector sheaves shall have grooves sufficiently deep to prevent the ropes from being displaced.

13.6.5.5 Wire rope

13.6.5.5.1 General
Unless otherwise specified in this Section, the manufacturing requirements for wire rope shall be in accordance with CSA G4.

13.6.5.5.2 Grade
The wire for the ropes shall be either Grade 1770 or Grade 110/120 steel, bright finish, with a minimum tensile strength of 1770 MPa and 246 000 psi, respectively, and a maximum tensile strength as specified in Tables 13.1 and 13.2, respectively.

Table 13.1
Maximum tensile strength for Grade 1770 bright wire
(See Clause 13.6.5.5.2.)

Wire diameter, mm	Maximum tensile strength, MPa
0.20 to less than 0.50	2160
0.50 to less than 1.00	2120
1.00 to less than 1.50	2090
1.50 to less than 2.00	2060
2.00 to less than 5.00	2030

Table 13.2
Maximum tensile strength for Grade 110/120 bright wire
(See Clause 13.6.5.5.2.)

Wire diameter, in	Maximum tensile strength, psi
0.007–0.019	302 000
0.020–0.038	298 000
0.039–0.058	293 000
0.059–0.078	289 000
0.079–0.097	284 000
0.098–0.147	284 000
0.148–0.196	284 000

13.6.5.5.3 Construction
The ropes shall be 6 × 19 classification and the construction shall be 6 × 25 filler with fibre core.

13.6.5.5.4 Splicing
Ropes or strands shall not be spliced. Wire splices shall be made by electric welding, and no two joints in any one strand shall be less than 7.5 m apart, except for filler wires.

13.6.5.5.5 Lay
All wire ropes, unless otherwise specified by the Owner, shall be right regular lay, and the maximum length of rope lay shall be 6.75 times the nominal operating rope diameter or 7.5 times the nominal counterweight rope diameter.

The lay of the wires in the strands shall be such as to make the wires approximately parallel to the axis of the rope where they would come in contact with a cylinder circumscribed on the rope.

13.6.5.5.6 Lubrication
All portions of the wire ropes, including wires, strands, and cores, shall be thoroughly lubricated during fabrication with a lubricant containing a rust inhibitor. The lubricant shall be approved by the Engineer.

13.6.5.5.7 Testing, inspection, and sampling

The manufacturer shall provide proper and adequate facilities for testing, inspecting, and sampling wire ropes as specified in CSA G4. All tests shall be made in the presence of an inspector appointed by the Owner.

13.6.5.5.8 Tensile test on whole rope

For the tensile test on whole rope, a test specimen of unused and undamaged wire rope shall be cut from each end of each continuous length of wire rope used for the finished lengths ordered. Each specimen shall have sockets of the same type and lot as those of the finished product attached at the ends. Additional test specimens may be required by the Engineer, but the total number of test specimens shall not exceed 10% of the total number of finished lengths of rope ordered, not including rejected specimens. The distance between sockets shall be at least 1.0 m for ropes less than 26 mm (1.0 in) in diameter and at least 1.5 m for ropes 26 mm (1.0 in) in diameter or larger.

Each specimen shall be tested by applying a load not more than 60% of the ultimate strength specified in Table 13.13 with unrestricted crosshead speed; thereafter, the load shall be increased, with crosshead speed limited to 15 mm/min (0.5 in/min), until breakage occurs. If the specimen breaks within two wire diameters of the socket before the ultimate strength specified in Table 13.13 is attained, the tensile test shall be rejected and the test repeated.

All test specimens shall develop the ultimate strength specified in Table 13.13.

Failure of any valid specimen to pass the test shall be cause for rejection of the entire length of rope from which the specimen was taken, which shall be replaced by the manufacturer with a new length of rope of the same type.

Note: *See Clause 13.6.5.5.15 for testing of sockets.*

13.6.5.5.9 Prestressing counterweight ropes

Each counterweight rope shall be prestressed using the following procedure:

(a) load the rope in tension to 40% of the ultimate strength specified in Table 13.13 and hold that load for 5 min;
(b) reduce the load to 5% of the ultimate strength;
(c) repeat this load-unload cycle two more times; and
(d) release the load.

Prestressing shall be carried out using rope lengths containing not more than two counterweight ropes. The rope shall be supported throughout its entire length at points not more than 7.5 m apart.

13.6.5.5.10 Length measurement of counterweight ropes

13.6.5.5.10.1

The length of each counterweight rope shall be taken as follows:

(a) for open sockets: the distance between the centres of pins;
(b) for closed sockets: the distance between bearings; and
(c) for block sockets: the distance between bearing faces.

13.6.5.5.10.2

The following procedure shall be used in determining the correct rope length:

(a) the rope shall be supported throughout its entire length at points not more than 7.5 m apart;
(b) it shall be twisted until the lay is correct; and
(c) it shall be measured under a tension of 12% of its ultimate strength (which corresponds to approximately the direct load on the rope).

13.6.5.5.10.3

Where rope attachment assemblies do not provide for rope length adjustment after installation, the length of all ropes shall be rechecked after socketing in accordance with the procedures specified in Clause 13.6.5.5.10.2. The checked rope length shall be corrected, if necessary, to bring it within the permissible variations in length by resocketing or other means approved by the Engineer.

A durable tag or label shall be attached to each coil or reel of rope to indicate length, diameter, grade, construction, manufacturer's name, order number, and destination.

Each rope shall have a stripe painted along its entire length at the time that the length is measured. This stripe shall be straight after the rope is erected.

13.6.5.5.11 Length tolerance

A maximum variation of ± 6 mm for every 30 m of rope length shall be allowed.

13.6.5.5.12 Operating ropes

Operating ropes shall not be prestressed unless otherwise specified by the designer. When not socketed, the ends of operating ropes shall be seized and the ends of the rope wires shall be welded together. If necessary, the seizing may be removed before the ropes are attached to the drums.

13.6.5.5.13 Shipping

Wire ropes shall be shipped on reels or coiled on wooden crosses. The diameter of the reel or the inside diameter of the coil shall be at least 25 times the diameter of the wire rope. The wire ropes shipped on reels shall be removed by revolving the reels.

Ropes shipped on wooden crosses shall be securely lashed to one side of the cross and wood blocks shall be attached to the four arms so that each block makes contact with the inside of the coil to prevent movement during transit.

Each rope shall be coiled on a separate reel of the same diameter for shipment.

13.6.5.5.14 Sockets

Counterweight ropes shall have socketed end connections. Sockets, except block types, should be forged without welds from steel with an ultimate tensile stress of 445 to 515 MPa (65 000 to 75 000 psi) and be normalized. Cast steel sockets of open and closed types may be used for the larger sizes.

Block sockets that provide for the direct load on the rope to be transmitted by bearing on the front face of the socket may be made from hot-rolled bars; retaining lugs may be welded on where necessary.

The dimensions of all sockets shall be such that no part under tension will be stressed more than 445 MPa (65 000 psi) when the rope is loaded to its ultimate strength. All sockets shall be attached to the rope by a proven method that will not permit the rope to slip appreciably in its attachments.

13.6.5.5.15 Testing of sockets

The rope test specified in Clause 13.6.5.5.8 shall also be used to test the sockets. If appreciable slipping in the sockets occurs during the test, the method of fastening the sockets to the rope shall be changed to the satisfaction of the Engineer to minimize the amount of slippage.

The sockets shall be stronger than the rope to which they are attached. If a socket breaks during a test, other sockets shall be selected, attached to the rope, and the test repeated. This process shall be continued until the Engineer is satisfied as to the reliability of the sockets. If, however, 10% or more of the sockets tested break at a load less than the minimum ultimate strength of the rope specified in Table 13.13, the lot shall be rejected.

13.6.5.6 Wire rope attachment

13.6.5.6.1 Counterweight rope connections

The connections of the counterweight ropes to the lift span and to the counterweights shall be such as to permit ready replacement of any one rope without disturbing the others. Provision shall also be made for the replacement of all ropes simultaneously. The connections of all ropes shall be such as to load all ropes of a group equally.

13.6.5.6.2 Alignment of sockets
The axis of the rope shall at all times be at right angles to the axis of the pin for open and closed sockets and to the bearing face for block sockets.

13.6.5.6.3 Operating rope connections
Each operating rope shall have at least two full turns of the rope on the operating drum when the span is in the fully open or closed position. The end of the rope shall be clamped to the drum to avoid sharp bends in the wires. The dead end of each operating rope shall be attached to a device for taking up the slack in the rope.

13.6.5.6.4 Slapping ropes
On the lift span side, the counterweight ropes shall be sufficiently separated to prevent objectionable slapping against each other while the span is in the closed position. This shall be accomplished by using widely spaced grooves on the sheaves, by deviating alternate ropes in vertical planes, or by another means approved by the Engineer.

13.7 Structural analysis and design

13.7.1 General
Clauses 13.7.2 to 13.7.19 apply to bridges for which the moving span is normally left in the closed position. When the bridges are in the closed position, all of the requirements of this Code relating to fixed bridges shall apply.

Clauses 13.7.2 to 13.7.19 apply specifically to the design of movable bridges when they are in the operating mode or in the open position, to swing bridges that are closed but whose ends are just touching, and to bascule and vertical lift bridges that are closed but whose counterweights are supported for repairs. The special load combinations in Tables 13.3 to 13.5 do not include load combinations for substructure designs.

13.7.2 Design theory and assumptions
The design theory and assumptions specified in other Sections of this Code shall apply to movable bridges, except that the loads, load factors, and load combinations specified in Clauses 13.7.3 to 13.7.19 shall also be considered.

13.7.3 Wind loads

13.7.3.1 General
The loads and related areas specified in Clauses 13.7.3.2 to 13.7.3.10 shall be used in proportioning members and determining stability.

13.7.3.2 Horizontal transverse wind, normal to centreline
For girder spans, the surface area shall be considered to be 1.5 times the vertical projection of the span, including the deck and railing, plus the vertical projection of any counterweight. For truss spans, the surface area shall be considered to be the vertical projection of the floor system and any counterweight, plus twice the vertical projection of the members of one truss.

13.7.3.3 Horizontal longitudinal wind, parallel to centreline
For bascule (including rolling lift) bridges, the surface area shall be considered to be the vertical projection of the floor plan area and those portions of the vertical projection of the counterweight, where applicable, that are not shielded by the floor plan area. The floor plan area of a bridge with an open deck shall not be

considered to be shielding other parts of the structure. For vertical lift bridges, the total longitudinal wind force acting on the moving span shall be assumed to be 50% of the total transverse wind force acting on the span and to act through the same centre of gravity.

13.7.3.4 Vertical wind, normal to the floor plan area
For swing bridges, the surface area shall be the floor plan area of the larger arm. For vertical lift bridges, the surface area shall be the floor plan area.

13.7.3.5 Floor plan area
The floor plan area exposed to wind or ice shall be taken as a quadrilateral whose length is equal to that of the floor of the moving span and whose width is that of the distance out-to-out of trusses, girders, or sidewalks, whichever gives the greatest width.

For bridges decked with open steel grating, the floor plan area of the grating shall be assumed to be 85% of the floor plan area of a solid deck.

13.7.3.6 Operator's house and machinery house
If the operator's house, the machinery house, or both are located on the moving span, their projected areas shall be included in the surface area for wind, except for portions shielded by the floor plan area. Open decks shall not be considered to be shielding.

13.7.3.7 Towers and their bracing
Exposed areas for transverse and longitudinal wind loads on towers and their bracing shall include the vertical projections of all columns and bracing not shielded by the counterweights and houses.

13.7.3.8 Swing bridges
The horizontal transverse wind pressure shall be 1.2 kPa on one arm and 1.70 kPa on the other arm. The vertical wind pressure shall be 0.25 kPa on the floor plan area of one arm; in the case of unequal arm bridges, the floor plan area of the longer arm shall be used.

13.7.3.9 Bascule (including rolling lift) bridges
The horizontal transverse wind pressure shall be 1.50 kPa and the horizontal longitudinal wind pressure shall be 1.50 kPa.

13.7.3.10 Vertical lift bridges
The horizontal transverse wind pressure shall be 1.50 kPa and the horizontal longitudinal wind pressure shall be 1.50 kPa, except as specified in Clause 13.7.3.3. The vertical wind pressure shall be 0.25 kPa on the floor plan area.

13.7.4 Seismic loads
For design with the movable spans in the closed position, seismic loads shall be as specified in Sections 3 and 4. For design with the movable spans in the open position, the seismic loads shall be one-half of those specified in Sections 3 and 4.

13.7.5 Reaction due to temperature differential
For swing bridges, provision shall be made for an end reaction due to the following temperature differentials:
(a) between the top and bottom chords of a truss: 10 °C; and
(b) between the top and bottom flanges of a girder: 8 °C.
Load combinations ULS 2, ULS 3, and ULS 4 of Section 3 shall apply.

13.7.6 Hydraulic cylinder connections

The loads on the structural connections to the cylinders shall be based on the maximum of
(a) wind, ice, inertia, or other structural loads, assuming the cylinder as a rigid link; and
(b) driving and braking mechanical loads, assuming a cylinder force developed by 150% of the setting of the pressure-relief valve that controls the maximum pressure available at the cylinder.

13.7.7 Loads on end floor beams and stringer brackets

The end floor beams and stringer brackets of the moving span shall be proportioned for at least the factored dead load and factored live load plus twice the factored dynamic load allowance.

13.7.8 Swing bridges — Ultimate limit states

13.7.8.1 Closed position

In the closed position, the bridge ends are lifted to give a positive reaction equal to 150% of the maximum negative reaction due to live load and dynamic load allowance. For this position, load combinations ULS 1 to ULS 8 of Section 3 shall apply.

13.7.8.2 Special load combinations and load factors

In addition to load combinations ULS 1 to ULS 8 of Section 3, the special load combinations and load factors specified in Table 13.3 shall be considered.

Table 13.3
Swing bridges — Special load combinations and load factors
(See Clauses 13.7.1 and 13.7.8.2.)

Load combination	D_o*	L_s	L_c	W_o	I_o	M_o
ULS S1	α_D	0	0	0	1.20	1.55
ULS S2	α_D	1.70	0	0	0	0
ULS S3	α_D	0	1.70	0	0	0
ULS S4	α_D	1.40	0	1.30	0	0
ULS S5	α_D	0	1.40	1.30	0	0
ULS S6	α_D	0	0	1.30	1.20	1.25
ULS S7	α_D	0	0	1.65	1.20	0

**See Table 13.6 for values of α_D.*

Notes:

(1) *When the ends are being lifted, the loading combination and load factors are similar to ULS S1 except that there are different machinery forces and there are forces at the ends.*

(2) *For any combination, the minimum or maximum value of the load factor, α_D, shall be used so as to maximize the total force effect.*

Legend:

D_o = dead load; bridge open in any position or closed with ends just touching

L_s = live load (including dynamic load allowance) on one arm as a simple span; bridge closed with ends just touching

L_c = live load (including dynamic load allowance); bridge closed with ends just touching, with bridge considered as a continuous structure; reactions at both ends to be positive

W_o = wind load; bridge open in any position or closed with ends just touching

I_o = operating impact of 20% applied to the maximum dead load effect in all members that are in motion and to the load effect on a stationary member caused by the moving dead load

M_o = maximum forces on structural parts caused by the operation of machinery, or by forces applied for moving or stopping the span, increased 100% as an allowance for impact

13.7.8.3 Stationary in open position

In the event that a movable bridge is required to remain open or can be stuck in the open position, the requirements of Section 3 for wind loads on a fixed bridge shall apply.

13.7.9 Bascule (including rolling lift) bridges — Ultimate limit states

13.7.9.1 Closed position

When the bridge is not in operating mode and the counterweight is not temporarily supported for repairs, load combinations ULS 1 to ULS 8 of Section 3 shall apply.

13.7.9.2 Special load combinations and load factors

In addition to load combinations ULS 1 to ULS 8 of Section 3, the special load combinations and load factors specified in Table 13.4 shall be considered.

Table 13.4
Bascule (including rolling lift) bridges —
Special load combinations and load factors
(See Clauses 13.7.1 and 13.7.9.2.)

Load combination	D_o*	D_t*	L_t	H_o	W_o	I_o	M_o
ULS B1	α_D	0	0	0	0	1.2	1.55
ULS B2	α_D	0	0	0	1.30	1.2	1.25
ULS B3	α_D	0	0	0	1.60	1.2	0
ULS B4	α_D	0	0	1.55	0	1.2	1.55
ULS B5	0	α_D	1.70	0	0	0	0

**See Table 13.6 for values of α_D.*

Notes:

(1) *Combination ULS B4 applies only to those parts of the structure that support trunnions of the moving span and/or counterweight.*

(2) *For any combination, the minimum or maximum value of the dead load factor, α_D, specified in Section 3 shall be used to maximize the total force effects.*

Legend:

D_o = dead load; bridge open in any position or closed with ends just touching

D_t = dead load; bridge closed; counterweight supported for repairs

L_t = live load (including dynamic load allowance); bridge closed; counterweight supported for repairs

H_o = horizontal force, taken as 5% of the total moving load carried by the steel towers and/or special parts of the structure that support the counterweight assembly; bridge open in any position; this force shall be applied in any direction, through the centre of gravity of the moving load

W_o = wind load; bridge open in any position or closed with ends just touching

I_o = operating impact of 20% applied to the maximum dead load effect in all members that are in motion and to the load effect on a stationary member caused by the moving dead load

M_o = maximum forces on structural parts caused by the operation of machinery, or by forces applied for moving or stopping the span, increased 100% as an allowance for impact

13.7.10 Vertical lift bridges — Ultimate limit states

13.7.10.1 Closed position

When the bridge is not in operating mode and the counterweight is not temporarily supported for repairs, load combinations ULS 1 to ULS 8 of Section 3 shall apply.

13.7.10.2 Special load combinations and load factors

In addition to load combinations ULS 1 to ULS 8 of Section 3, the special load combinations and load factors specified in Table 13.5 shall be considered.

Table 13.5
Vertical lift bridges — Special load combinations and load factors
(See Clauses 13.7.1 and 13.7.10.2.)

Load combination	D_o*	D_t*	L_t	W_o	I_o	M_o
ULS V1	α_D	0	0	0	1.2	1.55
ULS V2	α_D	0	0	1.30	1.2	1.25
ULS V3	α_D	0	0	1.65	1.2	0
ULS V4	0	α_D	1.70	0	0	0

**See Table 13.6 for values of* α_D.

Note: *For any combination, the minimum or maximum value of the dead load factor,* α_D*, shall be used to maximize the total force effects.*

Legend:

D_o = dead load; bridge open in any position or closed with ends just touching

D_t = dead load; bridge closed; counterweight supported for repairs

L_t = live load (including dynamic load allowance); bridge closed; counterweight supported for repairs

W_o = wind load; bridge open in any position or closed with ends just touching

I_o = operating impact of 20% applied to the maximum dead load effect in all members that are in motion and to the load effect on a stationary member caused by the moving dead load

M_o = maximum forces on structural parts caused by the operation of machinery, or by forces applied for moving or stopping the span, increased 100% as an allowance for impact

13.7.11 Dead load factor

The dead load factor, α_D, in Tables 13.3 to 13.5 shall be as specified in Table 13.6.

Table 13.6
Dead load factor, α_D
(See Clause 13.7.11 and Tables 13.3–13.5.)

Dead load type	Maximum	Minimum
D1 — Factory-produced components	1.20	0.90
D2 — Cast-in-place concrete and wood	1.40	0.80
D3 — Wearing surface	1.40	0.75

13.7.12 All movable bridges — Ultimate limit states
Longitudinal wind shall be included in the design of the superstructure for load combinations ULS 3, ULS 4, and ULS 7 of Section 3. For seismic loading with the bridge open, load combination ULS 5 of Section 3, as modified by Clause 13.7.4, shall apply. For vessel collisions with the bridge operating, load combination ULS 8 of Section 3 shall apply.

13.7.13 Special types of movable bridges
The analysis of special types of movable bridges shall be carried out for all applicable load conditions. The members shall be proportioned for total factored load effects in accordance with the requirements specified in this Section for the design of fixed, swing, bascule, and vertical lift bridges.

13.7.14 Load effects
A drawing shall be prepared showing the load effects for the various analyses and the total factored load effects for the applicable combinations in the different primary members at appropriate locations.

13.7.15 Fatigue limit state
The stress range arising from the operation of the span from the fully closed to the fully open position and back to the fully closed position, including the effect of wind, shall be less than the allowable stress range specified in Section 10, based on the estimated number of load cycles.

This Clause shall also apply to members and/or steel that are embedded in or encase counterweights and either support the mass or transfer the load to the main structure, and to the connections of such members.

13.7.16 Friction
Consideration shall be given to bending stresses arising from pin, journal, trunnion, and other friction.

13.7.17 Machinery supports
All structural parts supporting machinery shall be of ample strength and rigidity and be designed to minimize vibration.

13.7.18 Vertical lift bridge towers
The lateral bracing of vertical lift bridge towers shall be designed for 2.5% of the total compression in the columns in addition to the specified wind loads.

13.7.19 Transitory loads
Transitory loads, e.g., operating impact, shall be included in the loading combinations only if their inclusion increases the total factored load effect.

13.8 Mechanical system design

13.8.1 General
Because mechanical system design in North America is based mainly on working stress design, the requirements of Clauses 13.8.2 to 13.8.20 are specified in terms of working stress even though other clauses of this Code are based on limit states design.

Most North American textbooks on mechanical system design use foot/pound units. Accordingly, where a value was originally expressed in foot/pound units, Clause 13.8 provides the foot/pound value in parentheses, accompanied by a soft-converted SI value.

13.8.2 Operating machinery
The design and construction of operating machinery shall be such that it requires minimum maintenance. All working parts shall be arranged so that they can be easily erected, adjusted, and dismantled. Fastenings shall be designed so that all machinery parts, after they are set, aligned, and adjusted, will be securely and rigidly connected.

All machinery shall have moving parts fitted with the guards or other safety devices required by applicable safety codes.

13.8.3 Power sources
Movable bridges should be operated by electric power. If two independent sources of electric power are procurable, both should be made available for bridge operation. Movable bridges may also be operated by internal combustion engines or human effort, depending on local conditions.

13.8.4 Prime mover

13.8.4.1 Emergency prime mover
A movable bridge may be provided with an emergency prime mover for operation of the bridge in case of failure of the prime mover or power supply normally used.

The emergency prime mover may be one of the following types:

(a) an electric motor that has controls independent of those provided for normal operation and a power source independent of that normally used for bridge operation, e.g., a generator driven by an internal combustion engine or a motor-generator set powered by a storage battery unit (its power characteristics may be different from those of the normal power source);
(b) an internal combustion engine;
(c) an air motor;
(d) hydraulics; or
(e) human effort.

The emergency prime mover may operate the bridge at a slower speed than the prime mover.

No emergency prime mover need be furnished where two independent and normally reliable sources of electric power with identical characteristics are made available and twin electric motors with independent controls are provided for each set of operating machinery. For twin motors, the capacity of each shall be such that, in the event of the failure of one, the other can still drive the bridge through the independent control in about the same time under the same conditions.

13.8.4.2 Locks, wedges, lights, signals, and traffic gates
Where two independent and normally reliable sources of electric power with identical characteristics are made available, no additional source of power need be provided for operating locks, wedges, lights, signals, and traffic gates.

If only one electric power service is available, an engine-generator set shall also be supplied. It shall be of sufficient capacity to provide power for the operation of the following electrically powered devices:

(a) span locks;
(b) traffic gates and their lights;
(c) traffic signal lights;
(d) pier lights;
(e) navigation lights on the span;
(f) vital indicating lights on the control desk;
(g) a sufficient number of lights in the control house, in the machinery room, and on stairways to enable the operator to move about safely; and
(h) release of electric brakes for main machinery.

The starting up of this engine-generator set may be automatic on the failure of the main electrical power supply. A transfer switch shall be provided to enable the operator to switch over from the main to the emergency power supply and vice versa. An indicating light with a suitable label shall be supplied and installed on the operator’s desk to indicate that electric power is being supplied by the emergency engine-generator set. A holding circuit shall also be supplied to bypass the automatic start-up feature, enabling the set to supply power until stopped by the operator pressing a "stop" push-button switch.

If driven by an electric or any other type of mechanical prime mover, the machinery for operating span locks, wedges, traffic gates, or any other devices essential to the operation of the bridge shall also be capable of being operated by human effort or other independent means if the normal prime mover fails.

13.8.5 Power requirements for main machinery

13.8.5.1 General

Power shall be provided, and machinery shall be designed to operate the bridge within the times specified in Clauses 13.8.5.2 and 13.8.5.3 and to hold the bridge in any position, under the conditions specified in Clauses 13.8.5.2 to 13.8.5.4.

13.8.5.2 Case A

In the normal time for opening, as follows:

(a) Swing bridges: against frictional resistances, inertia, and a horizontal wind pressure of 0.12 kPa on one arm and a wind pressure of 0.24 kPa on the other (the longer arm, if the arms are unequal) acting on the vertical surfaces specified in Clause 13.7.3.

(b) Bascule bridges (including rolling lift bridges): against frictional resistances, inertia, unbalance (if any), and a wind pressure of 0.12 kPa acting normal to the floor acting on the applicable surfaces specified in Clause 13.7.3.

(c) Vertical lift bridges: against frictional resistances, inertia, unbalance (if any), rope bending, and a wind pressure of 0.12 kPa acting normal to the floor, acting on the applicable surfaces specified in Clause 13.7.3. The wind load shall be considered to include the frictional resistances from span and counterweight guides caused by horizontal wind on the moving span.

13.8.5.3 Case B

In excess of the normal time for opening, as follows:

(a) Swing bridges: against frictional resistances, inertia, and a horizontal wind pressure of 0.24 kPa on one arm and a wind pressure of 0.48 kPa on the other (the longer arm, if the arms are unequal) plus an ice loading of 0.12 kPa on the floor, acting on the applicable surfaces specified in Clause 13.7.3.

(b) Bascule bridges (including rolling lift bridges): against frictional resistances, inertia, unbalance (if any), and a horizontal wind pressure of 0.24 kPa acting on the open bridge, plus an ice loading of 0.12 kPa on the floor, acting on the applicable surfaces specified in Clause 13.7.3.

(c) Vertical lift bridges: against frictional resistances, inertia, unbalance (if any), rope bending, and a wind pressure of 0.12 kPa plus an ice loading of 0.12 kPa, all acting normal to the floor, acting on the applicable surfaces specified in Clause 13.7.3. Frictional resistances from the span and counterweight guides shall be considered to be included in the loads specified in this Item.

13.8.5.4 Case C

Bridge held in any open position or operating position, as follows:

(a) Swing bridges: against a horizontal wind pressure of 0.96 kPa acting on one arm and a wind pressure of 1.44 kPa on the other (the longer arm, if the arms are unequal) acting on the applicable surfaces specified in Clause 13.7.3.

(b) Bascule bridges (including rolling lift bridges): against a horizontal wind pressure of 0.96 kPa acting on the open bridge on the applicable surfaces specified in Clause 13.7.3.

(c) Vertical lift bridges: against the wind and ice loads specified in Clause 13.8.5.3(c).

13.8.6 Wedges

The end-lift machinery of swing bridges shall be capable of lifting and supporting the span sufficiently to ensure a positive reaction under all conditions of live load and to remove any deflection due to temperature variations.

The centre-wedge machinery of swing bridges shall be capable of driving the wedges to a position where they will provide an adequate reaction for the live load.

13.8.7 Brakes

13.8.7.1 General

13.8.7.1.1 General
Movable bridge spans shall have at least one set of brakes.

Bridges that are manually operated only may be provided with only one set of brakes. This set shall consist of two units. One brake unit shall be considered equivalent to a motor brake and the other brake unit shall be proportioned to assist in dynamic braking for emergency stopping or to assist in static braking or “parking” the span in any position and shall be considered equivalent to a machinery brake.

Each drive unit on a power-operated movable bridge shall have at least two sets of brakes. One set shall be a motor brake in accordance with Clause 13.8.7.2 and the other shall be either a motor brake in accordance with Clause 13.8.7.2 or a machinery brake in accordance with Clause 13.8.7.3.

For the purposes of this Clause, a set of brakes may consist of one or more individual braking units.

Hydraulically operated bridges shall be provided with equivalent means for motion control.

13.8.7.1.2 Operating
The motor brakes for controlling the motion of the moving span shall have sufficient capacity to stop the span in 10 s under the loading conditions specified in Clause 13.8.5.2 for bridge operation in the normal time for opening or closing.

13.8.7.1.3 Holding
The braking systems shall also be capable of holding the span against movement in any open position under the loads specified in Clause 13.8.5.4.

13.8.7.1.4 Gradual application
Brakes, whether electrically, mechanically, hydraulically, or manually operated, shall be designed so that the retarding torque is applied gradually and is consistent with the deceleration time assumed for design in order to minimize shock loading.

13.8.7.1.5 Sequencing
When two sets of brakes are used, they shall be sequenced so that under normal operation they cannot be applied simultaneously.

13.8.7.1.6 Frictional assistance
In calculating the necessary brake capacity, frictional resistances that assist the brake may be included. Coefficients of friction that are 40% of those related to motion may be used for this condition.

13.8.7.2 Motor brakes
Motor brakes shall be provided for all movable bridges.

Where only one set of brakes is fitted, the motor brakes shall be capable of controlling the span for both the operating and the holding conditions.

Motor brakes shall be operated either electrically or mechanically. On electric motor installations, they should be electrically operated and mounted on the motor shaft. On internal combustion engine and manually operated installations, they shall be mounted as near to the high-speed shaft as practicable.

13.8.7.3 Machinery brakes
When machinery brakes are supplied, the motor brakes shall have sufficient capacity to stop the span in 10 s and the machinery brakes shall have a capacity, as measured at the shafts of the motor brakes, equal to 50% of that of the motor brakes. The combined capacity of the motor and machinery brakes shall be sufficient to hold the span under the conditions specified in Clause 13.8.5.

The machinery brakes shall normally be held in release during the entire operating cycle but shall be capable of being applied in an emergency at the discretion of the operator. They shall be designed to be held in release indefinitely.

The machinery brakes shall be mounted as near as practicable to the operating ropes or main pinion.

13.8.7.4 Brakes for emergency power

When emergency power by means of an internal combustion engine is used, a manually operated brake that is capable of being applied by the operator from the point at which the engine is being operated shall be provided. Brakes shall not be required for emergency manual operation.

13.8.7.5 Brakes for locks and wedge motors

Span lock and wedge motors shall each have one electrically operated brake.

13.8.8 Frictional resistance

13.8.8.1 Machinery

The frictional resistances of the moving span and its machinery parts shall be determined using the coefficients specified in Tables 13.7 and 13.8.

Table 13.7
Coefficients of friction
(See Clause 13.8.8.1.)

	Coefficient of friction	
	For starting	**For moving**
For trunnion friction, plain bearings		
Less than one complete revolution	0.18*	0.12*
More than one complete revolution	0.13*	0.09*
For trunnion friction, anti-friction bearings	0.004	0.003
For friction on centre discs	0.15	0.10
For collar friction at ends of conical rollers	0.15	0.10
For rolling friction		
Bridges rolling on segmental girders	0.009	0.006
Rollers with flanges	0.009	0.006
Rollers without flanges		
r measured in millimetres	$0.04/\sqrt{r}$	$0.04/\sqrt{r}$
r measured in inches	$0.08/\sqrt{r}$	$0.08/\sqrt{r}$
For sliding surfaces, intermittently lubricated (e.g., span guides of vertical lift bridges)	0.12	0.08

For manually operated bridges, this coefficient shall be increased by 25%. For proprietary bearing materials, the coefficients of friction shall be as specified by the manufacturer.

Note: *For wire rope bending through a 180° wrap, the loss per sheave is the direct tension multiplied by $0.3(d_r/D)$ for starting and moving.*

Legend:

r = radius of roller, mm (in)
d_r = rope diameter, mm (in)
D = pitch diameter of sheaf or drum, mm (in)

Table 13.8
Machinery losses and efficiency coefficients
(See Clause 13.8.8.1.)

		Coefficient
Machinery losses	For journal friction, plain bearings	0.05
	For journal friction, anti-friction bearings	0.01
	For friction at thrust collars*	0.10
	Screw gearing, bronze on steel	0.10
Efficiency coefficients	Journal friction included, for efficiency of any pair of gears	
	Plain bearings	
	Spur gears	0.95
	Bevel gears, collar friction included	0.87
	Anti-friction bearings	
	Spur gears	0.98
	Bevel gears, collar friction included	0.90
	Worm gearing, collar friction not included	$Np/(Np + R)$

**Where anti-friction thrust collars are used, the thrust bearing friction may be neglected.*

Legend:
N = number of threads of lead of worm
p = circular pitch of teeth on wheel
R = radius of worm, mm (in)

13.8.8.2 Locks and Wedges
For sliding span locks and end and centre wedges, the coefficients of friction specified in Table 13.9 shall be used for steel on bronze.

Table 13.9
Coefficients of friction for sliding span locks and end and centre wedges
(See Clause 13.8.8.2.)

	For starting	For moving
Top surfaces	0.15	0.10
Bottom surfaces	0.20	0.15

13.8.9 Torque

13.8.9.1 Torque at prime mover for main machinery
The sum of all resistances specified in Clause 13.8.8, with the addition of the machinery resistances, shall be reduced to a starting, accelerating, and running torque on the prime mover (referred to as “bridge torque” in Clause 13.8.9).

13.8.9.2 Starting torque
In calculating the bridge torque for starting conditions, the torque required to overcome inertia need not be included except for swing spans.

13.8.9.3 Time for determining torque required for acceleration

A period of 10 to 15 s shall be allowed for determining the torque required for acceleration.

13.8.9.4 Torque at prime mover for locks and wedges

For span lock and wedge machinery, the sum of all resistances to be overcome shall be reduced to a single equivalent torque at the prime mover.

13.8.9.5 Torque of prime mover for main machinery

13.8.9.5.1 Electric motors

Where electric motors are used as prime movers, they shall be capable of developing the following minimum torques at voltage within ±10% of normal voltage, starting cold, for the loading conditions and time intervals specified in Clause 13.8.5:

(a) Single-motor prime mover:
   - (i) For operation against the loading conditions specified in Clause 13.8.5.2, the bridge torque for starting or accelerating, whichever is larger, shall not be more than 125% of the rated full-load torque of the motor.
   - (ii) For operation against the loading conditions specified in Clause 13.8.5.3, the bridge torque for running after acceleration shall be not more than 125% of the rated full-load torque of the motor or more than 180% of the rated full-load torque for starting or accelerating, whichever is larger.

(b) Twin-motor prime mover:
   - (i) Where twin motors are used for joint operation, the two motors together shall meet the requirements specified for single-motor installations in Item (a).
   - (ii) Where twin motors are used for alternate operations, each motor shall meet the requirements specified for single-motor installations in Item (a).

13.8.9.5.2 Internal combustion engines

The rated engine torque (i.e., the torque measured at the flywheel, at the speed to be used for operation, with the radiator, fan, housings, and all other power-consuming accessories in place), shall be not more than 85% of the manufacturer's torque rating of the stripped engine.

Where engines are used as the prime mover, they shall be capable of developing minimum rated engine torques that exceed the maximum bridge torques by the following percentages:

(a) 33% for engines of four or more cylinders; and
(b) 50% for engines of three or fewer cylinders.

13.8.9.5.3 Worker power

The gear ratio shall be such that the number of persons assumed to be available for bridge operation is capable of developing the required maximum bridge torque.

13.8.9.6 Torque for lock and wedge machinery

Where span locks and wedges are operated by electric motors, the torque for starting or running shall not be more than 150% of the full-load torque of the motor, but in no case shall a motor of less than 1.5 kW (2 hp) be used.

Where span locks and wedges are operated by human effort, the gear ratio shall be such that the number of persons available for bridge operation shall be capable of delivering a torque equal to the maximum torque.

Where span locks and wedges are operated by hydraulic systems, the hydraulic systems shall be capable of providing 150% of the maximum torque or an equivalent force at the normal operating pressure.

13.8.10 Application of worker power

Worker power may be applied by capstans, cranks, hand chains, or levers, assuming that each worker employed thereon shall do work continuously as follows:

(a) Push 0.135 kN (30 lb) on a capstan handle while walking at a rate of 60 m (200 ft) per minute, or push 0.175 kN (40 lb) on a capstan handle while walking at a rate of 50 m (160 ft) per minute. The force shall be assumed to be applied 250 mm (10 in) from the end of the capstan handle, and the spacing of additional worker on the same handle shall be assumed to be 600 mm.
(b) Turn a crank having a radius of 375 mm (15 in) by exerting a force of 0.135 kN (30 lb) at a rate of 15 revolutions per minute.
(c) Exert a pull of 0.260 kN (60 lb) on a hand chain at a rate of 20 m (70 ft) per minute.
(d) Apply a force of 0.440 kN (100 lb) on the extreme end of a brake lever or a force of 0.590 kN (130 lb) on a foot pedal.

For starting conditions, it shall be assumed that a worker will, for a short time, exert twice the forces specified in Items (a) to (c).

13.8.11 Machinery loads

All machinery parts whose failure could interfere with the operation of the bridge or impair its safety shall be proportioned for the following loads, as shall the connections of such parts, the members to which such parts can be attached, and any other members affected by such parts:

(a) machinery driven by electric motors shall be designed for 150% of the rated full-load torque of the motor or motors at normal unit stresses;
(b) machinery driven by internal combustion engines shall be designed for 100% of the rated engine torque at normal unit stresses;
(c) machinery operated by worker power or machinery parts under the action of manually operated brakes shall be designed for 133% of the torque specified under Clause 13.8.10 at normal unit stresses; and
(d) machinery operated by hydraulic systems shall be designed for 100% of the maximum hydraulic system relief valve pressure.

For the wind and ice loads specified in Clause 13.8.5.4, the allowable unit stresses specified in Clause 13.8.12 may be increased by 50%.

13.8.12 Allowable stresses for machinery and allowable hydraulic pressures

13.8.12.1 Machinery materials

The maximum allowable stresses specified in Tables 13.10 and 13.11 shall be used for the design of machinery and those parts of the structure directly affected by vibrational or shock loads from the machinery, e.g., machinery supports.

Where materials of different strengths are in contact, the fixed bearing and shear values of the weaker material shall govern.

For rotating parts and frames, pedestals, and other units that support rotating parts, the calculated stresses shall be multiplied by an impact factor K, as follows:

(a) for trunnions and counterweight sheaves, $K = 1.0$; and
(b) for other rotating parts, $K = 1.0 + 0.03n^{0.5}$, where n is the number of revolutions per minute of the rotating part.

All of the stresses specified in Tables 13.10 and 13.11 include allowances for reversal, stress concentration factors up to 1.4, keyways of normal proportions, and good design details.

Table 13.10
Maximum allowable stresses in trunnions, MPa (psi)

(See Clause 13.8.12.1 and Table 13.12.)

	Rotation more than 180°	Rotation 90° or less	Fixed trunnions
Forged carbon steel, ASTM A 668/A 668M, Class D	69 (10 000)	103 (15 000)	117 (17 000)
Forged alloy steel, ASTM A 668/A 668M, Class G	69 (10 000)	138 (20 000)	152 (22 000)

Table 13.11
Maximum allowable stresses for machinery parts other than trunnions, MPa (psi)

(See Clauses 13.8.12.1, 13.8.16.1, and 13.8.17.4.3 and Table 13.12.)

	Specification					
Material	CSA	ASTM	Tension	Compression	Fixed bearing	Shear
Structural steel	G40.21, Grade 300W or 300WT	A 36M	83	83 – 0.38l/r	110	41
	G40.21, Grade 300W or 300WT	A 36	(12 000)	(12 000 – 55l/r)	(16 000)	(6000)
Forged carbon steel (except keys)		A 668M, Class D A 668, Class D	103 (15 000)	103 – 0.45l/r (15 000 – 65l/r)	124 (18 000)	52 (7500)
Forged carbon steel keys		A 668M, Class D A 668, Class D	— —	— —	103 (15 000)	52 (7500)
Forged alloy steel		A 668M, Class G A 668, Class G	110 (16 000)	110 – 0.48l/r (16 000 – 70l/r)	145 (21 000)	55 (8000)
Cast steel		A 27M, Grade 485-250 A 27, Grade 70-36	62 (9000)	69 – 0.31l/r (10 000 – 45l/r)	90 (13 000)	34 (5000)
		A 148M, Grade 620-415 A 148, Grade 90-60	103 (15 000)	103 – 0.45l/r (15 000 – 65l/r)	145 (21 000)	55 (8000)
Cast iron		A 48M, Class 200 A 48, Class 30	14 (2000)	69* (10 000)*	— —	— —
Bronze		B 22, Alloy 905	48 (7000)	48 (7000)	— —	— —
Hot-rolled steel bars		A 675M, Grade 515 A 675, Grade 75	83 (12 000)	83 – 0.38l/r (12 000 – 55l/r)	110 (16 000)	41 (6000)

**For struts whose l/r is 20 or less.*

13.8.12.2 Hydraulic systems and components

13.8.12.2.1 Allowable system pressures

The hydraulic system shall be designed and the hydraulic components proportioned in such a manner that the maximum system pressures shall not exceed the following, except as approved in writing by the Engineer:
(a) normal operation: 6.9 MPa (1000 psi);
(b) operation against maximum specified wind loads: 13.8 MPa (2000 psi); and
(c) holding against maximum specified wind loads: 20.7 MPa (3000 psi).

Normal operation shall be defined as operation against the Case A loads specified in Clause 13.8.5.2. Operation against maximum specified loads shall be defined as operation against the Case B loads specified in Clause 13.8.5.3. Holding against maximum specified wind loads shall be defined as holding the movable span against the Case C loads specified in Clause 13.8.5.4.

13.8.12.2.2 Pressure ratings for hydraulic components

13.8.12.2.2.1

The minimum working pressure ratings for hydraulic components shall be as follows, except as approved in writing by the Engineer:
(a) pipes, tubing, and their fittings: 20.7 MPa (3000 psi); and
(b) flexible hose and hose fittings:
  (i) for pressure lines: 34.5 MPa (5000 psi);
  (ii) for drain lines: 13.8 MPa (2000 psi); and
  (iii) for cylinders, pumps, valves, and all other components: 20.7 MPa (3000 psi).

13.8.12.2.2.2

The working pressure rating shall be defined as the maximum allowable continuous operating pressure for the component. For pipes, tubing, flexible hose, and fittings, the working pressure rating shall be the burst pressure rating divided by a minimum safety factor of 4. For cylinders, the working pressure rating shall be equal to the National Fluid Power Association theoretical static failure pressure rating required by Article 6.5.37.11 of Chapter 15 of the AREMA *Manual for Railway Engineering,* divided by a minimum safety factor of 3.33. For pumps, valves, and other components, the working pressure rating shall be the maximum allowable peak (intermittent) pressure rating divided by a minimum safety factor of 1.5.

13.8.12.2.2.3

The minimum safety factors specified in Clause 13.8.12.2.2.2 shall apply to systems with light-to-moderate operating shock loads during operation, resulting in short-duration peak system pressures not greater than twice the allowable maximum operating pressure against Clause 13.8.5.3 Case B loads or Clause 13.8.5.4 Case C loads, whichever are greater. For systems with higher shock load pressures, the safety factors shall be increased proportionally.

13.8.13 Bearing pressures (moving surfaces)

13.8.13.1 Maximum bearing pressures

The maximum bearing pressures specified in Table 13.12 shall be used in proportioning rotating and sliding surfaces. Bearing pressures greater than the maximum values specified in Table 13.12 may be used where the maximum loading occurs only during a small part of the motion cycle or in other cases deemed appropriate, provided that special precautions are taken with respect to surface finish and lubrication. The greater bearing pressures shall be subject to the approval of the Engineer.

13.8.13.2 Determination of bearing pressures

For the slow-moving parts specified in section A of Table 13.12, bearing pressures shall be calculated on the net projected area, i.e., after deducting the area of oil grooves, etc. For the higher-speed moving and rotating parts specified in sections B, C, and D of Table 13.12, bearing pressures shall be calculated on the gross projected area.

Table 13.12
Maximum bearing pressures
(See Clauses 13.8.13.1, 13.8.13.2, and 13.8.17.4.3.)

Condition	Parts	Material*	Maximum bearing pressure, MPa (psi)
A. Motion: speeds 15 m/min (50 ft/min) or less	Pivots for swing bridges	Hardened steel on ASTM B 22 Alloy 911 bronze Hardened steel on ASTM B 22 Alloy 913 bronze	17 (2500) 21 (3000)
	Trunnion bearings of bascules and counterweight sheave bearings of vertical lifts	Rolled or forged steel on ASTM B 22 Alloy 911 bronze	For loads in motion: 10 (1500) For loads at rest: 14 (2000)
	Wedges	Cast steel on ASTM B 22 Alloy 911 bronze Cast steel on ASTM B 22 Alloy 913 bronze	8 (1200) 10 (1500)
B. Motion: speeds over 15 m/min (50 ft/min) but less than 30 m/min (100 ft/min)	Bearings for main pinion shafts and other heavily loaded shafts	Rolled or forged steel on ASTM B 22 Alloy 937 bronze	7 (1000)
	Other bearings	Steel journals on babbitt Steel journals on ASTM B 22 Alloy 937 bronze	2.8 (400) 4 (600)
	Step bearings for vertical shafts	Hardened steel shaft end on ASTM B 22 Alloy 937 bronze ASTM B 22 Alloy 911 bronze	 4 (600) 8 (1200)
	Thrust collars	Rolled or forged steel on ASTM B 22 Alloy 937 bronze	1.4 (200)
	Acme screws that transmit motion	Rolled or forged steel on ASTM B 22 Alloy 905 bronze	10 (1500)
C. Motion: speeds of 30 m/min (100 ft/min) and more†	Journals	Rolled or forged steel on bronze	43.8/*nd* (250 000/*nd*)
	Step bearings	Hardened steel on bronze	10.5/*nd* (60 000/*nd*)
	Thrust collars	Rolled or forged steel on bronze	8.8/*nd* (50 000/*nd*)
	Acme screws that transmit motion	Rolled or forged steel on bronze	38.5/*nd* (220 000/*nd*)
D. Alternating motion	Crank pins and similar parts with alternating application and release of pressure	—	‡

**The materials specified in this column shall comply with the Standards specified in Tables 13.10 and 13.11. Alternative bearing materials may be considered. The maximum bearing pressures of such alternative materials shall conform to the manufacturer's recommendations.*
†*n = number of revolutions per minute and d = diameter of journal or step bearing or mean diameter of collar or screw, mm (in). To prevent heating and seizing at higher speeds, the pressures derived from the formulas in section C of this Table shall not be exceeded and shall never be greater than 75% of the values permitted by section B of this Table.*
‡*The limiting bearing pressure determined in accordance with the formula for journals in section C of this Table may be doubled.*

13.8.14 Line-bearing pressure

13.8.14.1 Rollers or rockers

The maximum line-bearing pressure in newtons per millimetre (pounds per inch) on rollers or rockers shall be as follows:

(a) For diameters less than 635 mm (25 in):

$$\frac{(p-90)2.76d}{138} \quad \left(\frac{(p-13\ 000)400d}{20\ 000}\right)$$

(b) For diameters of 635 to 3200 mm (25 to 125 in):

$$\frac{(p-90)2.22\sqrt{d}}{138} \quad \left(\frac{(p-13\ 000)2000\sqrt{d}}{20\ 000}\right)$$

where

p = the least of the values of the yield strength of the material in the roller, rocker, roller bed, or track, MPa (psi)

d = diameter of roller or rocker, mm (in)

Where the rollers could be subjected to live load with the bridge closed, e.g., on a rim-bearing swing bridge, or for balance wheels subjected to wind loads, the maximum bearing pressures may be increased by 50%.

13.8.14.2 Segmental girders

The maximum line-bearing pressure in newtons per millimetre (pounds per inch) on the treads of segmental girders rolling on flat surfaces for diameters of 3 m (10 ft) or more shall be as follows:

$$\frac{(p-90)(2.10+0.55d)}{138} \quad \left(\frac{(p-13\ 000)(12\ 000+80d)}{20\ 000}\right)$$

where

p = the lesser of the values of the yield strength of the steel in the segmental girder tread or track, MPa (psi)

d = diameter of segmental girder, mm (in)

Those portions of the segmental girder and the track or tread that are in contact when the bridge is closed shall be designed for the sum of the dead load and live load (including dynamic load effects). Under this loading, the maximum line-bearing pressure may be increased by 50%.

13.8.15 Design of wire ropes

13.8.15.1 Bending formula

For counterweight ropes, the maximum stress from the combined effect of direct loads and bending shall not exceed 0.22 of the ultimate stress of the rope specified in Table 13.13. The stress from the direct load shall not exceed 0.125 of the ultimate strength of the rope specified in Table 13.13. For operating ropes, the limits shall be 0.30 and 0.16, respectively.

Where ropes are bent over sheaves or drums, the extreme fibre stress, f, in megapascals (pounds per square inch) shall be calculated as follows:

$$f = 0.8\frac{Ed_w \cos^2 L \cos^2 B}{D}$$

where

E = modulus of elasticity of wire
= 197 000 MPa (28 500 000 psi)

d_w = diameter of largest individual wire, mm (in)

L = angle of helical wire with axis of strand, radians (degrees)

B = angle of helical strand with axis of rope, radians (degrees)

D = pitch diameter of sheave or drum, mm (in)

Table 13.13
Ultimate stress and ultimate strength of steel wire rope of 6 × 19 classification and 6 × 25 filler construction
(See Clauses 13.6.5.5.8, 13.6.5.5.9, 13.6.5.5.15, and 13.8.15.1.)

Grade 1770				Grade 110/120			
Rope diameter, d, mm	Approx. area of section (= $0.4d^2$), mm^2	Ultimate stress, MPa	Ultimate strength of entire rope, kN	Rope diameter, d, in	Approx. area of section (= $0.4d^2$), in^2	Ultimate stress, psi	Ultimate strength of entire rope, lb
12	57.6	1460	84.0	1/2	0.100	212 000	21 000
14	78.4	1460	114.0	5/8	0.156	212 000	33 000
16	102.4	1460	149.0	3/4	0.225	204 000	46 000
18	129.6	1460	189.0	7/8	0.306	209 000	64 000
20	160.0	1460	234.0	1	0.400	209 000	83 000
22	193.6	1460	283.0	1 1/8	0.506	209 000	106 000
24	230.4	1460	336.0	1 1/4	0.625	210 000	131 000
26	270.4	1460	395.0	1 3/8	0.756	214 000	162 000
28	313.6	1460	458.0	1 1/2	0.900	213 000	192 000
32	409.6	1460	600.0	1 5/8	1.056	214 000	226 000
36	518.4	1460	755.0	1 3/4	1.225	212 000	260 000
40	640.0	1460	935.0	1 7/8	1.406	216 000	304 000
44	774.4	1460	1130.0	2	1.600	211 000	338 000
48	921.6	1460	1350.0	2 1/8	1.806	208 000	376 000
52	1081.6	1460	1580.0	2 1/4	2.025	207 000	420 000
56	1254.4	1460	1830.0	2 3/8	2.256	211 000	476 000
60	1440.0	1460	2100.0	2 1/2	2.500	218 000	520 000
64	1638.4	1460	2390.0				

13.8.15.2 Sheaves and drums — Minimum diameters

The minimum pitch diameters of sheaves and drums shall be as follows:

(a) counterweight sheaves: not less than 72 times the rope diameter;
(b) operating rope sheaves and drums: not less than 45 times the rope diameter; and
(c) auxiliary counterweight sheaves: not less than 60 times the rope diameter.

13.8.15.3 Short arc contact

Where operating ropes have an arc of contact with a deflector sheave of 45° or less, a minimum sheave diameter of 20 times the rope diameter may be used.

13.8.15.4 Limiting rope sizes

The diameter of counterweight ropes shall normally be not less than 22 mm (0.875 in) and not greater than 64 mm (2.5 in). The use of diameters outside of this range shall require approval by the Engineer.

The diameter of operating ropes shall be not less than 16 mm (0.625 in).

13.8.15.5 Limiting rope deviations

For counterweight ropes, the transverse deviation from a vertical plane through the centre of the sheave groove shall not exceed 1 in 40. The longitudinal deviation measured from a vertical plane tangent to the pitch diameter of the sheave shall not exceed 1 in 30.

For operating ropes, the transverse deviation from a vertical or horizontal plane through the centre of the sheave or drum groove shall not exceed 1 in 30.

The deviations specified in this Clause shall not be exceeded for any position of the moving span.

13.8.15.6 Initial tension of operating ropes

The initial tension in each operating rope, I, measured in kilonewtons (pounds), shall be calculated as follows:

$$I = (T_i/2) + P$$

where

T_i = maximum operating tension in the rope (including unbalance, if any) kN (lb)

P = minimum tension in the slack rope, kN (lb)

P should be not less than $0.1\,T_i$.

13.8.16 Shafting

13.8.16.1 General

Shafting shall be designed for combined bending and torsion loads in accordance with the following formulas:

$$S = 16\frac{K}{\pi d^3}\sqrt{M^2 + T^2}$$

$$f = 16\frac{K}{\pi d^3}\left[M + \sqrt{M^2 + T^2}\right]$$

where

S = shear stress, MPa (psi)

f = extreme fibre stress in tension or compression, MPa (psi)

K = the applicable impact factor specified in Clause 13.8.12.1

M = simple bending moment calculated for the distance centre-to-centre of bearings, N•mm (in•lb)

T = simple torsional moment, N•mm (in•lb)

d = diameter of shaft at the section considered, mm (in)

Bending due to the tooth load of bevel gears shall be calculated as for a spur gear having a pitch diameter equal to the mean pitch diameter of the bevel gear.

Shafting may be made of carbon or alloy steel forgings, or of structural steel bars, with the maximum allowable stresses specified in Table 13.11. Cold-rolled steel bars shall not be used for shafting of the main or auxiliary operating machinery. All gears or other components attached to shafts shall be located adjacent to bearings.

The allowable stresses specified in Table 13.11 include the effects of keyways with a width of not more than 0.25 and a depth not more than 0.125 of the shaft diameter. In the absence of keyways, higher stresses may be used.

For shafts supporting their own mass only, the unsupported length of the shaft between bearings shall not exceed $235\sqrt[3]{d^2}$ mm or $80\sqrt[3]{d^2}$ in, where d is the diameter of the shaft in millimetres (inches).

13.8.16.2 Speed of line shafts
Line shafts connecting the machinery at the centre of the bridge with machinery at the ends shall be designed to run at a high speed, the speed reduction being made in the machinery at the ends.

13.8.16.3 Minimum size of shafts
Shafts transmitting power for the operation of the bridge, and shafts 1200 mm (48 in) or more in length forming part of the operating machinery of bridge locks, shall be not less than 65 mm (2.5 in) in diameter.

13.8.16.4 Turning and balancing
Shafting shall be turned as required for journals, gear seats, etc. When the speed exceeds 400 rpm, the shaft shall be turned full length. All gear shaft assemblies running over 600 rpm shall be dynamically balanced. Cold-rolled shafting need not be turned at journals.

13.8.16.5 Longitudinal movement
Effective means for preventing longitudinal movement of shafting shall be provided (e.g., a split collar clamped in a cut groove or a substantial pin or bolt passing through a collar or through the hub of an attached part). Collars with set screws may be used only where there is no definite longitudinal force to be resisted.

13.8.16.6 Angular deflection
Shafts shall be proportioned so that the angular deflection in degrees per metre (degrees per foot) of length under maximum loads will not exceed the following limits:
(a) for all shafts: $50/d$ ($0.6/d$), where d is the shaft diameter in millimetres (inches); and
(b) for more rigid drives where less spring is desirable, e.g., shafts driving end-lifting devices: 0.26 (0.08).
When d exceeds 190 mm (7.5 in), Item (a) shall apply.

13.8.16.7 Alignment shafts and bearings
Provision shall be made for field adjustment of the alignment of all shafts or bearings that cannot be assembled and fitted in the shop.

13.8.16.8 Change of section
All shafts and trunnions shall have generous fillets where changes in section occur. Suitable stress concentration factors shall be used for unusual configurations.

13.8.16.9 Trunnions
All trunnions more than 200 mm (8 in) in diameter shall have a hole whose diameter is about 0.2 times the outside diameter bored lengthwise through the centre.

13.8.17 Machinery fabrication and installation

13.8.17.1 Shaft keys

13.8.17.1.1 General
All parts transmitting torsion to shafting shall be fastened thereto by keys. Keys shall not be wider than 0.25 times the diameter of the shaft. Their thickness shall be not more than 0.25 times the diameter of the shaft and their dimensions shall be such that the allowable stresses in shear and bearing will not be exceeded.

If the keyed parts are also connected by a shrink or press fit, 25% of the transmitted torque may be assumed to be absorbed by this fit, and the keys shall be designed to take the remaining torque at the normal unit stresses specified in Clause 13.8.13.

Keys shall be parallel faced, square or rectangular, and fitted into keyways cut into the hub and shaft. The keyway in the shaft and the key should have semi-circular ends. Keyways shall have filleted corners in accordance with ANSI B17.1.

13.8.17.1.2 Multiple keys
When two keys are used to connect a rotating part to a shaft, they shall be placed at an angle of 120° to each other, except in cases where the keyed part is required to slide along the shaft, in which case two parallel keys shall be used, placed at an angle of 180° to each other. Each key shall be designed to carry 60% of the transmitted torque.

13.8.17.1.3 Trunnion keys
For trunnions and similar parts that are designed for bending and bearing, the keys and keyways shall be proportioned simply to hold the trunnion from rotating. The force tending to cause rotation shall be taken as 0.2 times the load on the trunnion, acting at the circumference of the trunnion.

13.8.17.1.4 Locking
Where necessary, keys shall be held by set screws or other effective means. In vertical shafts, bands clamped about the shaft, or other key-retaining devices, shall be placed below the key.

13.8.17.2 Shaft couplings
All couplings shall be made of cast or forged steel. All couplings shall be standard manufactured flexible couplings and be placed close to bearings.

Couplings between machinery components shall be gear type and provide angular or angular and offset misalignment capabilities as necessary.

Couplings between prime movers and machinery components shall be flexible couplings transmitting the torque through metal parts and providing for both misalignment and shock.

Couplings with non-metallic parts shall be used only for secondary mechanisms and shall function even after failure of the non-metallic elements.

Rigid couplings may be used where self-aligning couplings are not required.

All couplings shall be keyed to their shafts. Bolts in coupling housings shall be shrouded.

13.8.17.3 Bearings

13.8.17.3.1 Alignment
When final alignment cannot be performed in the shop, supports for bearings shall provide for field alignment.

13.8.17.3.2 Material
Steel shall be used for the following parts of all bearings unless otherwise specified by the Engineer:
(a) the cap and base of plain bearings; and
(b) the housing of anti-friction bearings.

Cast iron housings may be used for light-duty bearings.

13.8.17.3.3 Bevel gear bearings
The bases of the bearings for mating bevel gears shall be made in one solid piece. The hubs of bevel gears, worms, or worm gears shall bear against adjacent shaft bearings through suitable thrust collars or anti-friction thrust bearings.

13.8.17.4 Plain bearings

13.8.17.4.1 Proportions
The length of a bearing shall not be less than the diameter of the journal and should be 1.5 times that diameter except for counterweight sheaves, where the length shall not be more than 1.2 times the diameter.

13.8.17.4.2 Adjustment
On all bearings, adjustment for height shall be provided in order to allow for wear. Adjustment of caps by means of laminated brass liners shall be provided.

13.8.17.4.3 Bushings
Bearings shall have bronze bushings unless otherwise approved by the Engineer. Alloys for various types of service shall be in accordance with Table 13.12.

The bronze linings shall be effectively locked against rotation. The force tending to cause rotation shall be taken as 0.06 times the load on the bearing, acting at the outer radius of the lining.

The inside corners of the bushings shall be rounded or chamfered, except for a distance of 10 mm from any joint.

13.8.17.4.4 Journal bearings
Journal bearings shall have split housings. The cap shall be recessed into the base and fastened by bolts, with the heads recessed into the base. Nuts shall be hexagonal and lock nuts shall be provided.

Both heads and nuts shall bear on finished bosses or spot-faced seats. Bearings shall be designed to facilitate cleaning.

13.8.17.4.5 Step bearings
The bearing ends of vertical shafts running in step bearings shall be of hardened steel and shall run on bronze discs.

13.8.17.5 Anti-friction bearings

13.8.17.5.1 General
Anti-friction bearings may be used for applications where good commercial practice would indicate their suitability and economy.

Anti-friction bearings shall be sized for an American Bearing Manufacturers Association B-10 life of 40 000 h under design running conditions.

Anti-friction bearings mounted in pillow blocks shall be self-aligning and shall have seals suitable for the conditions under which they operate. Housings shall be steel and may be split on the centreline. Bases shall be solid and shall be drilled for mounting bolts at assembly. Positive alignment shall be provided between the cap and the base on split housings. The alignment system shall be adequate for the design bearing loads.

13.8.17.5.2 Thrust bearings
The bearing ends of vertical shafts shall run in ball or roller thrust bearings or in radial bearings of types capable of carrying both radial and thrust loads.

13.8.17.5.3 Trunnion bearings
Where roller bearings are used to support the trunnions of counterweight sheaves of vertical lift bridges and similar shafts carrying heavy loads, they shall receive special design consideration in establishing the most suitable size and type. Only manufacturers who have experience in producing bearings for this type of service shall be considered.

13.8.17.6 Gearing

13.8.17.6.1 Material
Power-driven gears shall be made of steel.

13.8.17.6.2 Tooth type
Power-driven gears shall have straight spur teeth of full depth. They shall be of the involute type, with a pressure angle of 20°, and be machine cut.

13.8.17.6.3 Rack and pinion gearing
For rack and pinion gearing, stub teeth or special forms of teeth designed for greater strength may be used.

The circular pitch for rack and main pinion gearing shall not be less than 40 mm.

Pinions shall have at least 15 teeth for standard full-depth teeth. Main pinions shall have at least 17 teeth. The pitch line shall be inscribed on both ends of all cut teeth for racks, gears, and pinions.

The backs or sides of racks that bear on metal surfaces and surfaces in contact with them shall be finished.

13.8.17.6.4 Enclosed gear speed reducers
For enclosed gear speed reducers, the following shall apply:

(a) Speed reducers shall be models from one manufacturer unless otherwise approved by the Engineer. The reducers shall have the gear ratios, dimensions, construction details, and American Gear Manufacturers Association (AGMA) ratings as shown on the drawings. Ratings shall be based on a service factor of 1.0.

(b) The AGMA strength rating shall be based on a torque equal to 300% of full-load motor torque. Gears shall have helical or herringbone teeth, bearings shall be of the anti-friction type, and housings shall be welded steel plate or steel castings. The insides of the housings shall be sandblast-cleaned before assembly and protected from rusting. Exact ratios shall be furnished where specified.

(c) Each unit shall have a means for filling and draining the case, an inspection cover, and a dipstick and sight glass to show the oil level. Sight glasses shall be of rugged construction and protected against breakage. Drains shall have shut-off valves to minimize spillage. Each unit shall have a moisture trap breather.

(d) Lubrication of the gears and bearings shall be automatic when the unit is in operation.

(e) If a pressurized lubrication system is required for the reducer, a backup lubrication system shall be provided. The backup system shall operate whenever the reducer is operating.

(f) A remote sensor shall be provided to indicate when a pressurized lubrication system malfunction occurs.

(g) Reducers shall be manufactured in accordance with AGMA requirements and shall have nameplates indicating the rated horsepower, ratio, speed, service factor, and AGMA symbols.

(h) Reducer bases shall extend sufficiently past the body of the reducers to allow for mounting bolt hole reaming and bolt installation from above the unit.

(i) Inspection covers shall be sized and located to allow for inspection of all gears and bearings.

(j) Gearing shall conform to AGMA Quality No. 8 or higher.

(k) The reducer design calculations and shop drawings showing each gear box component shall be submitted to the Engineer before construction of the unit.

13.8.17.6.5 Details of design of teeth
The face width of cut spur gears shall not exceed three times the circular pitch. The face width of bevel gears shall not exceed 0.33 times the slant height of the pitch cone.

13.8.17.6.6 Permissible loads on gear teeth

In the design of spur, bevel, helical, and herringbone gears, the full load shall be taken as applied to one tooth. For spur gears, the permissible load on teeth shall be determined by the Lewis equations, including the velocity factors.

13.8.17.6.7 Permissible stresses in gear teeth

The permissible stresses for gear teeth shall be as specified in Table 13.14.

13.8.17.6.8 Teeth strength factors

Allowable stresses for racks and pinions and other gear sets not mounted in a common frame shall be reduced 20% from the values listed in Table 13.14.

13.8.17.6.9 Worm gearing

Except for the operation of wedges, span locks, and other secondary mechanisms, worm gearing shall not be used to transmit power unless approved by the Engineer.

13.8.17.7 Welded parts

13.8.17.7.1 Machinery supports

Machinery bases, support frames, chassis, platforms, seats, brackets, and similar items may be of welded construction.

13.8.17.7.2 Welded components

Primary machinery components, e.g., counterweight sheaves, rope drums, gears, gear cases, and bearing housings, may be of welded construction from structural steel.

Table 13.14
Permissible stresses in gear teeth
(See Clauses 13.8.17.6.7 and 13.8.17.6.8.)

Material	ASTM Specification	Permissible stress for cut teeth, MPa (psi)
Cast steel	A 27M, Grade 480-250 A 27, Grade 70-36	110 (16 000)
Forged carbon steel	A 668/A 668M, Class C A 668/A 668M, Class D	138 (20 000) 155 (22 500)
Forged alloy steel	A 668/A 668M, Class G and higher	60% of yield strength but not more than 33% of ultimate tensile strength
Cast bronze	B 22, Alloy 905	62 (9000)
Cast bronze (high strength)	B 22, Alloy 863	138 (20 000)

13.8.17.7.3 Design of welded connections

The design of welded connections shall be in accordance with Section 10. To allow for impact, the static design load shall be increased by 100%.

The stress range for welds on machinery components, bases, support frames, etc. that are subject to vibrational or shock loads shall not exceed the constant amplitude threshold stress range specified in Table 10.4 for the detail category involved.

Structures or components that are to be welded shall be designed so that distortion or residual stresses resulting from welding operations are minimized. In cases of complicated weldments requiring large deposits of weld metal, the welding procedure shall be clearly defined and carefully controlled in practice. When necessary, weldments shall be stress-relieved or peened.

Fracture control in accordance with Clause 10.23 shall be considered during material selection and structural design.

13.8.17.7.4 Welding

All welding shall be in accordance with Clause 10.24.

13.8.17.8 Bolts and nuts

Bolts and nuts shall comply with the following requirements:

(a) Bolts for connecting machinery parts to each other or to steel supporting members shall be one of the following types:
  (i) finished high-strength bolts;
  (ii) turned bolts, turned cap screws, and turned studs; and
  (iii) high-strength turned bolts, turned cap screws, and turned studs.

(b) Finished high-strength bolts shall meet the requirements of ASTM A 449. High-strength bolts shall have finished bodies and regular hexagonal heads. Holes for high-strength bolts shall be not more than 0.25 mm (0.01 in) larger than the actual diameter of individual bolts, and shall be drilled to match the tolerances for each bolt. The clearance shall be checked with 0.28 mm (0.011 in) wire. The hole shall be considered too large if the wire can be inserted into the hole together with the bolt.

(c) Turned bolts, turned cap screws, and turned studs shall have turned shanks and cut threads. Turned bolts shall have semi-finished, washer-faced, hexagonal heads and nuts. Turned cap screws shall have finished, washer-faced, hexagonal heads. All finished shanks of turned fasteners shall be 1.6 mm (0.063 in) larger in diameter than the diameter of the thread, which shall determine the head and nut dimensions. The shanks of all turned fasteners shall have Class LT1 fit in the finished holes in accordance with ASME B4.1. The material for the turned shank fasteners shall meet the requirements of ASTM F 568M, Class 4.6 (ASTM A 307, Grade A).

(d) High-strength turned bolts, turned cap screws, and turned stud details shall be as specified in Item (c), except that the material shall meet the requirements of ASTM A 449.

(e) Elements connected by bolts shall be drilled or reamed assembled to ensure accurate alignment of the hole and accurate fit over the entire length of the bolt within the specified limit.

(f) The dimensions of all bolt heads, nuts, castle nuts, and hexagonal head cap screws shall be in accordance with the applicable ASME Standard (B18.2 series).

(g) Heads and nuts for turned bolts, screws, and studs shall be heavy series.

(h) ASTM A 449 bolts shall be tightened to at least 70% of their required minimum tensile strength.

(i) The dimensions of socket-head cap screws, socket flathead cap screws, and socket-set screws shall be in accordance with ASME B18.3. The screws shall be made of heat-treated, cadmium-plated alloy steel and furnished with a self-locking nylon pellet embedded in the threaded section. Unless otherwise called for on the drawings or specified in this Section, set screws shall be of the headless safety type, shall have threads of the coarse thread series, and shall have cup points. Set screws shall not be used to transmit torsion or as fastenings or stops for any equipment that contributes to the stability or operation of the bridge.

(j) Threads for bolts, nuts, and cap screws shall be of the coarse thread series and shall have a Class 2 tolerance for bolts and nuts or a Class 2A tolerance for bolts and Class 2B tolerance for nuts in accordance with ASME B1.10.

(k) Bolt holes through unfinished surfaces shall be spotfaced for the head, nut, and washer, square with the axis of the hole.

(l) Unless otherwise called for on the drawings, all bolt holes in machinery parts or connecting such parts to the supporting steelwork shall be subdrilled at least 0.8 mm (0.031 in) smaller in diameter than the bolt diameter. The steelwork shall be subdrilled after the machinery is correctly and finally assembled and aligned, and then the holes shall be reamed for the proper fit with the bolts.

(m) Holes in shims and fills for machinery parts shall be reamed or drilled to the same tolerances as the connected parts at final assembly.
(n) Positive locks of a type approved by the Engineer shall be furnished for all nuts, except those of ASTM A 449 bolts. Double nuts shall be used for all connections requiring occasional opening or adjustment. If lock washers are used for securing, they shall be made of tempered steel and shall conform to Society of Automotive Engineers (SAE) regular dimensions. The material shall meet the SAE tests for temper and toughness.
(o) High-strength bolts shall be installed with a hardened plain washer in accordance with ASTM F 436 at each end.
(p) Wherever possible, high-strength bolts connecting machinery parts to structural parts or other machinery parts shall be inserted through the thinner element into the thicker element.
(q) Cotters shall conform to SAE standard dimensions and shall be made of half-round stainless steel wire meeting the requirements of ASTM A 276, Type 316.
(r) Anchor bolts connecting machinery parts to masonry shall be of ASTM F 568M, Class 4.6 (ASTM A 307, Grade A) material, hot-dipped galvanized in accordance with ASTM A 153/A 153M unless otherwise specified by the designer. Bolts shall be as shown on the structural drawings. Anchor bolts for new construction shall be cast-in-place and not drilled. The designer shall specify the material and loading requirements for the given design condition. When anchor bolts connect a mechanical component directly to the concrete, there shall be a filler material in the annular area between the bolt and the bolt hole in the machinery component. The filler material may be a non-shrink grout, babbitt metal, or zinc.
(s) Bolts and nuts shall be North American manufacture and shall be clearly marked with the manufacturer's designation unless otherwise Approved.

13.8.17.9 Wrenches

A set of wrenches to fit those bolts and nuts that
(a) are 30 mm or larger;
(b) are actually used on the machinery; and
(c) might require tightening, adjustment, or dismantling
shall be furnished by the supplier of the machinery.

13.8.17.10 Set screws

Set screws shall not be used for transmitting torsion loads. They may be used for holding light parts such as keys in place.

13.8.17.11 Dust covers

Dust covers shall be provided to protect sliding and rotating surfaces and prevent dirt from mixing with the lubricant.

13.8.17.12 Drain holes

Proper provision shall be made for draining at places where water is likely to collect.

13.8.17.13 Cams

Cams and similar devices shall not normally be used for transmitting power by line or point contact.

13.8.18 Lubrication

A combined diagram and chart covering all machinery parts that require lubrication, with recommendations or the type of lubricant to be used for each part and the frequency of lubrication, shall be included in the operating and maintenance manual. Non-fading prints of the diagram and chart shall be framed under glass near each machinery area.

Provision shall be made for effective lubrication of all sliding surfaces and of ball and roller bearings. Lubricating devices shall be easily accessible.

Grease grooves shall be machine cut in the bushings of the bearings. Grooves shall be straight for large bearings, number at least three, and spaced so that the entire surface will be swept by lubricant in one cycle of opening or closing the bridge. Grooves in the shape of a figure eight shall be acceptable for shafts making more than one revolution per opening or closing cycle. Grooves shall be smoothly transitioned into the bushings and shall be of such a size that an 8 mm diameter wire will lie wholly within the groove. Grooves shall have inlet and outlet ports to facilitate cleaning and purging.

A high-pressure system of lubrication shall be provided for journal bearings and sliding surfaces (where practicable). Where lubrication points are not readily reached, the fittings shall be made accessible by extension pipes. Grooves shall be provided where necessary for the proper distribution of the lubricant.

Disc bearings shall be provided with oil bath lubrication.

Where anti-friction gear cases are used, the gearing should be oil bath lubricated and the bearings splash lubricated. Where plain bearing gear cases are used, the gearing should be oil bath lubricated and the journals grease lubricated.

Where bearings are small, the unit-bearing pressures are low, and the motion is slow and intermittent, self-lubricating bushings may be used. These bushings shall be of a type that will not be injured by the application of oil and shall have protected oil holes for emergency use. Self-lubricating bushings shall not be used for the main machinery.

The sliding surfaces of span guides, locks, etc. shall be hand lubricated with a suitable grease.

Two hand-operated grease guns, including adapters, shall be provided as necessary to service all lubrication fittings.

For special devices, the manufacturer’s recommendations for lubrication shall be followed.

13.8.19 Power equipment

13.8.19.1 Tests of machines

Machines that are of the usual manufactured types, e.g., gasoline or diesel engines, electric motors, oil motors, pumps, and air compressors, shall be tested for the specified performance requirements to the satisfaction of the Engineer and shall be guaranteed by the supplier to fulfill these requirements for one year.

Note: *“Specified performance requirements” can mean requirements specified in this Code, by the Owner, or in the contract, as applicable.*

13.8.19.2 Brakes

13.8.19.2.1 General

Brakes shall be provided in accordance with Clause 13.8.7.

13.8.19.2.2 Electrically operated brakes

Motor brakes for the main motors shall be thruster- or motor-operated spring-set shoe brakes with a torque and a time rating that meets the load requirements of Clause 13.8.7.

Machinery brakes for the main machinery shall be thruster- or motor-operated spring-set shoe brakes that are continuously rated, with a torque and a time rating that meets the load requirements of Clause 13.8.7.

Brakes shall be electrically interlocked with the main motors to prevent motor operation if the brakes are applied.

Brakes shall be arranged for hand release. Interlocking switches shall be provided to prevent electric power operation when the brakes are in the hand-released position.

Enclosing covers that are weatherproof and easily removable shall be supplied if the brakes are not in a machinery enclosure.

Electrically operated brakes located on the moving leaf of a bascule bridge shall be spring set and function in any position of bridge rotation.

13.8.19.2.3 Mechanically operated brakes

13.8.19.2.3.1 General
Mechanically operated brakes should be of the shoe type. If the main function of these brakes is to stop and hold the moving span in any position, e.g., for machinery brake usage, they shall be arranged so that the brake is applied by a mass or spring and released by hand or foot.

13.8.19.2.3.2 Air brakes
Air brakes shall be controlled from the operator's house.

13.8.19.2.3.3 Hydraulic brakes
Hydraulic brakes shall be operated by a foot pedal in the operator's house.

13.8.19.2.3.4 Mechanically operated hand or foot brakes
If the brakes are to be mechanically operated by hand or foot, the operating lever in the operator's house shall be suitably connected to the brake mechanism at the brake wheel by levers and connecting rods.

13.8.19.2.3.5 Material for brake wheels
Brake wheels should be made from ductile cast iron or another material with characteristics suitable to the application. Ordinary cast iron shall not be used for brake wheels.

13.8.19.3 Internal combustion power equipment

13.8.19.3.1 Gasoline or diesel electric power
For bridge locations where adequate electric power is not available, electric generator sets driven by diesel or gasoline engines may be provided. Such power units shall be of the type currently used for commercial industrial service.

13.8.19.3.2 Gasoline or diesel engine power

13.8.19.3.2.1 General
Gasoline or diesel engines, where used, shall be of the industrial, automotive, or marine type; only substantial or heavy-duty models shall be used. The operational speed shall be limited to 1800 rpm and should be not more than 1400 rpm. Engines should have at least four cylinders, shall be equipped with a suitable speed governor, and shall be effectively water cooled by a radiator and fan. An exhaust pipe that discharges outside the engine room and is fitted with an industrial-type muffler and moisture trap shall be provided for each engine. The engines shall be tested by the manufacturer at its plant to prove that they will develop the specified torque rating.

13.8.19.3.2.2 Clutch and overload protection
A friction clutch of a design approved by the Engineer shall be provided between the engine and the driven machinery. It shall be capable of being gradually applied and shall be designed so that it will slip at a predetermined torque to protect either the engine or the driven machinery from overload damage at all times and under all conditions of operation. If it is not practicable to supply a clutch with these built-in safety features, an additional device of a design approved by the Engineer, e.g., a friction or hydraulic coupling, shall be installed between the engine and the driven machinery to provide the required overload protection.

13.8.19.3.2.3 Reversing gear

Engines shall be equipped with a reversing gear unit mounted on a common frame and shall be designed so that the bridge drive machinery can be run in either direction with the engine running continuously. This reversing gear shall be capable of transmitting the full torque of the engine in either direction.

13.8.19.3.2.4 Control board

A small control board for mounting the throttle and choke controls, ignition switch, starter button, oil and temperature gauge, and any other operational equipment shall be provided at the engine and should be mounted integrally with it.

13.8.19.3.2.5 Arrangement of controls

The controls for operating the bridge under internal combustion power shall be positioned so that one operator can conveniently and quickly perform, from one location, all functions necessary for the operation of the bridge, e.g., the starting and stopping of the engine and the release and resetting of electric brakes where emergency electric power is available. The operator shall also be able to operate any drive clutch, reversing clutch, foot brake, or other vital device and be able to see the engine control board and the span position indicator (if any).

13.8.19.3.2.6 Fuel tanks

Fuel tanks shall be made of corrosion-resistant metal. When gasoline is used for fuel, the fuel tanks shall be located outside of the machinery house and be protected from the direct rays of the sun. If the engine is the primary power unit, the fuel tanks shall have sufficient capacity for 30 d normal operation of the bridge.

If the engine is used for auxiliary power only, the fuel tanks shall have a minimum capacity of 76 L. Tanks shall be equipped with an automatic gauge to indicate the quantity of fuel on hand, a sump, and a drain cock. All pipes and fittings connecting the tanks to the engine shall be made of copper or brass.

13.8.19.3.2.7 Spare parts

Spare parts for the ignition system and other necessary spares shall be furnished in accordance with a list specified by the Engineer.

13.8.20 Quality of work

13.8.20.1 General

The machinery shall be manufactured, finished, assembled, and adjusted in accordance with best industry practice.

Machinery components in contact with each other, or in contact with machinery supports, shall be machined to provide true contact surfaces. Surfaces that are in moving contact with other surfaces shall be machined true to dimension and with the grade of finish specified on the drawings.

Castings shall be clean and all fins and other irregularities shall be removed. Unfinished edges of flanges and ribs shall have rounded corners. All inside angles shall have suitable fillets. Drainage holes of suitable sizes shall be provided where necessary to prevent collection of water.

13.8.20.2 Fits and tolerances

The limits of accuracy for machining the work and the tolerances on all metal fits shall be shown on the shop drawings. Fits and tolerances shall be in accordance with CSA B97.3. The following six classes of fits, selected from CSA B97.3, shall be used in movable bridge applications:

(a) FN1 (light drive fit);
(b) FN2 (medium drive fit);
(c) LC1 (locational clearance fit no. 1);
(d) LC4 (locational clearance fit no. 4);
(e) LT1 (locational transition fit no. 1); and
(f) RC6 (medium running fit).

These six classes shall be applied in accordance with Table 13.15.

The machined work, where decimal dimensions are given with a tolerance indicated, shall be within the limits of this tolerance. The size that will obtain the most satisfactory mating of parts will be halfway between these limits, and this result shall be sought whenever practicable.

The standard fits listed in this Clause apply only where both fitting parts are machined.

Table 13.15
Fits and finishes
(See Clauses 13.8.20.2 and 13.8.20.3.)

Part(s)	Fit	Finish: Micro-inches	Finish: Microns
Machinery base on steel	—	250	6.3
Machinery base on concrete	RC6	500	12.7
Shaft journal	RC6	8	0.2
Journal bushing	LC1	16	0.4
Split bushing in base	FN1	125	3.2
Solid bushing in base (up to 6.4 mm wall)	FN2	63	1.6
Solid bushing in base (over 6.4 mm wall)	FN2	63	1.6
Hubs on shafts (up to 50.8 mm bore)	FN2	32	0.8
Hubs on shafts (over 50.8 mm bore)	FN2	63	1.6
Hubs on main trunnions	LT1	32	0.8
Turned bolts in finished holes	RC6	63	1.6
Sliding bearings	LC4	32	0.8
Keys and keyways	—	6	1.6
Machinery parts in fixed contact	—	125	3.2
Teeth of open spur gears	—		
Circular pitch under 25 mm		32	0.8
Circular pitch 25–44 mm		63	1.6
Circular pitch over 44 mm		125	3.2

Note: *The fits for cylindrical parts specified in this Table shall also apply to the major dimension of non-cylindrical parts.*

13.8.20.3 Surface finishes

The American National Standards Institute (ANSI) system of surface finishes shall be used for indicating the various degrees of roughness allowed for machine-finished surfaces. Such finishes shall be in accordance with Table 13.15.

13.9 Hydraulic system design

Because movable bridges at many locations in Canada do not operate in winter, necessitating a close-down and start-up operation, the designer of the hydraulic system shall consider the effect of this inactivity on maintenance requirements for the system. The designer shall also consider the need to prevent contamination of the environment by the hydraulic fluid.

The detailed design requirements for hydraulic design shall be as specified in Article 6.5.37 of Chapter 15 of the AREMA *Manual for Railway Engineering*, subject to the following:

(a) any reference to "Company" in the *Manual* shall be taken to mean "Owner"; and
(b) where the *Manual* refers to parts of the *Manual* other than Article 6.5.37 of Chapter 15, these shall be taken to refer to the applicable clauses of this Code.

13.10 Electrical system design

13.10.1 General

The requirements specified in Clause 13.10 are based on the use of either dc or 60 Hz ac motors. Movable bridges should be operated by dc motors using variable-voltage control or adjustable-voltage control, by ac squirrel-cage induction motors using variable-voltage control or variable-frequency control, or by ac wound-rotor induction motors with the appropriate control system. The use of other motors or motor-controller methods shall be in accordance with the requirements specified by the Owner.

For the operation of a vertical lift bridge, the requirements of Clause 13.10 are based on the use of one hoisting machine to operate the bridge or on the use of two hoisting machines mechanically connected. Special requirements are specified for tower-drive vertical lift bridges that use independent hoisting machines at the ends of the span operated by ac or dc motors electrically connected by synchronizing motors or synchronizing controls to maintain the bridge in level position during operation. When required by the Engineer, such independent hoisting machines shall maintain the span in level position during operation by means of special synchronizing controls and the requirements of Clause 13.10 shall be as subject to such modifications as are required by the Engineer.

13.10.2 *Canadian Electrical Code, Part I*

The construction and installation of all electrical materials and devices shall be in accordance with the *Canadian Electrical Code*, *Part I*, and local ordinances, except as otherwise specified in Clauses 13.10.3 to 13.10.50.

13.10.3 General requirements for electrical installation

The drawings shall indicate the electric power service that is available and the location of the point at which such service shall be obtained. The contractor shall provide the electrical installation complete from this service point, including all equipment, wiring, and cables, except as specified by the Engineer.

The electrical equipment shall comply with the Standards of the following organization, as applicable: CSA, the Electrical and Electronic Manufacturers Association of Canada (EEMAC), the Institute of Electrical and Electronics Engineers (IEEE), the National Electrical Manufacturers Association (NEMA).

The voltage characteristics of the power supply shall be determined by the Owner. Where necessary, suitable stabilization equipment shall be included in the system.

Total voltage drops shall not exceed 5% at rated load within the electrical installation.

The contractor shall provide all grounding devices required for the electrical equipment and service.

To prevent deterioration due to corrosion of parts of the electrical installation other than electrical apparatus, all bolts, nuts, studs, pins, screws, terminals, springs, and similar fastenings and fittings shall be, where practicable, of a corrosion-resisting material, e.g., stainless steel or bronze, approved by the Engineer or of a material treated in a manner approved by the Engineer to render it adequately resistant to corrosion. Hot-dip galvanizing of materials in compliance with CSA Standards for such materials shall be considered an approved treatment. Corrosion-prevention treatment of electrical equipment shall be as specified by the Engineer to suit the conditions of exposure.

All metal parts of the electrical equipment, including all conduits not furnished with a fused coating of polyvinylchloride, shall be painted as specified by the Engineer for structural steel. For conduits and similar parts where it is not practicable or convenient to apply paint in the shop, the shop coat may be applied in the field and followed by the required field coats.

The contractor shall take insulation resistance readings on all installed circuits (with the electrical equipment disconnected) and shall furnish a complete record of the results. These circuits should include connected motors when tested. Circuits, feeders, and equipment up to 350 V shall be tested with a 500 V instrument. Circuits, feeders, and equipment from 350 to 600 V shall be tested with a 1000 V instrument. Resistance to grounding devices shall be checked before energizing. At least one megohm shall be registered. Defective circuits shall be replaced.

Requirements for emergency operation of power-operated bridges and for standby power for electrically operated bridges shall be as specified in Clause 13.8.

All electrical installations shall incorporate seismic restraints in accordance with applicable regional or local codes.

13.10.4 Working drawings

13.10.4.1

In addition to furnishing the data required by Clause 13.13, the contractor shall prepare and furnish complete working drawings for the electrical system. In these drawings the distance between adjacent lines and symbols of elements shall not be less than 10 mm. The number of required sets of drawings shall be specified by the Owner. Only drawings approved by the Engineer shall be used for construction. The tracings shall be revised to show the work as constructed and shall then become the property of the Owner.

13.10.4.2

Working drawings shall include the following:

(a) Wiring interconnection diagrams that provide termination identification of wires and cables, sizes and numbers of wires and cables, and the make and capacity of all apparatus, including the ratings of impedances. Schematic diagrams shall include two- or three-line power and control diagrams showing the connection schemes, including detailed equipment and control schematic diagrams, which shall include the control panels and console. The number of each wire and the designation for each electrical device or piece of equipment shall be shown on the control schematic diagram. This device designations shall be used to identify each piece of equipment on the assembly and installation drawings, which shall show locations to scale of all external and internal components, including terminal blocks for the control panels, terminal boxes, and control console.

(b) One-line diagrams showing the complete power and distribution system.

(c) Control block diagrams for complex systems, showing the control components in block form and the interconnection of blocks and direction of signal flow.

(d) Conduit drawings showing the routing and size of each conduit, the number and size of each wire in each conduit, and the location and method of support of all conduits, ducts, boxes, and expansion fittings. Each conduit shall be given an individual conduit designation.

(e) Installation drawings giving the location of all cables, conduits, control panels, control consoles, resistances, lamps, switches, and other apparatus.

(f) Sectional drawings of all cables, showing component parts, their dimensions, and the material used.

(g) Drawings showing the general construction and dimensions of the control console and all control panels and the arrangement of all equipment thereon.

(h) Certified dimension prints of all electrical equipment.

(i) Detailed construction drawings of all boxes, troughs, ducts, and raceways other than conduit.

(j) Detailed construction drawings of warning gates and barrier gates, navigation lights, and audible signalling devices.

(k) Curves for each span-driving motor, showing the variation in motor speed and motor currents with output torque, and within the torque intervals determined by test in accordance with Clause 13.10.5, for each power point on the controller.

The requirements of Items (a) and (b) for working drawings may be partially fulfilled by use of a suitably coordinated conduit and cable schedule.

13.10.5 Motor and generator tests

One span-driving motor of each size or type used shall be subjected to a complete test in accordance with CSA C22.2 No. 100 or ANSI/NEMA MG-1. At the option of the Owner, certified test data of a motor of identical design may be accepted in lieu of tests of the actual motors.

For ac motors the tests shall also include the determination of the variation in speed and motor currents with motor torques from zero to the maximum designed torque for the drive system. Where required by the Engineer, the speed-current-torque curve shall also be determined for overhauling torque, including the effects of the motor control equipment. In addition, for wound-rotor motors the speed-current-torque relationship shall be determined with a rotor-shorted condition.

All dc motors shall also be tested to determine the speed-current-torque relationship for each power point on the controller, from an overhauling torque of 100% of full load to a driving torque of 200% of full load.

Unless otherwise specified by the Engineer, all motors except span-driving motors tested to CSA C22.2 No.100 or ANSI/NEMA MG-1 requirements shall be subjected to a short commercial test. If the results indicate characteristics differing materially from those of the completely tested motor, the contractor shall make the necessary alterations and run complete tests to demonstrate the final characteristics.

The motors in tower-drive vertical bridges with synchronizing motors shall be subject to the test requirements for span-driving motors, except that where the synchronizing motors are of the same size and type as the span-driving motors, only the short commercial test shall be required.

Each generator shall be subjected to a short commercial test.

Except as otherwise approved by the Owner, all motor and generator tests shall be made in the presence of an inspector designated by the Owner.

The contractor shall furnish six certified copies of each test report.

13.10.6 Motors — General requirements

The following requirements shall be met:

(a) Motors shall be of the totally enclosed type unless otherwise specified by the Engineer.
(b) Motors subjected to atmospheric conditions shall be totally enclosed and non-ventilated or totally enclosed and fan cooled.
(c) Motors installed in weather-protected houses may be drip-proof or of the protected type.
(d) Unless otherwise specified by the Engineer, motor windings shall be impregnated with a moisture-resisting compound in order to increase their resistance to moisture, and span-drive motors shall have embedded winding temperature-sensitive devices.
(e) A drain hole shall be provided in the bottom of the motor frame and, where feasible, heaters shall be incorporated. A seal-type plug and sealed entrance for the heaters shall be required for motors subjected to atmospheric conditions. Heaters shall be thermostatically controlled and shall be de-energized when the motor is operating.
(f) Motors whose frames tilt during the operation of the bridge shall have ball or roller bearings arranged with provisions for flushing.
(g) Span motors shall be capable of stalled operation for 2 min with the motor control equipment functioning normally for seating torque.
(h) Primary and secondary conduit boxes for span-driving motors shall be split cast and fully gasketed.

All dc motors shall be series, compound, or shunt wound, as determined by the performance specified, and shall have commutating poles. Motors for dynamic or regenerative braking shall perform their function within the allowable temperature rise. Span-driving motors shall comply with ANSI/NEMA MG-1.

All ac motors shall be induction motors that are suitable for the specified service characteristics and comply with CSA C22.2 No. 100 or ANSI/NEMA MG-1. Preferably, all motors shall be of the wound-rotor type for ac variable-voltage silicon controlled rectifier (SCR) drives or of the squirrel cage type for ac variable-frequency drives.

13.10.7 Motor torque for span operation

The locked rotor and breakdown torques for ac motors shall be those specified by CSA C22.2 No. 100 or ANSI/NEMA MG-1.

Motor torques shall be as follows:

(a) For a one-motor installation, not more than 125% of the full-load motor torque shall be required to produce the maximum required bridge starting torque. The maximum torque peaks that occur when the bridge is accelerated to the required speed using the specified bridge control should not exceed 180% of the rated full-load motor torque.
(b) For a two-motor installation with no provision for operating the bridge with a single motor, the two motors jointly shall meet the requirements specified in Item (a).

(c) For each motor in a two-motor installation with provision for operating the bridge with a single motor in not more than 1.5 times the opening time specified in Clause 13.5.8, not more than 150% of the full-load motor torque shall be required to produce the maximum required bridge starting torque.

The maximum bridge-starting torque shall be determined in accordance with Clause 13.8.9.

13.10.8 Motor temperature, insulation, and service factor

All ac motors shall have Class B or better insulation, 1.15 service factors, and ambient temperature ratings of 40 °C unless otherwise specified by the Engineer. All dc motors shall have Class F insulation, 1.0 service factors, and ambient temperature ratings of 40 °C.

Note: *Classes of insulation are described in CSA C22.2 No. 100 and ANSI/NEMA MG-1.*

13.10.9 Number of motors

When the total power necessary at the motor shaft to move the bridge under Case A of Clause 13.8.5.2 at the required speed exceeds 37 kW (50 hp), the use of two similar span-driving motors, with provision for operation of the bridge by one motor, shall be considered, taking into account the importance of both the bridge and the waterway.

Warning gates, bridge locks, and the end and centre lifting devices of a swing span should be operated by one or more motors separate from and independent of the span-drive motors.

13.10.10 Synchronizing motors for tower-drive vertical lift bridges

Where synchronizing motors are used on a tower-drive vertical lift bridge to maintain the bridge in level position during operation, the total full-load rated torque of these motors on each tower shall not be less than 50% of the total full-load rated torque of the span-drive motors on each tower, and consideration shall be given to increasing this to 100% where practicable.

13.10.11 Speed of motors

The speed of span-driving motors shall not exceed 900 rpm. The speed of motors that operate bridge locks and wedges shall not exceed 1200 rpm. The speed of gear motors of 7.5 kW (10 hp) or less, fractional horsepower motors, motors driving hydraulic pumps, and motor-generator sets shall not exceed 1800 rpm.

13.10.12 Gear motors

Gear motors should have an extension of the high-speed shaft to allow hand operation. Gears shall be lubricated by immersion in the lubricant and effective seals shall be provided to prevent the lubricant from reaching the motor windings. Gear motors shall have at least a Class II rating as defined by AGMA and shall carry an AGMA nameplate stating the kilowatts (horsepower), service rating, and service factor.

13.10.13 Engine-generator sets

13.10.13.1 General

Engine-generator sets, whether for primary or emergency power, shall consist of an internal combustion engine and an electric generator direct connected and mounted on a common base. Separate units may be provided for supplying power for span operation and for auxiliary services such as lights and signals. Where used as an emergency power source, the lighting generator unit shall start automatically on failure of the normal power. The span-operating power unit shall be started manually by a remote control switch.

Engines shall meet the applicable requirements of Clauses 13.8 and 13.10.13.2. They shall develop adequate power to supply the maximum load, including the motor-starting load, while maintaining speed within the specified range. Consideration shall be given to the special requirements of non-linear and harmonic-producing loads.

13.10.13.2 Instruments and controls

Engine instruments and controls shall be mounted in a cabinet on the unit and shall include gauges indicating water temperature, oil pressure and temperature, and vacuum (for diesel engines); a throttle control; a start/stop switch for manual control; a manual emergency shutdown; indicating lights for low oil pressure, high water temperature, overspeed, and overcrank; and an alarm contact for sounding a remote alarm in case of high water temperature, low lubricating oil pressure, or failure to start after four cranking cycles.

13.10.13.3 Engine governor

The engine governor shall provide 3 to 5% speed regulation from no load to maximum load.

13.10.13.4 Generator and exciter

The generator power characteristics shall be capable of supplying the maximum load, including the motor-starting load, at suitable power characteristics, with a regulated voltage drop within the limits specified by the Engineer. It shall have a continuous rating with Class B or better insulation over 40 °C ambient, be of drip-proof construction, and comply with applicable CSA, ANSI, IEEE, and NEMA Standards.

The exciter shall be of the direct-connected brushless type and sized to furnish 10% more excitation than is required at full generator-operating load. When the generator supplies non-linear loads, the exciter shall be of the permanent magnet type.

13.10.13.5 Starting system

An automatic starting system, complete with suitable batteries and automatic charging, shall be provided and shall have

(a) a positive shift gear-engaging starter, 12 V dc;
(b) a cranking limiter to provide four cranking cycles of 10 s duration, each separated by 5 s rest;
(c) a storage battery with sufficient capacity to crank the engine for 3 min at 0 °C without using more than 25% of ampere-hour capacity;
(d) a solid state battery charger of constant voltage that is two stage from trickle charge at standby to boost charge after use; and
(e) the following regulation characteristics:
  (i) ±1% output for ±10% input variation;
  (ii) automatic boost for 6 h every 30 d; and
  (iii) equipped with a dc voltmeter, dc ammeter, battery charge meter, and on-off switch.

13.10.13.6 Generator control panel

The generator control panel shall contain the following devices:

(a) a three-position control switch ("Off-Auto-Manual") for automatic starting units;
(b) an air circuit breaker;
(c) a voltmeter and ammeter;
(d) a frequency meter;
(e) an elapsed time meter;
(f) an automatic voltage regulator and voltage adjustment rheostat;
(g) alarm contacts for remote indication;
(h) a device to automatically start the generator when the automatic transfer switch is in "Auto" mode; and
(i) a battery charge meter.

13.10.14 Automatic electric power transfer

Where two sources of electric power are available, power for continuous services such as lights and navigation signals may be transferred automatically from the normal feeder to the standby or emergency source on failure of the normal supply. On return of the normal power to at least 90% of rated voltage, the load may be retransferred after an adjustable time delay of not less than 5 min and after the bridge motion has ceased. If the emergency source fails, the retransfer shall be instantaneous on return of normal power. The automatic transfer switch shall be of the circuit breaker or contactor type, electrically operated and mechanically held, with a single solenoid or motor mechanism and separate arcing contacts. It shall be enclosed in a cabinet and a circuit diagram shall be provided on the inside of the door. It shall also comply with CSA C22.2 No. 178 and shall have a separate disconnect to de-energize the power source during maintenance.

Where both power sources are external, with one designated "Normal" and the other "Standby", an auxiliary switch shall be provided to permit using either as the preferred source.

Where the standby source is an engine-generator set, the automatic transfer switch shall be equipped with a pilot contact for remote automatic starting of the engine 3 s after normal source failure or after a drop of any phase to 70% or less of the rated voltage. The normal load circuits shall remain connected during this 3 s delay. When the standby generator delivers not less than 90% rated voltage and frequency, the load shall be automatically transferred. On transfer to normal, the engine shall run for a minimum of 5 min to permit engine cool-down and then automatically shut down. A time delay shall be provided to ensure that the transfer switch remains in the neutral position when transferring between normal and emergency positions. The transfer switch shall have a test button so that normal source failure can be simulated.

13.10.15 Electrically operated motor brakes

Motor brakes for the span-driving motors shall meet the requirements of Clause 13.8.7 and shall be fail-safe-type disc or shoe brakes. They shall be held in the set position by springs with such force as is needed to provide the required retarding torques. Disc hubs or brake wheels for the motor brakes shall be mounted on the motor pinion shaft or on a motor shaft extension.

Brakes shall be designed for intermittent duty for the required retarding torques and shall be designed to release when the current is on and to apply automatically when the current is cut off. Brakes for the span operation shall have hydraulic, mechanical, or electrical interlocks to ensure that all of the brakes will not be applied at the same time.

The brakes shall be equipped with a means for adjusting the torque and shall be set in the shop for the specified torque. Each brake shall have a nameplate that shall state the torque rating of the brake and the actual torque setting (where it differs from the torque rating). Shoe brakes shall be designed in such a manner that it is possible to adjust the brakes or replace the shoe linings without changing the torque settings.

All dc brakes shall be released by thruster units or shunt coil solenoids. Shunt coils shall have discharge resistors or surge suppressors so that opening the shunt coil circuit does not cause high transient voltage.

All ac brakes shall be released by thruster units or, if specified by the Engineer, motor operators. Thruster motors exposed to the atmosphere shall be totally enclosed, non-ventilated, and have weatherproof insulation for the motor and conduit box.

For shoe brakes the releasing mechanism shall be capable of exerting a force of at least 130% of the force actually required to release the brake when set at the specified torque and minimum expected ambient temperature.

Brakes for other motors shall be solenoid-released shoe brakes or dry-type disc brakes and shall have an intermittent rating not less than the full-load torque of the motors.

Brakes shall be of a construction that ensures uniform wear and shall have independent provisions for adjusting lining wear, equalizing clearance between friction surfaces, and adjusting the retarding torque. The brake linings shall not be affected by moisture. Solenoids, thruster units, and motor operators shall be moisture proof. Fittings shall be corrosion resistant. Thrusters for shoe brakes shall be provided with year-round oil.

Shoe brakes shall have a hand-release lever permanently attached to the brake mechanism and arranged so that one worker can operate the releases easily and rapidly. Means shall be provided for latching the lever in the set and released positions. Disc brakes shall have provisions for hand release and be arranged so that one worker can operate them easily and rapidly and so that they can be latched in the released position.

Where brakes are located outside the machinery house, they shall be of weatherproof construction or shall have a weatherproof housing. The housing shall be arranged to permit operation of the hand-release lever from outside the housing.

Brakes installed on the moving span shall operate satisfactorily with the span in any position.

Brakes shall have heating elements where needed to prevent the accumulation of moisture and frost. Brakes shall also provide for the addition of limit switches for control and lights to indicate the position of the brakes and their hand-release levers.

13.10.16 Electrically operated machinery brakes

Motor brakes and machinery brakes for the span-operating machinery shall meet the requirements of Clauses 13.8.7.2 and 13.8.7.3, respectively.

13.10.17 Design of electrical parts

Electrical parts for lift bridges, including wiring, switches, circuit breakers, controllers, and contactors, shall be designed for operation of the bridge using either normal or emergency power for the span loads specified in Clause 13.8 and for at least 30 min total continuous duration of the operating cycles with Clause 13.8.5.3 Case B loading. Electrical parts for bascule and swing bridges shall be similarly designed for bridge operation for the span loads specified in Clause 13.8 and for 30 min continuous operating cycles of Clause 13.8.5.3 Case B loading for bascule bridges and 30 min continuous operating cycles of Clause 13.8.5.2 Case A loading for swing bridges.

The temperature rise of electrical parts under such operation shall not exceed that for which the part is normally rated in accordance with Clauses 13.10.3 and 13.10.39 and any other applicable clauses.

13.10.18 Electrical control

13.10.18.1 General

Electrical control shall be classified as manual, semi-automatic, or automatic sequence control.

Separate controllers shall be provided for the span-driving, bridge lock, wedge, and gate motors. For control of motors in parallel, the switches shall be interconnected so that all switches will be operated simultaneously by one handle. The controllers shall be arranged in such a manner that the operation of one motor can be cut out without affecting the operation of any other motor.

When there are two main dc motors powering one output, the control shall be series, parallel, or series-parallel, as required, unless the current is furnished by a storage battery, in which case the control shall be of the series-parallel type.

For parallel operation for ac, and for constant potential parallel or series-parallel operation for ac, there shall be mechanically interlocked reversing contactors and separate resistors for each motor. When two motors are connected to one hoisting machine, accelerating contactors shall be common to both motors, unless otherwise specified by the Engineer. For three-phase ac, each phase shall have its own resistors, so designed to give balanced current in all three phases. Some of the acceleration contactors shall be controlled by acceleration relays to ensure that the torques specified in Clause 13.10.7 are not exceeded. When common accelerating contactors are not used, the accelerating contactors shall be electrically or mechanically connected or designed in such a manner that the corresponding circuits in each motor control will be made simultaneously and, in the event of one motor being cut out, the control for the motor in service will operate satisfactorily.

Controls for span-driving motors shall provide multi-speed (stepped) or variable-speed (stepless) control. Multi-speed controls shall be of the full-voltage magnetic, reduced voltage, wound rotor master switch (drum controller), or wound rotor face plate controller type. Variable-speed controls shall be of the following types:
(a) solid state, of the four-quadrant type (for dc motors);
(b) solid state variable-voltage silicon controlled rectifier (SCR) (for ac and dc motors); or
(c) variable frequency (for ac motors).

Motor controllers shall be constructed, selected, and installed in accordance with applicable CSA and NEMA requirements.

Controls shall be arranged in such a manner that all motor brakes shall be held released when power is applied to the span-driving motors.

13.10.18.2 Manual sequence control

For non-emergency manual sequence control, it shall be possible for the operator to initiate each interlocked function in sequence by push button, selector switch, and/or master switch control.

Note: *The steps in such a sequence may include*

(a) actuate traffic signals;
(b) actuate approach gates;
(c) actuate exit gates;
(d) actuate barriers or retarders;
(e) pull span locks;
(f) release brakes;
(g) open span by manually accelerating and decelerating span-driving motors; and
(h) set brakes.

For emergency control, actions by the operator can include operation of bypass switches, selection of the emergency mode of span operation, and skew correction.

When two motor brakes are used on a hoisting machine, a point of control for each motor brake shall be provided for each direction of travel so that the motor brakes can be applied separately.

For tower-drive vertical lift bridges, two points of motor brake control for each direction of travel shall be provided when two motor brakes are used for the hoisting machine in each tower.

Electrically operated machinery brakes may be controlled through contacts on the master switch or by a separate switch.

If the machinery brakes are controlled by the master switch, the contacts shall be arranged in such a manner that all machinery brakes will be held released when power is applied to the span-driving motors, except when the seating switch described in this Clause is used. The sequence of the master switch contacts shall be arranged in such a manner that the machinery brakes can be applied by the operator whenever the span is coasting. One point of machinery brake control shall be provided for each direction of travel for all machinery brakes on a hoisting machine.

If the machinery brakes are controlled by a separate switch, they shall normally be set and arranged in such a manner that they will be released by the operator before the bridge is put in motion. They shall be held in release during the entire operation unless the operator desires to use them while the bridge is coasting or there arises an emergency requiring brake power exceeding that offered by the motor brakes, at which time they shall be capable of being applied instantly by the operator. The machinery brakes shall be designed in such a manner that they will not be injured if left in release indefinitely. When specified by the Engineer, brakes shall have at least three steps of retarding torque to permit partial application of the brakes. The machinery brake circuits shall be independent of the general interlocking system and there may be an electrically operated interlocking device that will prevent the use of the span-driving motor and the machinery brakes against each other except by use of the seating switch described in this Clause.

A seating switch for applying the machinery brakes with power still on the motors shall be provided to enable the span to be drawn tightly to its seat and held in that position. The seating switch shall be convenient to the operator and shall be hand or foot operated.

For tower-drive vertical lift bridges, one point of control for each direction of travel shall be provided for all machinery brakes. All brakes shall be applied automatically if the span attains a predetermined skew.

Motors for bridge locks, wedges, and other devices associated with the movement of the span shall be controlled through magnetic contactors energized by control switches or push buttons independent of the span-driving motor controls.

13.10.18.3 Semi-automatic sequence control

Procedures for semi-automatic sequence control shall be the same as those for manual sequence control, except that the span-driving motors shall be automatically accelerated and decelerated by the single operation of a push-button or selector switch.

13.10.18.4 Automatic sequence control

For non-emergency automatic sequence control, it shall be possible for the operator to initiate each interlocked function in sequence by one movement of a push button or selector switch.

Note: *The steps in such a sequence may include*
(a) actuate traffic signals;
(b) actuate approach gates;
(c) actuate exit gates;
(d) actuate barriers or retarders;
(e) pull span locks;
(f) release brakes;
(g) open span by manually accelerating and decelerating span-driving motors; and
(h) set brakes.

For emergency control, actions by the operator can include operation of bypass switches, selection of the emergency mode of span operation, and skew correction.

Span motor controls shall include all components needed to provide motor protection against abnormal conditions, automatic controlled acceleration and deceleration, modulated speed control (where applicable) (e.g., tower drives without power-synchronizing motors) and four-quadrant control to accommodate overhauling loads involving negative torque or regenerative braking loads, and any other feature needed to ensure satisfactory performance following a single movement of the initiating control switch.

Motor and machinery brake types, and control arrangements, shall be selected so as to ensure time-sequenced brake application under all conditions.

Two modes of stopping span movement shall be provided for variable-speed motor controls, as follows:
(a) normal stop, with controlled electrical deceleration followed by brake application; and
(b) immediate power cut-off and application of brakes initiated by an emergency stop button.

Limit switch, resolver, or encoder actions shall initiate deceleration before the nearly open and nearly closed span positions are reached and the control system shall be designed to accomplish a reduction to slow speed when those positions are passed. Speed limit switches or some other means shall be provided to detect span speed at the nearly open and nearly closed positions. If the span speed is within the normal limit of the span, movement shall continue to completion; if not, power shall be cut off, brakes shall be applied, and a reset operation of the overspeed circuit shall be required before span movement can be resumed. During final seating, the motor torque shall be reduced and the brakes shall remain in released position until the span is tightly seated, after which the brakes shall set and the motors shall be disconnected.

Tower-drive lift bridges arranged for automatic sequence control shall have two independent skew limit switches, resolvers, or encoders connected in series for each span mode of operation.

13.10.19 Speed control for span-driving motors

Multi-speed (stepped) motor controls shall provide at least six steps of acceleration. These steps shall be such that
(a) the motor torque will differ as little as practicable from the average torque required for uniform acceleration from zero speed to full speed; and
(b) the bridge shall accelerate and decelerate smoothly
    (i) under the lowest friction conditions in the absence of wind or other extraneous unbalanced loads; and
    (ii) when the motors are carrying their maximum loads.

Separate resistors shall be provided for each motor.

Variable-speed (stepless) drives shall provide smooth variable-speed control with a minimum speed range of 10 to 1, independent acceleration and deceleration ramps field adjustable from 2 to 20 s minimum, speed regulation of ±1.5% (or better), and motor slip. Variable-speed motor controls shall also provide four-quadrant control to accommodate overhauling loads, provide dynamic braking, and enable compatibility with programmable controllers.

13.10.20 Master switches and relays for span-driving motors

Master switches for the span-driving motors shall be cam-operated reversing switches with a single handle and shall have the necessary contacts and contact fingers for operating the magnetic contactors. The contacts and wearing parts shall provide for speed control of the motors.

Adjusting plugs, screws, and nuts, including time-limit adjustments, shall be easily accessible so that acceleration relays can be adjusted for the proper timing intervals between acceleration steps. The contacts shall be removable without disturbing the setting of the relays.

13.10.21 Programmable logic controllers

Programmable logic controllers (PLCs) shall be manufactured and tested to meet the requirements of the applicable CSA, IEEE, and NEMA Standards and shall be installed and grounded in accordance with the *Canadian Electrical Code, Part I*.

The PLC system power supply and the input-output (I/O) system shall use a common ac source in order to minimize line interference and reduce the possibility of the PLC receiving faulty input signals. Unless otherwise specified by the Engineer, the ac source shall feed an uninterruptible power supply (UPS) inverter, complete with batteries and battery charger, that provides power to the PLC and I/O systems. The UPS shall be able to provide power for least 1 h.

One or more regulation/isolation transformers shall be placed on the incoming ac power line to the PLC to stabilize the voltage to the PLC power supply and reduce the possibility of noise and interference.

A properly rated power disconnect switch shall be placed in the power circuits feeding the PLC power supply and in the I/O system to remove power from the PLC system during an emergency.

Master control relay (MCR) circuits shall be provided as a safe and convenient means for removing power from the I/O system during periods when PLC operation is intentionally halted, when PLC operation is unintentionally halted because of power loss or fault, or when there is an emergency stop condition.

Grounding connections to the grounding electrodes and structural steel shall be provided using the exothermic method or brazing.

Shielded cable shall be used to protect low-level signals from interference and the shields shall be continuous and connected to ground at one point only. The degree and type of shielding, the shield ground location, and the I/O surge protection shall be as recommended by the PLC manufacturer.

A computer and appropriate software shall be provided for use as a diagnostic tool.

The PLC may have a communication card installed to allow remote communication if required by the Owner.

13.10.22 Resistances and reactors

Resistors for motor control shall, unless otherwise specified by the Engineer, be non-breakable, corrosion-resistant, and edgewise-wound or punched-grid resistor units. Resistors for the span-operating motors shall, unless otherwise specified by the Engineer, be of a capacity equal to a NEMA ICS 9 intermittent cycle rating providing for 15 s on out of every 30 s. The resistors shall be mounted on a steel frame or protected in another appropriate manner to ensure that they are free from injurious vibration, mounted in such a manner that free circulation of cooling air is permitted, and furnished in such a manner that any unit or part of a unit can be removed and replaced without disturbing the others. The units shall be insulated from their supports.

For wound rotor motors with secondary resistance control, the controller shall be arranged in such a manner that a small amount of resistance shall always be left in the rotor circuits of each motor. This permanent resistance section shall be adjustable after installation and shall be proportioned for continuous duty.

Reactors for secondary control of wound rotor motors shall be arranged to present the same reactance to each motor phase, mounted on a steel frame or protected in another appropriate manner to ensure that they are free from injurious vibration, and mounted in such a manner that free circulation of cooling air is permitted and they are protected from dripping liquids.

13.10.23 Limit switches

Limit switches shall be provided for the bridge locks, end and centre lifting devices, and gate motors and shall stop the motors and set the brakes automatically at each end of travel.

Limit switches for the movable span shall have a master switch control that will cut off the current from the span-driving motors and set the brakes to stop the span in the nearly closed and nearly open positions. To fully close or fully open the bridge, it shall then be necessary to return the controller handle to the off position in order to bypass the limit switch contacts and regain control of the span. Where specified by the Engineer, relays that will prevent the bypass from functioning until a predetermined time after the brakes have set shall be provided. Additional limit switch contacts shall be provided to stop the span in the fully open position, and for swing bridges, where specified by the Engineer, in the fully closed position. Unless otherwise specified by the Engineer, the nearly closed and nearly open positions shall be taken to be 2 m from the fully closed and fully open positions, respectively.

Fully seated switches that indicate to the operator when the bridge is fully closed shall be provided for vertical lift and bascule bridges.

Tower-drive vertical lift bridges shall have skew limit switches mechanically connected to the machinery on the two towers, or equally effective switches of other types, that will cut off the current from the main motors and set the brakes to stop the span whenever it is more than a prescribed amount out of level. An ultimate skew limit switch, or a grouping of switches providing this function, shall also be provided to serve as a backup for the skew limit switch and stop the span at a larger amount out of level.

Limit switches, resolvers, and encoders exposed to the weather shall be watertight and all exposed parts shall be corrosion resistant. Where plunger-type limit switches are used for fully seated switches, they shall be weatherproof and shall be provided with cast or malleable iron enclosures and stainless steel operating rods. Where specified by the Engineer, each fully seated limit switch shall be provided with a ball plunger to minimize bending stresses on the plunger rod.

Electrically operated swing and bascule bridges shall include an overspeed limit switch to stop the span whenever normal span speed is exceeded. Lift bridges should also include such a switch.

Movable spans shall have an overtravel switch that will prevent excessive travel beyond the fully open position.

13.10.24 Interlocking

The operating mechanisms of all movable bridges shall be interlocked in such a manner that all devices can be operated only in the prescribed sequence.

Emergency bypass switches that will free the motors from the prescribed interlocking in case of emergency shall be provided. These switches shall be conveniently mounted on the control desk or main switchboard. Each such emergency switch shall be sealed in the off position.

Auxiliary and main power units shall be interlocked in such a manner that one is inoperative while the other is in service.

Motor and machinery brakes shall have limit switches arranged in such a manner that the bridge shall be inoperable whenever any brake or combination of brakes is released by hand and the available braking torque left in service is insufficient to meet the requirements of Clause 13.8.7.

13.10.25 Switches

An enclosed service-rated fused switch or circuit breaker with a pole for each ungrounded conductor shall be provided as a disconnect for the power system supply feeder. A similar switch, or a circuit breaker without the service rating to make it capable of being operated as a switch, shall be provided as a disconnect for each motor, light, signal, or other circuit.

Main disconnect switches shall be of not less than 60 A capacity and shall have their enclosure doors interlocked with the switch mechanism.

Toggle and tumbler switches shall be of corrosion-resistant construction and shall be of not less than 20 A capacity.

13.10.26 Circuit breakers and fuses

An automatic circuit breaker shall be placed in the supply line and have undervoltage release or trip coils to permit provision of undervoltage, phase reversal, and loss of phase protection. Where the supply has very large short-circuit capability, suitably rated current-limiting fuses may be provided in a disconnect switch ahead of the automatic circuit breaker, or otherwise incorporated into its design to accomplish alternative suitability. Where practicable, circuit breakers shall be used to provide short-circuit protection for all wiring circuits. The moulded-case circuit breaker selection process shall include a comparison of the short-circuit-interrupting ability of the I^2t rating (the integral of the square of the short circuit current, I, with respect to time, t, for the period of interruption) with the corresponding short-circuit capacity of the I^2t rating of the supply source connection. Moulded-case circuit breakers shall not be applied to circuits with possible short-circuit duty exceeding 60% of their rated interrupting ability or, if preceded by current-limiting fuses, their permissible I^2t source rating shall be at least 125% of the rated I^2t let-through of the preceding fuses for the particular application. A moulded-case circuit breaker or fuse shall be provided in each motor, brake, light, signal, indicator, or other circuit.

For circuits above 600 V, air-break, vacuum-break, or oil-immersed circuit breakers shall be used (as service conditions dictate). Breakers shall have a pole for each phase wire feeding through the breaker, an overload device consisting of a thermal or magnetic element for each pole, and a common trip.

Circuit breakers shall not be used for motor overload protection or for limiting the travel of any mechanism.

13.10.27 Contact areas

For custom-designed electrical equipment such as slip rings for swing bridges, line contacts shall be avoided where practicable. The current per square inch (645 mm^2) of contact area shall not exceed 50 A for spring-held contact or 100 A for bolted or clamped contact.

13.10.28 Magnetic contactors

Magnetic contactors shall have an 8 h current rating not less than the current through the contactor when the connected apparatus is operating at rated load. Magnetic contactors shall be of the shunt type and shall be quick acting. Contacts shall be well shielded to prevent arcing between them and other metal parts nearby and shall be designed to be readily accessible for inspection and repair. Copper contacts shall have a wiping action. Contactors shall have double-break features or magnetic blowouts or an equivalent means for rapidly quenching the arc and shall have a minimum number of parts. All steel parts shall be corrosion resistant. Magnetic motor starters shall have not less than a 25 A rating unless otherwise specified by the Engineer.

13.10.29 Overload relays

Overload relays (automatic or hand reset, as specified by the Engineer) shall be used in each phase or dc circuit for overload protection of all motors. Instantaneous magnetic overcurrent relays shall also be provided in motor circuits to de-energize all motors when the safe torque is exceeded, unless other means are provided for limiting the maximum torque.

13.10.30 Shunt coils

Where shunt coils are used, the insulation shall be capable of withstanding the induced voltage caused by cutting off the current.

13.10.31 Instruments

A line voltmeter, ammeters for span-driving motors, and a power bus wattmeter shall be provided and mounted on the control console. A voltmeter switch shall be provided for measuring the voltage between any two phases and between any phase and ground. Instruments shall be of the rectangular illuminated type, flush mounted, and back connected. Where specified by the Engineer, each hoisting machine at a tower-drive lift span shall have a wattmeter to determine the power required to operate each end of the span. All instruments mounted on the control console shall have a terminal voltage not greater than 120 V.

13.10.32 Protection of electrical equipment
Electrical equipment shall be protected from the weather and accumulation of debris.

13.10.33 Cast iron in electrical parts
Where cast iron is used in switches and small electrical parts, it shall be of the malleable type.

13.10.34 Position indicators and meters
Synchronous moving-span position indicators, skew indicators for tower-drive lift bridges, or electrical digital meters of the high-accuracy type guaranteed within $\pm 1°$ for angular measurements and ± 50 mm for linear measurements shall be provided. Transmitters, resolvers, or encoders shall be geared to trunnion shafts, counterweight sheave shafts, or machinery shafts, whichever are most suitable for the particular installation, and the receivers in the control console shall be geared to the indicators. Gearing shall be arranged so as to give the greatest practicable accuracy.

13.10.35 Indicating lights
Indicating lights of suitable colours shall be installed on the control console to show span positions (at a minimum, the fully closed, fully open, nearly closed, and nearly open positions), and the positions of the traffic gates, bridge locks, and end lifting devices. Indicating lights shall also be provided to show the released position of each span brake, the overload or overheat tripping of span-drive motors, and the status of other functions. All indicating lights shall operate at not more than 120 V ac. Indicating lights shall be of the oil-tight push-to-test type. A testing capability for all indicating lights shall be provided by a spring-return selector switch or a single momentary contact push button.

13.10.36 Control console

13.10.36.1 General
The span-control console shall contain switches for the span-operating motors and for the lock, end lift, and wedge motors; seating switches; bypass switches; switches for traffic gates and traffic signals; position indicators; indicating lights; and all other control devices and apparatus necessary for or pertinent to the proper operation and control of the span and its auxiliaries by the operator.

The control console shall be located in a manner that affords the operator a clear view in all directions. The console shall be of cabinet-type construction, with a horizontal front section about 0.9 m above the floor and an inclined rear instrument panel set at such a slope that the meters can be read from average eye level without parallax and without reflection from the glass instrument cover. The console plan dimensions and the arrangement of equipment shall be such that all control devices are within easy reach. Edges shall be bevelled and neatly finished. Unless otherwise specified by the Engineer, the top of the console shall be stainless steel that is at least 3 mm thick and has a non-reflecting finish.

13.10.36.2 Construction
The console frame shall be constructed of sheet steel at least 3 mm thick. All corners and edges of the console shall be rounded and the sheet steel shall be reinforced by flanging the metal into angle and channel sections. Connecting sections shall be properly joined by continuous seam welding or spot welding to provide a rigid free-standing structure. Outside surfaces shall be smooth and without visible joints, seams, or laps. The bottom of the console shall be left open. The supporting flange on the inside of the console frame at the bottom shall have suitable holes for bolting the console to the floor. Suitable brackets and angles shall be provided on the inside of the console in order to support the top and the equipment mounted thereon.

The control console shall have hinged doors on the front and, in accordance with the requirements of the installation, doors, removable panels, or fixed panels on the back and sides. Doors shall be fitted with sturdy three-point latches operated by flush-type, chromium-plated handles, shall be assembled accurately, and shall have a clearance not exceeding 3 mm at any point. A toe space shall be provided on the front of the console at the bottom.

The console, when finished, shall be given one coat of moisture-resisting primer and one coat of filler on all surfaces. The outside surfaces shall be given a non-reflecting finished coat of a colour as specified by the Engineer. The stainless steel console top shall be left unpainted.

The console interior shall be equipped with suitable lights controlled from a switch on the console.

Each piece of equipment and each indicating light on the control console shall have a properly engraved metal or lamicoid nameplate with black characters on a white background. The designations on the nameplates shall correspond with those shown on the wiring diagrams and in the operating instructions.

13.10.36.3 Controls

The off position of master switch handles shall be located toward the front of the console. For bascule and swing bridges, the direction of rotation of each master switch shall be such that when it is moved from the off position, the span, as seen by the operator, will move in the same direction as the master switch handle. For double-leaf bascule bridges, the switches shall rotate in opposite directions. For vertical lift bridges, clockwise rotation shall raise the bridge.

For bridges that have automatic controls, push buttons, industrial-type touch screens, or menu-driven graphical interfaces may be provided. Bridges that have automatic controls shall also be provided with controls for manual operation, which may be of the push button type.

Foot-operated seating switches may be supported by the outside of the console or set in a suitable recess at the bottom of the console. This foot recess shall be rounded at the top to a 30 mm radius. Outgoing control connections from the console shall be brought to suitably marked barrier-type terminal boards supported on straps securely attached to the console frame. Terminal boards shall be located in such a manner that they do not interfere with door access to the inside of the console. Wires shall be brought from the terminal boards to their respective terminals in an orderly arrangement, properly bunched and tied.

13.10.37 Control panels

Control panels shall be of enclosed, dead front, free-standing construction, EEMAC Type 1 or better, and should be of standard industrial motor control centre-type construction. Motor control centres shall be constructed in accordance with the *Canadian Electrical Code, Part I*, applicable NEMA and EEMAC Standards, and UL 845. All disconnect switches, circuit breakers, contactors, relays, rectifiers, instrument transformers, and other electrical equipment for the control of the span and its auxiliaries shall be mounted on or in the control panels. Equipment mounted at the bottom of the panel boards shall clear the floor by at least 150 mm. Open control panels shall not be used.

Control panels shall be front wired. Interconnections shall be made by copper bus bars or insulated cables of equivalent current-carrying capacity. Board wiring shall terminate in terminal strips supported in a substantial manner and all conductors shall be copper.

Each piece of equipment on the board shall have a properly engraved nameplate that meets the requirements of Clause 13.10.36.2.

13.10.38 Enclosures for panel boards

Enclosures for panel boards shall, unless otherwise specified by the Engineer, be general-purpose enclosures that comply with *Canadian Electrical Code, Part I*, and EEMAC requirements for Type 1 general-purpose enclosures. They shall be provided with suitably arranged doors to give access to the front of the board, shall be made of sheet steel at least 3 mm thick (welded and flanged in a manner that will result in a rigid free-standing structure), and shall be finished in the manner specified in Clause 13.10.36.2.

Note: *The sheet steel composition and finishing requirements of this Clause include the doors and panels of enclosures.*

13.10.39 Electrical wires and cables

Electrical wires and cables (including their insulation and covering), shall be of a quality that satisfies applicable CSA, EEMAC, and Insulated Power Cable Engineers Association (IPCEA) Standards. Where these requirements do not apply, electrical wires and cables shall conform to ASTM requirements.

Unless otherwise specified by the Engineer, wires external to the control console and control panels shall be protected by conduit or armour or be suitably jacketed. They shall be rubber-insulated, rubber-jacketed wires with an insulation of a quality at least equal to synthetic rubber moisture-resisting 60 °C or Type THWN or XHHW.

Insulated wire for connections on the backs of control panels and in control consoles shall be thermoplastic insulated wire that complies with UL requirements for Type THWN wire, 600 V. Insulated wire for connections to motor resistance grids shall have insulation rated for 250 °C (Types TFE, TGGT, and TKGT meet this requirement).

All wires shall be stranded copper. No wires smaller than No. 12 AWG shall be used, except that No. 14 AWG shall be permitted for connection to internal control components where the use of No. 12 AWG would be impracticable for control console, control panel, or interlocking device wiring.

The ends of all wires that are No. 8 AWG or smaller shall have solderless high-compression indent-type terminals where they terminate at control panels, control consoles, terminal strips, lighting panels, junction boxes, and similar locations. The ends of larger wires shall be similar and shall terminate in pressure lugs or screw-type solderless connectors.

13.10.40 Tagging of wires

To enable any wire to be traced from terminal to terminal, wires shall be numbered and the numbers permanently marked on durable fibre tags, on metal or plastic bands with protective heat-shrunk sleeving, or on such other material specified by the Engineer. The numbers shall correspond to those shown on the wiring diagrams.

13.10.41 Wire splices and connections

Wire splices and connections shall be made only on terminals and within enclosures intended for the purpose. Wires shall be continuous from terminal to terminal.

13.10.42 Raceways, metal conduits, conduit fittings, and boxes

13.10.42.1 General

Except as otherwise specified by the Engineer, conduits shall be hot-dip galvanized schedule 40 steel or alloy steel pipe, with a factory-fused and bonded polyvinylchloride plastisol coating and shall be not less than 3/4 NPS (National Pipe Size). All couplings, locknuts, and bushings shall be standard screw type; setscrew-type couplings, locknuts, and bushings shall not be used. Bushings shall be of the insulating type. Conduit entrances to sheet metal enclosures shall have sealing O-rings or liquid-tight hub fittings.

13.10.42.2 Conduit size

The conduit size shall be such that the total areas of the wires, including insulation, shall not exceed the percentage of the area of the conduit specified by the *Canadian Electrical Code, Part I*. Phase wires in ac motor circuits shall be placed close together in one conduit to lessen the inductive effects. The circuits for not more than three ac motors may be in one conduit.

13.10.42.3 Boxes and fittings

Suitable conduit outlet boxes, junction and pull boxes, ells, and other fittings shall be used with conduits. Boxes, outlets, and other fittings shall be of cast iron or malleable iron of sufficient thickness to permit the conduit to be threaded into the fitting and shall be hot-dip galvanized. Boxes and other fittings shall be weatherproof throughout, free from rough edges and rough surfaces, and unless otherwise specified by the Engineer, of *Canadian Electrical Code, Part I*, and EEMAC Type 4 construction, unless housed in a room. Large boxes for which cast iron or malleable iron is not practicable may be built of steel plates and angles at least 4.7 mm thick, with all joints continuously welded, and shall be provided with drain holes.

13.10.42.4 Bends
Bends in conduits shall be used sparingly. The total angle of all bends in one conduit run should not exceed 180°. Where the conduit is bent, the radius of the bend to the centre of the conduit shall be at least eight times the inside diameter of the conduit (except for factory ells). Conduits shall have drain holes placed in tee-connections located at the low points. So far as possible, conduits shall be run in lines parallel and perpendicular to the principal lines of the house and structure. Embedded conduits shall be carefully rodded after placing with a device that will ensure that the whole interior surface of the conduit is free of obstructions. The conduit shall be temporarily protected by conduit closures or pipe caps until the wires are pulled and the conduit is permanently closed.

13.10.42.5 Conduit supports
Conduits shall be placed so that dirt will not accumulate around them and shall be firmly clamped to the structure by supports on not more than 2 m centres to prevent rattling. There shall be at least 25 mm clearance between conduits and at least 100 mm clearance between conduits and the supporting structure. Adequate provision for conduit movement shall be made wherever conduits cross expansion joints in the supporting structure. Conduit runs between the bridge and solidly based structures, such as piers and operator’s houses, shall include at least 300 mm of liquid-tight flexible metal conduit at the interface with bonding jumpers.

13.10.42.6 Conduit connections
Conduit connections to motors, generators, limit switches, brakes, and other devices specified by the Engineer shall include at least 500 mm of slack liquid-tight flexible metal conduit.

13.10.42.7 Wireways and cable trays
Where bridges have a relatively large amount of equipment and an extensive control system, consideration shall be given to the use of wireways or continuous rigid cable supports instead of the exclusive use of conduits above the control panels and with control console connections.

Where wireways are used, they shall be of the full-lay type with a cross-section of at least 200 x 200 mm (preferably 300 x 300 mm) to adequately accommodate the recommended bending radii of all cables. Where continuous rigid cable supports are used, all cables supported by such supports shall meet *Canadian Electrical Code, Part I*, requirements. Wireways and trays shall not be used outside the operator’s house except with armoured cables.

13.10.43 Electrical connections between fixed and moving parts

13.10.43.1 General
Electrical connections for carrying current between fixed and moving parts shall be made using the flexible cables or collector rings described in Clauses 13.10.43.2 and 13.10.43.3 or another suitable method, as specified by the Engineer.

13.10.43.2 Flexible cables
Conductors in flexible cables shall have extra-flexible stranding. The cables shall be connected to terminal strips in junction boxes at which the wiring in conduits terminates. Short cables with relatively small movement of the moving part with reference to the fixed part, e.g., cables extending from a fixed pier to a fender not rigidly attached to the pier, shall be extra-flexible round portable cable covered with a neoprene jacket or protected with corrosion-resistant metal armour. Long cables with relatively large movement of the moving part with reference to the fixed part, e.g., vertical cables hanging in a loop between the end of a vertical lift span and a tower, shall be rubber-insulated flexible cables covered with an internally reinforced neoprene jacket. Such cables shall be suspended from segmental supports arranged to prevent sharp bends in the cables as the span moves.

13.10.43.3 Collector rings

On swing bridges, the connection between the fixed part and the swing span may be made through shoes sliding on circular collector rings attached to the centre pivot. The collector rings shall be protected by a removable metal casing.

13.10.44 Electrical connections across the navigable channel

Electrical connections for carrying current across the navigable channel shall be made as specified by the Engineer. They shall be made using submarine or overhead cables (submarine cables are preferred, but, particularly for vertical lift bridges, overhead cables can be a more suitable choice). The voltage, number of conductors in each cable, size and number of strands in each conductor, construction of the cable, and other such characteristics shall be as specified by the Engineer. Each cable shall provide a number of spare conductors. The following requirements shall also apply:

(a) Submarine cables shall be armoured with spiral-wound galvanized steel wire armour and, if specified by the Engineer, covered with a neoprene jacket. Individual wires shall meet the requirements of Clause 13.10.39. Submarine cables shall be provided with conductor insulation suitable for submarine use. Unless otherwise specified by the Engineer, submarine cables shall be placed at least 1.5 m below the bed of the channel. Cables shall be long enough to provide ample slack.

(b) Overhead cables shall be jacketed with neoprene or another superior jacketing compound resistant to weather and aging. Individual wires shall meet the requirements of Clause 13.10.39. Overhead cables shall be installed in accordance with CAN/CSA-C22.3 No. 1.

Each cable shall be suspended from a messenger strand at intervals of at least 500 mm. Messenger strands shall be strung with a sag required to safely support the entire construction under the conditions of ice, wind, and temperature applicable to the location of the bridge, shall be of high-strength material, and shall be adequately anchored to steel framework at their ends. Messenger strands, cable hangers, and all accessories shall be protected against corrosion in a manner that ensures a service life not less than that of the overhead cable.

13.10.45 Service lights

A complete electric lighting system shall be installed for the operator's house, machinery house, stairways, vertical lift span tower tops, signals, machinery, and end lifting and locking apparatus and at all other points where periodic inspection or maintenance of equipment is required.

Lighting systems shall be designed to produce at least the following intensities measured 1.0 m above floor level:

(a) operator's house: 300 lx;
(b) machinery house: 200 lx;
(c) unhoused machinery: 150 lx; and
(d) walkways and stairways: 200 lx.

Lighting may be fluorescent, incandescent, or mercury vapour, or any other suitable type. All fixtures fitted with incandescent lamps smaller than 100 W shall be equipped so that lamps up to 100 W can be used. Conductors shall be at least No. 12 AWG in size. The lights in the operator's room should have dimming adjustment from the control console. In machinery houses, there shall be fixed pendants of suitable length, with enclosed fixtures or fire-enamelled steel dome reflectors. Vapour-tight fire-enamelled steel dome reflectors or enclosed fixtures shall be provided for exterior lighting. Lampholders shall have shock-absorbing porcelain sockets.

Convenience outlets shall be provided in each room of the operator's house, in machinery houses, at bridge lock and wedge machinery, at submarine cable terminal cabinets, and at all locations where occasional inspection or maintenance of equipment is required. They shall be of the twin-receptacle three-wire grounding type. Convenience outlets exposed to the weather shall be weatherproof and have a ground fault circuit interrupter installed.

13.10.46 Navigation lights

Navigation and other light fixtures on the movable span and on fenders shall be capable of withstanding shocks and rough treatment, completely weatherproof, and made of bronze or aluminum. Light fixtures shall have shock-absorbing porcelain sockets and lamps rated between 100 and 150 W. LED technology lighting may be considered if permitted by the authority having jurisdiction.

13.10.47 Aircraft warning lights

Aircraft warning lights shall be provided when required by Transport Canada or other Regulatory Authority.

13.10.48 Circuits

Circuits shall be classified as follows:
(a) power circuits:
  (i) motors; and
  (ii) other;
(b) control circuits:
  (i) span;
  (ii) bridge locks;
  (iii) wedges;
  (iv) gates; and
  (v) other; and
(c) lighting circuits:
  (i) navigation and marking or warning lights;
  (ii) service lights;
  (iii) convenience outlets; and
  (iv) other.

An independent circuit shall be provided for each motor, each control circuit, the navigation lights, each group of service lights, and each group of convenience outlets. Common neutral return wires shall not be used. Each circuit shall be protected and controlled by its own circuit breakers, fuses, and switches located on the panelboards or at an equally convenient point.

13.10.49 Grounding and lightning protection

Movable bridges shall have grounding and lightning protection systems that meet or exceed the requirements of the *Canadian Electrical Code, Part I*, and the provincial or territorial lightning rods statute (or CAN/CSA-B72 if these is no provincial or territorial statute). The power supply shall be one of the following types (in order of preference):
(a) solidly grounded (most preferred);
(b) resistance grounded; or
(c) ungrounded (least preferred).

Ground-indicating lights shall be provided, except for solidly grounded systems.

The metal portions of the bridge shall have grounding conductors connected to low-resistance grounding electrodes. An electrical system ground bus, and connections to all major electrical equipment (including each motor, each brake, the aircraft warning lights, and the land-based navigation lights) shall be provided.

13.10.50 Spare parts

In the absence of Owner-specified requirements, the following list of spare parts may be included in the contract for supply by the contractor:
(a) six fuses of each size and kind;
(b) one complete set of stationary and moving contacts for each size of each device;
(c) one indicating light unit, complete with lamp and fitted with a coloured cap, for each size, type, and colour used;
(d) one spare circuit breaker for each size and kind used;

(e) one control relay and two extra sets of contacts for each type;
(f) one complete set of contacts and one operating coil for each size and type of magnetic contactor and motor starter;
(g) one brake coil or thruster motor for each size of brake, or one complete brake;
(h) one set of brushes and one set of motor bearings for each size of motor; and
(i) spare parts for engines, engine-generator sets, skew and positioning indicating devices, electronic control components, tachometers, motor secondary impedance elements, and other parts.

13.11 Construction

13.11.1 Shop assemblies

All shafts, gears, pinions, and other parts supported by common machinery frames or cases shall be assembled in the shop and tested by operation where possible in order to prove fits and clearances before shipping to the field ready to be set in place.

Where practicable, machinery parts shall be assembled in the supporting structural members in the shop. They shall be aligned and fitted, and holes in the supports shall be drilled, with all components in their correct relative positions. The members shall be match-marked to the supports and to each other and erected in the field in the same relative positions. When this assembly method is not possible, the holes in the machinery parts shall be drilled in the shop and holes in the supports shall be left blank for drilling in field assembly after final alignment.

If any undersize holes are shop drilled to aid field alignment of machinery, they shall be reamed to fit the permanent bolts after all other holes have been drilled and their bolts placed.

The complete centre pivot arrangement of swing bridges, including rim girders, centre pivot, radial members, rack, track, and rollers, shall be shop assembled. All parts shall be aligned, fitted, and match-marked before disassembly.

The tread and track components of rolling lift bridges shall be shop assembled to the segmental and track girders, respectively; aligned, fitted, and drilled; and the parts match-marked for field assembly.

Built-up counterweight sheaves shall be assembled, welded, and stress-relieved before finish machining is performed. Where journal bearings are used, each trunnion shaft shall be shop assembled to its bearings and the linings shall be scraped to a true fit in the journals.

Parts that are likely to be damaged by weather or handling shall be suitably crated or otherwise protected for shipment. The rubbing surfaces of trunnions, shafting, and machinery bearings shall be protected by suitable anti-rust compounds and lagged with wood.

13.11.2 Coating

The surfaces of machinery parts, except rubbing surfaces, shall be cleaned and coated, in the shop and in the field, as specified by the Engineer. The colours shall be the Approved safety colours for machinery components. A minimum three-coat system shall be used.

13.11.3 Erection

13.11.3.1 Structural

Erection shall be performed in accordance with Clause 10.24 and the applicable requirements of this Section. Movable spans may be erected in either the closed or the open position, depending on navigational requirements, the season of the year, and other site conditions indicated on the drawings.

13.11.3.2 Machinery

The installation and adjustment of machinery shall be carried out by millwrights skilled in this type of work. Final alignment and adjustment of machinery parts whose relative position is affected by the deflection or movement of the supports under full dead load, or of the span under full dead load, shall not occur until such deflection or movement has taken place.

Machinery parts previously aligned and assembled in the shop shall be erected according to the match-marks. Where final alignment and the drilling of holes in supporting members in the field is required, the machinery components shall be adjusted to the proper elevation and aligned through the use of shims. The holes in the supporting steel shall be drilled while the parts are assembled. Shims shall not be smaller than the bearing area shimmed.

The trunnion bearings of bascule bridges and the counterweight sheave bearings of vertical lift bridges shall be aligned with special care and due allowance shall be made for the deflection of the bearing seats that can result from the working load on the bearings. Deflections of this nature shall have been previously calculated and the method to be used for securing proper alignment shall be shown on the contractor's erection drawings.

Before the ropes are placed over counterweight sheaves, the bearings shall be lubricated and each sheave shall be turned to prove that it runs freely in its bearings.

The tension in counterweight ropes on each sheave shall be adjusted so that the maximum and minimum tension does not vary more than 5% from the average tension in all ropes on any one sheave.

All open gearing shall be aligned so that backlash is within tolerance and at least the centre 50% of the face width of each pair of meshing teeth is in contact. The cross-mesh shall not exceed 0.25 mm per 150 mm (0.01 in per 6 in) of face width. All open gear measurements, including main pinion and racks, shall be submitted to the Engineer for review and approval. The measurements shall include backlash, cross-mesh alignment, tooth valley gap, and face contact. The type of bluing or lubricant used for face contact measurements shall be submitted to the Engineer for approval before any measurements. The measurements shall be performed at a minimum of eight equally spaced span positions ranging from fully open to fully closed.

13.11.3.3 Protection from damage

Electrical components and other parts that are protected from the weather in the finished structure shall be protected in the field during erection by temporary housing or other suitable means.

All machinery parts shall be carefully protected from damage during transit, unloading, and storing while awaiting erection. All finished surfaces that were shop coated with protective rust-inhibiting grease or another medium shall be cleaned of the protective coating(s) immediately before erection.

Wire ropes shall be housed and stored at least 460 mm above the ground. The ropes shall be kept free of dirt, cinders, and sand. Wire ropes shall be carefully removed from reels or coils by revolving them and shall be erected in a manner that avoids sharp kinks or bends. They shall not be dragged across the ground. The stripe painted on each counterweight rope shall be straight after the rope is erected.

13.11.3.4 Items for initial operation

The contractor shall furnish grease, oil, fuel, and any other item necessary for the preliminary operation of the movable bridge until it has been accepted by the Owner, except for electric power, which shall be supplied by the Owner. The supplied grease and oils shall be subject to the approval of the Engineer.

13.11.3.5 Treatment of ropes

As soon as the movable span can be operated, the contractor shall thoroughly remove any foreign material from ropes and apply one coat of approved wire rope lubricant. The types of lubricant to be used on the operating and counterweight ropes and the method of application shall be specified by the Engineer. The application shall be made to the satisfaction of the Engineer.

13.11.3.6 Counterweights

The contractor shall adjust the counterweight so that the moving span is balanced according to the drawings. Approval of any balance calculations, or of any materials or processes, shall not relieve the contractor from the entire responsibility for obtaining a satisfactory balance.

For satisfactory balance, the movable span shall have a slight closing force present when it is seated and either a neutral or a very slight opening force present when it is fully open.

Note: *Balance can be checked in the field using the following procedures:*
(a) compare motor currents during opening and closing of the span;
(b) compare power meter (kilowatt) readings during opening and closing of the span;

(c) run a drift test from the mid-position of travel in both the opening and the closing direction. Compare the drift in each direction with power off and the brakes released;
(d) measure the torque in the drive train during opening and closing of the bridge;
(e) compare the grease patterns on the main pinion teeth; and
(f) for vertical lift bridges, weigh the imbalance between the span and the counterweights.

13.11.3.7 Testing

Before the main operating machinery is connected for transmitting power, it shall be given an idle run for at least 4 h to the satisfaction of the Engineer.

When the entire installation has been completed, the movable span, including all accessories, shall be operated through not fewer than three complete cycles using normal power, prime movers, and controls, and through at least two complete cycles using emergency power, emergency prime movers, and controls. If inspection during and after these tests shows that any components are defective, inadequate, or functioning improperly, the contractor shall make the necessary corrections, adjustments, or replacements.

13.12 Training and start-up assistance

For power-operated bridges, the contractor shall provide specialist personnel for 30 d after acceptance, on the basis of one 8 h shift per day, plus emergency service if required. These personnel shall supervise the operation of the bridge and instruct the operator in its proper operation and maintenance.

13.13 Operating and maintenance manual

Every movable bridge shall have an operating and maintenance manual. The bridge contractor shall supply the number of copies of the manual specified by the Owner. The manual shall include a schedule of normal and emergency testing of bridge operations and a step-by-step explanation describing the sequence and interlocks of each operation. It shall also state the maximum operating wind speed. It shall include instructions for the maintenance, lubrication, and adjustment of mechanical and electrical parts and copies of all instruction manuals, bulletins, etc. relating to any commercial devices that form part of the mechanical and electrical systems that operate the bridge. It shall be revised as needed to include updated information if changes are made to mechanical and electrical systems.

The manual shall also include the following:
(a) a schematic wiring diagram;
(b) “as fitted” wiring diagrams;
(c) “as fitted” conduit and wire layouts;
(d) “as built” machinery erection drawings;
(e) machinery shop drawings;
(f) a machinery lubrication checklist and drawings;
(g) a list of the names and addresses of local distributors of all standard components;
(h) where PLC controls are used, a hard copy and soft copy of the software program;
(i) a detailed listing of all of the interlocks employed in the electrical controls of the movable bridge;
(j) where control devices are employed in the control of the movable bridge and require calibration and setting of set points, a listing of such devices, along with their set points;
(k) a copy of the testing and commissioning records; and
(l) hydraulic schematic diagrams.

13.14 Inspection

The Engineer shall specify the frequency and types of inspections recommended for long-term durability and trouble-free operation of the movable bridge.

The frequency and types of inspections shall be consistent with the use of the facility, i.e., seasonal or year-round operation.

Scheduled inspections shall include the following at a minimum:

(a) An annual visual and aural inspection. In the case of seasonal use, this should be at the start of the season. This inspection shall take into consideration the adequacy of the maintenance of the bridge.

(b) A comprehensive inspection at not more than two-year intervals. This inspection shall include structural safety, wear and alignment of mechanical components, and any breaks, defects, or hot spots in the electrical system.

All inspections shall be followed by a written report. The design shall incorporate the necessary means to facilitate inspection of all components requiring inspection.

Section 14 — Evaluation

Annexes

Section 14 Evaluation

14.1 Scope

This Section specifies methods of evaluating an existing bridge to determine whether it will carry a particular load or set of loads.

14.2 Definitions

The following definitions apply in this Section:

Capacity — the unfactored nominal resistance of an element or joint.

Evaluation — determination of a bridge's capacity to carry traffic loads.

> **Evaluation Level 1** — evaluation of a bridge to determine its load-carrying capacity for vehicle trains (in normal traffic).
>
> **Evaluation Level 2** — evaluation of a bridge to determine its load-carrying capacity for two-unit vehicles (in normal traffic).
>
> **Evaluation Level 3** — evaluation of a bridge to determine its load-carrying capacity for single-unit vehicles (in normal traffic).

Evaluator — a qualified Engineer responsible for evaluating a bridge.

Normal traffic — vehicular traffic that does not include any vehicle operating under a permit for weights, dimensions, or both that do not meet regulatory limits.

Posting — signing of a bridge for load restrictions in accordance with regulations.

Single-unit vehicles — trucks, buses, cars, and other vehicles consisting of a single unit.

Two-unit vehicles — tractor–semi-trailers, car-trailers, truck-trailers, and other vehicles consisting of two units.

Vehicle trains — tractor-trailer-trailers, tractor–semi-trailer–trailers, tractor–semi-trailer–semi-trailers, and other vehicles consisting of three units.

14.3 Symbols

The following symbols apply in this Section:

A = force effects due to additional loads (including wind, creep, shrinkage, temperature, and differential settlement) that may be considered in the evaluation

$A_{s\ell}$ = area of longitudinal tensile reinforcing steel in the bottom of concrete deck slabs, mm^2

A_{st} = area of transverse tensile reinforcing steel in the bottom of concrete deck slabs, mm^2

A_r = nominal area of a rivet, mm^2

A_v = area of transverse shear reinforcement perpendicular to the axis of a member within a distance s, mm^2

a = length of end split, crack, or check measured into the span from the centreline of the support to the tip of the split, mm
B_r = factored bearing resistance of a riveted connection, N
b = width of a component, mm
b_v = effective web width within depth d_v, mm (see Clause 8.9.1.6)
CR = loss of prestress due to creep of concrete, MPa
D = nominal (unfactored) dead load effect
$\overline{D}$ = mean dead load effect
d = diameter of a rivet, mm; depth of a wood member, mm
d_ℓ = depth from the top of a slab to the centroid of bottom longitudinal tensile reinforcing steel, mm
d_t = depth from the top of a slab to the centroid of bottom transverse tensile reinforcing steel, mm
d_v = effective shear depth, mm
e = edge distance, mm
F = live load capacity factor
F_c = correction factor for concrete deck punching shear capacity as a function of f_c'
F_q = correction factor for concrete deck punching shear capacity as a function of q
F_u = specified tensile strength of structural or rivet steel, MPa
F_y = nominal value of yield strength of steel, MPa
$\overline{f}_c$ = average of measured strengths of 100 mm diameter concrete cores after modification in accordance with Clause A14.1.2, MPa
f_c' = specified compressive strength of concrete, MPa
f_{cr} = cracking strength of concrete, MPa
f_{ps} = calculated stress in prestressing steel at ultimate limit state, MPa
f_{pu} = specified tensile strength of prestressing steel, MPa
f_{py} = specified yield strength of prestressing steel, MPa
f_{se} = effective stress in prestressing steel after losses, MPa
f_{sj} = stress in prestressing steel at jacking, MPa
f_{st} = stress in prestressing steel at transfer, MPa
f_y = specified yield strength of reinforcing bars, MPa
$\overline{f}_y$ = average of measured values of yield strength of reinforcing steel or structural steel, MPa
h = overall thickness or depth of a component, mm
I = nominal (unfactored) dynamic component of the live load, expressed as a fraction of the nominal static live load effect (dynamic load allowance)
k_c = factor to modify coefficient of variation of concrete core strengths (Table A14.1.2)
k_s = factor to modify coefficient of variation of steel coupon strengths (Table A14.1.1)
k_{sv} = size effect factor for shear of wood members (see Clause 14.14.1.7.2)
L = nominal (unfactored) static live (traffic) load effect
$\overline{L}$ = mean static and dynamic live (traffic) load effect
L_a = factored total load effect in masonry arches
L_t = unfactored load effect from test loading
L_{wf} = factored wheel load (including dynamic load allowance), kN
m = number of shear planes in a riveted joint (equal to one for rivets in single shear and two for rivets in double shear)
n = number of rivets; number of strength tests

P = posting factor

P_n = unfactored resistance of a masonry arch, kN

P_r = factored resistance of a masonry arch, kN

q = average percentage of tensile reinforcement in the two directions in which steel is placed at the midspan of a slab panel (see Clause 14.14.1.3.3); uniformly distributed portion of lane load, kN/m

R = nominal unfactored resistance, kN, calculated using the material strengths as specified in Clause 14.7.1 and in accordance with the requirements of Sections 6 to 12 and Clause 14.14

$\overline{R}$ = mean resistance, kN

REL_1 = loss of prestress due to relaxation of prestressing steel prior to transfer, MPa

REL_2 = loss of prestress due to relaxation of prestressing steel after transfer, MPa

R_d = nominal resistance of a concrete deck slab, kN

R_n = unfactored (nominal) resistance of a concrete deck slab, kN

R_r = factored resistance of structural component, kN

S_D = standard deviation of dead load force effects

S_L = standard deviation of live load force effects

SH = loss of prestress due to shrinkage of concrete, MPa

s = spacing of stirrups measured parallel to the longitudinal axis of a component, mm

s_{m1} = maximum allowable spacing below which a section is considered to have full transverse reinforcement, mm

s_{m2} = maximum allowable spacing above which a section is considered not to have transverse reinforcement, mm

s_{r1} = maximum allowable spacing, as a fraction of shear depth, below which the section is considered to have transverse reinforcement

s_{r2} = maximum allowable spacing, as a fraction of shear depth, above which the section is considered not to have transverse reinforcement

T_f = tensile force in a member or component at the ultimate limit state, N

T_r = factored tensile resistance of a riveted joint, N

t = thickness of a steel component, mm; thickness of a concrete deck slab, mm; time, d

U = resistance adjustment factor

V = coefficient of variation

V_{AD} = coefficient of variation for dead load analysis method

V_{AL} = coefficient of variation for live load analysis method

V_D = coefficient of variation for dead load

V_I = coefficient of variation for dynamic load allowance

V_L = coefficient of variation for live load

V_R = coefficient of variation for resistance

V_S = coefficient of variation for total load

V_f = shear force at the ultimate limit state, 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, the resistance factor for tendons in Clause 8.4.6 (taken as positive if resisting the applied shear), N

V_r = factored shear resistance of a riveted joint, N

W = gross vehicle weight, kN

α_A = load factors for force effects due to additional loads (including wind, creep, shrinkage, temperature, and differential settlement) that may be considered in the evaluation

α_D = load factors for force effects due to dead loads

α_L = load factors for force effects due to live loads

β = target reliability index

δ_{AD} = bias coefficients (ratio of mean to nominal effects) for dead load analysis method

δ_{AL} = bias coefficients (ratio of mean to nominal effects) for live load analysis method

δ_D = bias coefficients (ratio of mean to nominal effects) for dead load

δ_I = bias coefficients (ratio of mean to nominal effects) for dynamic load allowance

δ_L = bias coefficients (ratio of mean to nominal effects) for live load

δ_R = bias coefficients (ratio of mean to nominal effects) for resistance

μ_p = ratio of area of prestressed reinforcement to area of concrete

ρ = reinforcement ratio of tensile reinforcement

ρ_b = reinforcement ratio producing balanced conditions

σ_A = unfactored stress due to additional loads (including wind, creep, shrinkage, temperature, and differential settlement) that may be considered in the evaluation, MPa

σ_D = unfactored dead load stress, MPa

σ_L = unfactored live load stress, MPa

σ_{SLS} = serviceability limit state stress, MPa

ϕ = resistance factor

ϕ_c = resistance factor for concrete (see Clause 8.4.6)

ϕ_{mc} = member resistance factor for a riveted connection

ϕ_{md} = member resistance factor for a reinforced concrete deck slab

ϕ_{mm} = member resistance factor for a masonry component

ϕ_r = material resistance factor for rivet steel

ω_p = mechanical reinforcement ratio for prestressing steel

14.4 General requirements

14.4.1 Exclusions

This Section shall not be used to determine whether a bridge or bridge design complies with the design requirements of Sections 1 to 13 and 16.

Pedestrian bridges, railings, barrier walls, foundations, and retaining walls shall be evaluated in accordance with the design requirements of this Code.

This Section does not address loads caused by earthquakes, fires, floods, ice, and vehicle and vessel collisions.

This Section shall not be used unless the bridge is secure against causes of failure other than traffic loading. Loads other than traffic shall be considered in accordance with other Sections of this Code, except as required by Clause 14.9.5.

This Section shall not be used for buried structures.

14.4.2 Expertise

Evaluations in accordance with this Section shall be performed and checked by suitably qualified Engineers, and shall be reviewed by an experienced bridge Engineer, who may also be one of the persons performing or checking the evaluation.

14.4.3 Future growth of traffic or future deterioration
No allowance is made in this Section for future growth of traffic or for future deterioration of the bridge. If such changes are anticipated, they shall be considered in the evaluation. If any change in traffic or in the condition of the bridge that has not been accounted for occurs, the evaluation shall be reviewed and, if necessary, the bridge shall be re-evaluated.

14.5 Evaluation procedures

14.5.1 General
The procedures in this Section shall be used when bridges are to be evaluated for load limit restrictions, serviceability, or fatigue loadings.

A bridge shall be evaluated in accordance with one or more of the following methods:

(a) ultimate limit states methods (except for masonry abutments, masonry piers, and masonry retaining walls). The following shall be considered acceptable methods:
   (i) ultimate limit states methods in accordance with Clauses 14.15.2.1 and 14.15.2.2, using load and resistance adjustment factors specified in Clauses 14.13 and 14.14;
   (ii) the mean load method for ultimate limit states specified in Clause 14.15.2.3; and
   (iii) the load testing method specified in Clause 14.16;

(b) serviceability limit states methods; and

(c) other Approved methods.

14.5.2 Limit states

14.5.2.1 General
The limit states for which a bridge is to be evaluated shall be selected from Clauses 14.5.2.2 to 14.5.2.5.

14.5.2.2 Ultimate limit states
Ultimate limit states shall be used in determining the load-carrying capacity, stability, and load posting of bridges, except as specified in Clause 14.5.2.3.

14.5.2.3 Serviceability limit states
Serviceability limit states shall be used in determining the load-carrying capacity, stability, and load posting of masonry abutments, masonry piers, and masonry retaining walls, in accordance with Clause 14.15.3.

For a bridge where cracking detrimental to the structure, deformation, stresses, or vibrations are expected or evident, the bridge and its affected components shall be evaluated for serviceability limit states requirements in accordance with the applicable Sections of this Code.

Where there is no evidence of serviceability-related defects, the evaluation need not consider the serviceability limit state if neither the use nor the behaviour of the bridge is changed.

14.5.2.4 Fatigue limit state
Evaluations for the fatigue limit state shall be carried out in accordance with Clause 14.18.

14.5.2.5 Earthquake loading
Evaluation for earthquake loading shall be carried out in accordance with Clause 4.11.

14.5.3 Condition data
The dimensions, member sizes, geometry, strength properties, extent, and location of deterioration, distress, and permanent distortion shall, to the extent that they will affect the evaluation, be verified in accordance with Clause 14.6.

14.5.4 Procedures

14.5.4.1 General

The following requirements shall apply to the evaluation of bridges in accordance with this Section:
(a) material strengths shall be determined in accordance with Clause 14.7;
(b) loads shall be defined in accordance with Clauses 14.8 to 14.10; and
(c) lateral distribution of live load shall be categorized in accordance with Clause 14.11.

14.5.4.2 Evaluation at ultimate limit states using load and resistance factors

The following requirements shall apply to the evaluation at ultimate limit states using load and resistance factors:
(a) the target reliability index, β, shall be selected in accordance with Clause 14.12.1;
(b) the load factors shall be selected in accordance with Clause 14.13;
(c) the resistances shall be calculated in accordance with Clause 14.14; and
(d) the live load capacity factor, F, shall be determined in accordance with Clause 14.15.

14.5.4.3 Evaluation at serviceability limit states

For evaluation at serviceability limit states, the live load capacity factor, F, shall be determined in accordance with Clause 14.15.3.

14.5.4.4 Evaluation by use of mean load method

The following requirements shall apply to evaluation by use of the mean load method:
(a) the target reliability index, β, shall be selected in accordance with Clause 14.12.1;
(b) nominal (unfactored) resistances shall be calculated in accordance with Sections 6 to 12 or Clause 14.14 by taking all resistance factors, ϕ, as having a value of 1.0; and
(c) the live load capacity factor, F, shall be determined in accordance with Clause 14.15.2.3.

14.5.4.5 Evaluation by load testing

The following requirements shall apply to evaluation by load testing:
(a) the bridge shall be evaluated in accordance with Clause 14.5.4.2, 14.5.4.3, or 14.5.4.4;
(b) the bridge shall be tested in accordance with Clause 14.16; and
(c) the live load capacity factor, F, shall be determined in accordance with Clause 14.16.4.2.

14.5.5 Bridge posting

If a bridge needs to be posted for load restriction based on the load capacity factor calculated in accordance with Clause 14.15 or 14.16, the posting loads shall be in accordance with Clause 14.17.

14.6 Condition inspection

14.6.1 General

A condition inspection of the bridge shall be carried out to the satisfaction of the evaluator. Inspection records shall be sufficiently detailed to allow changes in condition to be assessed during future inspections.

14.6.2 Plans

The evaluator shall verify that the available Plans accurately represent the structure. If no Plans are available, measurements shall be made with sufficient precision to suit the intended purpose.

14.6.3 Physical features

All physical features of a bridge that affect its structural integrity shall be examined.

14.6.4 Deterioration
All flawed, damaged, or deteriorated regions shall be identified, and sufficient data shall be collected so that these defects can be properly considered in the evaluation.

14.7 Material strengths

14.7.1 General
The strengths of materials that do not have visible signs of deterioration shall be determined using one of the following methods:
(a) review of original construction Plans and documents in accordance with Clause 14.7.2;
(b) analysis of tests of samples obtained from the bridge or from specific bridge components in accordance with Clause 14.7.3;
(c) estimation by considering the date of bridge construction in accordance with Clause 14.7.4; or
(d) an Approved method.

14.7.2 Review of original construction documents

14.7.2.1 General
The Plans and other relevant contract documents may be reviewed to determine
(a) the specified minimum yield strength of structural steel;
(b) the specified compressive strength of concrete;
(c) the specified minimum yield strength of reinforcing steel;
(d) the specified tensile strength of prestressing steel;
(e) the species and grade of wood; and
(f) the type of stone and grade of mortar used in masonry construction.

14.7.2.2 Mill certificates
Actual values of yield and ultimate tensile stresses reported on mill certificates shall not be used for evaluation. Instead, the strength used shall be the guaranteed minimum value specified for the grade of steel shown on the certificate.

14.7.3 Analysis of tests of samples

14.7.3.1 General
Nominal material strengths to be used in the calculation of member resistances may be determined by testing samples obtained from the bridge. Samples shall not be removed from locations where the strength, stability, or integrity of the member might be adversely affected. The location and orientation of each sample shall be recorded, as well as any additional information that could later be useful in interpretation of the test results.

Material strength values obtained by testing shall not be directly substituted into the equations for resistance specified in this Code. Test results shall be converted to nominal material strengths in accordance with Annex A14.1 or an Approved method.

14.7.3.2 Prestressing steel
Removal of prestressing steel specimens for testing shall not jeopardize the safety of the structure or be hazardous to the personnel involved. Specimens shall be tested in accordance with CSA G279.

14.7.3.3 Wood
In lieu of obtaining wood samples for testing, the species and grade shall be identified by a wood grader.

14.7.3.4 Masonry mortar
Mortar in joints shall be sampled or tested to obtain the compressive strength. The mortar shall be classified as one of the following types:
(a) hard mortar — compressive strength greater than 7 MPa;
(b) medium mortar — compressive strength between 2 and 7 MPa; or
(c) soft mortar — compressive strength less than 2 MPa.

The allowable limit state stress at the serviceability limit state in shear and compression shall be determined using a rational method based on the classification of the mortar, the thickness of the mortar joint, and the type of stone used in the masonry.

14.7.4 Strengths based on date of construction

14.7.4.1 General
In the absence of more specific information, preliminary evaluation may be based on material strengths estimated by considering the date of bridge construction.

14.7.4.2 Structural steel
If Plans and mill certificates are not available, and coupons have not been taken for testing, the values specified in Table 14.1 shall be used for structural steel.

Table 14.1
Properties of structural steel
(See Clause 14.7.4.2.)

Date of bridge construction	Specified F_y, MPa	Specified F_u, MPa
Before 1905	180	360
1905–1932	210	420
1933–1975	230	420
After 1975	250	420

14.7.4.3 Concrete
If construction Plans and specifications are not available and cores have not been obtained, the compressive strength of concrete with no visible signs of deterioration shall be taken as 15 MPa for the substructure, 20 MPa for the superstructure, and 25 MPa for prestressed concrete components.

14.7.4.4 Reinforcing steel
If Plans and mill certificates are not available and specimens have not been tested, the values specified in Table 14.2 shall be used for reinforcing steel.

Table 14.2
Minimum yield strengths of reinforcing steel, MPa
(See Clause 14.7.4.4.)

Date of bridge construction	Grade			
	Structural	Medium or intermediate	Hard	Unknown
Before 1914	—	—	—	210
1914–1972	230	275	345	230
1973–1978	275	345	415	275
After 1978	300	350	400	300

14.7.4.5 Prestressing steel
If Plans and mill certificates are not available and specimens have not been tested, the tensile strength of prestressing steel shall be taken as 1600 MPa for bridges constructed before 1963 and 1725 MPa for bridges constructed later.

14.7.4.6 Rivets
If Plans and mill certificates are not available, the following values for the ultimate tensile strength of rivets shall be used:
(a) rivets constructed to ASTM A 7-39, constructed before 1936, or of unknown origin: F_u = 320 MPa; and
(b) rivets constructed to CESA S42-1935, CSA G40.2-1959, ASTM A 141-33, ASTM A 141-39, or ASTM A 502-65, or constructed after 1935 but of unknown origin: F_u = 360 MPa.

14.7.5 Deteriorated material
Deteriorated material shall be assessed in accordance with Clause 14.14.3. Non-destructive test methods, such as ultrasonic pulse velocity and surface hardness methods, may be used to correlate the concrete strength in damaged and sound regions of a structure. If compressive strengths are estimated using non-destructive methods, calibration factors shall be determined using concrete cores from the structure, and their uncertainty shall be accounted for in the estimate of predicted strengths.

14.8 Permanent loads

14.8.1 General
The evaluation of the load-carrying capacity of existing bridges shall take into consideration all permanent loads except as specified in Clause 14.8.4.

14.8.2 Dead load

14.8.2.1 General
Dead load shall include the weight of all components of the bridge, fill, utilities, and other materials permanently on the bridge. Dead loads shall be determined from bridge drawings and verified with field measurements in accordance with Clause 14.6.

Dead load shall be apportioned to three categories, D1, D2, and D3, as follows:
(a) D1: dead load of factory-produced components and cast-in-place concrete, excluding decks;
(b) D2: cast-in-place concrete decks (including voided decks and cementitious concrete overlays), wood, field-measured bituminous surfacing, and non-structural components; and
(c) D3: bituminous surfacing where the nominal thickness is assumed to be 90 mm for the evaluation.

14.8.2.2 Dead load distribution

The transverse distribution of dead load shall be in accordance with Section 5.

14.8.3 Earth pressure and hydrostatic pressure

Earth pressure and hydrostatic pressure shall be considered in the evaluation, treated as permanent loads, and multiplied by a load factor in accordance with Clause 14.13.2.2.

14.8.4 Shrinkage, creep, differential settlement, and bearing friction

Shrinkage, creep, differential settlement, and bearing friction need not be considered in evaluation at ultimate limit states if their effects induce ductile behaviour. When their effects induce non-ductile behaviour, they shall have load factors determined in accordance with Clause 14.13.2.3.

14.8.5 Secondary effects from prestressing

Secondary effects from prestressing shall be considered as permanent loads and multiplied by load factors in accordance with Clause 3.5.1.

14.9 Transitory loads

14.9.1 Normal traffic

14.9.1.1 General

Bridges shall be evaluated for the following:

(a) a vehicle train, a two-unit vehicle, and a single-unit vehicle, as specified in Clause 14.9.1.2, 14.9.1.3, and 14.9.1.4, respectively; or

(b) alternative loading based on local traffic, as specified in Clause 14.9.1.6.

Truck axles and portions of uniformly distributed lane load that reduce the load effect shall be neglected.

14.9.1.2 Evaluation Level 1 (vehicle trains)

A bridge required to carry vehicle trains in normal traffic shall be evaluated to Evaluation Level 1, for which the live load model shall be the CL1-W Truck or Lane Load shown in Figure 14.1, where W is the gross vehicle weight in kilonewtons of a vehicle train legally permitted on bridges without a permit.

The value of W shall be taken as 625 unless the Regulatory Authority uses lesser or greater values of W where traffic conditions are expected to differ from the norm.

In Ontario, the load used for Evaluation Level 1 shall be the CL1-625-ONT Truck Load or CL1-625-ONT Lane Load specified in Annex A14.2.

14.9.1.3 Evaluation Level 2 (two-unit vehicles)

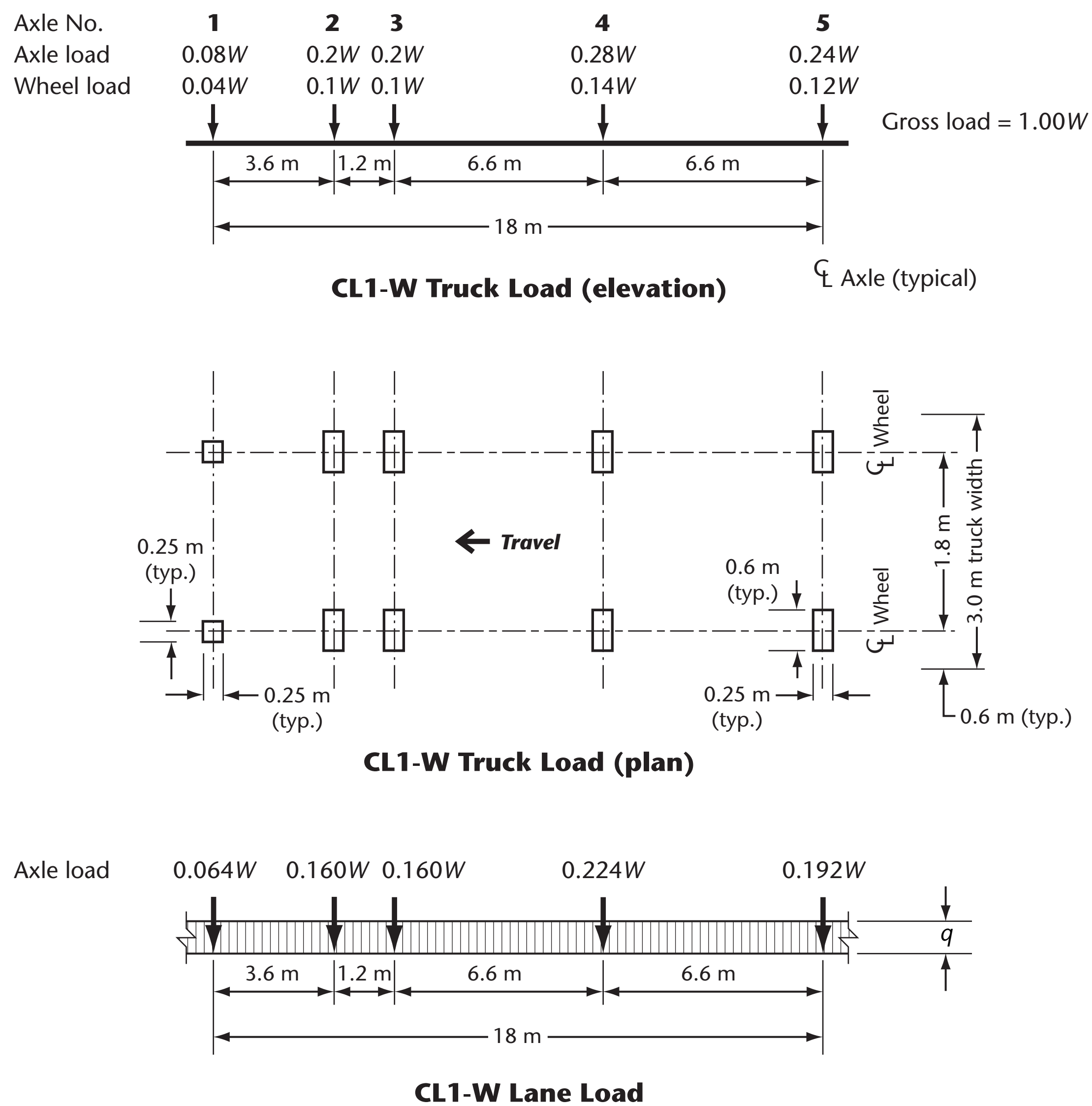
A bridge shall be evaluated to Evaluation Level 2 when load restrictions are to be applied and the bridge is required to carry two-unit vehicles. The live load model shall be the CL2-W Truck or Lane Load shown in Figure 14.2, where W is as specified in Clause 14.9.1.2.

In Ontario, the load used for Evaluation Level 2 shall be the CL2-625-ONT Truck Load or the CL2-625-ONT Lane Load specified in Annex A14.3.

14.9.1.4 Evaluation Level 3 (single-unit vehicles)

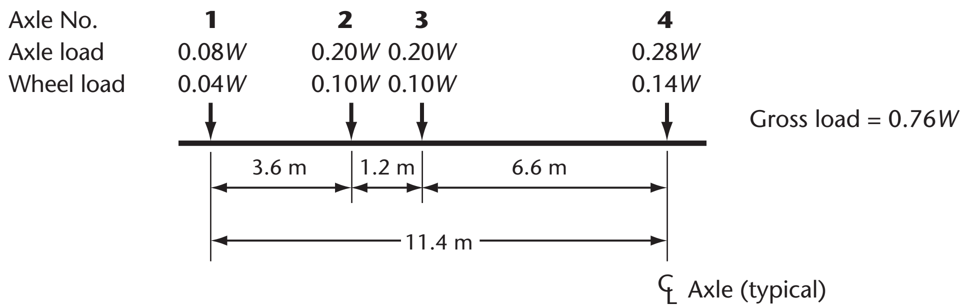
A bridge shall be evaluated to Evaluation Level 3 when load restrictions are to be applied and the bridge is required to carry single-unit vehicles. The live load model shall be the CL3-W Truck or Lane Load shown in Figure 14.3, where W is as specified in Clause 14.9.1.2.

In Ontario, the load used for Evaluation Level 3 shall be the CL3-625-ONT Truck Load or the CL3-625-ONT Lane Load specified in Annex A14.4.

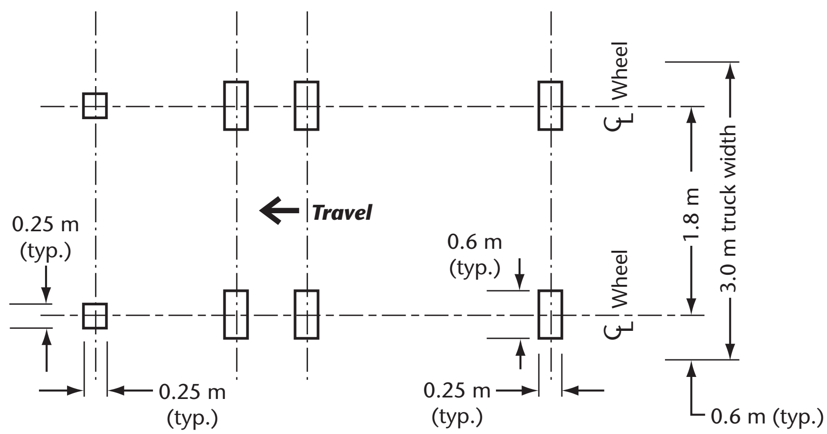


Note: *The values of the uniformly distributed load, q, for each highway class (see Section 1) are as follows:*
(a) Class A: 9 kN/m*;*
(b) Class B: 8 kN/m*; and*
(c) Class C or D: 7 kN/m.

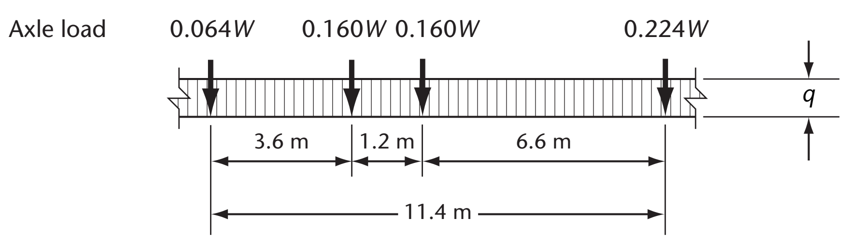
Figure 14.1
Level 1 evaluation loads with CL1-W Truck
(See Clauses 14.9.1.2 and 14.9.1.7.)



CL2-W Truck Load (elevation)



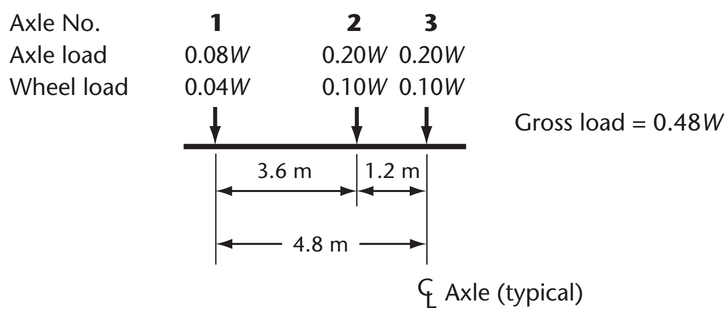
CL2-W Truck Load (plan)



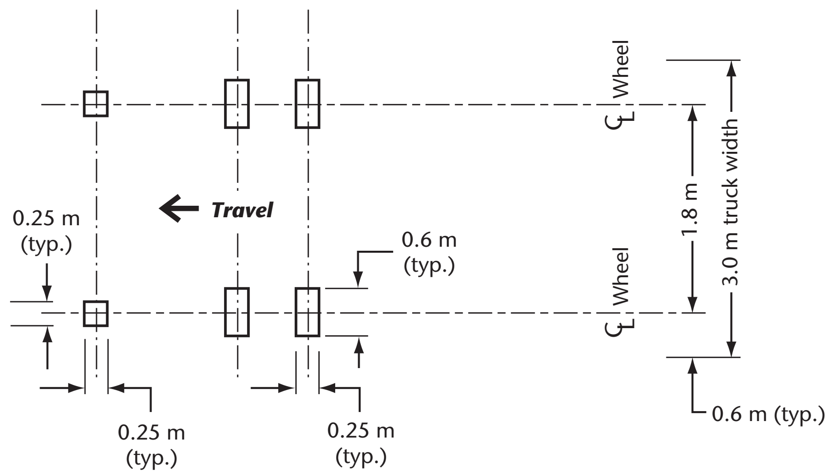
CL2-W Lane Load

Note: *The values of the uniformly distributed load, q, for each highway class (see Section 1) are as follows:*
(a) Class A: 9 kN/m*;*
(b) Class B: 8 kN/m*; and*
(c) Class C or D: 7 kN/m.

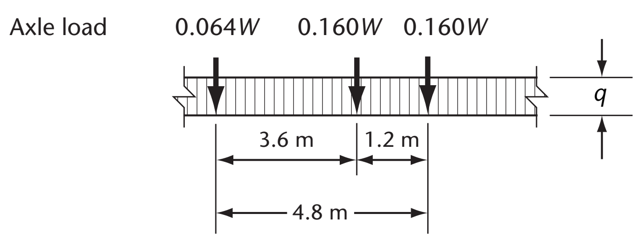
Figure 14.2
Level 2 evaluation loads with CL2-W Truck
(See Clauses 14.9.1.3 and 14.9.1.7.)



CL3-W Truck Load (elevation)



CL3-W Truck Load (plan)



CL3-W Lane Load

Note: *The values of the uniformly distributed load, q, for each highway class (see Section 1) are as follows:*
(a) Class A: 9 kN/m*;*
(b) Class B: 8 kN/m*; and*
(c) Class C or D: 7 kN/m.

Figure 14.3
Level 3 evaluation loads with CL3-W Truck
(See Clauses 14.9.1.4 and 14.9.1.7.)

14.9.1.5 Configuration of evaluation load models

The wheel spacings, wheel footprints, and clearances of the Evaluation Level 1, 2, and 3 Trucks shall be as specified in Clause 14.9.1.2, 14.9.1.3, and 14.9.1.4, respectively. The uniformly distributed load in a lane load shall occupy a width of 3.0 m in a traffic lane.

14.9.1.6 Alternative loading

As an alternative to the CL1-W, CL2-W, and CL3-W loadings in Clauses 14.9.1.2 to 14.9.1.4, the traffic load for evaluation may be based on local vehicle and traffic conditions.

The load factor, α_L, to be used with the alternative loading shall be in accordance with Table 14.9.

The alternative loading to be used for evaluation shall be the more severe of

(a) the vehicle for which the bridge is being evaluated, with dynamic load allowance in accordance with Clause 3.8.4.5; or

(b) 80% of the vehicle for which the bridge is being evaluated, plus a superimposed uniformly distributed load of 9, 8, 7, and 7 kN/m for highway classes A, B, C, and D, respectively, without dynamic load allowance for either vehicle or uniformly distributed loads.

14.9.1.7 Dynamic load allowance for normal traffic

For the Truck models shown in Figures 14.1 to 14.3, A14.2.1, A14.3.1, and A14.4.1, the dynamic load allowance shall be in accordance with Clause 3.8.4.5.

For the Lane Load models shown in Figures 14.1 to 14.3, A14.2.1, A14.3.1, and A14.4.1, no dynamic load allowance shall be applied to the Truck or the uniformly distributed load.

14.9.2 Permit — Vehicle loads

14.9.2.1 General

Vehicles operating under permit shall be classified as PA, PB, PC, or PS in accordance with Clauses 14.9.2.2 to 14.9.2.5.

14.9.2.2 Permit — Annual or project (PA)

PA traffic shall include the vehicles authorized by permit on an annual basis or for the duration of a specific project to carry an indivisible load, mixed with other traffic without supervision. Individual axle loads and the gross vehicle weight may exceed the non-permit legislated limits.

For the lane carrying the PA vehicle, the load effect shall be calculated from the more severe of

(a) the permit vehicle alone in the lane with dynamic load allowance, in accordance with Clause 14.9.3; or

(b) 85% of the permit vehicle, plus a superimposed uniformly distributed load of 9, 8, 7, and 7 kN/m for highway classes A, B, C, and D, respectively, without dynamic load allowance for either Truck or uniformly distributed loads.

14.9.2.3 Permit — Bulk haul (PB)

PB traffic shall include bulk haul divisible load traffic authorized by permit programs for many trips, mixed with general traffic. Axle loads shall not exceed the non-permit legislated limits, but gross vehicle weights may exceed such limits. Axle spacings may be less than the legislated limits. Permit limits on axle loads and gross vehicle weights shall be strictly enforced.

For the lane carrying the PB vehicle, the load effect shall be calculated from the more severe of

(a) the permit vehicle alone in the lane with dynamic load allowance, in accordance with Clause 14.9.3; or

(b) 80% of the permit vehicle, plus a superimposed uniformly distributed load of 9, 8, 7, and 7 kN/m for highway classes A, B, C, and D, respectively, without dynamic load allowance for either Truck or uniformly distributed loads.

14.9.2.4 Permit — Controlled (PC)

PC traffic shall include the vehicles authorized by permit to carry an indivisible load on a specified route under supervision and specified travel conditions. The weights and spacings of the axles shall be verified by measurement.

Lane load need not be considered for PC traffic if other traffic is excluded from the bridge during passage of the PC vehicle.

14.9.2.5 Permit — Single trip (PS)

PS traffic shall include vehicles authorized by permit for a single trip to carry an indivisible load, mixed with other traffic without supervision. Axle loads and the gross vehicle weight may exceed the non-permit legislated limits.

For the lane carrying the PS vehicle, the load effect shall be calculated from the more severe of

(a) the permit vehicle alone in the lane with dynamic load allowance, in accordance with Clause 14.9.3; or

(b) 85% of the permit vehicle, plus a superimposed uniformly distributed load of 9, 8, 7, and 7 kN/m for highway classes A, B, C, and D, respectively, without dynamic load allowance for either Truck or uniformly distributed loads.

14.9.3 Dynamic load allowance for permit vehicle loads and alternative loading

The dynamic load allowance for permit vehicle loads shall be as specified in Clause 3.8.4.5, except that an "axle" in Clause 3.8.4.5.3 shall be interpreted as an "axle group", i.e., single axle, tandem, or tridem, for the purposes of this Clause.

For a permit vehicle crossing the bridge at a restricted speed, the dynamic load allowance so calculated shall be multiplied by

(a) 0.30 for a vehicle speed of 10 km/h or less;

(b) 0.50 for a vehicle speed greater than 10 km/h and less than or equal to 25 km/h;

(c) 0.75 for a vehicle speed greater than 25 km/h and less than or equal to 40 km/h; and

(d) 1.00 for a vehicle speed greater than 40 km/h.

14.9.4 Multiple-lane loading

14.9.4.1 Design lanes

The number of loaded lanes shall be determined in accordance with the current or intended use of the bridge. Where the traffic lanes are clearly designated on the bridge, they shall be used as design lanes, except that outer lanes shall include the adjacent shoulders.

14.9.4.2 Normal traffic

The modification factors for multiple-lane loading shall be as specified in Table 14.3.

Table 14.3
Modification factors for multiple-lane loading
(See Clause 14.9.4.2.)

Number of lanes loaded	Highway class A	B	C or D
1	1.00	1.00	1.00
2	0.90	0.90	0.85
3	0.80	0.80	0.70
4	0.70	0.70	—
5	0.60	—	—
6 or more	0.55	—	—

14.9.4.3 Permit vehicle with normal traffic
When the permit vehicle is allowed to travel with normal traffic, the loading to be applied in the other lanes shall be taken as a fraction of the CL1-W loading or CL1-625-ONT loading, as specified in Table 14.4.

Table 14.4
Fraction of CL1-W loading to be applied in the other lanes
(See Clause 14.9.4.3.)

	Highway class A	B	C or D
Second loaded lane	0.7	0.6	0.5
Third and subsequent loaded lanes	0.4	0.4	0.4

14.9.5 Loads other than traffic

14.9.5.1 Sidewalk loading
Except for sidewalk components, sidewalk loading shall not be considered coincident with traffic loading unless the evaluator has reason to suppose that significant sidewalk loading will occur coincident with maximum traffic loading, in which case the pedestrian loading specified in Clause 3.8.9 shall be used with the same load factor specified in Clause 14.13.3 for traffic.

14.9.5.2 Snow loads
If significant snow loading on sidewalks is expected, it shall be considered in the evaluation.

14.9.5.3 Wind loads
Wind loads are not specifically considered in this Section. If the evaluator considers that significant wind forces could occur simultaneously with the maximum traffic loads, these wind forces shall be considered in accordance with Clause 3.10.

14.9.5.4 Temperature effects
Temperature effects need not be considered at ultimate limit states for any element that will behave in a ductile manner. When non-ductile behaviour is expected, temperature effects shall be considered in accordance with Clause 3.9.4.

14.9.5.5 Secondary effects
Secondary effects (excluding secondary effects from prestressing) need not be considered in evaluation at ultimate limit states if their effects induce ductile behaviour. When secondary effects induce non-ductile behaviour, they shall be considered and multiplied by a load factor in accordance with Clause 3.5.1.

14.9.5.6 Seismic loads
Bridges in regions prone to earthquakes shall be evaluated for seismic conditions in accordance with Section 4.

14.10 Exceptional loads
Loads (other than traffic loads) that occur on rare occasions and are of significant magnitude shall be considered exceptional loads and shall be evaluated in accordance with Sections 1 to 13 and 16 or, when not covered by Sections 1 to 13 and 16, in accordance with good engineering practice.

14.11 Lateral distribution categories for live load

14.11.1 General
The method to be used in calculating the lateral distribution of live loads to the elements considered shall be categorized as statically determinate, sophisticated, or simplified in accordance with Clauses 14.11.2 to 14.11.4.

14.11.2 Statically determinate method
In this method the lateral distribution is statically determinate.

14.11.3 Sophisticated method
In this method the lateral distribution is statically indeterminate and is calculated in accordance with a sophisticated method of analysis such as the grillage analogy, orthotropic plate theory, finite element, finite strip, or folded plate method.

14.11.4 Simplified method
In this method the lateral distribution is calculated in accordance with the simplified methods of Section 5. However, it is possible that the methods specified in Section 5 will not be suitable for non-standard bridges or permit vehicle loads (especially those that are wider than the CL-W vehicles), in which cases such methods shall not be used.

14.12 Target reliability index

14.12.1 General
For all evaluation levels, the target reliability index, β, shall be taken from Table 14.5 for PA, PB, and PS traffic and Table 14.6 for PC traffic. In both cases, the system behaviour, element behaviour, and inspection level shall be as specified in Clauses 14.12.2 to 14.12.4.

14.12.2 System behaviour

System behaviour shall take into consideration the effect of any existing deterioration and shall be classified into one of the following categories:

(a) Category S1, where element failure leads to total collapse. This includes failure of main members with no benefit from continuity or multiple-load paths, e.g., a simply supported girder in a two-girder system.

(b) Category S2, where element failure probably will not lead to total collapse. This includes main load-carrying members in a multi-girder system or continuous main members in bending.

(c) Category S3, where element failure leads to local failure only. This includes deck slabs, stringers, and bearings in compression.

14.12.3 Element behaviour

Element behaviour shall take into consideration the effect of any existing deterioration and shall be classified into one of the following categories:

(a) Category E1, where the element being considered is subject to sudden loss of capacity with little or no warning. This can include failure by buckling, concrete in shear and/or torsion with less than the minimum reinforcement required by Clause 14.14.1.6.2(a), bond (pullout) failure, suspension cables, eyebars, bearing stiffeners, over-reinforced concrete beams, connections, concrete beam-column compression failure, and steel in tension at net section.

(b) Category E2, where the element being considered is subject to sudden failure with little or no warning but will retain post-failure capacity. This can include concrete in shear and/or torsion with at least the minimum reinforcement required by Clause 14.14.1.6.2(a), and steel plates in compression with post-buckling capacity.

(c) Category E3, where the element being considered is subject to gradual failure with warning of probable failure. This can include steel beams in bending or shear, under-reinforced concrete in bending, decks, and steel in tension at gross section.

14.12.4 Inspection level

Evaluation shall not be undertaken without inspection. Inspection levels shall be classified as follows:

(a) Inspection Level INSP1, where a component is not inspectable. This can include hidden members not accessible for inspection, e.g., interior webs of adjacent box beams.

(b) Inspection Level INSP2, where inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator.

(c) Inspection Level INSP3, where the evaluator has directed the inspection of all critical and substandard components and final evaluation calculations account for all information obtained during this inspection.

14.12.5 Important structures

For structures that can affect the life or safety of people under or near a bridge, are essential to the local economy, or are designated as emergency route bridges (in accordance with Clause 4.4.2), a value of β greater than that specified in Table 14.5 or 14.6 shall be used if directed by the Regulatory Authority.

Table 14.5
Target reliability index, β, for normal traffic and for PA, PB, and PS traffic
(See Clauses 14.12.1 and 14.12.5.)

System behaviour category	Element behaviour category	Inspection level		
		INSP1	INSP2	INSP3
S1	E1	4.00	3.75	3.75
	E2	3.75	3.50	3.25
	E3	3.50	3.25	3.00
S2	E1	3.75	3.50	3.50
	E2	3.50	3.25	3.00
	E3	3.25	3.00	2.75
S3	E1	3.50	3.25	3.25
	E2	3.25	3.00	2.75
	E3	3.00	2.75	2.50

Table 14.6
Target reliability index, β, for PC traffic
(See Clauses 14.12.1 and 14.12.5.)

System behaviour category	Element behaviour category	Inspection level		
		INSP1	INSP2	INSP3
S1	E1	3.50	3.25	3.25
	E2	3.25	3.00	2.75
	E3	3.00	2.75	2.50
S2	E1	3.25	3.00	3.00
	E2	3.00	2.75	2.50
	E3	2.75	2.50	2.25
S3	E1	3.00	2.75	2.75
	E2	2.75	2.50	2.25
	E3	2.50	2.25	2.00

14.13 Load factors

14.13.1 General

The unfactored load effects for each element under consideration shall be multiplied by the appropriate load factors specified in Clauses 14.13.2 and 14.13.3 for the value of β determined in accordance with Clause 14.12 for the element under consideration.

14.13.2 Permanent loads

14.13.2.1 Dead load

When the dead load effect counteracts the effect due to transitory load, the minimum dead load factors specified in Section 3 shall be used for all dead load categories at any β value. Otherwise, the dead load factors specified in Table 14.7 shall apply.

Table 14.7
Maximum dead load factors, α_D
(See Clause 14.13.2.1.)

	Target Reliability Index, β								
Dead load category	**2.00**	**2.25**	**2.50**	**2.75**	**3.00**	**3.25**	**3.50**	**3.75**	**4.00**
D1	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10	1.11
D2	1.06	1.08	1.10	1.12	1.14	1.16	1.18	1.20	1.22
D3	1.15	1.20	1.25	1.30	1.35	1.40	1.45	1.50	1.55

14.13.2.2 Earth pressure and hydrostatic pressure

The load factors for earth pressure and hydrostatic pressure shall be in accordance with Clause 3.5.1.

14.13.2.3 Temperature, shrinkage, creep, differential settlement, and bearing friction

If consideration of temperature, shrinkage, creep, differential settlement, or bearing friction is required by Clause 14.8.4 or 14.9.5.4, the load factors shall be in accordance with Clause 3.5.1.

14.13.2.4 Secondary effects from prestressing

The load factors for secondary effects from prestressing shall be in accordance with Clause 14.9.5.5.

14.13.3 Transitory loads

14.13.3.1 Normal traffic

The live load factors for normal traffic Evaluation Levels 1, 2, and 3 shall be as specified in Table 14.8. The live load factors for alternative loading as specified in Clause 14.9.1.6 shall be as specified in Table 14.9. For Table 14.9, "Short span" load factors shall be used for shear effects in beams with a span up to 6 m, moment effects in beams with a span up to 10 m, and shears and moments in floor beams where the tributary spans are up to 6 m. For all other conditions, "Other span" load factors shall be used.

Table 14.8
Live load factors, α_L, for normal traffic (Evaluation Levels 1, 2, and 3) for all types of analysis
(See Clause 14.13.3.1.)

	Target reliability index, β						
Spans	**2.50**	**2.75**	**3.00**	**3.25**	**3.50**	**3.75**	**4.00**
All Spans	1.35	1.42	1.49	1.56	1.63	1.70	1.77

Table 14.9
Live load factors, α_L, for normal traffic (alternative loading) for all types of analysis
(See Clauses 14.9.1.6 and 14.13.3.1.)

	Target reliability index, β						
Spans	**2.50**	**2.75**	**3.00**	**3.25**	**3.50**	**3.75**	**4.00**
Short spans	1.80	1.90	2.00	2.10	2.20	2.30	2.40
Other spans	1.35	1.42	1.49	1.56	1.63	1.70	1.77

14.13.3.2 Permit vehicle loads

The live load factors for permit vehicles shall be as specified in Tables 14.10 to 14.13, with "Short spans" and "Other spans" as specified in Clause 14.13.3.1.

Table 14.10
Live load factors, α_L, for PA traffic
(See Clause 14.13.3.2.)

		Target reliability index, β						
Type of analysis	**Spans**	**2.50**	**2.75**	**3.00**	**3.25**	**3.50**	**3.75**	**4.00**
Statically determinate	Short spans	1.42	1.48	1.53	1.59	1.65	1.71	1.77
	Other spans	1.27	1.32	1.37	1.42	1.48	1.53	1.59
Sophisticated	Short spans	1.45	1.51	1.58	1.64	1.71	1.78	1.85
	Other spans	1.29	1.34	1.39	1.45	1.51	1.57	1.63
Simplified (Section 5)	Short spans	1.48	1.55	1.62	1.70	1.78	1.87	1.96
	Other spans	1.28	1.34	1.40	1.47	1.53	1.60	1.67

Table 14.11
Live load factors, α_L, for PB traffic
(See Clause 14.13.3.2.)

Type of analysis	Spans	Target reliability index, β						
		2.50	2.75	3.00	3.25	3.50	3.75	4.00
Statically determinate	Short spans	1.15	1.19	1.23	1.28	1.33	1.38	1.43
	Other spans	1.10	1.12	1.16	1.21	1.26	1.30	1.36
Sophisticated	Short spans	1.17	1.22	1.27	1.32	1.38	1.43	1.49
	Other spans	1.10	1.13	1.18	1.23	1.28	1.33	1.39
Simplified (Section 5)	Short spans	1.19	1.25	1.31	1.37	1.44	1.50	1.57
	Other spans	1.10	1.13	1.19	1.24	1.30	1.36	1.42

Table 14.12
Live load factors, α_L, for PC traffic
(See Clause 14.13.3.2.)

Type of analysis	Spans	Target reliability index, β						
		2.00	2.25	2.50	2.75	3.00	3.25	3.50
Statically determinate	Short spans	1.11	1.15	1.19	1.24	1.28	1.33	1.38
	Other spans	1.10	1.10	1.10	1.13	1.18	1.23	1.28
Sophisticated	Short spans	1.12	1.17	1.22	1.27	1.32	1.37	1.43
	Other spans	1.10	1.10	1.10	1.14	1.19	1.24	1.30
Simplified (Section 5)	Short spans	1.13	1.18	1.24	1.30	1.36	1.42	1.49
	Other spans	1.10	1.10	1.10	1.13	1.19	1.25	1.31

Table 14.13
Live load factors, α_L, for PS traffic
(See Clause 14.13.3.2.)

Type of analysis	Spans	Target reliability index, β						
		2.50	2.75	3.00	3.25	3.50	3.75	4.00
Statically determinate	Short spans	1.34	1.39	1.44	1.49	1.55	1.61	1.67
	Other spans	1.20	1.24	1.29	1.34	1.39	1.44	1.50
Sophisticated	Short spans	1.36	1.42	1.48	1.54	1.60	1.67	1.74
	Other spans	1.21	1.26	1.31	1.36	1.42	1.48	1.54
Simplified (Section 5)	Short spans	1.38	1.45	1.52	1.60	1.67	1.75	1.84
	Other spans	1.20	1.26	1.32	1.38	1.44	1.51	1.57

14.14 Resistance

14.14.1 General

14.14.1.1 General

The factored resistances of concrete, structural steel, and wood components shall be determined in accordance with the applicable Sections of this Code. Components that do not meet the limitations on which the resistance calculations of this Code are based shall have their resistances calculated in accordance with alternative procedures based on established and generally recognized theories, analyses, and engineering judgment.

14.14.1.2 Prestressed concrete using stress-relieved strand or wire

14.14.1.2.1 General

The requirements of Section 8 for low-relaxation strand or wire shall be followed for the evaluation of prestressed concrete bridges using stress-relieved strand or wire, except as modified by Clauses 14.14.1.2.2 to 14.14.1.2.4.

14.14.1.2.2 Prestressing steel stress limitations

Stresses at jacking or transfer shall be based on data given on the Plans. In the absence of such data, the following stress limitations, for both pretensioning and post-tensioning, shall be used:

(a) at jacking: $0.80f_{pu}$; and

(b) at transfer: $0.70f_{pu}$.

14.14.1.2.3 Loss of prestress

14.14.1.2.3.1 At transfer

In pretensioned components, the relaxation loss in prestressing steel, REL_1, initially stressed in excess of $0.5f_{pu}$ shall be calculated as follows:

$$REL_1 = \frac{\log(24t)}{10}\left[\frac{f_{sj}}{f_{py}} - 0.55\right]f_{sj}$$

14.14.1.2.3.2 After transfer

The loss of prestress due to relaxation after transfer, REL_2, shall be calculated as follows:

$$REL_2 = \left[\left[\frac{f_{st}}{f_{pu}} - 0.52\right]\left[0.42 - \frac{CR + SH}{1.25f_{pu}}\right]\right]f_{pu} \geq 0.01f_{pu}$$

14.14.1.2.4 Prestressing steel stress at the ultimate limit state

The steel stress, f_{ps}, at the ultimate limit state in bonded prestressing steel shall be calculated using a method based on sectional strain compatibility and using stress-strain curves representative of the steel, except that if $f_{se} \geq 0.5f_{pu}$, the value of f_{ps} may be calculated as follows:

$$f_{ps} = f_{pu}\left[1 - 0.5\frac{\mu_p f_{pu}}{f_c'}\right]$$

14.14.1.3 Concrete deck slabs

14.14.1.3.1 General

When the concrete deck slab is at least 175 mm thick and the requirements for the empirical design method in accordance with Clause 8.18.4 are satisfied, the deck slab shall be deemed to have adequate resistance for the CL loadings specified with a value of W not greater than 625 in Clauses 14.9.1.2 to 14.9.1.4. When the concrete deck slab is less than 175 mm thick or the requirements of Clause 8.18.4 are not satisfied, an evaluation of the deck slab shall be carried out in accordance with Clauses 14.14.1.3.2 and 14.14.1.3.3.

14.14.1.3.2 Rigorous method

If any of the conditions specified in Items (a) to (e) are not satisfied, the factored resistance shall be determined in accordance with Section 8 and expressed as an equivalent wheel load:

(a) the centre-to-centre spacing of the supporting beams for a slab panel does not exceed 4.5 m and the slab extends sufficiently beyond the external beams to provide full development length for the bottom transverse reinforcement;

(b) the ratio of the spacing of the supporting beams to the thickness of the slab does not exceed 20;

(c) the minimum slab thickness of sound concrete is at least 150 mm (with the minimum slab thickness used for slabs of variable thickness);

(d) all cross-frames or diaphragms extend throughout the cross-section of the bridge between external girders and the maximum spacing of such cross-frames or diaphragms is in accordance with Clause 8.18.5; and

(e) edge stiffening is in accordance with Clause 8.18.6.

14.14.1.3.3 Simplified method

If all of the conditions of Clause 14.14.1.3.2 are satisfied, the value of the factored resistance, R_r, shall be calculated as follows:

$$R_r = \phi_{md} R_n$$

where

ϕ_{md} = 0.5

The values of R_n for both composite and non-composite concrete deck slabs shall be calculated as follows:

$$R_n = R_d F_q F_c$$

where R_d is taken from Figure 14.4 or 14.5, as applicable, for the deck thickness, d, and the deck span being considered; F_q is a correction factor based on q, where $q = 50\,(A_{s\ell}/bd_\ell + A_{st}/bd_t)$; and F_c is a correction factor based on f_c'.

The values of F_q and F_c shall be taken from Figure 14.4 or 14.5, as applicable, or obtained from those figures by linear interpolation.

For deck thicknesses other than those shown in Figures 14.4 and 14.5, the value of R_n shall be obtained by linear interpolation.

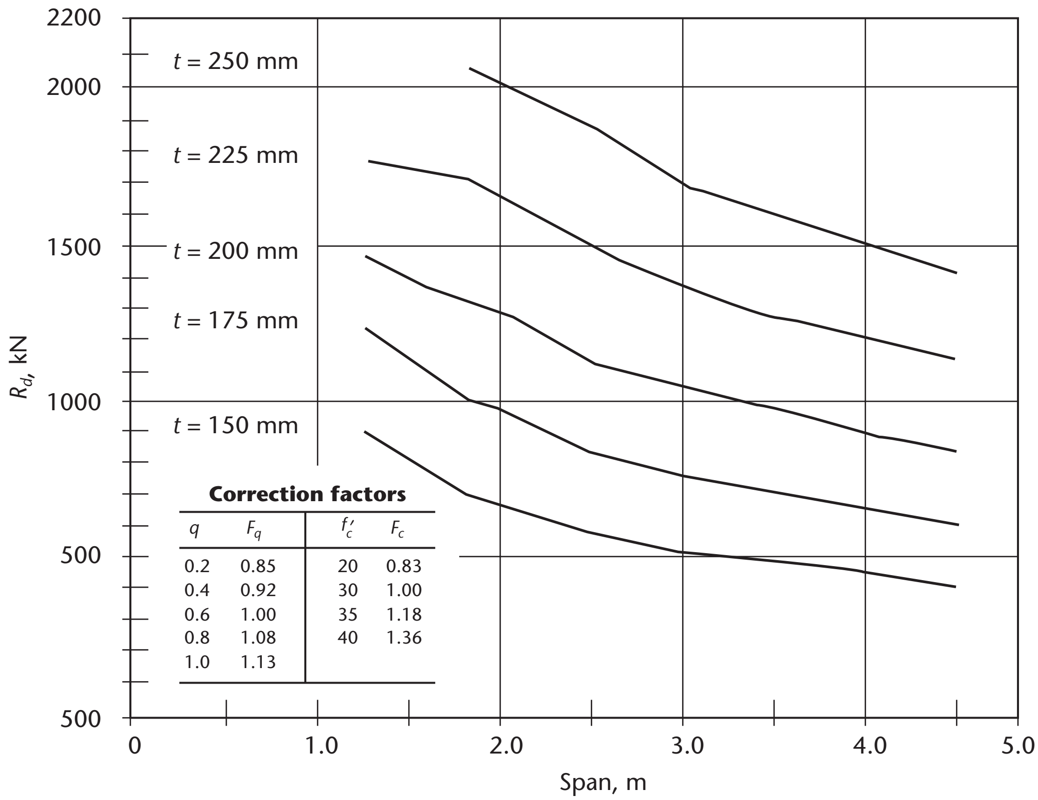


Figure 14.4
Deck punching shear for composite slabs
(See Clause 14.4.1.3.3.)

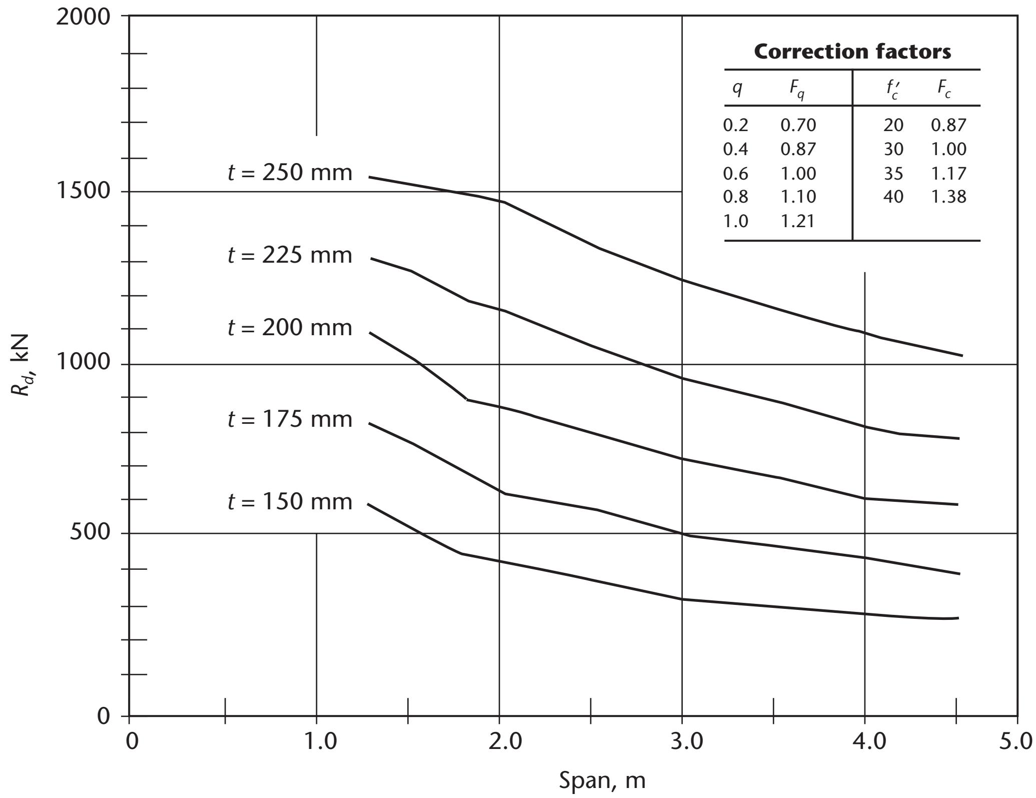


Figure 14.5
Deck punching shear for non-composite slabs
(See Clause 14.14.1.3.3.)

14.14.1.4 Rivets

14.14.1.4.1 Rivets in tension

The factored tensile resistance, T_r, of a riveted joint in tension shall be taken as

$$T_r = \phi_r n A_r F_u$$

where

F_u = specified tensile strength of the rivet steel

ϕ_r = 0.67

Rivets shall be able to resist the sum of the external load and any additional tensile load caused by deformation of the connected parts.

14.14.1.4.2 Rivets in shear

The factored resistance of a riveted connection subject to shear shall be taken as the lesser of the following:

(a) the factored bearing resistance, B_r, calculated as follows:

$$B_r = \phi_{mc} t n e F_u \leq 3\phi_{mc} t n d F_u$$

where

F_u = smaller of the specified tensile strengths of the connected parts

ϕ_{mc} = 0.67

(b) the factored shear resistance, V_r, calculated as follows:

$$V_r = 0.75\phi_r nmA_r F_u$$

where

F_u = specified tensile strength of the rivet steel

ϕ_r = 0.67

14.14.1.4.3 Rivets in shear and tension

A rivet that is required to develop resistance simultaneously to a tensile force and a shear force that result from loads at the ultimate limit state shall satisfy the following relationship:

$$V_r^2 + 0.56T_f^2 \leq 0.56(\phi_r A_r F_u)^2$$

where

F_u = specified tensile strength of the rivet steel

ϕ_r = 0.67

14.14.1.5 Masonry

The unfactored resistance, P_n, of a masonry arch shall be calculated as the axle load that alone or in combination with other axle loads results in the minimum value of resistance of the arch based on a mechanism analysis of the arch. The passive resistance of the fill within the spandrel shall be taken into account in the analysis.

The factored resistance, P_r, of a masonry arch shall be calculated as follows:

$$P_r = \phi_{mm} P_n$$

The value of ϕ_{mm} shall be taken as 0.80. However, for arches with soft mortar in the joints and in which the joint width exceeds 5% of the stone depth, ϕ_{mm} shall be taken as 0.70.

14.14.1.6 Shear in concrete beams

14.14.1.6.1 General

Concrete beams shall have their shear resistance calculated in accordance with Clause 8.9.3, except as modified in Clauses 14.14.1.6.2 and 14.14.1.6.3.

14.14.1.6.2 Transverse reinforcement area and spacing

In lieu of the requirements of Clauses 8.9.1.3 and 8.14.6, the following transverse reinforcement requirements shall apply for use in Clauses 8.9.3.6 and 8.9.3.7:

(a) The section shall have the shear resistance calculated as a section satisfying the minimum transverse reinforcement if

(i) $A_v \geq 0.15 f_{cr} \dfrac{(b_v s)}{f_y}$;

(ii) $s \leq s_{m1}$, where s_{m1} is obtained from Figure 14.6; and

(iii) $s/d_v < s_{r1}$, where s_{r1} is obtained from Figure 14.7.

(b) The section shall have the shear resistance calculated as a section with no transverse reinforcement if

(i) $A_v \le 0.05 f_{cr} \dfrac{(b_v s)}{f_y}$;

(ii) $s \ge s_{m2}$, where s_{m2} is obtained from Figure 14.6; or

(iii) $s/d_v \ge s_{r2}$, where s_{r2} is obtained from Figure 14.7.

(c) The section shall have the shear resistance calculated by linear interpolation of the shear resistances obtained from Items (a) and (b) if

(i) $A_v \ge 0.05 f_{cr} \dfrac{(b_v s)}{f_y}$;

(ii) $s \le s_{m2}$, where s_{m2} is obtained from Figure 14.6; and

(iii) $s/d_v \le s_{r2}$, where s_{r2} is obtained from Figure 14.7.

The interpolation shall be based on the most severe of A_v between 0.15 and 0.05, s between s_{m1} and s_{m2}, and s/d_v between s_{r1} and s_{r2}.

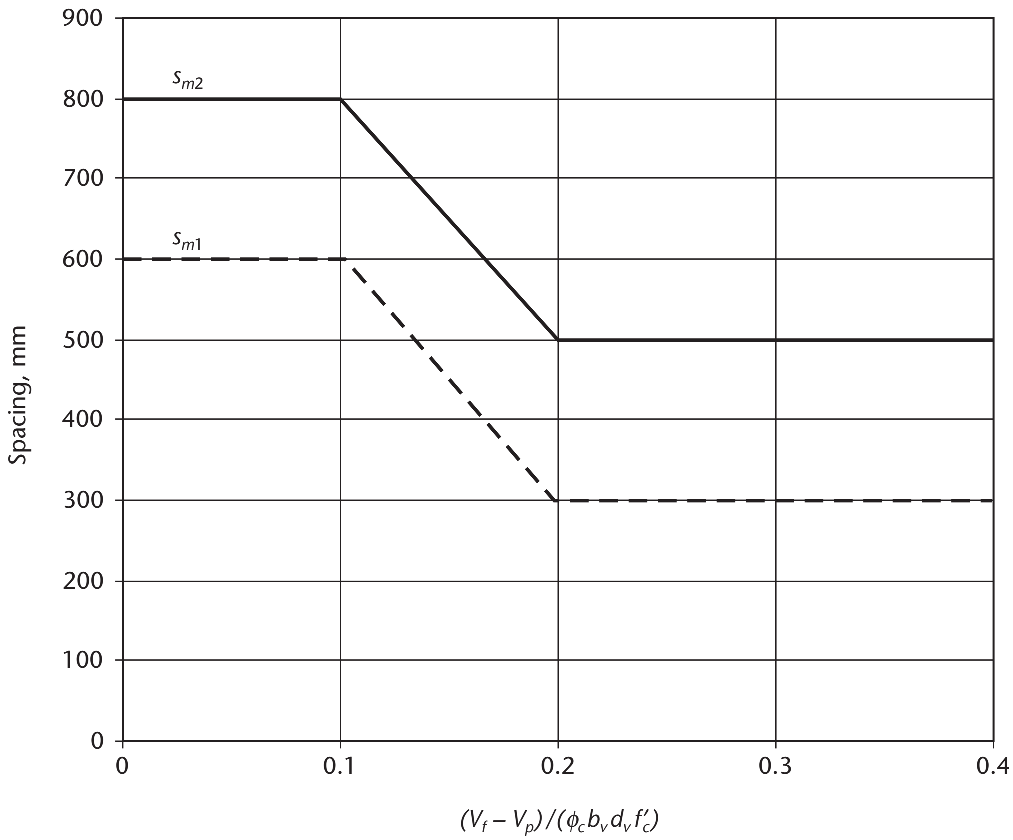


Figure 14.6
Spacing requirements for minimum reinforcement, mm
(See Clause 14.14.1.6.2.)

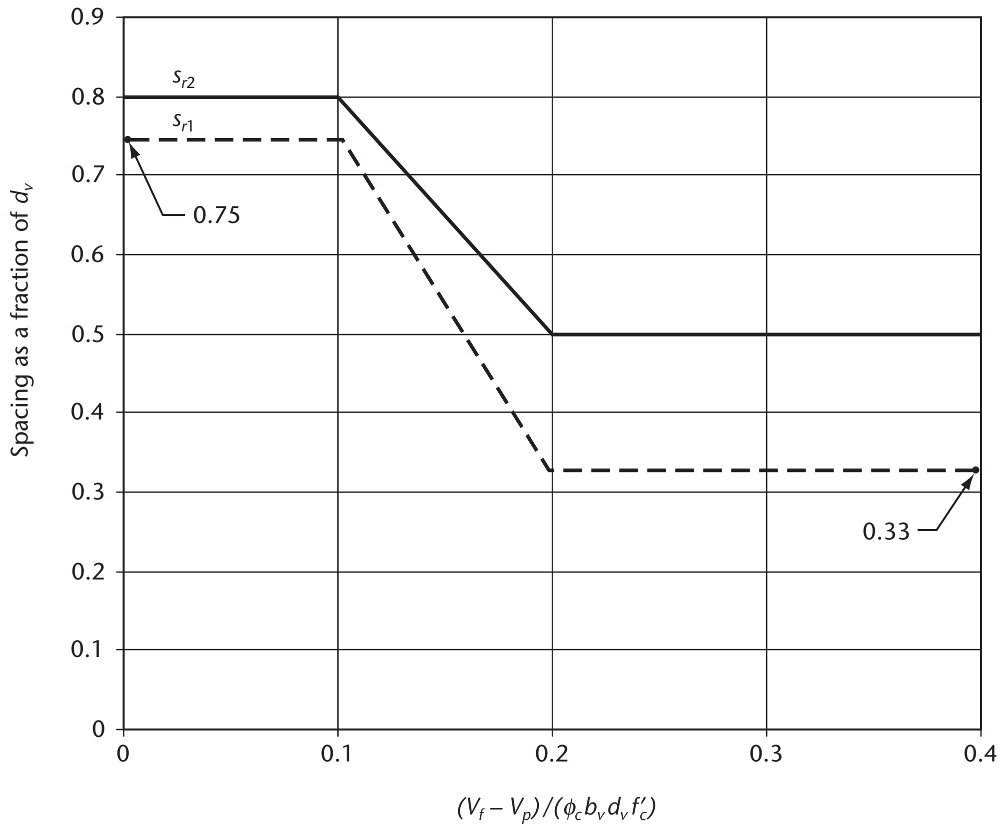


Figure 14.7
Spacing requirements for minimum reinforcement as a fraction of shear depth
(See Clause 14.14.1.6.2.)

14.14.1.6.3 Proportioning of transverse reinforcement

14.14.1.6.3.1 General

For the purpose of evaluation, Clause 14.14.1.6.3.2 shall apply in lieu of Clause 8.9.3.9.

14.14.1.6.3.2 Amount of transverse reinforcement

Provided that the cross-section does not change abruptly within a length equal to the member depth, h, and the load is applied through the top face of the beam, the amount of transverse reinforcement, $A_v h/s$, may be taken as the total amount calculated within the length h. This length shall be measured from the section of interest toward the support.

14.14.1.7 Wood

14.14.1.7.1 General

The resistances for beam and stringer grade and post and timber grade wood members of Select Structural Grade and Grade 1 quality shall be determined in accordance with Clauses 14.14.1.7.2 and 14.14.1.7.3. All other wood resistances shall be determined in accordance with Section 9.

14.14.1.7.2 Shear

The shear resistance in wood shall be taken from Clause 9.7, with f_{vu} taken from Table 14.14 and the size effect factor, k_{sv}, taken as follows:

$$k_{sv} = \frac{75}{\sqrt{d}} \frac{1}{(1+2a/d)} \leq 2.5$$

where

d = member depth

For members older than five years, a shall be the distance measured from the centreline of the support to the tip of the end split. Where the split does not extend past the centreline of the support into the span, a shall be taken as zero.

For members that are not older than five years, or where the end split length has not been measured, a shall be assumed to be $0.33d$ for Select Structural Grade and $0.75d$ for Grade 1.

14.14.1.7.3 Specified strengths and moduli of elasticity

The specified strengths and moduli of elasticity shall be obtained from Table 14.14.

Table 14.14
Specified strengths and moduli of elasticity for beam and stringer grades and post and timber grades, MPa
(See Clauses 14.14.1.7.2 and 14.14.1.7.3.)

Species Combination	Grade	Bending at extreme fibre, f_{bu}	Longitudinal shear, f_{vu}	Compression parallel to grain, f_{pu}	Compression perpendicular to grain, f_{qu}	Tension parallel to grain, f_{tu}	Modulus of elasticity E_{50}	Modulus of elasticity E_{05}
Douglas fir–Larch	SS	24.0	1.1	16.5	4.7	13.0	11 000	7 500
	No. 1	20.0	1.1	9.0	4.7	9.0	9 500	6 500
Hem-Fir	SS	20.0	0.8	14.5	3.1	13.0	11 000	7 500
	No. 1	18.0	0.8	10.5	3.1	9.0	10 500	7 000
Spruce-Pine-Fir	SS	18.5	1.0	14.5	3.6	13.0	10 000	7 000
	No. 1	13.0	1.0	10.5	3.6	9.0	9 000	6 000
Northern species	SS	13.0	0.8	10.0	2.3	10.0	7 000	5 000
	No. 1	9.0	0.8	7.0	2.3	7.0	6 000	4 000

Note: *See Clause 9.3 for the symbols used in this Table.*

14.14.1.8 Shear in steel plate girders with intermediate transverse stiffeners

Clauses 10.10.5 and 10.10.6 shall be used to calculate the shear resistance of steel plate girders with intermediate transverse stiffener plates on one side of the web if the width-to-thickness ratio of the plate does not exceed $400/\sqrt{F_y}$.

14.14.2 Resistance adjustment factor

For all components that have no visible sign of defect or deterioration, the factored resistance, as calculated in accordance with Clause 14.14.1, shall be multiplied by the appropriate resistance adjustment factor, U, specified in Table 14.15. Where no value for U is specified in Table 14.15 and in lieu of better information, a value of $U = 1.0$ may be used.

14.14.3 Effects of defects and deterioration

The effects of defects and deterioration on the factored resistance of a member shall be considered. These effects include changes in member strength, stability, and stiffness. The design net area of a deteriorated section shall include sound material only. The distribution of the section loss around the critical section shall also be considered.

When it is possible that the member has also lost ductility or post-failure capacity, appropriate adjustments to the reliability index, β, shall be made.

Redistribution of load effects between members due to defects and deterioration shall be considered.

Any increases in member defects and deterioration expected before the next bridge evaluation shall be accounted for.

If a prestressing tendon is significantly corroded, the contribution of the entire tendon to the strength of a component shall be neglected.

Table 14.15
Resistance adjustment factor, *U*
(See Clause 14.14.2.)

Resistance category	Resistance adjustment factor, *U*
Structural Steel (ϕ per Clause 10.5.7**)**	
Plastic moment	1.00
Yield moment	1.06
Inelastic lateral torsional buckling moment	1.04
Elastic lateral torsional buckling moment	0.96
Compression or tension on gross section	1.01
Tension on net section	1.18
Shear (stocky web)	1.02
Shear (tension field)	1.03
Bolts	1.20
Welds	1.32
Rivets	1.81
Composite — Slab on steel girder (ϕ per Clauses 8.4.6 **and** 10.5.7**)**	
Bending moment	0.96
Shear connectors	0.94
Reinforced concrete (ϕ per Clause 8.4.6**)**	
Bending moment	
$\rho \leq 0.4\rho_b$	1.02
$0.4\rho_b < \rho \leq 0.7\rho_b$	0.95
Axial compression	1.06
Shear (> min. stirrups)	1.05
Prestressed concrete (ϕ per Clause 8.4.6**)**	
Bending moment	
$\omega_p \leq 0.15$	1.01
$0.15 < \omega_p \leq 0.30$	0.94

14.15 Live load capacity factor

14.15.1 General

After the loads, load effects, load factors, and factored resistances multiplied by resistance adjustment factors, U, have been calculated, the live load capacity factor, F, shall be calculated as follows:

(a) for ultimate limit states, in accordance with Clause 14.15.2.1 or 14.15.2.2, as applicable, or in accordance with the alternative method specified in Clause 14.15.2.3;

(b) for serviceability limit states, in accordance with Clause 14.15.3; and

(c) for combined load effects, in accordance with Clause 14.15.4.

14.15.2 Ultimate limit states

14.15.2.1 General

For ultimate limit states, the value of the live load capacity factor, F, may be calculated as follows for all structural components, except, as specified in Clause 14.15.2.2, for concrete deck slabs and masonry arches:

$$F = \frac{UR_r - \Sigma\alpha_D D - \Sigma\alpha_A A}{\alpha_L L(1+I)}$$

For normal traffic, F shall be calculated for CL1-W or CL1-625-ONT loading and, if F is found to be less than 1.0 and posting of the bridge is an option, F shall also be calculated for CL2-W and CL3-W loading or CL2-625-ONT and CL3-625-ONT loading, as applicable, unless otherwise directed by the Regulatory Authority.

For permit traffic, F shall be calculated for the type of vehicle(s) for which a permit is sought.

14.15.2.2 Special cases for ultimate limit states

14.15.2.2.1 Concrete deck slabs

For wheel loads, the live load capacity factor, F, shall be calculated as R_r/L_{wf}, where R_r is as specified in Clause 14.14.1.3.3.

14.15.2.2.2 Masonry arches

For each lane of an arch, the live load capacity factor, F, shall be calculated as P_r/L_a, where P_r is as specified in Clause 14.14.1.5.

14.15.2.3 Mean load method for ultimate limit states (alternative method)

As an alternative to Clause 14.15.2.1, the live load capacity factor, F, at the ultimate limit state may be calculated as follows:

$$F = \frac{\bar{R}\exp\left[-\beta\left(V_R^2 + V_S^2\right)^{0.5}\right] - \Sigma\bar{D}}{\bar{L}}$$

where

$\bar{R} = \delta_R R$

$$V_S = \frac{\left(S_D^2 + S_L^2\right)^{0.5}}{\left(\Sigma\bar{D} + \bar{L}\right)}$$

where

$$S_D = \left[\sum\left[\left(V_D^2 + V_{AD}^2\right)\left(\delta_D \delta_{AD} D\right)^2\right]\right]^{0.5}$$

$$S_L = \left[V_{AL}^2 + V_L^2 + \left(V_I \delta_I I\right)^2 / \left(1 + \delta_I I\right)^2\right]^{0.5} \left[\delta_L \delta_{AL} L\left(1 + \delta_I I\right)\right]$$

$$\Sigma\bar{D} = \Sigma\delta_D \delta_{AD} D$$

$$\bar{L} = \delta_L \delta_{AL} L\left(1 + \delta_I I\right)$$

β shall be determined from Clause 14.12.1; D shall be calculated for dead loads in accordance with Clause 14.8.2.1; I (the dynamic load allowance) shall be calculated in accordance with Clause 14.9.3; and L shall be calculated for live loads as specified in Clause 14.9.

The bias coefficients and coefficients of variation to be used for calculating F may be taken from Clause C14.15.2.3 of CSA S6.1, from reported values in technical publications, or from field measurements.

14.15.3 Serviceability limit states

For serviceability limit states, the live load capacity factor, F, for the applicable loading shall be calculated using the following equation for all structural components:

$$F = \frac{\sigma_{SLS} - \sigma_D - \sigma_A}{\alpha_L \sigma_L (1 + I)}$$

where α_L is as specified in Clause 3.5.1.

14.15.4 Combined load effects

Where combined effects such as axial force and moment occur simultaneously in the same element such that the capacity for one is affected by the magnitude of the other, F shall be calculated by successive iteration or another suitable method.

14.16 Load testing

14.16.1 General

Bridges may be considered for load testing if the Engineer determines that the analytical evaluation does not accurately assess the actual behaviour of the bridge or there is otherwise a need to establish the actual behaviour of the bridge or its components.

When a load test is proposed as part of the evaluation procedure, such a test, including details of loads, loading pattern, instrumentation, condition survey, and analysis, shall be Approved.

Load testing shall not be carried out until a theoretical evaluation has been performed in accordance with this Section. This requirement may be waived only if no Plans of the bridge are available or could be made available, in which case testing shall be conducted with extreme care, taking into consideration the possibility of failure of the bridge during testing.

14.16.2 Instrumentation
Components of the bridge shall be instrumented and monitored during the test to the extent considered necessary for safety and detection of any damage or failure, or for verifying certain behaviour considered, or to be considered, in the analysis.

14.16.3 Test load

14.16.3.1 General
Testing shall be either static or dynamic, depending on the information required from the tests.

14.16.3.2 Static load test
Static test loads shall be applied in a manner that simulates the critical load effects due to the evaluation loads. The load shall be increased in gradual steps within the safe capacity of the loading equipment to at least a predetermined level, provided that no permanent movement of or damage to the bridge components results.

14.16.3.3 Dynamic load test
Dynamic testing to establish dynamic characteristics and behaviour of the bridge structure shall be conducted by
(a) running test vehicles with known axle loads across the bridge, with no other traffic on the bridge;
(b) carrying out the testing under normal traffic conditions, provided that the relevant response can be clearly recorded one vehicle at a time if the response to a single vehicle is required; or
(c) using Approved methods.

14.16.4 Application of load test results

14.16.4.1 Evaluation using observed behaviour
The bridge structure shall be evaluated taking the observed behaviour into account only if the evaluator is confident that this behaviour will be maintained at the limit state for which the evaluation is being performed.

If a dynamic test is performed on the structure to measure actual dynamic amplifications of the vehicle loads or load effects, the dynamic load allowance determined from the test may be used in the evaluation.

14.16.4.2 Live load capacity factors
When a live load capacity factor, F, is determined on the basis of load testing, it shall be calculated by dividing L_t by the load effects due to factored live loads.

The test results may be extrapolated to determine the live-load-carrying capacity if
(a) the maximum applied test load is limited by the capacity of the test equipment; and
(b) the stability of the bridge structure or its components is not of concern with any further increase in load.

Such an extrapolation, including methods of analysis, projected maximum load capacity, and determination of the live load capacity factors, shall be Approved.

14.17 Bridge posting

14.17.1 General

The calculations for the live load capacity factors for establishing posting limits shall be carried out in accordance with Clause 14.15 or 14.16. In certain cases, a single posting load, based on engineering judgment and experience, may be used, subject to Approval. Posting methods other than those specified in Clause 14.17, may be used, subject to Approval.

A reinforced concrete bridge need not be posted if it has been carrying normal traffic without signs of excessive cracking or deformation. Such a bridge shall be inspected at intervals recommended by the evaluator.

14.17.2 Calculation of posting loads

When Evaluation Levels 1, 2, and 3 are used as a basis for posting, the smallest value of F from Clause 14.15 or 14.16 shall be calculated and applied as follows:

(a) when $F \geq 1.0$ for Evaluation Level 1, posting shall not be required;

(b) when $1.0 > F \geq 0.3$ for Evaluation Level 1, triple posting shall be required, with the posting loads for each evaluation level being obtained from Figure 14.8 for the appropriate value of F for each evaluation level;

(c) when $F < 0.3$ for Evaluation Level 1 and $F \geq 0.3$ for Evaluation Level 3, single posting corresponding to Evaluation Level 3 shall be required, with the posting load being obtained from Figure 14.8 for Evaluation Level 3 only; and

(d) when $F < 0.3$ for Evaluation Level 3, consideration shall be given to closing the bridge.

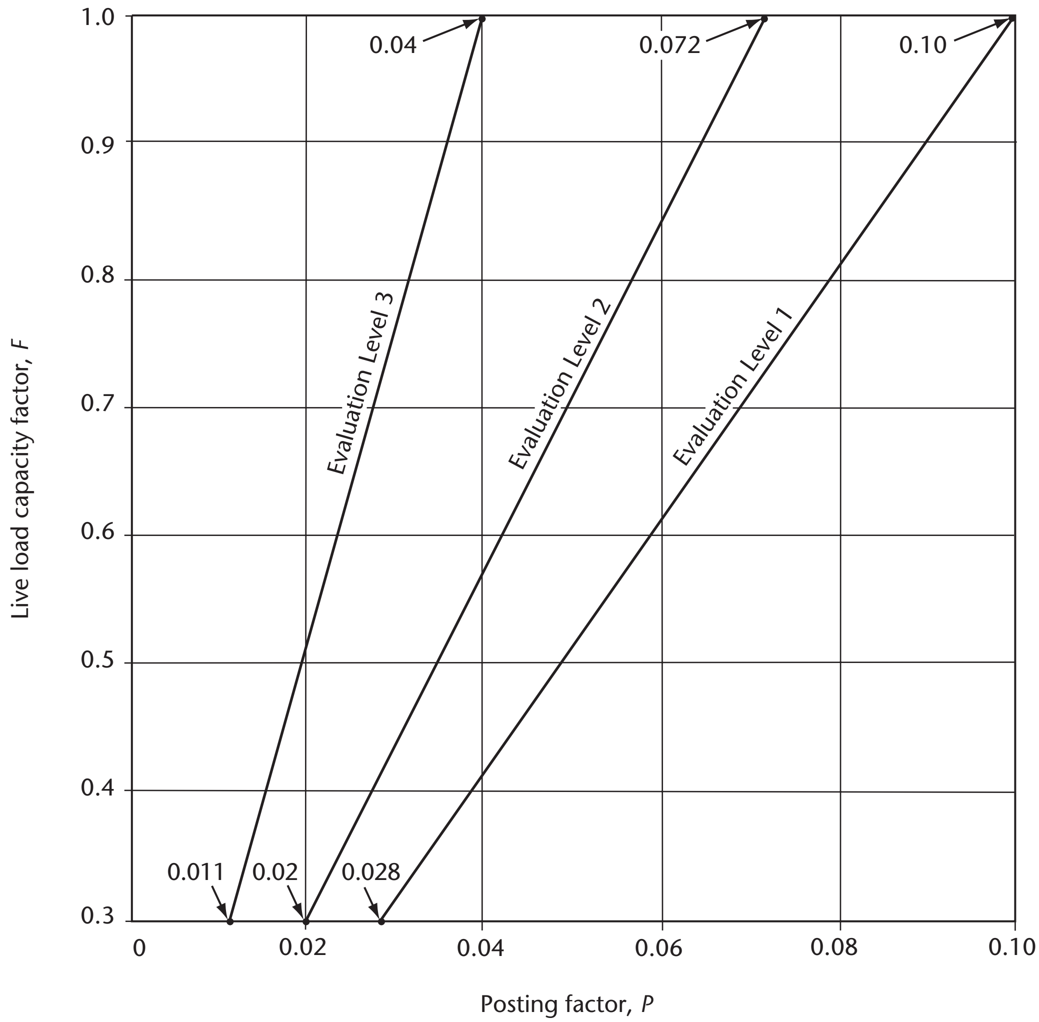


Figure 14.8
Posting loads for gross vehicle weight
(See Clauses 14.17.2 and 14.17.3.1.)

14.17.3 Posting signs

14.17.3.1 General

The posted weight limit(s) in tonnes shall be *PW*, where *P* is the posting factor shown in Figure 14.8 and *W* is in kilonewtons and as specified in Clause 14.9.1.2. For ONT loads specified for use in Ontario, *W* equals 625 kN.

14.17.3.2 Single posting signs for vehicles

Posting signs shall be in accordance with the regulations set by the Regulatory Authority. Posting shall show the gross vehicle weight to the nearest tonne.

14.17.3.3 Triple posting signs for vehicles

Posting signs shall be in accordance with the regulations set by the Regulatory Authority and shall show the following three types, from top to bottom, respectively, with the maximum gross vehicle weight to the nearest tonne permitted on the bridge for each type:
(a) single-unit vehicle corresponding to the Evaluation Level 3 loads;
(b) two-unit vehicle corresponding to the Evaluation Level 2 loads; and
(c) vehicle train corresponding to the Evaluation Level 1 loads.

14.17.3.4 Posting sign for axle weights

The posting sign shall be in accordance with the regulations set by the Regulatory Authority and may show one or more of the following axle types, with the weight limit to the nearest tonne permitted on the bridge for each axle type:
(a) single: weight limit = $9.1F$;
(b) tandem: weight limit = $17.0F$; and
(c) tridem: weight limit = $23.0F$.

In all cases, F shall be the live load capacity factor calculated for Evaluation Level 1.

14.18 Fatigue

Where there are fatigue-prone details or physical evidence of fatigue-related defects, the bridge and affected components shall be assessed for fatigue and remaining fatigue life at the fatigue limit state, using appropriate methods. As an alternative to assessment, the fatigue-prone details or fatigue-related defects may be monitored by regular inspections. Load combinations and load factors for assessment shall be in accordance with Section 3.

Where there are no fatigue-prone details or fatigue-related defects, the evaluation need not consider the fatigue limit state if neither the use nor the behaviour of the bridge is changed.

Annex A14.1 (normative) ***Equivalent material strengths from tests of samples***

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

A14.1.1 Structural steel

Coupon specimens for determination of the yield and ultimate tensile strengths of structural steel shall be tested in accordance with CSA G40.20. At least three coupons shall be obtained from the components being evaluated.

The "equivalent" yield strength of each coupon shall be its reported yield strength. If a coupon is obtained from the flange of a rolled member, its equivalent yield strength may be taken as 1.05 times the reported yield strength.

The yield strength, f_y, used for evaluation shall be calculated as follows:

$$f_y = \left(\bar{f}_y - 28\right)\exp\left(-1.3k_sV\right)$$

where $\bar{f}_y$ and V are, respectively, the average value and coefficient of variation of the yield strengths, and k_s is obtained from Table A14.1.1, in which n is the number of strength tests.

Table A14.1.1
Coefficient of variation modification factor, k_s
(See Clauses A14.1.1 and A14.1.3.)

n	k_s
3	3.46
4	2.34
5	1.92
6	1.69
8	1.45
10	1.32
12	1.24
16	1.14
20	1.08
25	1.03
30 or more	1.00

A14.1.2 Concrete

The compressive strength of sound concrete shall be determined from the strengths of cores obtained from the components being evaluated. The core tests shall be conducted in accordance with CAN/CSA-A23.2.

The strength of cores smaller than 100 mm diameter shall be adjusted to approximate the equivalent strengths of 100 mm diameter cores. The appropriate strength-correction factor shall be determined from cores of both diameters obtained from the components being evaluated.

It may be assumed that the strengths of 100 and 150 mm diameter cores are equivalent.

The equivalent strength of 100 mm diameter cores shall be increased by 8% for cores soaked 40 h in water or reduced by 5% for cores dried 7 d in air before testing.

The equivalent specified compressive strength, f_c', used for evaluation shall be calculated as follows:

$$f_c' = 0.9\bar{f}_c\left[1 - 1.28\left[(k_c V)^2 / n + 0.0015\right]^{0.5}\right]$$

where $\bar{f}_c$ is the average core strength, modified to account for the diameter and moisture condition of the core, V is the coefficient of variation of the core strengths, n is the number of cores tested, and k_c is obtained from Table A14.1.2.

Table A14.1.2
Coefficient of variation modification factor, k_c
(See Clause A14.1.2.)

n	k_c
2	2.40
3	1.47
4	1.28
5	1.20
6	1.15
8	1.10
10	1.08
12	1.06
16	1.05
20	1.03
25 or more	1.02

A14.1.3 Reinforcing steel

Coupon specimens for determining the yield and ultimate tensile strengths of reinforcing steel shall be tested in accordance with CAN/CSA-G30.18. At least three coupons, taken from different bars, shall be obtained from the components being evaluated.

The yield strength, f_y, used for evaluation shall be calculated as follows:

$$f_y = \left(\bar{f}_y - 24\right)\exp\left(-1.3 k_s V\right)$$

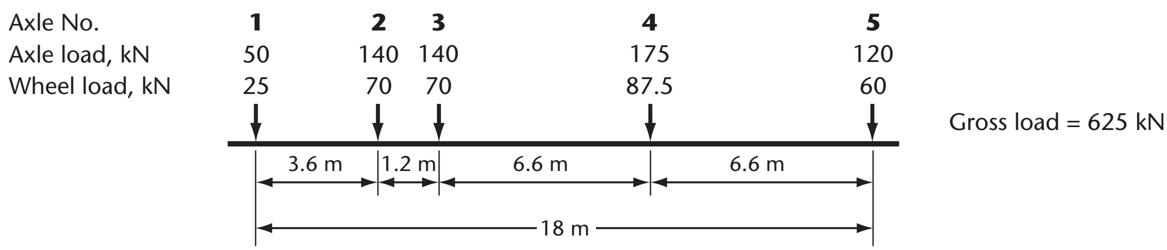
where $\bar{f}_y$ and V are, respectively, the average value and coefficient of variation of the yield strengths, and k_s is obtained from Table A14.1.1, in which n is the number of strength tests.

Annex A14.2 (normative)
Evaluation Level 1 (vehicle trains) in Ontario

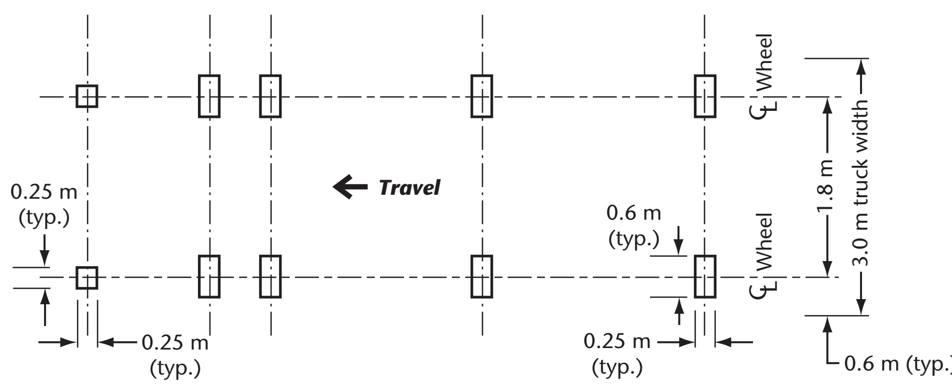
Note: *This Annex is a mandatory part of this Code.*

A14.2.1 General

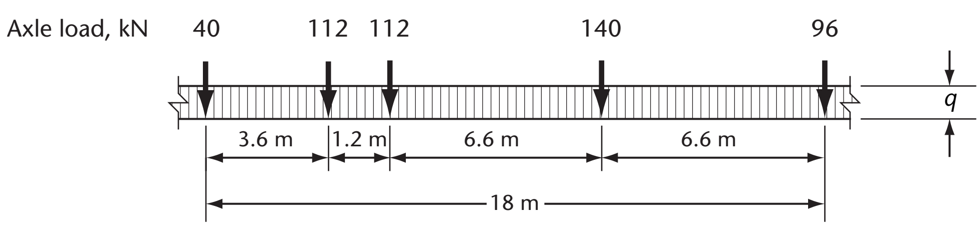
For Evaluation Level 1 in Ontario, the CL1-625-ONT Truck Load or the CL1-625-ONT Lane Load shown in Figure A14.2.1 shall be used instead of the CL1-W Truck Load and CL1-W Lane Load, respectively.



CL1-625-ONT Truck Load (Elevation)



CL1-625-ONT Truck Load (Plan)



CL1-625-ONT Lane Load

Note: *The values of the uniformly distributed load, q, for each highway class (see Section 1) are as follows:*
(a) *Class A:* 9 kN/m;
(b) *Class B:* 8 kN/m; *and*
(c) *Class C or D:* 7 kN/m.

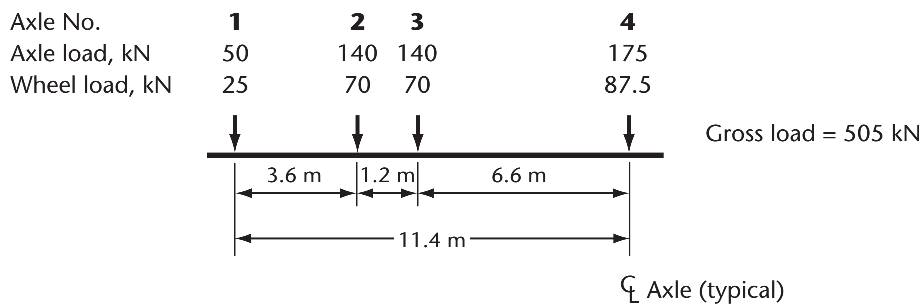
Figure A14.2.1
Evaluation Level 1 loads with CL1-625-ONT Truck
(See Clauses 14.9.1.7 and A14.2.1.)

Annex A14.3 (normative)
Evaluation Level 2 (two-unit vehicles) in Ontario

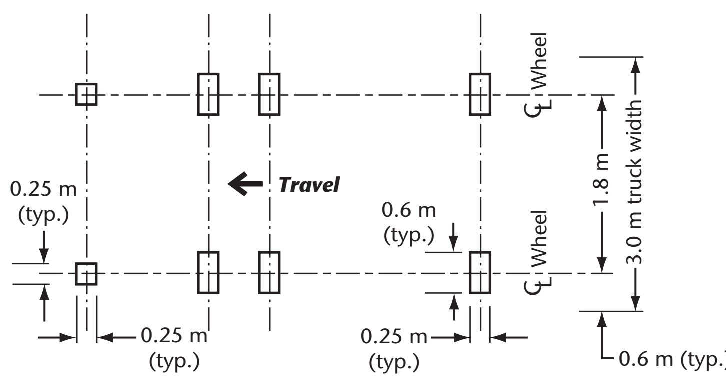
Note: *This Annex is a mandatory part of this Code.*

A14.3.1 General

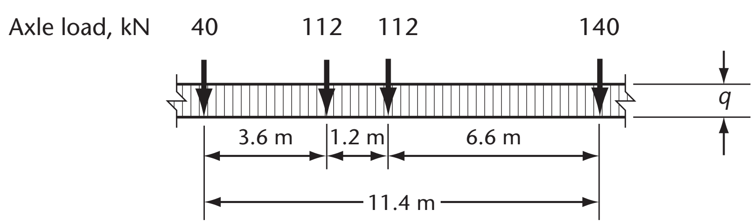
For Evaluation Level 2 in Ontario, the CL2-625-ONT Truck Load or the CL2-625-ONT Lane Load shown in Figure A14.3.1 shall be used instead of the CL2-W Truck Load and CL2-W Lane Load, respectively.



CL2-625-ONT Truck Load (Elevation)



CL2-625-ONT Truck Load (Plan)



CL2-625-ONT Lane Load

Note: *The values of the uniformly distributed load, q, for each highway class (see Section 1) are as follows:*
(a) Class A: 9 kN/m*;*
(b) Class B: 8 kN/m*; and*
(c) Class C or D: 7 kN/m.

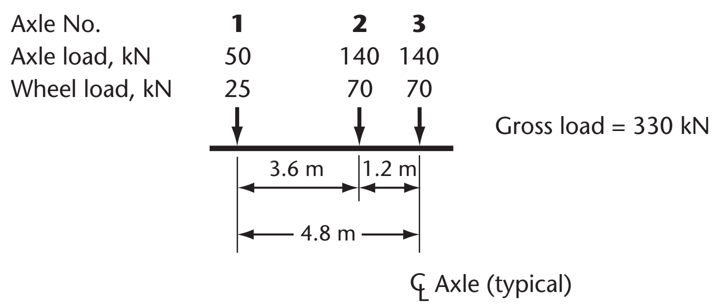
Figure A14.3.1
Evaluation Level 2 loads with CL2-625-ONT Truck
(See Clauses 14.9.1.7 and A14.3.1.)

Annex A14.4 (normative) ***Evaluation Level 3 (single-unit vehicles) in Ontario***

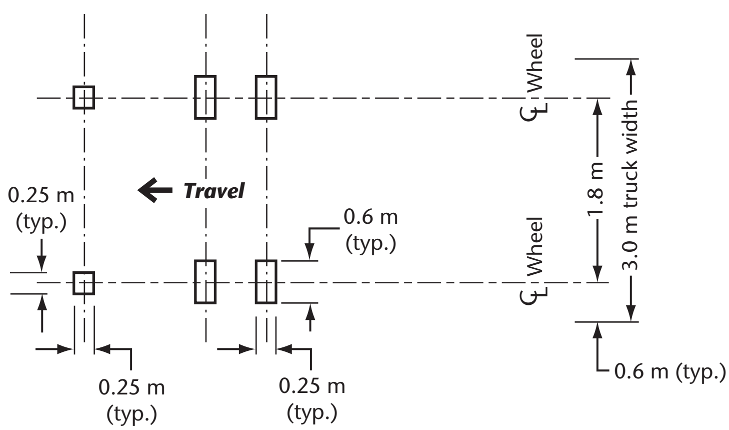
Note: *This Annex is a mandatory part of this Code.*

A14.4.1 General

For Evaluation Level 3 in Ontario, the CL3-625-ONT Truck Load or the CL3-625-ONT Lane Load shown in Figure A14.4.1 shall be used instead of the CL3-W Truck Load and CL3-W Lane Load, respectively.



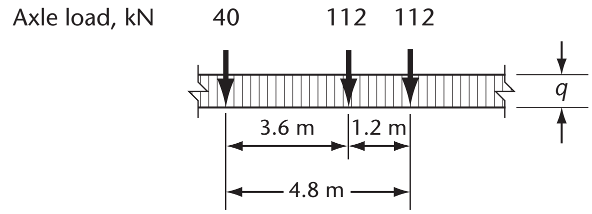
CL3-625-ONT Truck Load (Elevation)



CL3-625-ONT Truck Load (Plan)

Figure A14.4.1
Evaluation Level 3 loads with CL3-625-ONT Truck
(See Clauses 14.9.1.7 and A14.4.1.)

(Continued)



CL3-625-ONT Lane Load

Note: *The values of the uniformly distributed load, q, for each highway class (see Section 1) are as follows:*
(a) *Class A:* 9 kN/m*;*
(b) *Class B:* 8 kN/m*; and*
(c) *Class C or D:* 7 kN/m.

Figure A14.4.1 (Concluded)

Section 15 — Rehabilitation and repair

Section 15
Rehabilitation and repair

15.1 Scope

This Section specifies minimum requirements for the rehabilitation of bridges. The requirements specified in this Section relate only to loads, load factors, resistances, and other design criteria relevant to the rehabilitation of bridges. Material specifications and rehabilitation and maintenance procedures are not covered in this Section but should conform to accepted Canadian good practice.

15.2 Symbols

See Clause 14.3 for the symbols used in this Section.

15.3 General requirements

Note: *See Clause 15.4 for special considerations.*

15.3.1 Limit states

Unless otherwise specified by the Owner or required by this Section, all rehabilitated members shall satisfy the ultimate limit state and serviceability limit state requirements specified as part of the design requirements of Sections 1 to 13 and 16, except that if the purpose of the rehabilitation is to allow passage of a controlled vehicle, the only load combination that shall be considered is permanent loads plus the control vehicle, with the load factors specified in Section 14.

15.3.2 Condition data

Condition data on dimensions, member sizes, geometry, material strengths, extent and location of deterioration, distress, and permanent distortion shall be collected, to the extent that they will affect the rehabilitation design and preparation of Plans, in accordance with Clause 15.5.

15.3.3 Rehabilitation loads and load factors

The rehabilitation loads and load factors for which the bridge or its components are to be rehabilitated shall be selected in accordance with Clause 15.6.

15.3.4 Analysis

The structure shall be analyzed for the selected loads and limit states in accordance with Clause 15.7.

15.3.5 Factored resistances

The factored resistances for each element being considered shall be calculated in accordance with Clause 15.8.

15.3.6 Fatigue

The resistance of existing members and materials to fatigue loadings and imposed deformations shall not be impaired by the rehabilitation.

15.3.7 Bridge posting

If, after rehabilitation, the bridge needs to be posted for load restriction, posting loads shall be calculated in accordance with Clause 14.17.

15.3.8 Seismic upgrading

Seismic upgrading of the bridge shall be carried out in accordance with Section 4.

15.4 Special considerations

During the planning and design stages of bridge rehabilitation, special consideration shall be given to, but not limited to, the following:

(a) access;
(b) the aesthetics of the rehabilitation;
(c) architectural features;
(d) constructibility;
(e) the difference between as-built and as-designed information, including modifications made after initial construction;
(f) drainage;
(g) economics, including life cycle costs or phased rehabilitation to suit available cash flow;
(h) environmental impacts, including stream improvements;
(i) the extent of defects and deterioration;
(j) the geometry of approach and of the highway beyond the ends of the structure;
(k) heritage aspects;
(l) liaison with other agencies and individuals, including utility companies, railways, conservation authorities, municipalities, and private property owners;
(m) local expertise;
(n) the presence of utilities;
(o) provision for further rehabilitation at a later date;
(p) the remaining service life before and after rehabilitation;
(q) structural safety during all construction stages of rehabilitation;
(r) traffic conditions;
(s) waterproofing; and
(t) vibration.

15.5 Data collection

In addition to the condition survey and determination of material strengths required by Clauses 14.6 and 14.7, inspection and testing shall be carried out as necessary to ascertain that the planned rehabilitation is compatible with the geometry, material characteristics, and state of stress of the structure.

15.6 Rehabilitation loads and load factors

15.6.1 Loads

15.6.1.1 General

Loads, load factors, and their application shall be in accordance with Section 3, except as modified by Clauses 15.6.1.2 to 15.6.1.12.

15.6.1.2 Permanent loads

Permanent loads shall be based on the dimensions and information on available drawings and verified in the field. When drawings are not available, field measurements shall be used.

Addition, removal, or redistribution of permanent loads resulting from the rehabilitation shall be included in the rehabilitation design.

15.6.1.3 Rehabilitation design live loads

15.6.1.3.1 General

The rehabilitation design live loads specified in this Section do not include an allowance for future traffic growth beyond what is specified in Clause 15.6.1.3.2 for the appropriate value of *W* in Clause 14.9.1.

15.6.1.3.2 Normal traffic

Evaluation Level 1 Live Load CL1-W specified in Clause 14.9.1 shall be used for the rehabilitation design of bridges that are to carry unrestricted normal traffic after rehabilitation.

For restricted normal traffic, a suitable fraction of CL1-W, CL2-W, or CL3-W shall be used in accordance with Table 15.1 if rehabilitation to CL1-W is not economically justifiable.

For Ontario, ”625-ONT” shall be substituted for “-W” in this Clause.

Table 15.1
Rehabilitation design live loads for restricted normal traffic
(See Clause 15.6.1.3.2.)

Rehabilitation level	Proposed use
CL2	All trucks (excluding truck trains)
CL3	Urban buses, milk trucks, and single-unit trucks
75% of CL3	Light trucks, emergency vehicles of gross vehicle weight less than that of CL1-W, and school buses
50% of CL3	Passenger vehicles, light emergency vehicles, and maintenance vehicles
Pedestrian	Pedestrians only

Note: *Bridges rehabilitated for restricted normal traffic shall be posted in accordance with Clause 14.17.*

15.6.1.3.3 Vehicles operating under permit

For bridges to be rehabilitated for the passage of a vehicle operating under permit, the wheel loads, axle spacing, and other appropriate dimensions available from actual measurements or from available drawings of the loaded vehicle shall be used. The position and direction of the vehicle and the simultaneous application of other live loads shall be in accordance with controls imposed by the Owner of the bridge.

15.6.1.3.4 Design lanes

The number of design lanes and the modification factors for multiple lane loading shall be as specified in Clause 14.9.4.

15.6.1.4 Dynamic load allowance

15.6.1.4.1 Unrestricted highway live loads

For bridges to be rehabilitated for unrestricted highway live loads, the dynamic load allowance shall be as specified in Section 3.

15.6.1.4.2 Restricted highway live loads
For bridges to be rehabilitated and posted for restricted highway live loads, the dynamic load allowance specified in Section 3 shall be multiplied by the dynamic load allowance modification factor specified in Clause 14.9.3.

15.6.1.5 Thermal, shrinkage, and creep effects
Provision shall be made in the rehabilitation design for expansion and contraction due to temperature, shrinkage, and creep. Load effects induced by any restraints on these movements shall be included in the rehabilitation design.

15.6.1.6 Wind loads
Wind loads shall be as specified in Section 3.

15.6.1.7 Collision loads
Collision loads in accordance with Section 3 shall be considered unless effective measures are taken to protect the bridge and its components against these loads.

15.6.1.8 Settlement and permanent deformations
Load effects resulting from settlement and permanent deformations in the bridge or its components shall be included in the rehabilitation design.

15.6.1.9 Seismic loads
Seismic loads shall be considered in accordance with Section 4.

15.6.1.10 Stream flow and ice pressure loads
Load effects resulting from stream flow and ice pressure shall be considered in accordance with Section 3.

15.6.1.11 Component deterioration
Load effects resulting from redistribution of loads due to deterioration of components shall be included in the rehabilitation design.

15.6.1.12 Loads induced by the rehabilitation
Loads induced in the bridge or its components during rehabilitation construction and any changes in the bridge or its components resulting from the rehabilitation shall be included in the rehabilitation design.

15.6.2 Load factors and load combinations

15.6.2.1 General
Unless otherwise specified by the Regulatory Authority, all load factors and load combinations shall be in accordance with Section 3.

15.6.2.2 Minimum rehabilitation load factors
If the purpose of the rehabilitation is to allow passage of a permit vehicle (see Clause 14.9.2), the load factors shall be in accordance with Section 14. The reliability index, β, for all rehabilitated members shall be selected using Inspection Level INSP1.

If specified by the Regulatory Authority, load factors from Section 14 may be used for a bridge rehabilitation intended to carry normal traffic, but shall be selected using Inspection Level INSP1.

15.6.2.3 Total factored load effect
For each load combination, every load that is to be included shall be multiplied by the specified load factor and then added to the other loads to obtain the total factored load effect.

15.6.2.4 Overall minimum load factor
Except for PB or PC vehicles, the total factored load effect for ULS Combination 1 (see Table 3.1) shall not be less than 1.25 times the sum of the unfactored load effects. This requirement may be waived if the member provides a live load capacity factor, *F*, greater than or equal to 1.0 in accordance with the mean load method specified in Clause 14.5.2.3. For PC vehicles, the total factored load effect for ULS Combination 1 shall not be less than 1.15 times the sum of the unfactored load effects.

15.7 Analysis
Structural analysis shall be performed in accordance with this Section and the other applicable Sections of this Code.

The effects of member connections, connection continuity, support restraints, the contribution of the secondary components, and the interaction between the new and existing components of the bridge shall be considered in the analysis.

Structural analysis shall consider the extent of member deterioration and existing locked-in stresses due to system behaviour, their effect on the strength and stiffness characteristics of the member, and their effect on the strength of the structural system.

15.8 Resistance

15.8.1 Existing members

15.8.1.1 General
The factored resistances of existing members, including existing members strengthened with new material, shall be determined in accordance with Clauses 14.14.1 and 14.14.2.

The factored resistances of existing members shall be reduced to account for any member defects or deterioration in accordance with Clause 14.14.3.

In addition, the effects on member resistance and ductility of different stress levels in the new and existing portions of hybrid members shall be considered.

15.8.1.2 Strengthening using fibre-reinforced polymer
In addition to satisfying the requirements of Clause 15.8.1.1, members strengthened using fibre-reinforced polymer shall satisfy the durability requirements of Clause 16.4.

The design of concrete or wood members strengthened with fibre-reinforced polymer shall also satisfy the requirements of Clause 16.11 or 16.12, respectively.

15.8.2 New members
The factored resistances of new members shall be determined in accordance with Sections 5 to 13 and 16.

Section 16 — Fibre-reinforced structures

Annexes

Section 16
Fibre-reinforced structures

16.1 Scope

16.1.1 Components
The requirements of this Section apply to the following components containing fibre reinforcement:
(a) fully or partially prestressed concrete beams and slabs;
(b) non-prestressed concrete beams, slabs, and deck slabs;
(c) externally and internally restrained deck slabs;
(d) stressed wood decks;
(e) barrier walls;
(f) existing concrete elements with externally bonded fibre-reinforced polymer systems and near-surface-mounted reinforcement; and
(g) existing timber elements with externally or internally bonded glass-fibre-reinforced polymer systems and near-surface-mounted reinforcement.

16.1.2 Fibres
This Section covers fibre reinforcement in which the fibre comprises one or more of the following:
(a) glass;
(b) carbon;
(c) aramid;
(d) a low modulus polymer or polymers; and
(e) steel.

16.1.3 Matrices
This Section covers fibre-reinforced composites in which the matrix comprises one or more of the following:
(a) epoxy resin;
(b) saturated polyester resin;
(c) unsaturated polyester resin;
(d) vinylester resin;
(e) polyurethane; and
(f) Portland-cement-based mortar or concrete.

16.1.4 Uses requiring Approval
Uses of fibre-reinforced polymers in structures or strengthening schemes that do not meet the requirements of this Section require Approval.

16.2 Definitions
The following definitions apply in this Section:

Adhesive — a polymeric substance applied to mating surfaces to bond them together.

Bar — a non-prestressed FRP element with a nominally rectangular or circular cross-section that is used to reinforce a structural component.

Bond-critical applications — applications of FRP systems that rely on bond to the substrate for load transfer.

Contact-critical applications — applications of FRP systems that rely on continuous intimate contact between the substrate and the FRP system.

Continuous fibres — aligned fibres whose individual lengths are significantly greater than 15 times the critical fibre length.

Critical fibre length — the minimum length required to develop the full tensile strength of a fibre in a matrix.

Cure — the process of causing an irreversible change in the properties of a thermosetting resin by chemical reaction.

Deck slab — a concrete slab supported by girders, stringers, or floor beams.

Duct — a conduit for post-tensioning tendons.

Equivalent diameter — the diameter of a circular cross-section having the same cross-sectional area as that of the non-circular section.

Externally restrained deck slab — a deck slab with external straps or other confining systems designed in accordance with Clause 16.7.

Fibre-reinforced composite — an assembly of chemically dissimilar materials, being the matrix and fibres, whose properties in combination are more widely useful than those of the constituent materials.

Fibre-reinforced concrete (FRC) — a fibre-reinforced composite in which the matrix is Portland cement concrete or mortar and the fibres are discontinuous and uniformly and randomly distributed.

Fibre-reinforced polymer (FRP) — a fibre-reinforced composite with a polymeric matrix and continuous fibre reinforcement of aramid, carbon, or glass.

Fibres — small-diameter filaments of materials of relatively high strength, i.e., glass, carbon, aramid, low modulus polymer, or steel.

Fibre volume fraction — the ratio of the volume of the fibres to the volume of the fibre-reinforced composite.

Gel time — the time a material takes to become pseudo-plastic.

Glass transition temperature — the midpoint of the temperature range over which an amorphous material changes from a brittle and vitreous state to a plastic state or vice versa.

Grid — a prefabricated planar assembly consisting of bars in an orthogonal arrangement.

Impregnate — to saturate fibre assemblies with a resin.

Internally restrained deck slab — a deck slab containing embedded bottom transverse reinforcement designed in accordance with Clause 16.8.8.

Low modulus polymers — polymers with a modulus of elasticity less than 10 GPa, e.g., nylon, polyolefin, polypropylene, and vinylon.

Matrix — the material in a fibre-reinforced composite component that contains aligned or randomly distributed fibres.

Near-surface-mounted reinforcement (NSMR) — an FRP bar or strip bonded inside a groove near a surface of a structural component.

Open time — the maximum time for joining together adherents after adhesive has been applied in order to avoid surface alteration of the adhesive.

Plate — an FRP component whose thickness is significantly less than its other dimensions.

Pot life — the time one can work with primer, putty, and/or adhesive after mixing resin and a hardener before the primer, putty, and/or adhesive starts to harden in the mixture vessel.

Primary reinforcement — reinforcement provided mainly for strength.

Rope — an assembly of bundled continuous fibres.

Secondary reinforcement — reinforcement provided mainly for control of cracking.

Sheath — a protective encasement for a tendon or rope.

Sheet — a flexible component comprising fibres.

Shelf life — the length of time a material can be stored under specified environmental conditions and still continue to meet all applicable specifications for use.

Slab — a concrete slab that transfers load directly to the substructure.

Strand — a linear component that constitutes all or part of a tendon.

Strap — a linear component of steel or FRP that provides transverse restraint externally in a deck slab.

Stressed log bridge — a bridge deck made with logs that are trimmed to obtain two parallel faces and that are post-tensioned transversely.

Stressed wood deck — a stress-laminated wood deck or stressed log bridge.

Stress-laminated wood deck — a laminated wood deck that is post-tensioned perpendicular to the deck laminates.

Supporting beam — a stringer, floor beam, or girder.

Tendon — an FRP or high-strength steel element that imparts prestress to a structural component.

Thermoplastic matrix — a polymer capable of being repeatedly softened by an increase in temperature and hardened by a decrease in temperature.

Thermoset matrix — a polymer that changes into a substantially infusible and insoluble material when cured by heat, chemicals, or both.

Wet lay-up — a method of making an FRP-laminated product that involves applying a resin system as a liquid when the fabric is put in place.

16.3 Abbreviations and symbols

16.3.1 Abbreviations

The following abbreviations apply in this Section:

AFRP — aramid-fibre-reinforced polymer

CFRP — carbon-fibre-reinforced polymer

FLS — fatigue limit state

FRC — fibre-reinforced concrete

FRP — fibre-reinforced polymer
GFRP — glass-fibre-reinforced polymer
NSMR — near-surface-mounted reinforcement
SLS — serviceability limit state
ULS — ultimate limit state

16.3.2 Symbols

The following symbols apply in this Section:

A = area of cross-section of a strap or bar, mm^2

A_{FRP} = area of cross-section of an FRP bar, plate, sheet, or tendon, mm^2

A_f = area of cross-section of deck slab formwork along a section parallel to the beams, per unit length of the formwork, mm^2/mm

A_p = area of steel tendons in the tension zone, mm^2

A_s = area of cross-section of steel or FRP reinforcing bars used in edge stiffening of deck slabs, as shown in Figures 16.2 and 16.5, mm^2

A_v = area of transverse shear reinforcement perpendicular to the axis of a member, mm^2

$A_{v,min}$ = minimum required area of transverse shear reinforcement perpendicular to the axis of a member, mm^2

b = smallest log diameter, mm; height of the flat face of a trimmed log, as shown in Figure 16.10, mm; width of a rectangular section, mm

b_v = effective width of web within depth d_{long}, mm (see Clause 8.9.1.6)

b_w = width of web of a T-section, mm

D_g = diameter of a circular column or equivalent diameter of a rectangular column, mm (see Clause 16.11.2.5.6)

d = effective depth of a reinforced concrete component, being the distance from the compression face to the centroid of the tensile reinforcement, mm

d_{FRP} = effective shear depth for FRP, mm, calculated in the same manner as d_{long} for longitudinal reinforcement

d_b = equivalent diameter of a bar, tendon, or strand in a multiple-strand tendon, mm

d_c = distance from the centroid of the tension reinforcement to the extreme tension surface of concrete, mm

d_{cs} = smaller of the distance from the closest concrete surface to the centre of the bar being developed, or two-thirds the centre-to-centre spacing of the bars being developed, mm

d_{long} = effective shear depth for longitudinal reinforcement, mm

d_s = diameter of an FRP stirrup in Clause 16.8.7(c), mm; distance from the top of the slab to the centroid of the bottom transverse FRP bars in Clause 16.8.8.1(b), mm

E = modulus of elasticity, MPa

E_{FRP} = mean modulus of elasticity of FRP bars, plates, sheets, and tendons, MPa

E_{long} = modulus of elasticity of FRP or steel longitudinal reinforcement, MPa

E_p = modulus of elasticity of steel tendons, MPa

E_s = modulus of elasticity of steel, MPa

E_{vFRP} = modulus of elasticity of the FRP stirrups, MPa (see Clause 16.8.7)

EI = flexural rigidity, $N \cdot mm^2$

F = live load capacity factor (see Clause 14.15.2)

F_{SLS} = dimensionless factor (see Clause 16.8.3)

F_s = factor in Clauses 16.7.2(c) and 16.7.3(g), MPa

F_t = dimensionless factor (see Clause 16.5.2)

f_{FRP} = stress in the tension FRP reinforcement, MPa

$f_{FRPbend}$ = specified tensile strength of the straight portion of an FRP bent stirrup, MPa

f_{FRPu} = specified tensile strength of an FRP bar, grid, plate, sheet, tendon, or aramid rope, MPa

f_{bu} = specified bending strength of wood, MPa

f_c' = specified compressive strength of concrete, MPa

f_{cc}' = compressive strength of confined concrete, MPa

f_{cr} = cracking strength of concrete, MPa

$f_{\ell FRP}$ = confinement pressure due to FRP strengthening at the ULS, MPa

f_{po} = stress in tendons when the stress in the surrounding concrete is zero, MPa

f_{ps} = maximum permissible stress in a tendon at the ULS, MPa

f_{vu} = specified shear strength of timber (see Section 9)

f_y = specified yield strength of steel reinforcing bars, MPa

h = depth of a timber beam, mm

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

h_2 = distance from the extreme flexural tension surface to the neutral axis, mm

J = overall performance factor for a concrete beam or slab with a rectangular section or a T-section

K_{bFRP} = non-dimensional factor (see Clauses 16.12.2.1 and 16.12.2.2)

K_{tr} = transverse reinforcement index, mm (see Clause 8.15.2.2)

K_{vFRP} = non-dimensional factor (see Clauses 16.12.3.1 and 16.12.3.2)

k_b = coefficient depending on bond between FRP and concrete

k_1 = bar location factor (see Clause 8.15.2.4); concrete strength factor (see Clause 16.11.3.2)

k_2 = non-dimensional factor (see Clause 16.11.3.2)

k_4 = bar surface factor

L_e = effective anchorage length of an FRP sheet, mm (see Clause 16.11.3.2)

L_u = unsupported length of a transverse edge beam, mm

ℓ_a = minimum required anchorage length for externally bonded FRP beyond the point where no strengthening is required, mm

ℓ_d = development length of FRP bars and tendons, mm

M_c = moment at a section corresponding to a maximum compressive concrete strain of 0.001, 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, N•mm

M_{ult} = ultimate moment capacity of a section, N•mm

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 = number of test specimens

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

R_i = post-cracking residual strength index

r = radius of curvature of the bend of an FRP stirrup, mm; radius of curvature of the saddle for a deflected straight tendon, mm

S = centre-to-centre spacing of beams supporting a deck slab, mm

S_ℓ = spacing of straps, mm

s = spacing of shear or tensile reinforcement, mm

s_{FRP} = spacing of externally bonded FRP bands on concrete for shear strengthening, measured along the axis of the member, mm

T_{gw} = wet glass transition temperature, i.e., the glass transition temperature after moisture uptake by the polymer, °C

t = thickness of an externally restrained deck slab, including that of the stay-in-place formwork, if present, mm

t_{FRP} = total thickness of externally bonded FRP plates or sheets, mm

t_c = thickness of cast-in-place concrete in a deck slab cast on a stay-in-place formwork, if present, mm

t_s = projection of a shear connector or other shear-connecting device into a deck slab, as shown in Figure 16.1 for shear connectors, mm

V = coefficient of variation of the strength of FRP components, being the ratio of the standard deviation to the mean

V_{FRP} = factored shear resistance provided by the FRP shear reinforcement, N

V_c = factored shear resistance provided by tensile forces in concrete, 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 (see Clause 8.4.6), to be taken as positive if resisting the applied shear, N

V_r = factored shear resistance, N

V_s = factored shear resistance provided by the steel shear reinforcement, N

V_{st} = factored shear resistance provided by the shear reinforcement, N

w_{FRP} = width of an FRP sheet measured perpendicular to the direction of the main fibres, mm

w_{cr} = crack width at the tensile face of the flexural component, mm

α = angle of inclination of transverse reinforcement to the longitudinal axis of the member, degrees

β = angle of inclination of the internal or external transverse reinforcement to the longitudinal axis of a member, degrees

ε_{FRPe} = effective strain in FRP (see Clause 16.11.3.2)

ε_{FRPu} = ultimate strain in FRP

ε_v = strain in an FRP stirrup

ε_x = longitudinal strain calculated in accordance with Clause 16.8.7

θ = angle of inclination of the principal diagonal compressive stress to the longitudinal axis of the member, degrees

κ_V = bond reduction factor (see Clause 16.11.3.2)

ρ_s = ratio of the cross-sectional area of the longitudinal FRP reinforcement to the effective cross-sectional area of the beam

ρ_{vFRP} = ratio of the total cross-sectional area of the legs of an FRP stirrup to the product of the width of the beam and the spacing of the stirrups

σ_N = stress in concrete due to axial loads, MPa

σ_v = stress calculated in accordance with Clause 16.8.7

ϕ_{FRP} = resistance factor for FRP components (see Clause 16.5.3)

ϕ_c = resistance factor for concrete (see Clause 8.4.6)

ψ_c = curvature at a section when the moment is M_c, mm^{-1}

ψ_{ult} = curvature at a section when the moment is M_{ult}, mm^{-1}

16.4 Durability

16.4.1 FRP tendons, primary reinforcement, and strengthening systems

For FRP bars and grids (when used as primary reinforcement in concrete), FRP tendons, and FRP systems used in the strengthening of concrete and timber components, the matrices shall comprise only thermosetting polymers, except that thermoplastic polymers with proven durability may also be used with Approval. Matrices and the adhesives of FRP systems with a wet glass transition temperature, T_{gw}, of less than the sum of 20 °C and the maximum daily mean temperature specified in Section 3 shall not be used.

Tendons, fibre ropes, and primary reinforcement in concrete, being FRP bars or grids, shall be used only as permitted in Table 16.1, regardless of environmental conditions. Tendons shall also comply with Clause 16.8.6.2.

Subject to the conditions specified in Clauses 16.11 and 16.12, AFRP, CFRP, and GFRP shall be considered permissible reinforcement.

Table 16.1
Conditions of use for FRP tendons and primary reinforcement
(See Clause 16.4.1.)

	Component			
Application	AFRP*	CFRP	GFRP	Aramid rope*
Prestressed concrete beams and slabs				
Pre-tensioned	Permitted	Permitted	Permitted	—
Post-tensioned				
Grouted				
Non-alkaline grout	Permitted	Permitted	Permitted	—
Cement-based grout	Permitted	Permitted	Permitted	—
Ungrouted				
Internal	Permitted	Permitted	Permitted	Permitted
External	Permitted	Permitted	Permitted	Permitted
Non-prestressed beams and slabs	Permitted	Permitted	Permitted	—
Non-prestressed deck slabs	Permitted	Permitted	Permitted	Permitted
Stressed wood decks	Permitted	Not permitted	Permitted	Permitted
Barrier walls	Permitted	Permitted	Permitted	—

In dry and ultraviolet-protected conditions.

16.4.2 FRP secondary reinforcement

For FRP secondary reinforcement in concrete, FRPs with either thermosetting or thermoplastic polymers shall be permitted unless the matrix is susceptible to degradation from alkali.

16.4.3 Fibres in FRC

For FRC, the use of carbon, nylon, polypropylene, polyvinyl alcohol, steel, and vinylon fibres shall be permitted. The use of other fibres shall require Approval.

16.4.4 Cover to reinforcement

The minimum clear cover and its construction tolerance shall be 35 ± 10 mm for FRP bars and grids and 50 ± 10 mm for FRP tendons. For pretensioned concrete, the cover and construction tolerance shall not be less than the equivalent diameter of the tendon ±10 mm. For post-tensioned concrete, the cover and construction tolerance shall not be less than one-half the diameter of the duct ±10 mm.

16.4.5 Protective measures

Anchors for aramid fibre ropes and FRP tendons in concrete shall be of suitably durable materials.

For stressed wood decks, all steel components of the post-tensioning system shall be of stainless steel or suitably protected against corrosion.

Exposed tendons and FRP strengthening systems that are deemed susceptible to damage by ultraviolet rays or moisture shall be protected accordingly. Where the externally bonded FRPs are susceptible to impact damage from vehicles, ice, and debris, consideration shall be given to protecting the FRP systems.

Direct contact between CFRP and metals shall not be allowed.

Aramid ropes shall be protected against moisture ingress by suitably designed sheaths and anchors.

16.4.6 Allowance for wear in deck slabs

A deck slab without a wearing course shall have an additional thickness of 10 mm as an allowance for wear.

16.4.7 Detailing of concrete components for durability

Clauses 8.11.3.1 and 8.11.3.2 shall apply with respect to detailing of concrete components for durability.

16.4.8 Handling, storage, and installation of fibre tendons and primary reinforcement

To avoid damage to fibre tendons and primary reinforcement, instructions for careful handling, storage, and installation of primary reinforcement shall be specified in the Plans.

The specifications for FRP strengthening systems shall be in accordance with Annex A16.1.

16.5 Fibre-reinforced polymers

16.5.1 Material properties

The specified tensile strength, f_{FRPu}, of an FRP bar, grid, plate, sheet, tendon, or aramid rope used in the design shall be its fifth percentile tensile strength; the specified modulus of elasticity, E_{FRP}, shall be the mean modulus of elasticity. In the absence of test data, properties provided by the manufacturer may be used.

16.5.2 Confirmation of the specified tensile strength

The Plans shall specify that the value of f_{FRPu} shall be confirmed by the appropriate test method specified in CAN/CSA-S806.

The specified tensile strength shall be deemed to have been confirmed if the average test strength of the specimens multiplied by F_t is at least equal to the specified tensile strength. F_t, which depends on the coefficient of variation of the tensile strength, V, shall be calculated as follows:

$$F_t = \frac{1 - 1.645V}{1 + (1.645V)/\sqrt{n}}$$

where

n = number of test specimens (which shall not be less than five)

16.5.3 Resistance factor

The resistance factor, ϕ_{FRP}, for pultruded FRP and aramid fibre rope shall be as specified in Table 16.2. For non-pultruded FRP made by wet lay-up, ϕ_{FRP} shall be 0.75 times the corresponding value in Table 16.2. For non-pultruded FRP made in accordance with other factory-based controlled processes, ϕ_{FRP} shall be 0.85 times the corresponding value in Table 16.2. For bent GFRP bars subjected to vehicular impact loads, ϕ_{FRP} shall be taken as 0.75 regardless of the method of manufacture.

Table 16.2
ϕ_{FRP} for pultruded FRP and aramid fibre rope
(See Clause 16.5.3.)

Application	ϕ_{FRP}
AFRP reinforcement in concrete and NSMR	0.60
AFRP in externally bonded applications	0.50
AFRP and aramid fibre rope tendons for concrete and timber components	0.55
CFRP reinforcement in concrete	0.75
CFRP in externally bonded applications and NSMR	0.75
CFRP tendons	0.75
GFRP reinforcement in concrete	0.50
GFRP in externally bonded applications and NSMR	0.65
GFRP tendons for concrete components	0.50
GFRP tendons for timber decks	0.65

16.6 Fibre-reinforced concrete

16.6.1 General

Randomly distributed fibre reinforcement shall be permitted in deck slabs, barrier walls, and surfacing of stressed log bridges for the control of cracks that develop in concrete during its early life. Its use in other applications shall require Approval.

16.6.2 Fibre volume fraction

The fibre volume fraction shall be such that the post-cracking residual strength index, R_i, of the concrete is at least that specified by Table 16.3 for the particular application, where R_i is obtained in accordance with ASTM C 1399.

Table 16.3
Minimum values of R_i for various applications
(See Clause 16.6.2.)

Application	Minimum value of R_i
Barrier wall with one mesh of bars	0.10
Barrier wall with two meshes of bars	0.0*
Deck slab with one crack-control mesh	0.10
Deck slab with two crack-control meshes	0.0*
Surfacing of stressed log bridges	0.30

**Fibres not needed.*

16.6.3 Fibre dispersion in concrete

Fibres shall be mixed uniformly in concrete. The Plans shall specify that before a given mix of FRC is used in the structure, the uniformity of dispersion of the fibres shall be confirmed visually.

As an additional measure, the possibility of non-uniform fibre dispersion shall be estimated by comparing the compressive strength of FRC with that of the corresponding plain concrete using 7-day compressive strength tests in accordance with CAN/CSA-A23.1. If the FRC mix has a mean compressive strength less than 90% of that of the corresponding plain concrete, engineering judgment shall be exercised to determine corrective measures.

16.7 Externally restrained deck slabs

16.7.1 General

An externally restrained deck slab supported on parallel longitudinal beams that complies with Clause 16.7.2, 16.7.3, or 16.7.4 and satisfies the following conditions need not be analyzed except for negative transverse moments due to loads on the overhangs and barrier walls, and for negative longitudinal moments in continuous span bridges:

(a) The deck slab is composite, with parallel supporting beams in the positive moment regions of the beams.
(b) The spacing of the supporting beams, S, does not exceed 3000 mm.
(c) The total thickness of the deck slab, including that of the stay-in-place formwork, if present, t, is at least 175 mm and at least $S/15$.
(d) The supporting beams are connected with transverse diaphragms or cross-frames at a spacing of not more than 8000 mm.
(e) The deck slab is confined transversely by straps or a stay-in-place formwork in accordance with the applicable provisions of Clause 16.7.2, 16.7.3, or 16.7.4.
(f) When the deck slab is confined by straps, the distance between the top of the straps and the bottom of the slab is between 25 and 125 mm, as shown in Figure 16.1.
(g) The projection of the shear connectors in the deck slab, t_s, is at least 75 mm, as shown in Figure 16.1, or additional reinforcement with at least the same shear capacity as that of the shear connectors is provided and the projection of the additional reinforcement into the slab is at least 75 mm.
(h) The cover distance between the tops of the shear-connecting devices and the top surface of the deck slab is at least 75 mm when the slab is not exposed to moisture containing de-icing chemicals; otherwise, this cover distance is at least 100 mm or the shear-connecting devices have an Approved coating.

(i) The fibre volume fraction in the cast-in-place concrete is in accordance with Clause 16.6.2.
(j) The deck slab contains appropriate tensile reinforcement for transverse negative moments resulting from loads on deck slab overhangs and loads on railings or barrier walls.
(k) The transverse edges of the deck slab are stiffened by composite edge beams with a minimum flexural rigidity, EI, in the plane of the deck slab of $3.5L_u^4$ N•mm^2, where L_u is the unsupported length of the edge beam, or, for an unsupported length of edge beam less than 4250 mm, the details of the transverse edge beams are as shown in Figure 16.2, 16.3, 16.4, or 16.5.
(l) For continuous span bridges, the deck slab contains longitudinal negative moment reinforcement in at least those segments where the flexural tensile stresses in concrete for the SLS cases are larger than $0.6f_{cr}$, where f_{cr} is calculated as follows:

$$f_{cr} = 0.4\sqrt{f_c'}$$

When the conditions specified in this Clause and the requirements specified in Clause 16.7.1, 16.7.2, or 16.7.3, as applicable, are not satisfied, the design of the externally restrained deck slab shall require Approval.

16.7.2 Full-depth cast-in-place deck slabs

The design of an externally restrained deck slab with full-depth cast-in-place construction shall meet the requirements of Clause 16.7.1 and the following requirements:
(a) The top flanges of all adjacent supporting beams shall be connected by straps that are perpendicular to the supporting beams and either connected directly to the tops of the flanges, as in the case of the welded steel straps shown in Figure 16.6, or connected indirectly, as in the case of the partially studded straps shown in Figure 16.7; alternatively, the transverse confining system shall comprise devices that have been proved through Approved full-scale laboratory testing.
(b) The spacing of straps, S_ℓ, shall be not more than 1250 mm.
(c) Each strap shall have a minimum cross-sectional area, A, in mm^2, as follows:

$$A = \frac{F_s S^2 S_\ell}{Et}$$

where

F_s = 6.0 MPa for outer panels and 5.0 MPa for inner panels

E = modulus of elasticity of the strap material

In the case of FRP straps, the main fibres shall be in the direction perpendicular to the supporting beams.
(d) The direct or indirect connection of a strap to the supporting beams shall be designed to have a minimum shear strength in newtons of $200A$.
(e) In a negative moment region of a supporting beam, where the beam is not made composite, shear-connecting devices shall be provided on the beam in the vicinity of the straps, and have a minimum total shear strength in newtons of $200A$ (as shown, e.g., in Figure 16.8). As shown in Figure 16.7, such shear-connecting devices shall be within 200 mm of the nearest strap.
(f) The deck slab shall have a crack-control orthogonal assembly of GFRP bars placed near the bottom of the slab, with the area of the cross-section of the GFRP bars in each direction being at least $0.0015t$ mm^2/mm. In addition, the spacing of transverse and longitudinal crack-control bars shall be not more than 300 mm.

When steel straps are welded to steel girders in negative moment regions, the fatigue of the girder shall be considered.

When the conditions specified in Clause 16.7.1 and the requirements specified in this Clause are not satisfied, the design of the slab shall require Approval.

16.7.3 Cast-in-place deck slabs on stay-in-place formwork

The design of an externally restrained deck slab with cast-in-place construction on stay-in-place formwork shall meet the requirements of Clause 16.7.1 and the following requirements:

(a) The design of the formwork shall take into account its anticipated handling and anticipated conditions during construction.

(b) The effective span of the formwork shall be taken as the distance between the edges of the supporting beams plus 150 mm.

(c) The deflection of the formwork during construction shall not exceed 1/240 of the effective span of the formwork.

(d) The ends of the formwork shall be supported on beams in such a manner that after placement of concrete topping a support length of at least 75 mm shall be provided under the lower portions of the formwork. Such support shall be within 25 mm of the closer edges of the supporting beams.

(e) The top flanges of all adjacent supporting beams shall be connected by external straps or the formwork itself.

(f) When the deck slab is confined by straps, the straps and their connections shall be designed to satisfy the requirements specified in Items (a) to (e) of Clause 16.7.2.

(g) When the deck slab is restrained by a formwork, the concept shall have been verified by tests on full-scale models. In addition, the minimum area of cross-section of the formwork, in mm^2/mm, across a section parallel to the beams, A_f, shall be calculated as follows:

$$A_f = \frac{F_s S^2}{Et}$$

where

F_s = 6.0 MPa for outer panels and 5.0 MPa for inner panels

E = modulus of elasticity of the material of the formwork in the direction perpendicular to the supporting beams

(h) When the deck slab is restrained by a formwork, the direct or indirect connection of the formwork to the supporting beams shall have been proved by full-scale tests to have a shear strength in N/mm of at least $200A_f$.

(i) When the formwork is of precast concrete construction, it shall contain a crack-control orthogonal assembly of GFRP bars placed at its mid-depth, with an area of cross-section of GFRP bars in each direction equal to $0.0015t$ mm^2/mm. In addition, the spacing of transverse and longitudinal crack-control bars shall be not more than 300 mm.

(j) When it is of precast construction, the formwork panel shall have a maximum thickness of $0.5t$.

(k) When it is of precast construction, the upper surface of the formwork panel shall be clean, free of laitance, and roughened to an amplitude of 2 mm at a spacing of nearly 15 mm.

(l) The cast-in-place concrete shall have a crack-control orthogonal assembly of GFRP bars placed in the middle of the cast-in-place slab, with the area of the cross-section of the GFRP bars in each direction being at least $0.0015t_c$ mm^2/mm. In addition, the spacing of transverse and longitudinal crack-control bars shall be not more than 300 mm.

When the conditions specified in Clause 16.7.1 and the requirements specified in this Clause are not satisfied, the design of the slab shall require Approval.

16.7.4 Full-depth precast concrete deck slabs

The design of an externally restrained deck slab with full-depth precast concrete construction shall require Approval.

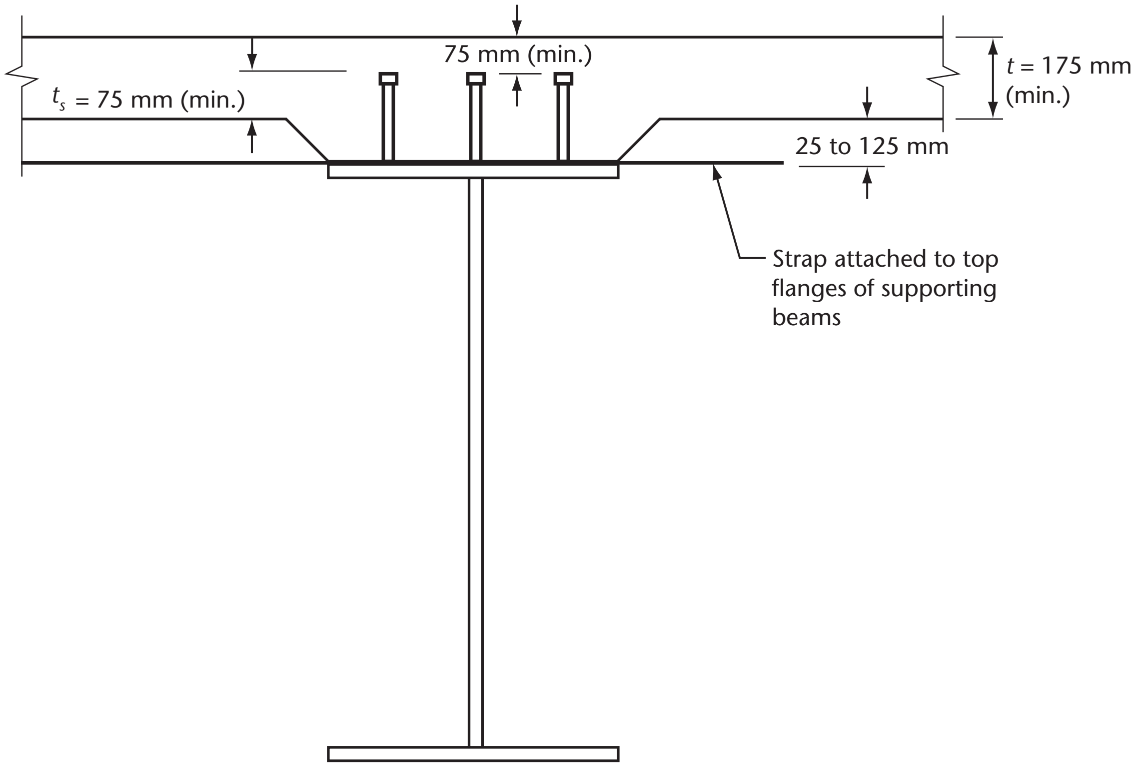


Figure 16.1
Distance between the deck slab and the top of the supporting beam
(See Clause 16.7.1.)

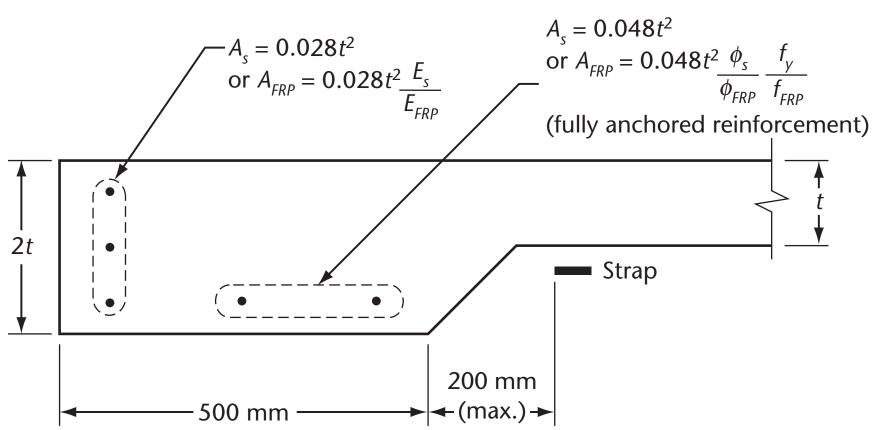


Figure 16.2
Detail of transverse edge stiffening
(See Clause 16.7.1.)

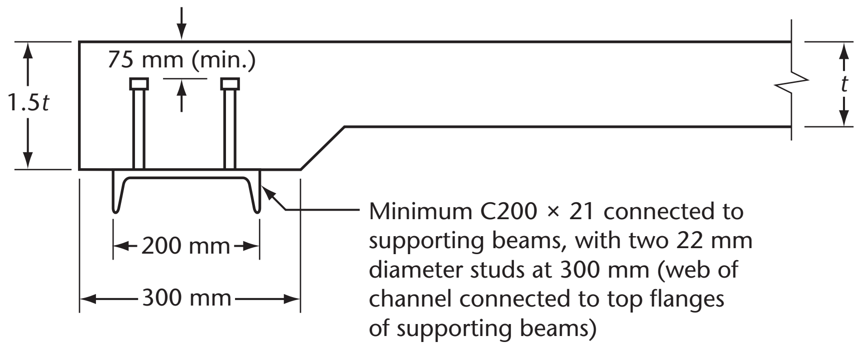


Figure 16.3
Detail of transverse edge stiffening
(See Clause 16.7.1.)

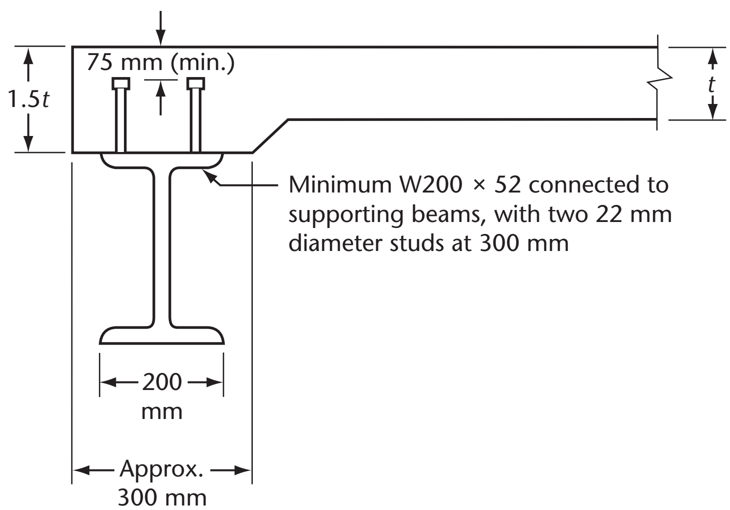


Figure 16.4
Detail of transverse edge stiffening
(See Clause 16.7.1.)

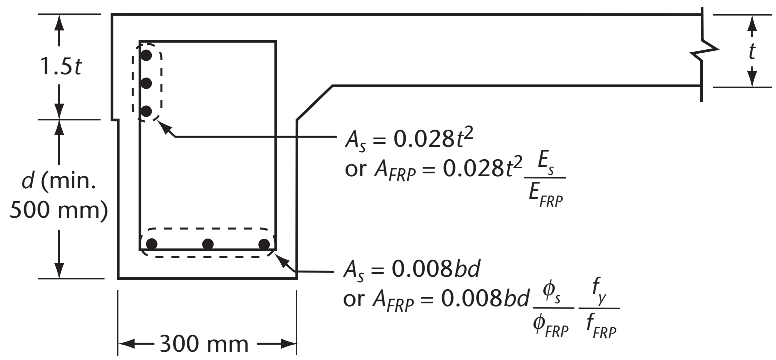


Figure 16.5
Detail of transverse edge stiffening
(See Clause 16.7.1.)

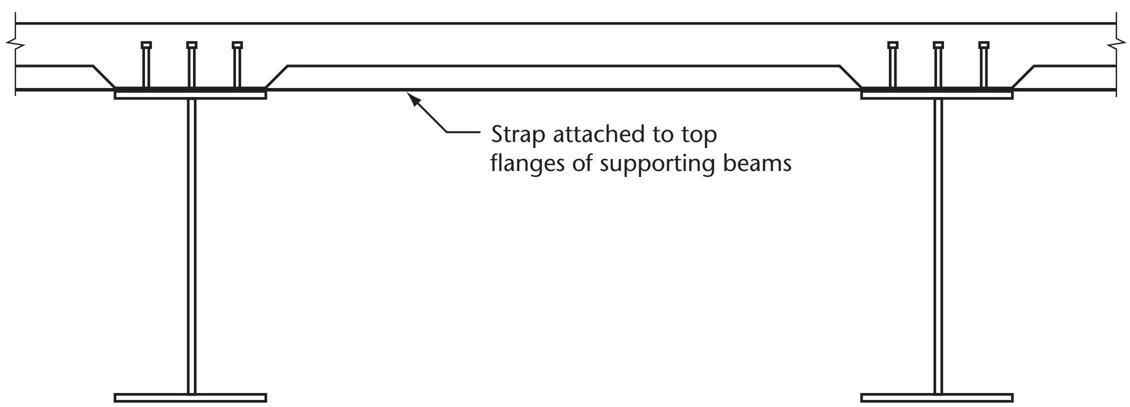


Figure 16.6
External transverse restraining system consisting of connected straps
(See Clause 16.7.2.)

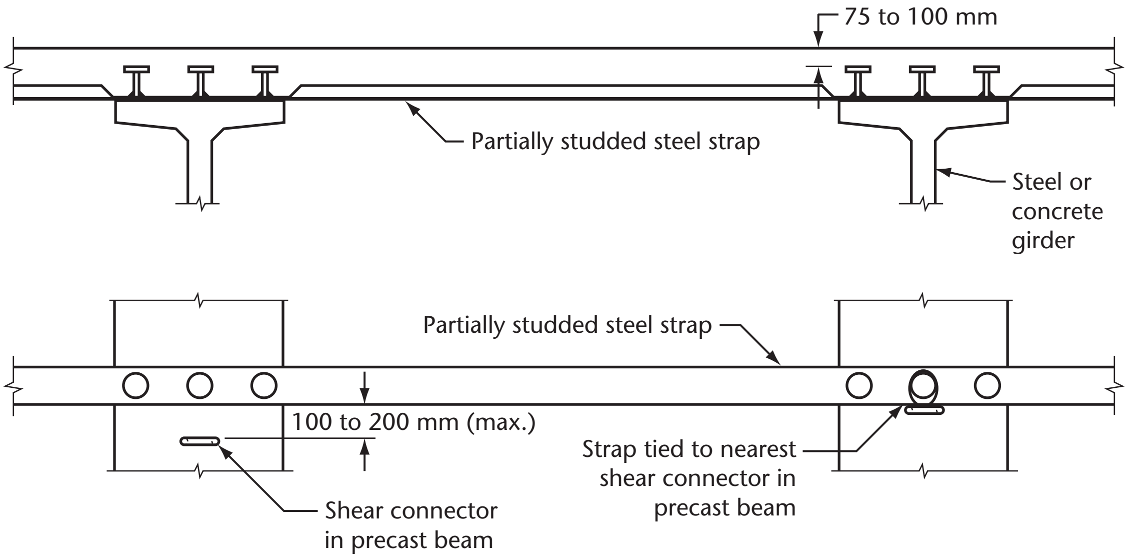


Figure 16.7
External transverse confining system consisting of indirectly connected partially studded straps
(See Clause 16.7.2.)

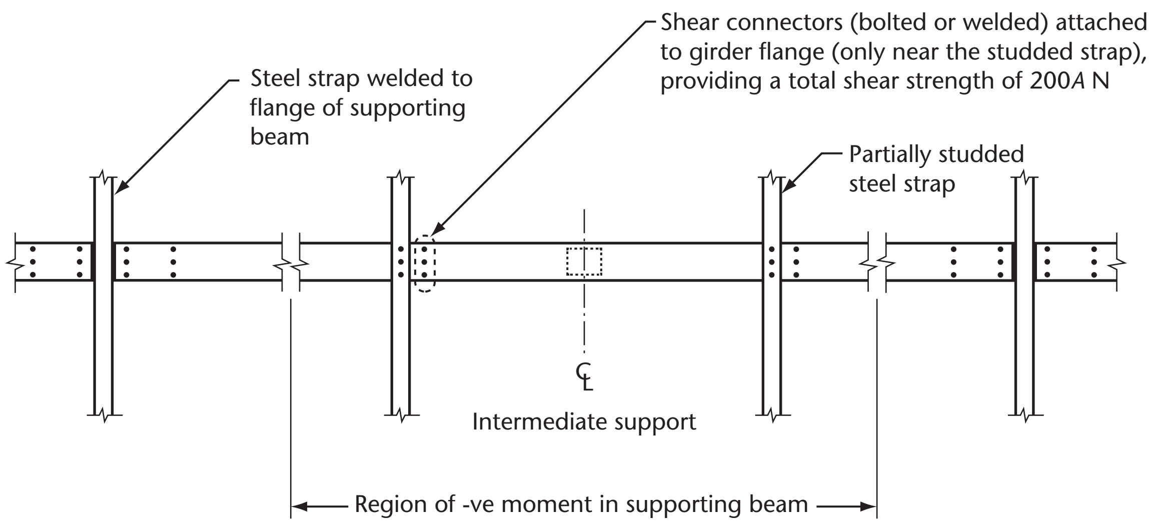


Figure 16.8
External transverse confining system in longitudinal negative moment regions
(See Clause 16.7.2.)

16.8 Concrete beams and slabs

16.8.1 General

Except as specified in this Section, the resistance and deformations of concrete beams, slabs, and deck slabs reinforced with FRP tendons, bars, or grids, corresponding to the various limit states, shall be calculated in accordance with Section 8.

The resistance at ULS for beams and slabs with FRP bars or grids in multiple layers shall be calculated by taking account of the linear variation of strain through the depth of the member, ensuring that the stresses in the reinforcement are consistent with Clause 16.8.3.

The maximum compressive concrete strains in beams and slabs due to factored loads shall not exceed the limiting strain specified in Section 8.

Internally restrained deck slabs shall be designed in accordance with Clause 16.8.8.

16.8.2 Deformability and minimum reinforcement

16.8.2.1 Design for deformability

For concrete components reinforced with FRP bars or grids, the overall performance factor, J, shall be at least 4.0 for rectangular sections and 6.0 for T-sections, with J calculated as follows:

$$J = \frac{M_{ult}\psi_{ult}}{M_c\psi_c}$$

where

M_{ult} = ultimate moment capacity of the section

ψ_{ult} = curvature at M_{ult}

M_c = moment corresponding to a maximum compressive concrete strain in the section of 0.001

ψ_c = curvature at M_c

16.8.2.2 Minimum flexural resistance

The factored resistance, M_r, shall be at least 50% greater than the cracking moment, M_{cr}, as specified in Clause 8.8.4.4. This requirement may be waived if the factored resistance, M_r, is at least 50% greater than the factored moment, M_f. If the ULS design of the section is governed by FRP rupture, M_r shall be greater than $1.5M_f$.

The principles for calculating M_{cr} and M_r shall be consistent with those specified in Clause 8.8, except that stresses in FRP bars at different levels, if present, shall be calculated by assuming a linear distribution.

16.8.2.3 Crack-control reinforcement

When the maximum tensile strain in FRP reinforcement under full service loads exceeds 0.0015, cross-sections of the component in maximum positive and negative moment regions shall be proportioned in such a way that the crack width does not exceed 0.5 mm for members subject to aggressive environments and 0.7 mm for other members, with the crack width calculated as follows:

$$w_{cr} = 2\frac{f_{FRP}}{E_{FRP}}\frac{h_2}{h_1}k_b\sqrt{d_c^{\,2} + (s/2)^2}$$

The value of k_b shall be determined experimentally, but in the absence of test data may be taken as 0.8 for sand-coated and 1.0 for deformed FRP bars. In calculating d_c, the clear cover shall not be taken greater than 50 mm.

16.8.3 Non-prestressed reinforcement

The maximum stress in FRP bars or grids under loads at SLS shall not be more than $F_{SLS} f_{FRPu}$, where F_{SLS} is as follows:
(a) for AFRP: 0.35;
(b) for CFRP: 0.65; and
(c) for GFRP: 0.25.

16.8.4 Development length for FRP bars and tendons

16.8.4.1 General

The development length, ℓ_d, for FRP bars in tension shall be calculated as follows:

$$\ell_d = 0.45 \frac{k_1 k_4}{\left[d_{cs} + K_{tr} \frac{E_{FRP}}{E_s} \right]} \left[\frac{f_{FRPu}}{f_{cr}} \right] A$$

All of the variables in this equation shall be in accordance with Clause 8.15.2.2, except as follows:
(a) The variable k_4 is the bar surface factor, being the ratio of the bond strength of the FRP bar to that of a steel deformed bar with the same cross-sectional area as the FRP bar, but not greater than 1.0. In the absence of experimental data, k_4 shall be taken as 0.8.
(b) The variable E_{FRP} is the modulus of elasticity of the FRP bar.
(c) The term ($d_{cs} + K_{tr} E_{FRP} / E_s$) shall not be taken greater than $2.5d_b$.
(d) The bond strength of the FRP bar shall be determined by testing or taken to be the bond strength specified by the manufacturer of the bar.

16.8.4.2 Splice length for FRP bars

The splice length for FRP bars in tension shall be $1.3\ell_d$, where ℓ_d is calculated in accordance with Clause 16.8.4.1. Spliced FRP bars shall not be separated by more than 150 mm.

16.8.5 Development length for FRP grids

For FRP grids in which the intersecting orthogonal bars have been demonstrated to be fully anchored, the development length shall include at least two transverse bars of the grid lying perpendicular to the direction of the force under consideration.

16.8.6 Tendons

16.8.6.1 Supplementary reinforcement

A structure incorporating concrete beams or slabs with FRP tendons shall contain supplementary reinforcement capable of sustaining the unfactored dead loads or have alternative load paths such that the failure of one beam or a portion of a slab will not lead to progressive collapse of the structure.

16.8.6.2 Stress limitations for tendons

For straight tendons, the maximum stress at jacking and transfer shall not exceed the values specified in Table 16.4. For curved tendons, the maximum stresses at jacking and transfer shall be those specified in Table 16.4, reduced by an amount determined from tests.

FRP tendons shall be stressed to provide a minimum effective prestress of 75% of the stresses at transfer.

The maximum SLS stresses after all prestress losses shall not exceed the F_{SLS} values specified in Clause 16.8.3.

The maximum stress in the tendons under factored loads at ULS, f_{ps}, computed using a method based on strain compatibility, shall not exceed $\phi_{FRP} f_{FRPu}$, where the resistance factor ϕ_{FRP} is as specified in Clause 16.5.3.

Table 16.4
Maximum permissible stresses in FRP tendons at jacking and transfer for concrete beams and slabs for pretensioning and post-tensioning systems
(See Clauses 16.8.6.2 and 16.9.4.)

Tendon	At jacking	At transfer
AFRP	$0.40f_{FRPu}$	$0.35f_{FRPu}$
CFRP	$0.70f_{FRPu}$	$0.65f_{FRPu}$
GFRP	$0.30f_{FRPu}$	$0.25f_{FRPu}$

16.8.6.3 Capacity of anchors

When tested in an unbonded condition, anchors for post-tensioning tendons shall be capable of developing a tendon force at least 50% higher than the jacking force (but not more than 90% of the specified tensile strength of the tendons) without exceeding the anticipated set. After tensioning and seating, anchors shall sustain applied loads without slippage, distortion, or other changes that result in loss of prestress. The Plans shall specify that at least two anchors are to be tested to confirm this requirement.

16.8.6.4 End zones in pretensioned components

The end zones of pretensioned concrete components shall be reinforced against splitting unless it can be demonstrated that such reinforcement is unnecessary.

16.8.6.5 Protection of external tendons

External tendons comprising glass or aramid fibres shall be protected against ultraviolet rays by encasing them in protective sheaths; carbon fibre external tendons with ultraviolet-susceptible matrices shall also be similarly protected. Aramid fibre ropes shall be protected by sheaths and watertight anchors against the ingress of moisture.

16.8.7 Design for shear

For concrete beams reinforced with steel or FRP longitudinal reinforcement, and with steel or FRP stirrups, the factored shear resistance, V_r, shall be calculated as follows:

$$V_r = V_c + V_{st} + V_p$$

where

$V_{st} = V_s$ or V_{FRP} (in accordance with the type of stirrups used in the beam)

Clause 8.9.3 shall be used to calculate V_c, V_s, and V_p, except as follows:

(a) Instead of calculating V_c in accordance with Clause 8.9.3.4, the following equation shall be used:

$$V_c = 2.5\beta\phi_c f_{cr} b_v d_{long}\sqrt{\frac{E_{long}}{E_s}}$$

The effective shear depth, d_{long}, shall be calculated in accordance with Clause 8.9.2.5.

(b) Instead of calculating ε_x in accordance with Clause 8.9.3.8, one of the following equations shall be used:

(i) $$\varepsilon_x = \frac{\dfrac{M_f}{d_{long}} + V_f - V_p + 0.5N_f - \left(A_p f_{po}\right)}{2\left(E_s A_s + E_p A_p\right)} \le 0.003$$

(ii) $$\varepsilon_x = \frac{\dfrac{M_f}{d_{long}} + V_f - V_p + 0.5N_f - \left(A_{FRP} f_{po}\right)}{2\left(E_s A_s + E_{FRP} A_{FRP}\right)} \le 0.003$$

(c) For the factored shear resistance carried by FRP shear reinforcement, V_{FRP}, the following equations shall be used:
  (i) for components with transverse reinforcement perpendicular to the longitudinal axis:
  $$V_{FRP} = \frac{\phi_{FRP} A_v \sigma_v d_{long} \cot\theta}{s}$$
  (ii) for components with transverse reinforcement inclined at an angle α to the longitudinal axis:
  $$V_{FRP} = \frac{\phi_{FRP} A_v \sigma_v d_{long} (\cot\theta + \cot\alpha)\sin\alpha}{s}$$

  For the equations in Items (i) and (ii), θ shall be obtained from Clause 8.9.3.6 for the simplified method or Clause 8.9.3.7 for the general method; the coefficient ϕ_{FRP} shall be as specified in Clause 16.5.3; and σ_v shall be the smaller of the values obtained from the following two equations:

  $$\sigma_v = \frac{(0.05r/d_s + 0.3) f_{FRPbend}}{1.5}$$

  $$= E_{vFRP} \varepsilon_v$$

  where

  $$\varepsilon_v = 0.0001 \left[f_c' \frac{\rho_s E_{FRP}}{\rho_{vFRP} E_{vFRP}} \right]^{0.5} \left[1 + 2\left[\frac{\sigma_N}{f_c'} \right] \right] \le 0.0025$$

(d) The minimum amount of shear reinforcement, $A_{v,min}$, shall be calculated as follows:

  $$A_{v,min} = 0.06\sqrt{f_c'}\,\frac{b_w s}{\sigma_v}$$

  where σ_v is as specified in Item (c).

16.8.8 Internally restrained cast-in-place deck slabs

16.8.8.1 Design by empirical method

The requirements of Clause 8.18 pertaining to cast-in-place deck slabs shall apply to cast-in-place deck slabs with FRP bars or grids, except that when the deck slab is designed using the empirical method of Clause 8.18.4, the following requirements shall be met in lieu of those specified in Items (a) and (c) of Clause 8.18.4.2:

(a) the deck slab shall contain two orthogonal assemblies of FRP bars, with the clear distance between the top and bottom transverse bars being at least 55 mm;
(b) for the transverse FRP bars in the bottom assembly, the minimum area of cross-section in mm^2/mm shall be $500d_s/E_{FRP}$; and
(c) the longitudinal bars in the bottom assembly and the transverse and longitudinal bars in the top assembly shall be of GFRP with a minimum ρ_s of 0.0035.

The edge-stiffening details shall be as shown in Figures 16.2 to 16.5.

16.8.8.2 Design for flexure

For cast-in-place deck slabs with FRP bars or tendons designed for flexure, the requirements of Clauses 16.8.2 to 16.8.6 shall apply. In addition, the requirements of Clause 8.18.5 shall be satisfied for diaphragms. The distribution reinforcement shall be designed in accordance with Clause 8.18.7.

16.9 Stressed wood decks

16.9.1 General

Clauses 16.9.2 to 16.9.6 shall apply to stress-laminated wood decks and stressed log bridges that are post-tensioned with FRP or fibre rope tendons.

16.9.2 Post-tensioning materials

16.9.2.1 Tendons

GFRP, AFRP, and aramid ropes shall be permitted as tendons in stressed wood decks. The design of stressed laminated wood decks shall comply with Section 9 (with the exception of Clause 9.23.3.4) and Clauses 16.9.2.2 to 16.9.6.

16.9.2.2 Anchors

Anchors shall be capable of developing a tendon force at least 50% higher than the jacking force. The Plans shall specify that at least two anchors are to be tested to confirm this requirement.

16.9.2.3 Stress limitations

At initial stressing, the stresses shall not exceed $0.35f_{FRPu}$ for GFRP tendons or $0.40f_{FRPu}$ for AFRP and aramid rope tendons.

16.9.3 Post-tensioning system

For stressed log bridges, the post-tensioning system shall be external or internal (the external system is shown in Figure 16.9). For stress-laminated wood decks, the post-tensioning system shall be as shown in Figure 9.5 or 9.6, except that the tendons shall comprise GFRP, AFRP, or aramid ropes. When lower tendons in decks with external post-tensioning are exposed to damage from flowing debris, they shall be suitably protected.

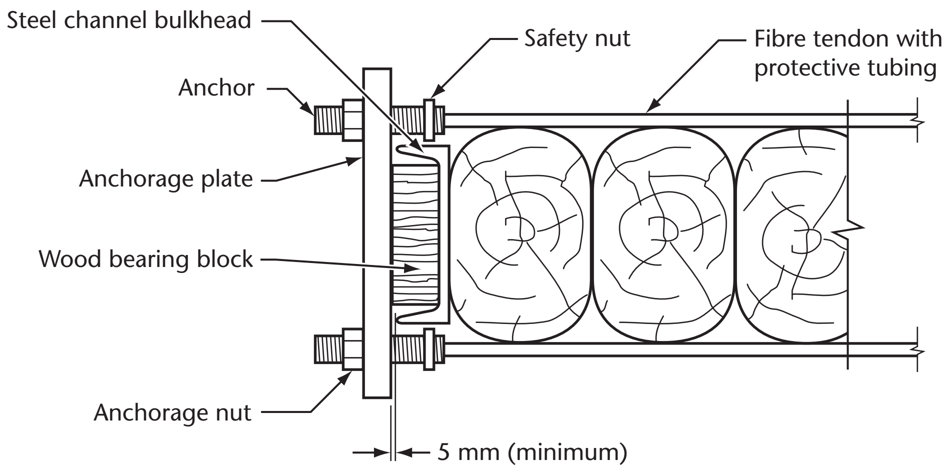


Figure 16.9
Post-tensioning system for stressed log bridges
(See Clause 16.9.3.)

16.9.4 Stressing procedure

The initial post-tensioning forces in the tendons shall be such as to bring the average interface pressure between wood laminates or logs to approximately 0.8 MPa, regardless of the species concerned. These forces shall be reduced 12 to 24 h after initial post-tensioning to establish an average interface pressure of 0.35 to 0.44 MPa, at which level the stresses in the tendons shall not exceed the relevant values specified in Table 16.4 for post-tensioning at transfer.

16.9.5 Design of bulkheads

Stressed wood decks shall incorporate steel distribution bulkheads, as specified in Clause 9.23.4, except that the factored bearing resistance, calculated in accordance with Clause 9.23.4.2, shall be for the reduced post-tensioning forces specified in Clause 16.9.4.

16.9.6 Stressed log bridges

16.9.6.1 General

A stressed log bridge shall be constructed of logs
(a) that are
  (i) new; or
  (ii) used but structurally sound; and
(b) whose exposed surfaces have been suitably treated in accordance with Section 9.

16.9.6.2 Log dimensions

The logs in a stressed log bridge shall meet the following requirements:
(a) the ratio of the largest to the smallest diameter of a log shall be not more than 1.10;
(b) the out-of-straightness of a log shall be not more than 0.003 times its length;
(c) the end faces of a log shall be perpendicular to its axis to within an angular tolerance of 5°; and
(d) the logs shall be trimmed longitudinally so as to provide, at any transverse cross-section, two opposed faces that are parallel within an accuracy of 2°.

16.9.6.3 Splicing at butt joints
As shown in Figure 16.10, all butt joints shall be spliced by hot-dipped galvanized steel nail-plates with a minimum thickness of 1.5 mm. The nail-plates shall be installed by using a hydraulic jack or other Approved means to apply uniform pressure.

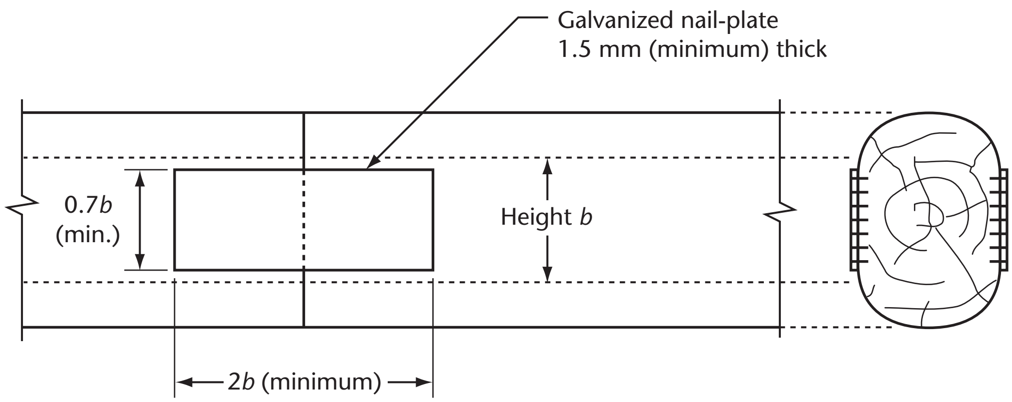


Figure 16.10
Spliced butt joint for logs
(See Clause 16.9.6.3.)

16.9.6.4 Frequency of butt joints
The butt joints shall be staggered in such a way that within any band with a width of 1 m measured along the logs, a butt joint shall not occur in more than one out of four adjacent logs on each side of the log with a butt joint.

16.9.6.5 Holes in logs for an internal system
The diameter of the holes drilled in the logs for an internal post-tensioning system shall be less than or equal to 20% of the minimum diameter of the logs.

16.9.6.6 Support anchorage
A stressed log bridge shall be secured to the substructure by steel bars in accordance with the requirements of Clause 9.23.5.5 for stress-laminated wood decks.

16.9.6.7 Surfacing
The top of a stressed log bridge shall be surfaced by hot-mix asphalt or FRC incorporating low modulus polymer fibres. A fibre-reinforced concrete surfacing may be assumed to be acting compositely with the logs if the smallest thickness of concrete is not more than 50 mm.

16.9.6.8 Flexural resistance and stiffness
The factored flexural resistance, M_r, of a stressed log bridge shall be calculated in accordance with Clause 9.6.1, except that the value of f_{bu} shall be obtained from Table 9.17, which shall also be used for obtaining the modulus of elasticity of the logs.

16.10 Barrier walls

A barrier wall that has the details shown in Figure 16.11 and satisfies the following conditions shall be deemed to have met the Performance Level 1 requirements of Section 12:

(a) on the traffic side, the wall has a GFRP grid or an orthogonal assembly of GFRP bars providing a factored strength of 330 N per millimetre length of the wall in the vertical direction and 240 N per millimetre length of the wall in the horizontal direction;

(b) the spacing of the bars in the GFRP grid or orthogonal assembly on the traffic side is not more than 300 mm;

(c) the wall is reinforced with
 (i) one planar GFRP grid or an orthogonal assembly of GFRP bars near the surface, as shown in Figure 16.11, and FRC is used in accordance with Clause 16.6; or
 (ii) two layers of planar GFRP grid or two orthogonal assemblies of GFRP bars (in which case FRC is not needed);

(d) if a planar GFRP grid or an orthogonal assembly of GFRP bars is provided away from the traffic side, it comprises bars of a diameter of at least 15 mm, at a spacing of 300 mm in the horizontal and vertical directions;

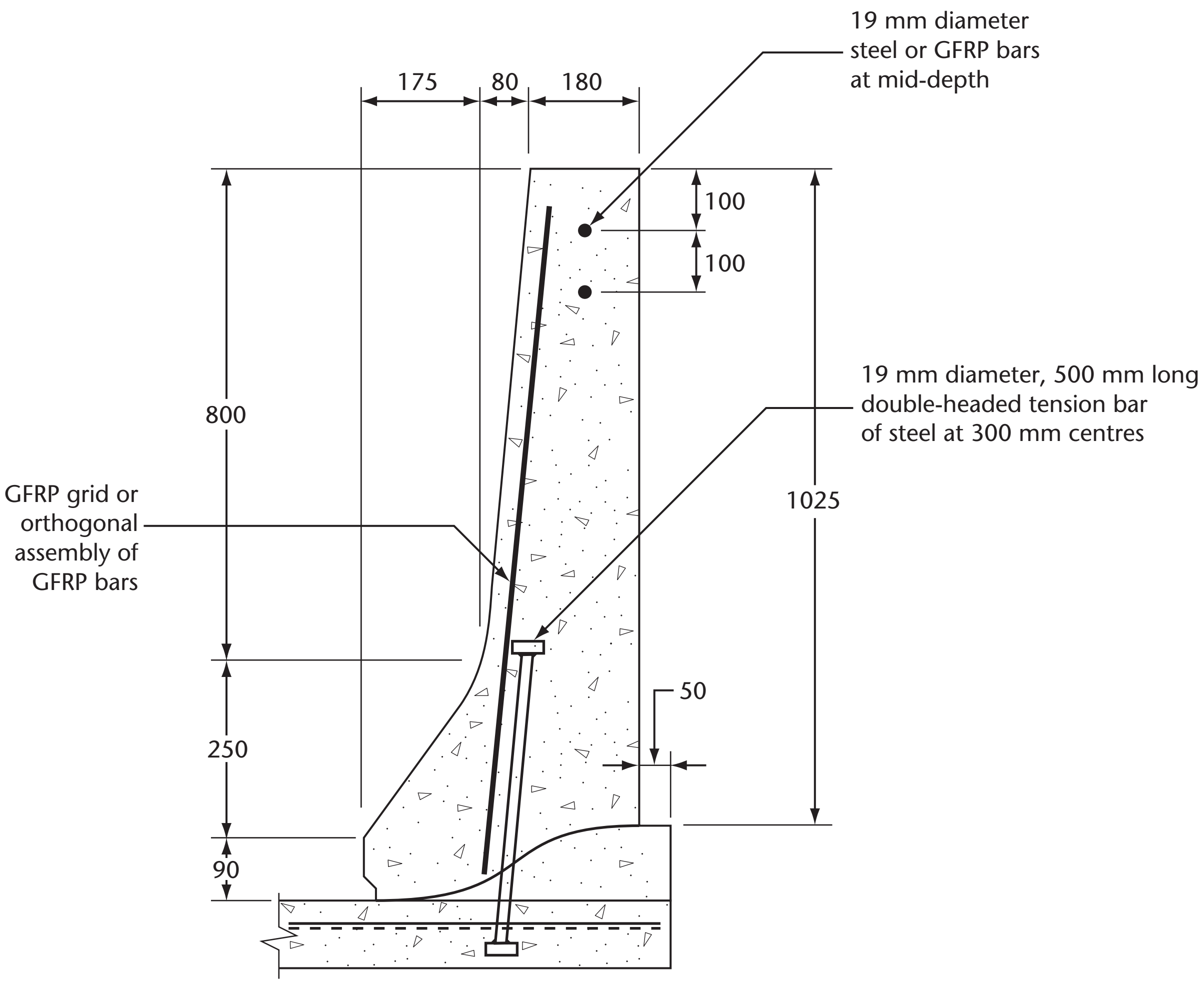
(e) if only one planar GFRP grid or orthogonal assembly of GFRP bars is used, the wall is provided at mid-thickness near its top with two 19 mm diameter steel bars or two 15 mm diameter GFRP bars, as shown in Figure 16.11;

(f) the wall is anchored to the slab by double-headed steel bars 19 mm in diameter and 500 mm long, or by bent GFRP bars whose performance has been established by Approved full-scale tests, at a spacing of 300 mm;

(g) the spacing of the bars and anchors is reduced by half for the following lengths of the wall:
 (i) 1.2 m on each side of a joint in the wall;
 (ii) 1.2 m on each side of a luminaire embedded in the wall; and
 (iii) 1.2 m from the free vertical edges of the wall; and

(h) the cover to GFRP bars (if any) meets the requirements of Clause 16.4.4 and the cover to double-headed bars (if any) meets the requirements of Clause 8.11.2.2.

Barrier walls for Performance Levels 1, 2, and 3 with details other than those shown in Figure 16.11 may be used if their performance has been established by Approved full-scale tests.



Note: *All dimensions are in millimetres.*

Figure 16.11
Cross-section of a barrier wall reinforced with GFRP
(See Clause 16.10.)

16.11 Rehabilitation of existing concrete structures with FRP

16.11.1 General

Clause 16.11 applies to existing concrete structures that have an f_c' of less than or equal to 50 MPa and are strengthened with FRP comprising externally bonded systems or NSMR. If the concrete cover is less than 20 mm, NSMR shall not be used. Rehabilitation of concrete structures with an f_c' of more than 50 MPa shall require Approval.

For situations where the concrete component contains corroded reinforcing steel, the causes of the corrosion shall be addressed and the corrosion-related deterioration shall be repaired before application of any FRP strengthening system.

For concrete structures strengthened with FRP, the Plans shall provide details and specifications relevant to the following and as specified in Annexes A16.1 and A16.2:
(a) identification of the FRP strengthening systems and protective coatings;
(b) concrete preparation;
(c) shipping, storage, and handling of the FRP strengthening systems;
(d) installation of the FRP strengthening systems, including
    (i) the spacing and positioning of the components;
    (ii) the locations of overlaps and multiple plies;
    (iii) installation procedures; and
    (iv) constraints for climatic conditions;
(e) the curing conditions of the strengthening systems;
(f) the quality control of the strengthening systems, as specified in Annex A16.2;
(g) staff qualifications;
(h) material inspection before, during, and after completion of the installation; and
(i) system maintenance requirements.

Before a rehabilitation strategy is developed, an assessment of the existing structure or elements shall be conducted in accordance with Section 14.

Only those structures that have a live load capacity factor, F, of 0.5 or greater as specified in Clause 14.15.2.1 shall be strengthened.

Consideration shall be given to the fact that FRP strengthening can result in a change in failure mode or in-service behaviour of a member or its adjacent members as a consequence of the increased loads or stresses.

16.11.2 Flexural and axial load rehabilitation

16.11.2.1 General

Clause 16.11.2 covers externally bonded FRP systems
(a) placed on or near the tension face of steel-reinforced concrete flexural members, with fibres oriented along the length of the member to provide an increase in flexural strength;
(b) placed on the external perimeter of concrete columns to enhance the axial load capacity of the columns; and
(c) for seismic upgrading.

Clause 16.11.2 shall not be used for seismic upgrading to enhance the flexural strength of members in the expected plastic hinge regions of ductile moment frames resisting seismic loads and shall not be used for the flexural strengthening of deep beams.

16.11.2.2 Assumptions for SLS and FLS calculations

In addition to being based on the conditions of equilibrium and compatibility of strains, SLS and FLS calculations shall be based on the assumptions of Clause 8.8.2 and on the assumption that strain changes in the FRP strengthening system are equal to the strain changes in the adjacent concrete.

16.11.2.3 Assumptions for ULS calculations

In addition to being based on the conditions of equilibrium and compatibility of strains, ULS calculations shall be based on the material resistance factors specified in Clauses 8.4.6 and 16.5.3, the assumptions of Clause 8.8.3, the assumption that strain changes in the FRP strengthening systems are equal to the strain changes in the adjacent concrete, and the assumption that the contribution of FRP in compression will be neglected.

For an externally bonded flexural strengthening system, the maximum value of the strain in the FRP shall not exceed 0.006.

16.11.2.4 Flexural components

16.11.2.4.1 Failure modes

For a section strengthened with FRP systems, the following flexural failure modes shall be investigated at ULS:

(a) crushing of the concrete in compression before rupture of the FRP or yielding of the reinforcing steel;
(b) yielding of the steel followed by concrete crushing before rupture of the FRP in tension;
(c) yielding of the steel followed by rupture of the FRP in tension;
(d) in the case of members with internal prestressing, additional failure modes controlled by the rupture of the prestressing tendons; and
(e) peeling failure or anchorage failure of the FRP system at the cut-off point.

16.11.2.4.2 Flexural resistance to sustained and fatigue loads

At SLS and/or FLS, stresses shall be calculated based on elastic analysis. The effect of the FRP system on the serviceability may be assessed using transformed section analysis. The initial stresses and strains related to unfactored dead loads in a beam before strengthening shall be considered.

In addition to satisfying the requirements of Clause 8.5.2 for SLS and Clause 8.5.3 for FLS, the stress level in the FRP system due to all dead loads after strengthening and SLS live loads shall not exceed $F_{SLS} f_{FRPu}$, where the value of F_{SLS} is obtained from Clause 16.8.3.

16.11.2.4.3 Factored flexural resistance

The factored flexural resistance shall be calculated in accordance with Clause 16.11.2.3.

16.11.2.4.4 Anchorage lengths for flexure

For externally-bonded FRP strengthening systems, the anchorage length beyond the point where no strengthening is required shall not be less than ℓ_a, calculated as follows:

$$\ell_a = 0.5\sqrt{E_{FRP} t_{FRP}}$$

In addition, the anchorage length shall be at least 300 mm or the FRP shall be suitably anchored.

For NSMR, the anchorage length, ℓ_d, beyond the point where no strengthening is required shall be calculated in accordance with Clause 16.8.4.1.

16.11.2.5 Compression components

16.11.2.5.1 General

For FRP-strengthened columns subjected to combined flexure and axial compression, the factored resistance shall be calculated in accordance with Clause 16.11.2.3.

16.11.2.5.2 Slenderness effects

The slenderness effects shall be accounted for in accordance with Clauses 8.8.5.2 and 8.8.5.3.

16.11.2.5.3 Maximum factored axial resistance

For columns with FRP systems bonded to the external perimeter that meet the requirements of Clause 16.11.2.3, the factored axial resistance, P_r, shall be less than or equal to $0.80P_o$.

The confined concrete compressive strength determined in accordance with Clause 16.11.2.5.6 may be used to evaluate P_o.

16.11.2.5.4 Biaxial loading

An analysis based on stress and strain compatibility for a loading condition of compression and biaxial bending shall be used to design the FRP strengthening system.

16.11.2.5.5 Transverse reinforcement
If the existing steel transverse reinforcement does not meet the requirements of Clause 8.14.4, FRP transverse reinforcement shall be provided in accordance with Clause 16.11.3.

16.11.2.5.6 Axial load capacity enhancement
The compressive strength of the confined concrete, f'_{cc}, shall be calculated as follows:

$$f'_{cc} = f'_c + 2f_{\ell FRP}$$

The confinement pressure due to FRP strengthening at the ULS, $f_{\ell FRP}$, shall be calculated as follows:

$$f_{\ell FRP} = \frac{2\phi_{FRP} f_{FRPu} t_{FRP}}{D_g}$$

For columns with circular cross-sections, D_g is the diameter of the column; for columns with rectangular cross-sections with aspect ratios less than or equal to 1.5 and a smaller cross-sectional dimension not greater than 800 mm, D_g is equal to the diagonal of the cross-section. For columns with other polygonal cross-sections, D_g is equal to the diameter of the inscribed circle.

The confinement pressure at the ULS shall be designed to be between $0.1f'_c$ and $0.33f'_c$.

16.11.3 Shear rehabilitation with externally bonded FRP systems

16.11.3.1 General
Clause 16.11.3 covers the proportioning of externally bonded FRP systems to increase the shear capacity of reinforced concrete beams and columns. The shear-strengthening scheme shall be of the type in which the fibres are oriented perpendicularly or at an angle β to the member axis.

The shear reinforcement shall be anchored by suitable means in the compression zone in accordance with one of the following schemes:

(a) the shear reinforcement shall be fully wrapped around the section, as shown in Figure 16.12;

(b) the anchorage to the shear reinforcement near the compression flange shall be provided by additional horizontal strips, as shown in Figure 16.12; and

(c) the anchorage shall be provided in the compression zone, as shown in Figure 16.12.

However, an alternative anchorage scheme may be used if Approved.

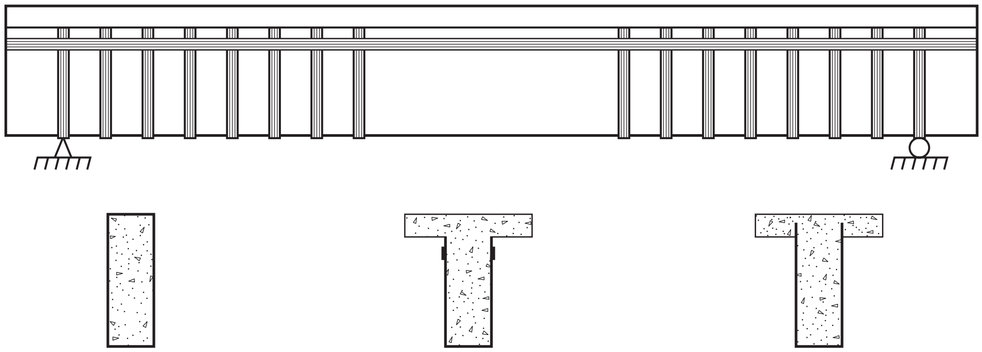


Figure 16.12
Anchorage methods in the compression zone of externally bonded FRP shear reinforcement
(See Clause 16.11.3.1.)

16.11.3.2 Factored shear resistance

For reinforced concrete members with rectangular sections or T-sections and FRP reinforcement anchored in the compression zone of the member, the factored shear resistance, V_r, shall be calculated as follows:

$$V_r = V_c + V_s + V_{FRP}$$

V_c and V_s shall be calculated in accordance with Clause 8.9.3 and V_{FRP} shall be calculated as follows:

$$V_{FRP} = \frac{\phi_{FRP} E_{FRP} \varepsilon_{FRPe} A_{FRP} d_{FRP} (\cot\theta + \cot\beta)\sin\beta}{s_{FRP}}$$

where

ε_{FRPe} = $0.004 \le 0.75\varepsilon_{FRPu}$ (for completely wrapped sections)

= $\kappa_V \varepsilon_{FRPu} \le 0.004$ (for other configurations)

For continuous U-shaped configurations of the FRP reinforcement, the bond-reduction coefficient, κ_V, shall be calculated as follows:

$$\kappa_V = \frac{k_1 k_2 L_e}{11900\varepsilon_{FRPu}} \le 0.75$$

where

$$k_1 = (f_c'/27)^{2/3}$$

$$k_2 = \frac{d_{FRP} - L_e}{d_{FRP}}$$

$$L_e = \frac{23300}{(t_{FRP}E_{FRP})^{0.58}}$$

For prestressed concrete components, V_r shall be the sum of V_c, V_s, V_p, and V_{FRP}. The equations in this Clause shall be used to calculate V_{FRP}. The general method of Clause 8.9.3 shall be used to calculate V_c, V_s, and V_p.

For components with non-rectangular or non-T cross-sections, a rigorous analysis or test shall guide the design.

16.11.3.3 Spacing and strengthening limits

The spacing of FRP bands shall be not more than s_{FRP}, calculated as follows:

$$s_{FRP} = w_{FRP} + \left(d_{FRP} / 4\right)$$

The total factored shear resistance subsequent to FRP strengthening, V_r, shall not exceed $0.66 b_w d (f'_c)^{0.5}$.

16.12 Rehabilitation of timber bridges

16.12.1 General

Clause 16.12 applies to beams of timber and stringer grades strengthened with GFRP sheets or bars. The bars, if present, shall be near-surface mounted or embedded in holes in timber. The empirical methods specified in Clause 16.12 may be used to determine the strength of timber beams strengthened with GFRP sheets or bars for flexure, shear, or both.

Although the requirements of Clause 16.12 are intended for GFRP bars and sheets, AFRP and CFRP bars and sheets may be used in place of GFRP bars and sheets. If the strength for either flexure or shear needs to be more than is provided by the empirical methods of Clause 16.12, experimental evidence shall be used to determine the amount of FRP reinforcement.

The procedures for handling, storage, and protection of FRP sheets and bars shall be the same as those specified in Clause 16.4.8.

The Plans shall provide details and specifications relevant to the following and as specified in Annexes A16.1 and A16.2:

(a) identification of the FRP strengthening systems and protective coatings;
(b) surface preparation;
(c) shipping, storage, and handling of the FRP strengthening systems;
(d) installation of the FRP strengthening systems, including
  (i) spacing and positioning of the components;
  (ii) locations of overlaps and multiple plies;
  (iii) installation procedures; and
  (iv) constraints for climatic conditions;
(e) the curing conditions of the strengthening systems;
(f) the quality control of the strengthening systems, as specified in Annex A16.2;
(g) staff qualifications;
(h) material inspection before, during, and after completion of the installation; and
(i) system maintenance requirements.

Before a rehabilitation strategy is developed, an assessment of the existing structure or elements shall be conducted in accordance with Section 14.

16.12.2 Strengthening for flexure

16.12.2.1 Flexural strengthening with GFRP sheets

When the following minimum conditions for strengthening with GFRP sheets are satisfied, the bending strength for beam and stringer grades used for the evaluation shall be $K_{bFRP} f_{bu}$, in which K_{bFRP} shall be obtained from Table 16.5 and f_{bu} from Table 9.13:

(a) The minimum fibre volume fraction of the GFRP system in the direction of the span of the beam is 30%.
(b) The GFRP sheet on the flexural tension face of the beam covers at least 90% of the width of the beam and has a minimum thickness of 0.1 mm.
(c) The adhesive used for bonding the GFRP sheets to the timber beam is compatible with the preservative treatment used on the timber.
(d) In the longitudinal direction of the beam, the GFRP sheets extend as close to the beam supports as possible.
(e) The adhesive used for bonding the GFRP sheets to the timber beam is compatible with the expected volumetric changes of the timber.

Table 16.5
Values of K_{bFRP}
(See Clause 16.12.2.)

Grade of original beam	K_{bFRP}
SS	*
No. 1	1.2
No. 2	1.5

**This value shall be 1.05 if the beam is not strengthened for shear and 1.1 if the beam is strengthened for shear.*

16.12.2.2 Flexural strengthening with GFRP NSMR

When the following minimum conditions for strengthening with GFRP NSMR are satisfied, the bending strength for beam and stringer grades used for the evaluation shall be $K_{bFRP} f_{bu}$, in which K_{bFRP} shall be obtained from Table 16.5 and f_{bu} from Table 9.13:

(a) The minimum fibre volume fraction of GFRP bars is 60%.
(b) There are at least two bars within the width of the beam.
(c) The total cross-sectional area for all bars on a beam is at least 0.002 times the cross-sectional area of the timber component.
(d) As shown in Figure 16.13, each bar is embedded in a groove (preferably with a rounded end). The depth of each groove is between 1.6 to 2.0 times d_b, the bar diameter; the width of each groove is not less than d_b plus 5 mm; the edge distance of the outer groove is not less than 25 mm and not less than $2d_b$; and the clear spacing between grooves is not less than 25 mm and not less than $3d_b$.
(e) The grooves in the beams are cleaned with pressurized air to remove any residue before the GFRP bars are embedded in them.
(f) The adhesive used for bonding the GFRP bars to the timber beam is compatible with the preservative treatment used on the timber and with the expected volumetric changes of the timber.
(g) In the longitudinal direction of the beam, the GFRP bar extends as close to the beam support as possible.
(h) Each GFRP bar is held in place as close to the tip of the groove as possible.

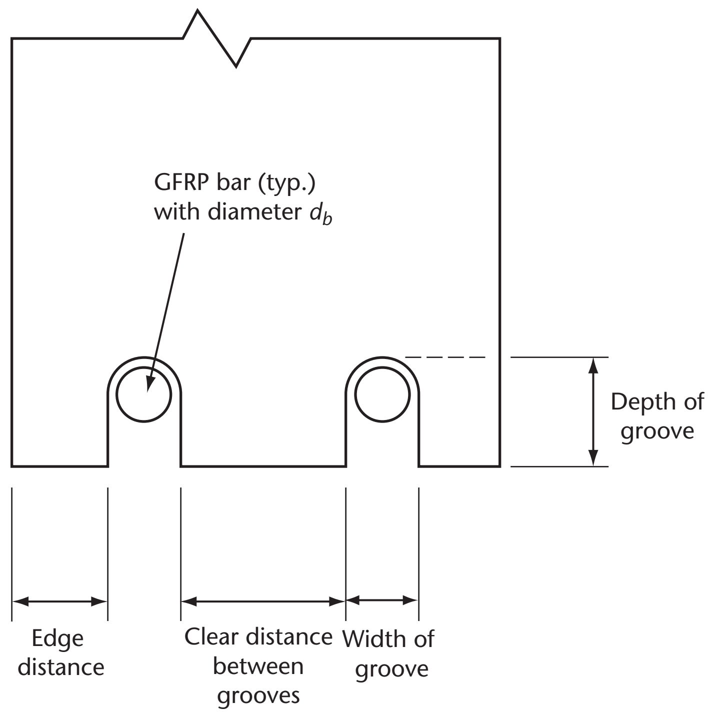


Figure 16.13
Cross-section of a timber beam with GFRP NSMR
(See Clause 16.12.2.2.)

16.12.3 Strengthening for shear

16.12.3.1 Shear strengthening with GFRP sheets

When the following minimum conditions for shear strengthening with GFRP sheets are satisfied, the shear strength for beam and stringer grades used for the evaluation shall be assumed to be $K_{vFRP} f_{vu}$, in which K_{vFRP} shall be taken as 2.0 and f_{vu} shall be obtained from Table 9.13:

(a) The minimum fibre volume fraction of the GFRP sheets along their axes is 30% and the sheets have a minimum thickness of 0.1 mm.
(b) Horizontal splits in beams, if present, are closed by a mechanical device before the application of the GFRP sheets.
(c) The GFRP sheets have at least the same width as the width of the cross-section of the beam (see Figure 16.14(a)).
(d) As shown in Figure 16.14(a), the GFRP sheet is inclined to the beam axis at an angle of 45 ± 10° from the horizontal.
(e) The top of the inclined GFRP sheet is as close to the centreline of the beam support as possible.
(f) The adhesive used for bonding the GFRP sheets to the timber beam is compatible with the preservative treatment used on the timber and with the expected volumetric changes of the timber.
(g) The top of the inclined GFRP sheet extends up to nearly the top of the beam.
(h) The lower end of the inclined GFRP sheet extends to the bottom of the beam if no dap is present (see Figure 16.14(a)). If there is a dap, the lower end is wrapped around the bottom and extends to at least half the width of the beam. In the latter case, the corner of the beam is rounded to a minimum radius of 12.5 mm to provide full contact of the sheet with the beam (see Figure 16.14(b)).

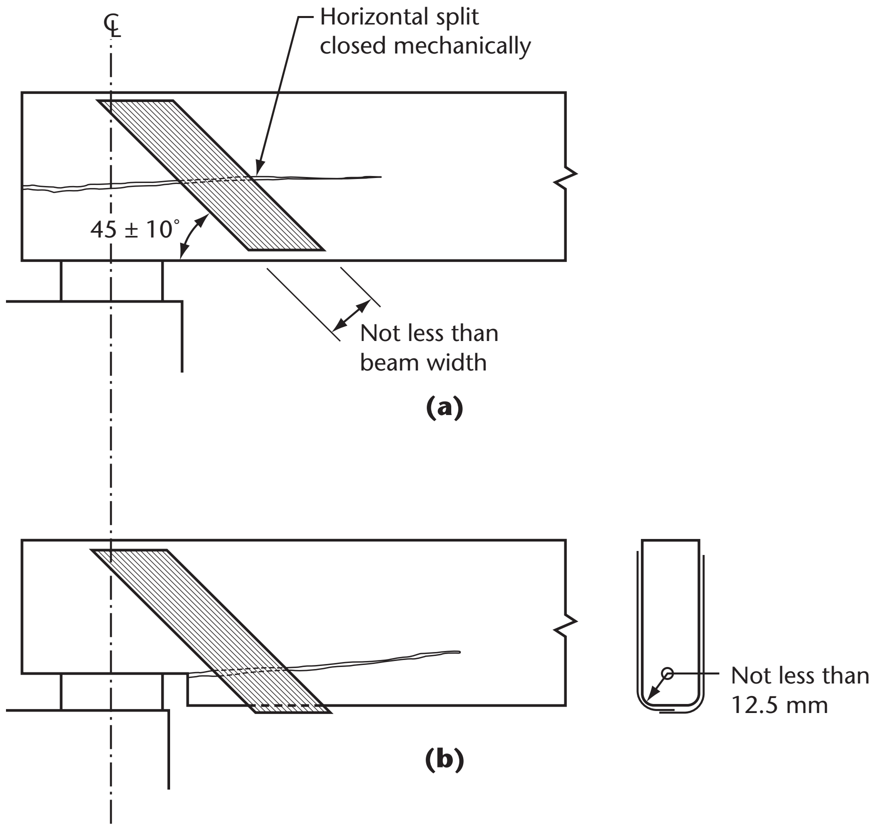


Figure 16.14
Elevation of timber beam with GFRP sheets for shear strengthening
(See Clause 16.12.3.1.)

16.12.3.2 Shear strengthening with GFRP embedded bars

When the following minimum conditions for shear strengthening with GFRP bars are satisfied, the shear strength for beam and stringer grades used for the evaluation shall be assumed to be $K_{vFRP} f_{vu}$, in which K_{vFRP} shall be taken as 2.2 and f_{vu} shall be obtained from Table 9.13:

(a) The minimum fibre volume fraction of the GFRP bars is 60%.
(b) Horizontal splits in beams, if present, are closed by a mechanical device before insertion of the GFRP bars.
(c) As shown in Figure 16.15, there are at least three GFRP bars at each end of the beam.
(d) The diameter of the GFRP bar, d_b, is at least 15 mm, and the minimum diameter of a hole containing a bar is d_b plus 3 mm.
(e) The spacing of bars along the length of the beam is h ± 25 mm.
(f) The adhesive used for bonding the GFRP bars to the timber beam is compatible with the preservative treatment used on the timber and with the expected volumetric changes of the timber.
(g) As shown in Figure 16.15, the GFRP bars are inclined to the beam axis at an angle of 45 ± 10° from the horizontal.
(h) The tops of the inclined GFRP bars are 10 to 25 mm from the top of the beam.
(i) When daps are present, the ingress of the drilled hole is 100 ± 10 mm from the edge of the dap.

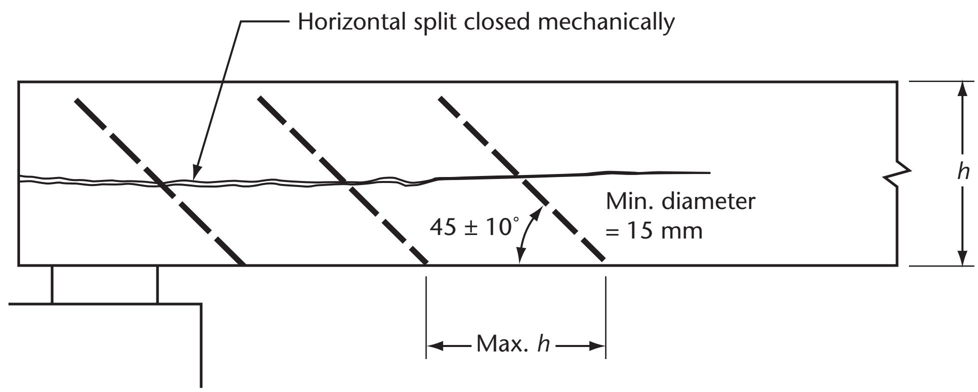


Figure 16.15
Elevation of timber beam with GFRP bars for shear strengthening
(See Clause 16.12.3.2.)

Annex A16.1 (normative) ***Installation of FRP strengthening systems***

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

A16.1.1 General

The selected material and its installation shall comply with the project specifications and drawings. The Plans, which include drawings, specifications, and submittals, shall comply with Clause 1.4.4.5.

Qualified, experienced, and properly educated workers shall perform the strengthening work under the supervision of qualified site engineers in accordance with Clause A16.2.2.

Because of the large variety of systems available for any given structural application, the installation procedure shall follow the manufacturer's recommendations.

A16.1.2 Shipping, storage, and handling of FRP systems

The shipping, storage, and handling of all fibre, resin, and FRP systems shall be performed in accordance with the manufacturer's specifications. The fibre and resin components shall be protected from water, humidity, and excessive cold and heat throughout storage, handling, placement, and curing. The resin components shall be stored and handled in well-ventilated areas. Materials and components that are damaged, past their shelf life, or contaminated shall not be used. The Plans shall indicate the appropriate specifications.

Safety measures in accordance with the applicable safety and environmental regulations shall be followed.

A16.1.3 Installation of FRP systems

A16.1.3.1 General

The FRP systems, including primer and putty when applicable, shall be installed on concrete surfaces approved by the Engineer and prepared in accordance with Clause A16.1.4.

A16.1.3.2 Spacing and positioning

The specified FRP positioning, ply orientation, and ply stacking sequence shall be followed.

Sheet and fabric materials shall be handled in a manner that maintains fibre straightness and orientation. Fabric with kinks, folds, or other forms of severe waviness shall be removed (if already installed) and discarded.

A16.1.3.3 Overlaps and multiple sheets

Overlaps and multiple sheets for an FRP system may be used only when permitted by the manufacturer.

Overlap splicing of FRP reinforcement shall be performed only as permitted by the Plans. Overlap length sufficient to prevent debonding in the overlapped area shall be provided.

Jacket-type FRP systems used for column members shall provide appropriate overlap length at splices, joints, and termination points to prevent failure of the spliced section.

Multiple sheets may be used if they are fully impregnated with the resin system and the installation of any particular sheet does not disturb the sheets already installed.

When an interruption of the FRP system laying-up process occurs, interlayer surface preparation such as cleaning or light sanding can be necessary. When interlayer surface preparation is necessary, the manufacturer's recommendations shall be followed.

A16.1.3.4 Installation procedure

The installation procedure for an FRP system shall follow the application steps recommended by the manufacturer.

When needed for externally bonded FRP systems, primer and putty shall be applied to an appropriate thickness and in an appropriate sequence. They shall also be allowed to cure as specified by the FRP manufacturer before subsequent materials are applied. The primer shall be applied to all areas on the concrete surface where the FRP system is to be placed. Rough edges or trowel lines of cured putty shall be smoothed before the installation is continued.

For hand-applied wet lay-up systems, the reinforcing fibre material shall be impregnated with the saturating resin in a manner recommended by the FRP manufacturer in order to obtain full saturation. The installation of the fibres of an externally bonded FRP section shall be completed within the pot life of the saturating resin.

The pre-cured plate surfaces to be bonded with an adhesive resin shall be cleaned and prepared in accordance with the FRP manufacturer's recommendations. The installation of an FRP reinforcing plate shall be completed within the pot life of the adhesive resin.

The primer, putty, and saturating and adhesive resins shall be installed within their respective pot lives and in accordance with the FRP manufacturer's recommended rate. Entrapped air under plates or sheets or between layers shall be released or rolled out before the resin sets.

A protective and aesthetic coating compatible with the proposed system shall be applied in accordance with the manufacturer's recommendations and the requirements of the Plans.

For NSMR, the grooves shall be cleaned after sawing; all concrete dust, wet concrete, or laitance shall be removed; and the grooves shall be dried before bonding.

A16.1.3.5 Climatic conditions

The following temperature and humidity requirements shall apply during installation of an FRP strengthening system:

(a) air and concrete surface temperature: more than 10 °C;
(b) concrete surface temperature above the actual dew point: more than 3 °C; and
(c) atmospheric relative humidity: less than 85%.

Free moisture shall not be present on the concrete surface. The surrounding temperature and relative humidity shall be continually recorded during the strengthening phase.

A16.1.4 Concrete preparation and condition for externally bonded FRP systems

The concrete surfaces shall be free of particles and pieces that no longer adhere to the structure. Oil residuals and contaminants shall be removed by cleaning. The Engineer and the manufacturer's representative shall inspect and approve the surfaces before installation proceeds.

The surface shall be blast-cleaned within an appropriate period of time and/or protected before FRP installation so that no additional materials that could interfere with the bond are redeposited on the surface. The surface roughness shall be in accordance with the manufacturer's specifications. All laitance, dust, dirt, oil, curing compounds, existing coatings, and any other matter that could interfere with the bond of the FRP shall be removed. The concrete surface to which the FRP will be applied shall be generally smooth. Small holes and voids shall be filled in accordance with the FRP manufacturer's specifications.

The concrete surfaces shall be repaired or reshaped in accordance with the original section with the material indicated in the Plans. Sections with sharp edges shall be rounded to a minimum radius of 35 mm before the FRP system is installed. The repaired surfaces shall be compatible, firmly adhere to the parent concrete, and be adequately cured before surface preparation and FRP system application. The repaired surfaces shall meet the requirements specified by the FRP system manufacturer.

For bond-critical applications, the concrete substrate shall have a minimum tensile strength of 1.5 MPa, as measured by a tension test in accordance with ASTM D 4541.

For contact-critical applications, the surface preparation shall ensure a continuous contact between the concrete and the FRP system.

The depth of the depressions on the concrete surface over a bond length of 0.3 or 2.0 m shall be not more than the applicable values specified in Table A16.1.1.

Table A16.1.1
Maximum depth of depressions on the concrete surface
(See Clause A16.1.4.)

Type of FRP	Maximum depth, mm, over a bond length of 0.3 m	Maximum depth, mm, over a bond length of 2.0 m
Plates ≥ 1.0 mm	4	10
Plates < 1.0 mm	2	6
Sheets	2	4
NSMR	—	—

A16.1.5 Curing conditions for FRP systems

FRP systems shall be cured in accordance with the manufacturer's recommendations. An FRP system shall be maintained in acceptable conditions for resin hardening during this period. During the cure, the temperature shall be maintained above the specified minimum; contamination and condensation on the surface shall be prevented.

Annex A16.2 (normative) **Quality control for FRP strengthening systems**

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

A16.2.1 General

Before construction, the designer shall decide whether the recommended design values and quality control documentation provided by the manufacturer are acceptable. If they are determined not to be acceptable, verification tests shall be carried out on the FRP material before use.

The following information shall be provided by the manufacturer:

(a) the identification of the FRP system, including restrictions or limitations on its use; and
(b) the results of quality control tests for verifying relevant properties, if required.

The quality control and inspection programs shall be carried out in accordance with the Plans.

A16.2.2 Staff qualifications

The strengthening work shall be performed by qualified personnel. The site engineer and the inspector shall also be qualified.

A16.2.3 FRP material inspection

A16.2.3.1 General

The FRP materials shall be inspected before, during, and after their installation. The inspection program shall cover such aspects as the presence and extent of delaminations, the cure of the installed system, adhesion, plate thickness, fibre alignment, and material properties.

A16.2.3.2 Before construction

The FRP material supplier shall submit a certification and identification of all FRP materials to be used. The quantity, location, and orientation of all FRP reinforcing materials to be used, as well as information concerning shelf life, pot life, and gel time, shall be provided.

Performance tests on the supplied materials shall be performed in accordance with the quality control and quality assurance test plan and shall meet the requirements specified in the Engineer's performance specifications.

Note: *These tests can include measuring such parameters as the tensile strength, glass transition temperature, and adhesive shear strength.*

A16.2.3.3 During construction

Special care shall be taken to keep all records on the quantity of mixed resin produced each day, the date and time of mixing, the components in the mixture, the mixture proportions, the ambient temperature, the humidity, and other factors affecting the resin properties. These records shall also identify the FRP material used each day, its location on the structure, the ply count and direction of application, and all other pertinent information.

Note: *It is possible that visual inspection of fibre orientation and waviness will be necessary for specific FRP systems with poor orientation, which implies misalignment of the entire system from the angles specified in the drawings.*

A16.2.3.4 On completion of the project

On completion of the project, a record of all final inspection and test results related to the FRP materials shall be retained and shall include delamination and repair, on-site bond tests, anomalies and correction reports, and mechanical and physical test results from the designated laboratories. Samples of the cured FRP materials shall be retained by the Engineer.

An inspection of the FRP repair system shall be conducted after the full cure. Delaminations or other anomalies that are detected shall be evaluated by considering their size and number relative to the overall application area, as well as their location with respect to structural load transfer. The inspection methods may include acoustic sounding (hammer sounding), ultrasonics, and thermography, and shall be capable of detecting delaminations of 1500 mm^2 or greater.

Approved methods for repairing FRP materials with delaminations may be used depending on the size and number of delaminations and their locations. Cutting away the affected sheet and applying an overlapping sheet patch of equivalent plies may be used in cases where delaminations are larger than 1500 mm^2 or exceed 5% of the total laminate area. The sheet shall be reinspected following the repairs and the resulting delamination map or scan shall be compared with that of the initial inspection to verify that the repairs were properly carried out.

In rehabilitation of concrete structures, tension bond testing of cored samples shall be conducted for FRP sheet systems. Tension bond strength values less than 1.5 MPa shall be considered unacceptable.

Proposition de modification

N'hésitez pas à nous faire part de vos suggestions et de vos commentaires. Au moment de soumettre des propositions de modification aux normes CSA et autres publications CSA prière de fournir les renseignements demandés ci-dessous et de formuler les propositions sur une feuille volante. Il est recommandé d'inclure

- le numéro de la norme/publication
- le numéro de l'article, du tableau ou de la figure visé
- la formulation proposée
- la raison de cette modification.

Proposal for change

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