1.5 Geometry

1.5.1 Planning
The much greater cost and increased difficulty of widening bridges as compared to roadways shall be considered in determining whether the bridge cross-section should suit that of the proposed roadway or that planned for the roadway at some time in the future.

Site conditions and requirements shall be considered in developing bridge alignment and plan geometry to accommodate existing and future traffic.

The design average annual daily traffic, design hourly volume, and directional split shall be those forecast for a period starting not less than ten years after the design life start. For structures that are not easily widened, a longer period shall be determined from an economic assessment.

Preference shall be given to straight horizontal alignments for bridges. The bridge deck longitudinal profile shall be continuous with the approach road profile.

1.5.2 Structure geometry

1.5.2.1 General
Roadway and sidewalk widths, curb widths and heights, and all other geometrical requirements not specified in this Code shall comply with the standards of the Regulatory Authority or, in their absence, with the Transportation Association of Canada’s Geometric Design Guide for Canadian Roads.

Sidewalks and bicycle paths shall be separated from traffic lanes by a barrier or guiderail, or by a curb with a face height of at least 150 mm and a face slope not flatter than 1 horizontal to 3 vertical. Sidewalks and bicycle paths not so separated shall be designed as part of the roadway.

1.5.2.2 Clearances

1.5.2.2.1 Roadways and sidewalks
Roadway and sidewalk clearances for structure openings and on structures shall comply with the standards of the Regulatory Authority or, in their absence, with the Transportation Association of Canada’s Geometric Design Guide for Canadian Roads.

1.5.2.2.2 Railways
Clearances for railways shall comply with the regulations of Transport Canada.

1.5.2.2.3 Waterways
Clearances for navigable waterways shall comply with Clause 1.9.7.1.

1.5.2.2.4 Construction
Clearances during Construction shall comply with the requirements of the agency with jurisdiction over the roadway, railway, or waterway passing through the opening.

1.6 Barriers

1.6.1 Superstructure barriers
The design of permanent barriers on structures shall comply with Section 12.

1.6.2 Roadside substructure barriers
Vehicular barriers or guiderails shall be provided in compliance with applicable roadside design requirements when the horizontal clearance from the roadway edge to a structure component is less than
the clear recovery zone width determined in accordance with the requirements of the Regulatory
Authority or, in their absence, with the Transportation Association of Canada’s Geometric Design Guide for
Canadian Roads.

Where a barrier is installed to protect a structural component, a minimum clearance of 125 mm shall be
provided between the barrier and the component. For flexible guiderails, the clearance shall be sufficient
to allow for their deflection under impact.

For barriers that are designed as an integral part of the structural component, no clearance shall be
required.

1.6.3 Structure protection in waterways
If the waterway allows passage of vessels large enough to cause damage to the structure, independent,
self-supporting fendering shall be provided to protect the structure and minimize damage to vessels. The
fendering or the structure shall be designed to resist the vessel collision load in compliance with Section 3.

1.6.4 Structure protection at railways
Protection shall be provided when specified by the railway authority.

1.7 Auxiliary components

1.7.1 Expansion joints and bearings
Expansion joints and bearings shall comply with Section 11.

1.7.2 Approach slabs
Unless otherwise Approved, bridges on paved roadways shall be provided with 6.0 m long reinforced
concrete approach slabs anchored to the abutments. The approach slabs shall extend transversely to the
limits of the roadway. The joints around the approach slabs shall be sealed.

1.7.3 Utilities on bridges

1.7.3.1 General
Utilities on bridges shall be corrosion resistant and designed not to cause corrosion or staining of the
structure.

1.7.3.2 Location and attachment
Unless otherwise Approved, utilities on bridges shall be located in or under the side and median areas. In
voided decks and box girders, utilities shall not pass through voids unless the voids are accessible for
inspection and maintenance.

Utilities shall be located and attached in such a manner that all primary components remain accessible.
Utilities and fittings shall not be attached to primary components in such a manner as to adversely affect
them structurally or reduce durability. Utility attachments to the flanges of steel girders shall not be
permitted.

All utilities on a bridge shall be designed for relocation to allow for future bridge maintenance.
Conduits embedded in concrete shall follow the deck alignment where possible. Drains shall be
provided at all low points.

At transition joints, e.g., expansion joints, couplings shall be provided that will allow all possible
movements without damage. At transition joints over or near structural bearings, the differential vertical
movement allowance provided for jacking shall not be less than 15 mm.
1.7.3.3 Highway utilities
The original bridge design shall provide for a highway illumination power supply, lighting standards, remote sensing cabling, and any other utilities likely to be necessary.

1.7.3.4 Public utilities
Provision shall be made for incorporating conduit in the superstructure for existing and planned utility cables that need to be carried across the structure.

1.7.3.5 Fluid-carrying utilities
Utilities that carry fluids, including gas and oil lines, sewers, and water pipes, shall not be allowed in or under the superstructure or on the bridge unless Approved.

1.8 Durability and maintenance

1.8.1 Durability and protection
Requirements for the durability of structures and for protective measures for structures (to ensure that the required design life is achieved) are specified in Section 2.

1.8.2 Bridge deck drainage

1.8.2.1 General
Bridge deck drainage shall be designed to remove water from the deck as completely and quickly as possible and to discharge the runoff harmlessly.

1.8.2.2 Deck surface

1.8.2.2.1 Crossfall and grades
Bridge deck drainage of the roadway shall be achieved by providing a minimum 2% transverse crossfall and by providing a minimum longitudinal grade of 0.5%, except where, for limited lengths, vertical curves or superelevation transitions preclude this.

   Except where unavoidable, bridges shall be located away from the low point of a sag curve in the vertical alignment of the road profile.

1.8.2.2.2 Deck finish
Deck finishing methods and acceptance criteria shall be specified on the Plans, preclude the occurrence of local depressions in the surface of the concrete, and ensure a surface acceptable either for the application of a waterproofing membrane or as a wearing surface with sufficient roughness for skid resistance.

1.8.2.3 Drainage systems

1.8.2.3.1 General
The spacing and capacity of bridge deck drains established by hydraulic design and testing shall be sufficient to ensure that for a ten-year design storm the runoff flowing in the swale or gutter will not encroach more than 1.50 m onto the traffic lane. Bridge deck drain inlets shall be provided only where this requirement would otherwise not be met.

   Where flat grades or sag curves are unavoidable, additional drainage shall be considered as a means of reducing local ponding.
1.8.2.3.2 Deck drain inlets
Drain inlets shall have grates with a clear spacing between bars of 40 to 75 mm. For highways from which cyclists are not excluded, grate bars shall be at an angle to the roadway centreline of between 45 and 90°.

The top surface of a drain inlet grate shall be a minimum of 15 mm and a maximum of 25 mm below the plane of the wearing surface. The wearing surface around the drain inlet shall be sloped toward it from the general plane of the wearing surface at a slope of approximately 1 in 20.

1.8.2.3.3 Downspouts and downpipes
The surface runoff collected at deck drain inlets shall be directed through the superstructure by individual vertical deck drain pipes with a minimum nominal inside diameter or width of 200 mm.

Downpipes shall be rigid and made of corrosion-resistant material. Where it is necessary to direct water laterally, this shall be accomplished by running pipes as nearly vertical as possible. Changes in direction shall be not greater than 45°.

Cleanouts shall be provided near bends or at intervals to permit access to all parts of a downpipe system.

The location and length of downspouts shall be such that drainage will not be discharged or blown against any structural component. For design purposes, water shall be assumed to spread from the outlet at an angle of 45° from the vertical. Downspouts shall project a minimum of 150 mm below any adjacent component, except where prohibited by minimum vertical clearance requirements.

Discharge from downpipes and downspouts shall be restricted to locations protected from erosion. Water shall not be discharged onto traffic or facilities such as roadways, pedestrian paths, and navigation channels. The probable performance of drainage systems at low temperatures shall be considered in the design, based on the behaviour of similar systems subject to local conditions, exposure, and maintenance standards, and shall take into account the consequences of freezing (including pipe bursting, deck flooding, and falling ice).

The involvement of open deck joints in deck drainage shall be considered. Open troughs provided to collect runoff passing through the joint shall be large enough to contain the discharge, shall slope at an angle of not less than 30° to the horizontal, and shall in other respects meet the requirements for downspouts.

1.8.2.4 Subdrainage of wearing surface
Provision shall be made for the drainage of water or for the release of pressure between waterproofing membranes and asphaltic concrete wearing surfaces. Drain holes with a minimum diameter of 15 mm shall be provided at this level in all deck drain inlets. At expansion joint dams or in other locations where a drainage pocket is formed, corrosion-resistant drainage tubes shall be installed to drain the trapped water and shall lead from the low point of the pocket to a location where the water can be discharged harmlessly.

1.8.2.5 Runoff and discharge from deck
Runoff from the bridge deck at the abutments shall be intercepted immediately beyond the end of the bridge approach slab or before the end of the curb or barrier by catch basins or other suitable means. The intercepted water shall be directed away from the embankment slopes and abutments to prevent embankment erosion.

Where approaches slope down toward a bridge, the runoff shall be intercepted by catch basins or other devices located on the approaches so as to minimize flow across the expansion joints and onto the bridge deck.
1.8.3 Maintenance

1.8.3.1 Inspection and maintenance access

1.8.3.1.1 General
The type of access needed for inspection and maintenance shall be considered in the design of all structures.

Types of structures that have inaccessible areas where undetected dangerous deterioration can occur shall be avoided.

Access to primary components requiring periodic inspection or maintenance shall be unhindered and not require equipment unlikely to be available.

1.8.3.1.2 Removal of formwork
Unless otherwise Approved, all formwork shall be removed from the underside of concrete deck slabs and from the inside of steel and concrete box sections that have an inside vertical dimension of 1.20 m or more.

1.8.3.1.3 Superstructure accessibility
The final structure site terrain and the need for access from below shall be reviewed during design to ensure ease of inspection of the primary components of superstructures.

High bridges and bridges over deep water with individual spans longer than 30 m and not more than 75 m long shall be designed to permit inspection by mobile inspection equipment. For structures with spans longer than 75 m, consideration shall be given to providing catwalks or travelling-scaffold equipment.

1.8.3.1.4 Access to expansion joints
A clear space of at least 200 mm shall be provided between the ballast walls and the superstructure end diaphragms and girders.

1.8.3.1.5 Access to primary component voids
For box or cellular girders that have voids with an inside vertical dimension of 1.20 m or more, access hatches shall have a minimum clear opening size of 600 × 800 mm if rectangular or 800 mm in diameter if round. All voids or cells shall have an access hatch or an interior connection to a void that does. The interior connection openings shall not be smaller than the hatch openings. All hatches shall have close-fitting lockable covers.

Where a void has access openings on the surface of structural components, the openings shall be fitted with cover plates of vandal-resistant design. All cover plates on manholes, hand holes, or other openings on the top surfaces of bridge components shall have weathertight seals. Drains shall be provided at all low points of voids and shall direct water to an area remote from structural components.

1.8.3.2 Maintainability
Where it can reasonably be expected that components will have to be replaced or modified during the design life of the structure, methods of replacement shall be investigated to ensure the feasibility, acceptable cost, and duration of the work and, where appropriate, the availability of alternative routes or detours for traffic. This investigation shall ensure the availability of access and the integrity of the structure during the work.

1.8.3.3 Bearing maintenance and jacking
Bridges with superstructures supported on bearings shall be designed to permit the jacking of the superstructure. Jack and shimming locations shall be shown on the drawings.

The design shall allow for movement at the permanent bearing locations sufficient to permit bearing replacement.
In the design of jack-bearing locations, the assumed factored jacking force shall not be less than twice the unfactored dead load.

When closure of the structure to traffic is not practicable, a backup system of shim supports independent of the jacking equipment shall be provided and all loads shall be considered at the ultimate limit states for the jacking and shimming locations.

1.9 Hydraulic design

1.9.1 Design criteria

1.9.1.1 General
The hydraulic design of bridges, culverts, and associated works shall comply with the requirements of the Regulatory Authority or, in their absence, with the Transportation Association of Canada’s Guide to Bridge Hydraulics.

Acceptability criteria for the performance of structures in withstanding a design flood shall be in accordance with the intended level of service and normal economic constraints.

Risks that are to be accepted shall be recorded as part of the design criteria.

1.9.1.2 Normal design flood
The normal design flood shall have a return period of 50 years unless otherwise specified by the Regulatory Authority.

Bridges and culverts shall be designed to accommodate the normal design flood without damage to the structure or the approaches. Relief flow over the roadway shall be allowed only with site-specific Approval and where geometric and level-of-service criteria permit. The site of the assumed relief flow shall be investigated to ensure that no significant damage or concealed hazards can arise.

1.9.1.3 Check flood
The check flood shall have a return period of at least twice the normal design flood unless otherwise specified by the Regulatory Authority.

Bridges and culverts shall be designed to withstand a check flood without endangering the integrity of the structure and without approach embankment failure.

1.9.1.4 Regulatory floods and relief flow
When it is required that a structure be designed for the regulatory flood, relief flow shall be considered to the extent permitted by geometric, level-of-service, and other site criteria.

The structure shall be designed for that part of the regulatory flood not accommodated by relief flow, without causing flooding of upstream property outside the established flood plain.

1.9.1.5 Design flood discharge
The design flood discharge shall be estimated by Approved methods.

1.9.1.6 High-water levels
Unless otherwise specified, the high-water levels used for design purposes shall be the water levels corresponding to the design flood discharge without ice jams. If the crossing is subject to abnormal flood conditions, the worst conditions for the particular design purpose likely to occur together with the design flood shall be assumed.
1.9.2 Investigations
For all water crossings, office studies and field surveys shall be carried out as part of the investigation of the proposed crossing site to determine
(a) hydraulic characteristics;
(b) geotechnical characteristics;
(c) the location of all utilities in the area;
(d) ownership of property that might be affected;
(e) past and potential problems in the vicinity;
(f) the hydraulic performance of existing structures near the site;
(g) the flood history of the site;
(h) the ice and debris history of the site;
(i) other features pertinent to the hydraulic design; and
(j) established and possible future land use trends and their effect on the waterway, flood plain, and watershed areas.
When appropriate, other investigations, e.g., archaeological surveys or environmental studies, shall be conducted.

1.9.3 Location and alignment
The selection of the location and alignment of a water crossing shall take into account the following factors:
(a) the stability of the channel;
(b) road geometrics and road user safety;
(c) geotechnical conditions;
(d) the effect of the crossing on adjacent structures and property;
(e) the effects of adjacent dams, bridges, and other structures;
(f) the interests of waterway users; and
(g) any known future developments.

1.9.4 Estimation of scour

1.9.4.1 Scour calculations
Depths of general scour, local scour, degradation, and artificial deepening of the channel shall be estimated at all structure sites.
Scour calculations shall be prepared for all potentially critical conditions, including maximum depth of flow, maximum velocity, and extreme ice conditions.
If abnormal flood conditions can occur at the site, a design flood discharge based on the lowest downstream water level likely to coincide with the design flood shall be considered.

1.9.4.2 Soils data
The properties of the material below the estimated depth of maximum scour shall be examined to reduce the possibility of large errors in scour predictions.

1.9.4.3 General scour

1.9.4.3.1 Average depth
The average depth of general scour shall be calculated using the competent velocity method or another Approved method and shall be referenced to the original stream bed.

1.9.4.3.2 Maximum depth
The maximum depth of general scour at any point in a structure opening shall be determined by redistributing the average depth of general scour as described in the Transportation Association of Canada’s Guide to Bridge Hydraulics or using other Approved methods. The maximum depth of general...
scour shall be assumed to occur at any point across the structure opening except where protective features would prevent it.

1.9.4.4 Local scour
The depth of local scour at a pier, abutment, or other obstruction shall be measured below the anticipated depth of general scour and shall be calculated using Approved methods. The possibility of local scour being caused or aggravated by ice jams or trapped debris shall be considered.

1.9.4.5 Total scour
The total depth of scour at a point across a structure opening shall be taken as the sum of the maximum depth of general scour and the depth of local scour.

1.9.4.6 Degradation
The probable depth of stream bed degradation during the design life of a structure shall be estimated by investigating the site and the characteristics and history of the channel.

1.9.4.7 Artificial deepening
The amount of artificial deepening of a channel anticipated during the design life of a structure shall be obtained from drainage plans, if available. Otherwise, it shall be estimated from an investigation of existing structures on the same channel.

1.9.4.8 Allowance for degradation or artificial deepening
The ultimate bed elevation at structures on channels subject to degradation or artificial deepening shall be taken as follows:
(a) for degrading channels not having a concrete or steel invert that complies with Clause 1.9.5.7, the expected amount of degradation plus one-half the total scour; and
(b) for channels likely to be artificially deepened, the expected amount of deepening plus the total scour.

1.9.5 Protection against scour

1.9.5.1 General
Scour protection requirements for structure foundations shall be determined on the basis of the normal design flood and shall be modified if necessary to ensure that structural failure will not occur as a result of the check flood.

1.9.5.2 Spread footings
The following shall apply to depth of footings:
(a) Minimum depth: except as specified in Items (b) to (e), the bottom surfaces of spread footings that could be exposed to stream flow shall be placed at the lowest of the following elevations:
   (i) a depth below the original bed not less than the following:
      (1) abutments other than arches: 1.50 m; and
      (2) piers and arch abutments: 2.00 m;
   (ii) a depth below the original bed not less than 1.7 times the estimated total depth of scour; and
   (iii) a depth not less than 0.50 m below the lowest level of existing or past scour.
(b) Bedrock: spread footings may be founded on scour-resistant durable bedrock at a higher elevation than that specified by Item (a) if the depth is sufficient to ensure that they remain unaffected by scour, freezing, weathering, degradation, or artificial deepening during the design life of the structure.
(c) Temporary structures: for temporary piers and abutments constructed of gabions or timber cribs, the depths specified in Items (a)(i) and (a)(iii) shall be reduced by half, and the factor specified in Item (a)(ii) shall be reduced from 1.7 to 1.3.

(d) Degrading channels: on degrading channels that are not stabilized with a paved invert or revetment complying with Clause 1.9.5.7, the footing depth specified in Item (a)(ii) shall not be taken as less than the expected amount of degradation plus the estimated total depth of scour.

(e) Artificial deepening: on channels likely to be artificially deepened, the footing depths specified in Items (a)(i) and (a)(ii) shall be measured below the expected stream bed elevation after deepening.

1.9.5.2.2 Protection of spread footings
Spread footings shall not be founded at a depth less than that specified in Clause 1.9.5.2.1 unless
(a) the structure opening has a concrete or steel invert complying with Clause 1.9.5.7; or
(b) the footings are protected against undermining by sheet piling along the inside face and ends of the footings or by other Approved means.

Spread footings adjacent to the stream channel shall not be founded at an elevation higher than the stream bed on material other than bedrock or rock fill unless protected by a concrete revetment complying with Clauses 1.9.5.7 and 1.9.9.3.

1.9.5.3 Piles

1.9.5.3.1 General
In sands and other highly erodible soils, piles shall be used in preference to spread footings to provide better protection against the effects of scour.

1.9.5.3.2 Penetration and strength
The penetration and structural strength of piles shall be sufficient to ensure their stability with the stream bed at its ultimate bed elevation.

1.9.5.3.3 Abutments supported on piles
The bottom of footing elevation of abutments exposed to flowing water shall be set at least 1.0 m below the ultimate stream bed elevation. This requirement may be waived if footing protection meeting the requirements of Clause 1.9.5.2.2 is provided.

1.9.5.4 Sheet piling
When required for scour protection, sheet piling shall comply with the following requirements:
(a) the piling shall be securely attached to the footing or otherwise anchored to prevent movement; and
(b) the penetration and structural strength of the piling shall be sufficient to ensure stability of the structure and piling with the stream bed at its ultimate bed elevation.

1.9.5.5 Protective aprons
Flexible aprons used for protecting piers and abutments against local scour shall comply with the recommendations of the Transportation Association of Canada’s Guide to Bridge Hydraulics or those of the Regulatory Authority.

Rip-rap stone sizes for aprons shall be determined by designing for a velocity 1.5 times the average velocity of the normal design flood discharge through the structure opening. The thickness of rip-rap aprons shall be not less than 1.5 times the median size of the stone.

1.9.5.6 Paved inverts and revetments
Concrete and steel inverts and revetments that are needed to stabilize a channel at a structure shall have cut-off walls of sufficient depth and strength to prevent undermining. The cut-off walls shall be integral with or securely attached to the invert or revetment.
1.9.5.7 Special protection against degradation
Where a paved invert or revetment is required to stabilize a degrading stream bed, the following shall be provided:
(a) Cut-off walls as specified in Clause 1.9.5.6. The downstream cut-off wall shall be designed to resist the maximum depth of degradation likely to occur during the design life of the structure or up to the time of scheduled stream bed maintenance.
(b) An apron, energy dissipator, or other device to control erosion caused by the discharge from the structure.
(c) If the invert is lower than the adjacent upstream bed, a sill, weir, or other effective control at the inlet.

1.9.6 Backwater

1.9.6.1 General
Backwater shall be calculated using Approved methods. Backwater shall be limited so as to preclude damage to upstream property and buildings during the design flood. Unless otherwise Approved, the design flood shall be the normal design flood.

1.9.6.2 High-water level
The high-water level used in the backwater calculations shall be as specified in Clause 1.9.1.6. If the crossing is subject to abnormal flood conditions, the highest downstream water level likely to occur together with the design flood shall be assumed.

1.9.6.3 Assumed depth of scour
Unless otherwise Approved, the depth of scour assumed to have occurred in backwater calculations shall not be more than one-half the average depth of general scour calculated in accordance with Clause 1.9.4.3.

1.9.6.4 Waterway modification
The enlargement of the natural channel cross-section at a bridge to reduce backwater shall be of such a nature as to provide a stable channel and preclude progressive sedimentation and growth of undesirable vegetation.

1.9.6.5 Reduction of backwater by relief flow
Backwater calculations shall include relief flow over the roadway where allowed by Clause 1.9.1.2.

1.9.7 Soffit elevation

1.9.7.1 Clearance
The clearance between the soffit of the structure and the high water level determined in accordance with Clause 1.9.7.2 shall be sufficient to prevent damage to the structure by the action of flowing water, ice floes, or debris, and unless otherwise Approved shall not be less than 1.0 m for freeways, arterial roads, and collector roads and not less than 300 mm for other roads. The clearance for a structure with an arched soffit shall be based on site-specific considerations.

Vertical clearance for a structure on a navigable waterway shall be measured from the highest water level at which usual navigation is likely to occur. This level, together with the vertical and horizontal clearances, shall be determined in accordance with the Government of Canada’s *Navigable Waters Protection Act*.

Unless otherwise Approved, the clearance between the lowest point of the soffit and normal water level shall be at least 1.0 m.
1.9.7.2 High-water level for establishing soffit elevation

1.9.7.2.1 General
Subject to Clause 1.9.7.2.2, and unless otherwise Approved for the site, the high-water level for establishing the minimum soffit elevation shall be the higher of:
(a) the high-water level determined in accordance with Clause 1.9.6.2; and
(b) the high-water level caused by ice jams and having a return period comparable to that of the design flood.

1.9.7.2.2 Flooding and relief flow
Use of the regulatory flood as the design flood to ensure that flooding of upstream property will not occur shall not require the soffit elevation to be higher than that required for the normal design flood, provided that adequate relief flow, as permitted by Clause 1.9.1.2, can occur.

1.9.8 Approach grade elevation

1.9.8.1 General
Where geometric and other considerations permit, the approach grade shall be set so that the requirements of Clause 1.9.8.2 are met. If consideration of a regulatory flood is required and relief flow is permitted, the grade shall be set to optimize relief flow.

1.9.8.2 Freeboard
Except when otherwise Approved, freeboard from the edge of through-traffic lanes to the high-water level determined in accordance with Clause 1.9.8.3 shall be 1.0 m for freeways, arterial roads, and collector roads and 300 mm for other roads.

1.9.8.3 High-water level for establishing approach grade
The high-water level for establishing the approach grade elevation shall be as specified for establishing the minimum soffit elevation in Clause 1.9.7.2. Unless otherwise Approved, the high-water level upstream of the opening shall be increased by the estimated backwater.

1.9.8.4 Freeboard for routes under structures crossing water
Freeboard for highways under bridges that cross water shall be in accordance with Clause 1.9.8.2. Freeboard for walkways, bicycle paths, and maintenance access roads under structures crossing water shall be at least 1.0 m above normal water level. This minimum value shall be increased when high maintenance costs are likely to result from its use.

1.9.9 Channel erosion control

1.9.9.1 Slope protection
Where necessary, embankments shall be protected against erosion to prevent damage to the structure, roadway, or property affected by the crossing. The design of protection works shall be in accordance with the Transportation Association of Canada’s Guide to Bridge Hydraulics or as otherwise Approved.

1.9.9.2 Stream banks
Stream banks shall be protected against erosion to the extent necessary to prevent damage to the highway or property affected by the crossing.
1.9.9.3 Slope revetments
Toe protection shall be provided to prevent undermining of slope revetments. Geotextile fabric, a graded granular filter blanket, or other Approved material shall be provided where necessary to prevent loss of underlying material.

Unless otherwise Approved, a concrete revetment for protecting structure footings shall comprise concrete paving reinforced, tied, or interconnected in a way that ensures that the underlying material remains protected for the design life of the structure.

1.9.9.4 Storm sewer and channel outlets
Outlet of storm sewers and channels discharging into or adjacent to a bridge or culvert opening shall have aprons, energy dissipators, drop structures, or other devices to prevent erosion that might endanger the bridge or culvert.

1.9.10 Stream stabilization works and realignment

1.9.10.1 Stream stabilization works
Stabilization works shall be considered if one or more of the following is necessary:
(a) stabilizing the channel location in the vicinity of the crossing;
(b) reducing the cost of the crossing;
(c) directing flow parallel to the piers and minimizing local scour;
(d) improving the hydraulics of the waterway or reducing erosion;
(e) protecting the roadway approaches from stream attack;
(f) permitting the construction of a square crossing by diverting the channel from a skewed alignment; or
(g) improving the location and geometry of the crossing.

Stabilization works shall be designed to suit the requirements of the site and comply with the Transportation Association of Canada’s *Guide to Bridge Hydraulics*.

1.9.10.2 Stream realignment
Stream realignment shall be considered only when no cost-effective alternative is possible. The design of stream realignments shall include an evaluation of environmental and hydraulic regime effects.

1.9.11 Culverts

1.9.11.1 General
Closed-invert culverts shall be used in preference to open-footing culverts except where site conditions dictate the use of open-footing culverts.

1.9.11.2 Culvert end treatment
End treatment shall be provided where there would otherwise be a possibility of uplift, piping, undermining, or damage due to ice or debris. End treatment in the form of an improved inlet shall be provided where there is a net benefit due to improved hydraulic efficiency. End treatments shall be of tested or established types.

1.9.11.3 Culvert extensions
Extensions to existing culverts shall be designed to prevent internal blockages caused by changes of direction, changes in the shape of the cross-section, or changes in the number of openings or cells.

1.9.11.4 Alignment of non-linear culverts
Unless otherwise Approved, changes of horizontal alignment shall be accomplished by gradual curves or by angular changes of direction not exceeding 15° at intervals of not less than 15 m.
1.9.11.5 Open-footing culverts

1.9.11.5.1 Inerodible inverts
An open-footing culvert with an inerodible invert shall be considered a closed-invert culvert.

1.9.11.5.2 Vertical clearance
Soffit elevations for open-footing culverts shall comply with Clause 1.9.7, except that the minimum clearance shall be 300 mm.

1.9.11.6 Closed-invert culverts

1.9.11.6.1 Invert elevation
The inverts of closed-invert culverts shall be located below the adjacent channel bed at an appropriate depth in order to
(a) reduce the likelihood of hanging outlets and the undermining of culvert ends;
(b) improve hydraulic efficiency;
(c) enhance fish passage;
(d) reduce the water velocity at the outlet; and
(e) encourage natural sedimentation of the culvert floor and replication of natural habitat.

1.9.11.6.2 Artificial deepening
If the channel is likely to be artificially deepened, the culvert invert elevation shall be based on the future channel elevation estimated in accordance with Clause 1.9.4.7.

1.9.11.6.3 Degrading channel
In a degrading channel, a sill or weir shall be provided to maintain any difference in elevation between the culvert invert and the upstream bed.

1.9.11.6.4 Piping
When soil properties and hydraulic conditions indicate that piping can occur along the barrel of a culvert, appropriate preventive measures, e.g., the use of clay seals, cut-off walls, or impermeable barriers, shall be taken.

1.9.11.6.5 Concrete box structures
A cut-off wall shall be provided at each end of a concrete box culvert in accordance with Clause 1.9.5.6.

1.9.11.6.6 Soil-steel structures
The following requirements shall apply to soil-steel structures:
(a) End treatment: a cut-off wall, headwall, collar, or other Approved device shall be provided at the ends of soil-steel structures where it is necessary to protect the culvert against uplift, piping, or undermining. Connections to the culvert shall be designed to resist all possible uplift and earth pressure forces. Embankment slopes shall be modified where necessary to provide sufficient weight of fill to prevent hydraulic uplift of the inlet end. If a weir is provided at the inlet end, piping shall be prevented as required by Clause 1.9.11.6.4 and the possibility of uplift due to buoyancy shall be considered.
(b) Camber: the camber requirements for all metal pipe culverts shall be calculated to accommodate longitudinal settlement and to prevent ponding within the culvert.
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Section 2
Durability

2.1 Scope
This Section specifies requirements for durability that need to be considered during the design process in addition to this Code’s requirements for strength and serviceability. The requirements of this Section apply to the design of new bridges as well as to rehabilitation and replacement work.

2.2 Definitions
The following definitions apply in this Section:

Design life — a period of time specified by the Owner during which a structure is intended to remain in service.

Durability — the capability of a component, product, or structure to maintain its function throughout a period of time with appropriate maintenance.

Predicted service life — an estimated period of time for the service life based on actual construction data, condition surveys, environmental characterization, or experience.

Service life — the actual period of time during which a structure performs its design function without unforeseen costs for maintenance and repair.

2.3 Design for durability

2.3.1 Design concept
The design shall ensure that the structure will be able to maintain its level of serviceability during its design life. The designer shall consider the environmental conditions that exist at the site or are likely to exist during the design life of the structure and shall assess their significance in relation to the possible mechanisms of deterioration in the structure. Structural site investigation shall include testing of soils, groundwater, local runoff water, atmospheric pollution levels, and, when applicable, drainage system discharge (to detect corrosive substances). When a structure is being designed for a new facility, environmental conditions shall be predicted from comparable existing facilities.

2.3.2 Durability requirements

2.3.2.1 General
The structural form, materials, and details shall be suitable for the design loads and environmental conditions that will be experienced during the design life of the structure.
2.3.2.2 Materials
The composition, properties, and performance of the materials selected for the structure shall be specified by taking into account the design loads and the expected environmental degradation during the design life of the structure.

Note: See also Clause 2.3.3.

2.3.2.3 Structural details
Members shall be designed to reduce the impact of environmental factors. Preference shall be given to structural details that provide free air circulation for all above-ground components.

Members shall be detailed to minimize exposed surface area and avoid pockets, crevices, recesses, re-entrant corners, and locations that collect and retain water, debris, and moisture.

2.3.2.4 Bearing seats
Bearing seats shall be designed so that contact with de-icing salts, salt-laden water runoff, leakage, and debris is prevented. The surfaces around and between bearing seats shall be sloped so that they are self-draining away from the bearings. Level areas for jacking of the superstructure for bearing replacement shall be provided.

2.3.2.5 Bridge joints

2.3.2.5.1 Expansion and/or fixed joints in decks
Wherever practical, expansion and/or fixed joints in decks shall be avoided or placed in the approach pavements. Where expansion joints cannot be avoided, they shall be detailed to prevent damage to components of the structure from water, de-icing salts, chemicals, and roadway debris.

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.

The end diaphragms of slab-on-girder bridges shall be detailed to prevent water from expansion joints travelling along girders.

2.3.2.5.2 Joints in abutments, retaining walls, and buried structures
Expansion and construction joints in abutments, retaining walls, and buried structures shall be sealed at the surface that is in contact with the backfill to prevent damage to components of the structure from water, de-icing salts, and chemicals.

2.3.2.6 Drainage
The longitudinal and transverse slopes on bridge decks and the number and location of deck drains shall be in accordance with Section 1.

Downspouts for deck drains shall be located in such a way that runoff water is discharged away from any part of the bridge. Downspouts shall extend at least 150 mm below adjacent members. Wherever practical, deck drains shall not pass through the box girders.

Box girders shall be made watertight at their ends and adequately drained so as to reduce the potential for moisture entrapment and accelerated corrosion.

Pockets and depressions that could retain water shall have effective drain holes or an alternative means of drainage.

Measures shall be taken to prevent erosion from the discharge of drainage water.

2.3.2.7 Utilities
All permanent iron and steel utility supports, fittings, and accessories shall be coated or galvanized. Utility supports shall be designed to prevent stray electrical currents between the structure and the utility supports.
2.3.2.8 Birds and other animals
In areas with large roosting bird populations, components shall be located and proportioned in a way that prevents the entry or roosting of birds in drain holes, expansion joints, and bearing cavities. In the absence of such measures, screening or other Approved methods shall be used to inhibit bird roosting. Voids shall be designed to prevent the entry of birds and other animals.

2.3.2.9 Access
Access for maintenance and inspection shall be provided for all components of the structure.

2.3.2.10 Construction
The quality of the materials, placement procedures, and construction details shall be specified on the Plans.

The testing and acceptance methods required at the site for quality assurance of materials and construction shall be specified on the Plans.

2.3.2.11 Inspection and maintenance
The design of bridges and other structural components shall be predicated on routine inspection and maintenance procedures being instituted.

2.3.3 Structural materials
The designer shall review the environmental conditions and deterioration mechanisms for the material used and shall apply the following durability requirements to achieve the design life of the structure:

(a) for concrete: Clause 8.11;
(b) for wood: Clause 9.17;
(c) for steel, including steel components of bearings, expansion joints, light poles, overhead sign supports, soil-steel structures, deck drains, and railings: Clause 10.6;
(d) for fibre-reinforced structures: Clause 16.4 and Annexes A16.1 and A16.2;
(e) for aluminum: Clause 2.4; and
(f) for other materials: Clauses 2.5 to 2.10.

2.4 Aluminum

2.4.1 Deterioration mechanisms
The deterioration mechanisms to be considered for aluminum components shall include, but not be limited to, corrosion.

2.4.2 Detailing for durability

2.4.2.1 Connections
Aluminum components shall be connected by welding, by stainless steel bolts, or by high-strength steel bolts galvanized in accordance with CAN/CSA-G164.

In hot and very humid conditions, the surface between the galvanized bolt and the aluminum shall be coated with paint or bitumastic materials.

2.4.2.2 Inert separators
Inert separators shall be provided where aluminum components are in contact with other metals (except stainless steel) or concrete.
2.5 Polychloroprene and polyisoprene
The properties specified for polychloroprene and polyisoprene shall ensure that the materials will not harden or crack in the environment in which they are used.

2.6 Polytetrafluoroethylene (PTFE)
PTFE surfaces in contact with stainless steel shall be free of dirt to prevent excessive friction.

2.7 Waterproofing membranes
Waterproofing membranes shall prevent the ingress of water and shall not crack during their service life. Only Approved waterproofing membranes shall be specified.
Where a hot applied rubberized asphalt waterproofing membrane is used, it shall be protected with an asphalt-impregnated protection board to prevent it from being punctured. The membrane shall terminate in a chase in the curb or barrier wall.
The top surfaces of a waterproofing membrane shall be drained to prevent ponding of water on the membrane.

2.8 Backfill material
Backfill material shall be free draining and shall not contain corrosive chemicals that could have a detrimental effect on structural components in contact with the backfill material.

2.9 Soil and rock anchors
Soil and rock anchors shall be protected from the detrimental effects of chemicals in the soil and rock or shall be made from inert materials.

2.10 Other materials
The composition, properties, and performance of materials not covered in this Section shall be specified by taking into account the design loads and expected environmental degradation during the design life of the structure.
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Section 3
Loads

3.1 Scope
This Section specifies loads, load factors, and load combinations to be used in calculating load effects for design. Resistance factors required to check ultimate limit states criteria in accordance with Clause 3.4.2 are specified elsewhere in this Code. Loadings provisions for evaluation of existing structures are covered in Section 14 and for rehabilitation in Section 15.

This Section includes requirements related to the vibration of highway and pedestrian bridges. It also includes requirements related to construction loads and temporary structures; these apply to partially completed structures and structures necessary for construction purposes. Snow loads are not specified because in normal circumstances the occurrence of a considerable snow load will cause a compensating reduction in traffic load.

3.2 Definitions
The following definitions apply in this Section:

Acceptance criterion — the acceptable frequency of collapse due to the design vessel collision.

Buffeting — the loads induced in a structure by the turbulence in the natural wind.

Damping — the dissipation of energy in a structure oscillating in one of its natural modes of vibration. It is normally expressed as a ratio of the actual value of damping to the critical value of damping. The critical value of damping is the lowest value at which an initial motion decays without oscillation.

Dead load — the load from material that is supported by the structure and is not subject to movement.

Debris torrent — a mass movement that involves water-charged inorganic and organic material flowing rapidly down a steep confined channel.

Design lane — a longitudinal strip that is a fraction of the deck width and within which a Truck or Lane Load is placed for the purpose of design or evaluation.

Divergence — an aerodynamic instability in torsion that usually occurs at wind speeds higher than those normally considered in design.

Drag — the load in the direction of the wind, induced by an airstream acting on a body.

Dynamic load allowance — an equivalent static load that is expressed as a fraction of the traffic load and is considered to be equivalent to the dynamic and vibratory effects of the interaction of the moving vehicle and the bridge, including the vehicle response to irregularity in the riding surface.

Effective temperature — the temperature that governs the thermally induced expansion and contraction of a superstructure.

Exceptional loads — the loads due to forces of nature or accident that would not be expected to occur more than once in the life of a bridge.
Exposed frontal area — the net area of a body, member, or combination of members as seen in elevation. For a superstructure, the sum of the areas of all members, including railings and the deck system, as seen in elevation at 90° to the longitudinal axis in the case of a straight structure or to an axis chosen to maximize wind effects in the case of a structure that is curved in plan.

Exposed plan area — the net area of an object as seen in plan from above. For a superstructure, the plan area of the deck and of any laterally protruding railings, members, or attachments.

Factored load — the product of a load specified in this Code and the corresponding load factor.

Factored load effect — the load effect caused by a factored load.

Falsework — a temporary structure used to support another structure or a part of the other structure.

Flexural frequency — a natural frequency of vibration of an unloaded bridge based on the longitudinal flexural stiffness and mass distribution of the superstructure.

Flutter — an instability caused by the interaction of the wind and the bridge structure involving either pure torsional motion or coupled vertical and torsional motion of a bridge deck.

Galloping — the cross-wind vibrations that arise from the aerodynamic instability of many slender structures.

Gust effect coefficient — the ratio of the peak wind-induced load on a structure or response of a structure, including both static and dynamic action, to the static wind-induced load or response. It is also referred to as the gust coefficient.

Ice accretion — the buildup of an ice layer on the exposed surfaces of a body due to freezing rain or in-cloud icing.

Live load — a load imposed by vehicles, pedestrians, equipment, or components that are subject to movement.

Load — a load, force, deformation, or volumetric change that is imposed externally on or internally within a structure.

Natural frequency — the frequency of vibration of one of the natural modes of a bridge, expressed in cycles per second, and being the inverse of the natural period.

Natural period — the duration of one complete cycle of free vibration of a normal mode of vibration of a structure.

Normal mode shape — the geometric configuration of a structure associated with vibration at one of its natural frequencies.

Pedestrian load — the load due to pedestrians on a bridge.

Permanent loads — the loads that do not vary unless physical changes are made to the bridge.

Reynolds number — the ratio of inertial forces to viscous forces of a fluid.

Service life — the number of years a structure is intended to be in use.

Slender structural element — a structural member with an aspect ratio of 20 or more.

Strouhal number — a non-dimensional parameter that characterizes the frequency of vortex shedding and represents the ratio of the width of a body placed in an airstream to the wavelength of vortices shed from the body.

Structural component — a component that influences the strength or stiffness of a structure.
Traffic barrier — a barrier that is intended to provide protection to vehicular traffic.

Traffic load — the load due to vehicles on a bridge.

Transitory loads — the loads due to traffic or equipment on a structure, or the seasonal effects of nature.

Travelled lane — a strip of roadway marked for use by a single line of vehicles.

Vessel collision — the impact of a ship with the substructure or superstructure of a bridge over a navigable waterway.

Vortex shedding — an instability of the wake behind a bluff (i.e., not streamlined) body in an airstream, comprising a more or less periodic shedding of vortices. The vortices are shed alternately from opposite sides of the body, producing an alternating lateral load normal to the wind direction.

Wake buffeting — the loads induced in a structure by the turbulence caused by the wake of an upwind structure.

Water loads — the loads from static or moving water, including pressure, buoyancy, waves, and debris torrents.

### 3.3 Abbreviations and symbols

#### 3.3.1 Abbreviations

The following abbreviations apply in this Section:

- **CL** — Canadian loading (see Clause 3.8.3)
- **FLS** — fatigue limit state
- **PL-1** — performance level 1 for traffic barriers
- **PL-2** — performance level 2 for traffic barriers
- **PL-3** — performance level 3 for traffic barriers
- **SLS** — serviceability limit state
- **ULS** — ultimate limit state

#### 3.3.2 Symbols

The following symbols apply in this Section:

- **$A$** = area of a pier or drift exposed to flowing water, projected parallel to the longitudinal axis of the pier onto a plane perpendicular to that axis, $m^2$ (see Clause 3.11.4.1); zonal acceleration ratio (dimensionless) (see Table A3.1.1); ice accretion load (see Tables 3.1 and A3.2.1)
- **$AF$** = annual frequency of collapse for a pier or span component susceptible to ship collision
- **$AF_{\text{max}}$** = maximum annual frequency of collapse for a whole bridge due to vessel collision
- **$a_i$** = modal coefficient of magnitude of the oscillatory displacement for the member mode of vibration, $i$, for a member with a constant diameter or frontal width, m
- **$a_i(x_1)$** = modal coefficient of magnitude of the oscillatory displacement due to vortex shedding excitation at location $x_1$ for the member mode of vibration, $i$, for a member with a tapered diameter or frontal width, m
- **$B$** = band width (a measure of the variability of the vortex shedding frequency) (see Table A3.2.4); width of ship, m (see Clause A3.3.3.3.5)
- **$BL$** = basic load (see Clause A3.2.3)
- **$BR$** = aberrancy base rate for vessels
- **$b$** = width of the traffic signal (see Figure A3.2.1); length of the member above or below location $x_1$ for which $D(x)$ is within a certain percentage of $D(x_1)$, m (see Clause A3.2.4.3.1(b))
\( C \) = a constant (see Clause A3.2.4.3.1)

\( C_D \) = horizontal wind drag coefficient of a cylindrical shape with a diameter of \( D \) (see Table A3.2.2); longitudinal drag coefficient for stream pressure (see Table 3.10)

\( C_H \) = hydrodynamic mass coefficient

\( C_L \) = lateral load coefficient for stream pressure (see Table 3.11)

\( C_L \) = root-mean-square (RMS) lift coefficient for the cross-sectional geometry (see Table A3.2.4)

\( C_a \) = coefficient allowing for the ratio of pier width to ice thickness when the ice fails by crushing

\( C_e \) = wind exposure coefficient

\( C_g \) = wind gust effect coefficient

\( C_h \) = horizontal wind drag coefficient

\( C_n \) = coefficient of pier nose inclination

\( C_v \) = vertical wind load coefficient

\( D \) = dead load (see Tables 3.1 and A3.2.1); width or diameter of member (see Table A3.2.2); constant diameter or frontal width of member (see Clauses A3.2.4.2 and A3.2.4.3.1(a))

\( D_B \) = total height of vessel, m

\( D_E \) = depth of earth cover between the riding surface and the highest point of a structure, m

\( D(x) \) = diameter or frontal width of a tapered member at location \( x \), m (see Clauses A3.2.4.2 and A3.2.4.3.1(b))

\( d \) = depth of superstructure, m

\( E \) = loads due to earth pressure and hydrostatic pressure, including surcharges but excluding dead load (see Tables 3.1 and A3.2.1)

\( E \) = earthquake load (see Tables 3.1 and A3.2.1)

\( e \) = eccentricity of wind load on a sign, luminaire, traffic signal support, m

\( F \) = loads due to stream pressure and ice forces or to debris torrents (see Table 3.1)

\( F_b \) = horizontal ice load caused when ice floes fail by flexure, kN; horizontal ice force caused by floes that fail to flex on impact and ride on the inclined pier nose, kN

\( F_c \) = horizontal ice load caused when ice floes fail by crushing, kN

\( F_h \) = horizontal wind load per unit exposed frontal area, Pa

\( F_i(x) \) = peak inertia load at location \( x \) for member mode of vibration, \( i \), N/m

\( F_t \) = transverse ice force, kN

\( F_v \) = vertical wind load per unit exposed plan area, Pa (see Clause 3.10.2.3); vertical force on a bridge pier due to ice adhesion, kN (see Clause 3.12.5)

\( F_w \) = force against a flat surface due to wave action, kN

\( G_{Mi} \) = generalized mass for mode of vibration, \( i \), kg

\( H \) = depth of flowing water at a pier, m (see Clause 3.11.4.2); collision load arising from highway vehicles or vessels (see Table 3.1); height above ground of the top of a superstructure, m (see Clause 3.10.1.4); length of member, m (see Clause A3.2.4.3.1); ultimate bridge element strength, MN (see Clause A3.3.3.3.6)

\( H_w \) = wave height, m

\( K \) = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains (see Tables 3.1 and A3.2.1)

\( KE \) = kinetic energy of a moving vessel, MNm

\( k \) = factor for calculation of load factor for wind effects, as determined by wind tunnel tests
**Symbols and Definitions:**

- \( L \) = length of a pier along the longitudinal axis, m (see Clause 3.11.4.2); live load (i.e., all applicable loads specified in Clause 3.8, including the dynamic load allowance, when applicable) (see Table 3.1); correlation length over which the vortices act in phase, m (see Table A3.2.4)
- \( L_c \) = width of horizontal clearance from pier(s), m
- \( L_p \) = perimeter of an oblong pier (excluding half-circles at the ends), m
- \( m(x) \) = mass per unit length of member at location \( x \), kg/m
- \( N \) = number of vessels passing under a bridge
- \( n \) = number of design lanes on a bridge
- \( n_e \) = frequency at which vortex shedding occurs for a member with constant diameter or frontal width, Hz
- \( n_e(x) \) = frequency at which vortex shedding excitation occurs at location \( x \) for a member with a tapered diameter or frontal width, Hz (see Clause A3.2.4.2)
- \( n_i \) = natural frequency of a member for mode of vibration, \( i \), Hz
- \( P \) = total load due to flowing water acting on a pier in the direction of its longitudinal axis, N (see Clause 3.11.4); secondary prestress effects (see Tables 3.1 and A3.2.1); vessel impact force, MN (see Clause A3.3.3.3.6)
- \( P_{BH} \) = ship bow collision force on an exposed superstructure, MN
- \( P_{DH} \) = ship deck house collision force on a superstructure, MN
- \( P_{MT} \) = ship mast collision force on a superstructure, MN
- \( P_s \) = ship collision force, MN
- \( P_p \) = total load due to flowing water acting on a pier in the horizontal direction perpendicular to its longitudinal axis, N
- \( P_A \) = probability of vessel aberrancy
- \( P_C \) = probability of bridge collapse due to a collision with an aberrant vessel
- \( P_G \) = geometric probability of a collision between an aberrant vessel and a bridge pier or span
- \( p \) = effective crushing strength of ice, kPa (see Clause 3.12.2); pedestrian load, kPa (see Clause 3.8.9)
- \( q \) = hourly mean reference wind pressure for the design return period, Pa
- \( R \) = radius of a circular pier, m; radius of half-circles at the ends of an oblong pier, m; radius of a circle that circumscribes each end of an oblong pier whose ends are not circular in plan at water level, m (see Clause 3.12.5)
- \( R_B \) = correction factor for bridge location to determine probability of vessel aberrancy
- \( R_BH \) = ratio of exposed superstructure depth to total bow depth
- \( R_C \) = correction factor for current acting parallel to vessel transit path to determine probability of vessel aberrancy
- \( R_D \) = correction factor for vessel traffic density to determine probability of vessel aberrancy
- \( R_{DH} \) = reduction factor for deck house collision force (see Clause A3.3.7.2)
- \( R_{XC} \) = correction factor for cross-currents acting perpendicular to vessel transit path to determine probability of vessel aberrancy
- \( R_e \) = Reynolds number (dimensionless) (see Table A3.2.4)
- \( r \) = radius of curve, m (see Clause 3.8.5); ratio of corner radius to radius of inscribed circle (see Table A3.2.2)
- \( S \) = Strouhal number (dimensionless) (see Clause A3.2.4.2); load due to differential settlement and/or movement of the foundation (see Tables 3.1 and A3.2.1)
- \( s \) = total loaded length of walkway, m
- \( T \) = torque on a support, N•mm
- \( t \) = thickness of ice expected to make contact with a pier, m (see Clause 3.12)
\( t_c \) = transverse component of wind load, N
\( V \) = hourly mean wind speed, m/s (see Clause A3.2.4.2); wind load on traffic (see Table 3.1); seismic zonal velocity ratio (dimensionless) (see Table A3.1.1); design collision velocity, m/s (see Clause A3.3.5)
\( V_C \) = current velocity component parallel to vessel transit path, m/s
\( V_T \) = typical vessel transit velocity, m/s
\( V_{XC} \) = current velocity component perpendicular to vessel transit path, m/s
\( V_{min} \) = minimum collision velocity, m/sec
\( V_{ref} \) = reference wind speed at deck height, m/s
\( V_w \) = coefficient of variation of wind effects, as determined by wind tunnel tests
\( \nu \) = design speed of a highway, km/h (see Clause 3.8.5); water velocity at the design flood, at SLS and ULS, m/s (see Clause 3.11.4)
\( W \) = gross load of the idealized Truck, kN (see Clause 3.8.3.2); vessel displacement tonnage, t (see Clause A3.3.6); wind load on structure (see Tables 3.1 and A3.2.1)
\( W_a \) = total normal wind load on a luminaire, sign panel, or traffic signal, N
\( W_c \) = deck width, m
\( W_e \) = width of design lane, m
\( W_h \) = total normal wind load on exposed horizontal supports, N
\( W_v \) = total normal wind load on exposed vertical supports, N
\( w \) = frontal pier width at the level of ice action where the ice is to be split or crushed, measured perpendicular to the direction of ice motion, m
\( X \) = distance to bridge element from centreline of vessel transit path, m
\( X_C \) = distance to edge of channel from centreline of vessel transit path, m
\( X_L \) = distance equal to three times the overall length of the design vessel from centreline of vessel transit path, m
\( x \) = coordinate describing length along the member, m
\( y_i(x) \) = peak member displacement due to vortex shedding excitation at location \( x \) for member mode of vibration, \( i \), m
\( Z_a \) = acceleration-related seismic zone (see Table A3.1.1)
\( Z_v \) = velocity-related seismic zone (see Table A3.1.1)
\( Z_W \) = width of zone of vessel collision, m
\( \alpha \) = pier nose angle to the horizontal plane, degrees, as shown in Figure 3.7; wind velocity profile exponent (see Clause A3.2.4.3.1)
\( \alpha_D \) = load factor for dead load
\( \alpha_E \) = load factor for earth pressure and hydrostatic pressure
\( \alpha_P \) = load factor for secondary prestress effects
\( \alpha_w \) = load factor for wind effects
\( \beta \) = subtended nose angle of an angular pier edge, degrees, as shown in Figure 3.7
\( \Delta T \) = temperature differential, °C
\( \delta_w \) = bias coefficient of wind effect, as determined by wind tunnel tests
\( \zeta_i \) = structural damping for the \( i \)th mode, expressed as a ratio of critical damping
\( \theta \) = angle between the direction of flow and the longitudinal pier axis, degrees (see Clause 3.11.4.2); angle of the turn of bend in channel, degrees (see Clause A3.3.3.3.1)
\( \theta_f \) = friction angle between ice and pier nose, degrees
\( \mu_i(x) \) = amplitude of the member mode shape at location \( x \) for mode of vibration, \( i \)
3.4 Limit states criteria

3.4.1 General
Bridge design shall be based on the limit states philosophy specified in Clause 1.4.2.1.

3.4.2 Ultimate limit states
Design shall provide a factored resistance that always exceeds the total factored load effect. Any structure where the total factored load effect could result in overturning, uplift, or sliding for any load combination shall be provided with anchorages.

3.4.3 Fatigue limit state
Structural components shall satisfy the requirements for the fatigue limit state specified in the applicable Sections of this Code for the appropriate loading combinations.

3.4.4 Serviceability limit states
Structural components shall satisfy the requirements for the serviceability limit states specified in the applicable Sections of this Code for the appropriate loading combinations.

Superstructure vibration limitations shall be considered a serviceability limit state.

Superstructures other than long-span bridges shall be proportioned so that the maximum deflection due to the factored traffic load, including the dynamic load allowance, does not exceed the limit shown in Figure 3.1 for the anticipated degree of pedestrian use. The deflection limit shall apply at the centre of the sidewalk or, if there is no sidewalk, at the inside face of the barrier. The traffic load shall be as specified in Clause 3.8.4.1(c).

An Approved method shall be used to ensure that vibration likely to occur in normal use will not cause discomfort or concern to users of a pedestrian bridge.
3.5 Load factors and load combinations

3.5.1 General

The loading combinations to be considered and the load factors to be used shall be as specified in Tables 3.1 and 3.2 unless otherwise specified in Clause 3.5.

Calibration of load factors and resistance factors shall be based on a minimum annual reliability index of 3.75 for CL-625 loading in accordance with Clause 3.8.3.

Every load that is to be included in a load combination shall be multiplied by the specified load factor and the resulting load effects shall be calculated. The factored load effects shall then be added together to obtain the total factored load effect.

If wind tunnel tests are used to derive wind loads, the wind load factors shall be as specified in Clause 3.10.5.2.

The load factors for the effects of elastic distortions shall be those of the loads causing the distortion. The load combinations for highway accessory supports and slender structural elements shall be as specified in Annex A3.2.

The total factored load effect used for each applicable load combination for construction loads shall not be less than 1.25 times the sum of the unfactored load effects included in the combination, unless otherwise Approved.
Table 3.1
Load factors and load combinations
(See Clauses 3.5.1, 3.10.1.1, 3.10.5.2, 3.13, 3.16.3, 4.10.7, 4.10.10.1,
7.6.3.1.1, 7.7.3.1.1, 9.4.2, and 15.6.2.4.)

<table>
<thead>
<tr>
<th>Loads</th>
<th>Permanent loads</th>
<th>Transitory loads</th>
<th>Exceptional loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>E</td>
<td>P</td>
</tr>
<tr>
<td>Fatigue limit state</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FLS Combination 1</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>SLS Combination 2†</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Serviceability limit states

<table>
<thead>
<tr>
<th>Loads</th>
<th>Permanent loads</th>
<th>Transitory loads</th>
<th>Exceptional loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>E</td>
<td>P</td>
</tr>
<tr>
<td>ULS Combination 1</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 2</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 3</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 4</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 5</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 6**</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 7</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 8</td>
<td>$\alpha_D$</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
<tr>
<td>ULS Combination 9</td>
<td>1.35</td>
<td>$\alpha_E$</td>
<td>$\alpha_P$</td>
</tr>
</tbody>
</table>

Ultimate limit states‡

*For the construction live load factor, see Clause 3.16.3.
†For superstructure vibration only.
§For wind loads determined from wind tunnel tests, the load factors shall be as specified in Clause 3.10.5.2.
**For long spans, it is possible that a combination of ice load F and wind load W will require investigation.

Legend:
A = ice accretion load
D = dead load
E = loads due to earth pressure and hydrostatic pressure, including surcharges but excluding dead load
EQ = earthquake load
F = loads due to stream pressure and ice forces or to debris torrents
H = collision load arising from highway vehicles or vessels
K = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains
L = live load (including the dynamic load allowance, when applicable)
P = secondary prestress effects
S = load due to differential settlement and/or movement of the foundation
V = wind load on traffic
W = wind load on structure
Table 3.2
Permanent loads — Maximum and minimum values
of load factors for ULS
(See Clauses 3.5.1, 3.5.2.1, 4.4.1, 4.4.9.3, and 7.8.7.1 and Table 3.1.)

<table>
<thead>
<tr>
<th>Dead load</th>
<th>Maximum $\alpha_D$</th>
<th>Minimum $\alpha_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factory-produced components, excluding wood</td>
<td>1.10</td>
<td>0.95</td>
</tr>
<tr>
<td>Cast-in-place concrete, wood, and all non-structural components</td>
<td>1.20</td>
<td>0.90</td>
</tr>
<tr>
<td>Wearing surfaces, based on nominal or specified thickness</td>
<td>1.50</td>
<td>0.65</td>
</tr>
<tr>
<td>Earth fill, negative skin friction on piles</td>
<td>1.25</td>
<td>0.80</td>
</tr>
<tr>
<td>Water</td>
<td>1.10</td>
<td>0.90</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dead load in combination with earthquakes</th>
<th>Maximum $\alpha_D$</th>
<th>Minimum $\alpha_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>All dead loads for ULS Combination 5 (see Table 3.1)</td>
<td>1.25</td>
<td>0.80</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Earth pressure and hydrostatic pressure</th>
<th>Maximum $\alpha_E$</th>
<th>Minimum $\alpha_E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passive earth pressure, considered as a load*</td>
<td>1.25</td>
<td>0.50</td>
</tr>
<tr>
<td>At-rest earth pressure</td>
<td>1.25</td>
<td>0.80</td>
</tr>
<tr>
<td>Active earth pressure</td>
<td>1.25</td>
<td>0.80</td>
</tr>
<tr>
<td>Backfill pressure</td>
<td>1.25</td>
<td>0.80</td>
</tr>
<tr>
<td>Hydrostatic pressure</td>
<td>1.10</td>
<td>0.90</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Prestress</th>
<th>Maximum $\alpha_P$</th>
<th>Minimum $\alpha_P$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Secondary prestress effects</td>
<td>1.05</td>
<td>0.95</td>
</tr>
</tbody>
</table>

*When passive earth pressure is considered as a resistance, it is factored in accordance with Section 6.

3.5.2 Permanent loads

3.5.2.1 General
Total factored load effects shall include those effects due to all permanent loads acting on the structure.

For ULS loading combinations, the maximum or minimum value specified in Table 3.2 for each load factor shall be used to maximize each total factored load effect. However, it is not normally necessary to consider load factors having different values in different spans.

Except as required by Clause 3.5.2.2, the minimum values for load factors specified in Table 3.2 shall not be used for some loads together with maximum values for other loads when the possibility of these loads having minimum and maximum values simultaneously can safely be excluded.

3.5.2.2 Overturning and sliding effects
When the maximum value of $\alpha_E$ for active pressure is used in calculating overturning for cantilever earth-retaining structures and for horizontal sliding, a value of 1.00 shall be used for $\alpha_D$.

In calculating backfill pressures that oppose one another or reduce load effects within a structure, all combinations of maximum and minimum earth pressure load factors shall be considered.

In calculating overturning moments occurring during balanced cantilever construction of segmental concrete bridges, maximum and minimum values of $\alpha_D$ equal to 1.05 and 1.0, respectively, may be used for the erected segments provided that construction controls are specified to ensure that the difference in weight between any two segments forming a balancing pair does not exceed 5%, and that all differences in weight are corrected before the addition of further segments.
3.5.3 Transitory loads
Transitory loads shall be included in the loading combinations only if there is a possibility of the loads being applied to the structure at the stage considered and their inclusion increases the total factored load effect.

3.5.4 Exceptional loads
Exceptional loads shall be included in the loading combinations only if there is a possibility of the loads being applied to the structure at the stage considered and their inclusion increases the total factored load effect.

3.6 Dead loads
Dead loads shall include the weight of all components of the structure and appendages fixed to the structure, including wearing surface, earth cover, and utilities.

In the absence of more precise information, the unit material weights specified in Table 3.3 shall be used in calculating dead loads.

The weight of water shall be considered dead load. Other static effects, including lateral or upward water pressure and buoyancy, shall be considered hydrostatic pressures.

The assumed water level shall be the maximum or minimum probable level, whichever produces the worst effect.

Table 3.3
Unit material weights
(See Clause 3.6.)

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit weight, kN/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminum alloy</td>
<td>27.0</td>
</tr>
<tr>
<td>Bituminous wearing surface</td>
<td>23.5</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
</tr>
<tr>
<td>Low-density concrete</td>
<td>18.1</td>
</tr>
<tr>
<td>Semi-low-density concrete</td>
<td>21.0</td>
</tr>
<tr>
<td>Plain concrete</td>
<td>23.5</td>
</tr>
<tr>
<td>Prestressed concrete</td>
<td>24.5</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>24.0</td>
</tr>
<tr>
<td>Coarse-grained (granular) soil</td>
<td>22.0</td>
</tr>
<tr>
<td>Crushed rock</td>
<td>22.0</td>
</tr>
<tr>
<td>Fine-grained sandy soil</td>
<td>20.0</td>
</tr>
<tr>
<td>Glacial till</td>
<td>22.0</td>
</tr>
<tr>
<td>Rockfill</td>
<td>21.0</td>
</tr>
<tr>
<td>Slag</td>
<td></td>
</tr>
<tr>
<td>Air-cooled slag</td>
<td>11.0</td>
</tr>
<tr>
<td>Water-cooled slag</td>
<td>15.0</td>
</tr>
<tr>
<td>Steel</td>
<td>77.0</td>
</tr>
<tr>
<td>Water</td>
<td></td>
</tr>
<tr>
<td>Fresh water</td>
<td>9.8</td>
</tr>
<tr>
<td>Salt or polluted water</td>
<td>10.5</td>
</tr>
<tr>
<td>Wood</td>
<td></td>
</tr>
<tr>
<td>Hardwood</td>
<td>9.5</td>
</tr>
<tr>
<td>Softwood</td>
<td>6.0</td>
</tr>
</tbody>
</table>
3.7 Earth loads and secondary prestress loads

3.7.1 Earth loads
Earth loads, other than those applied as dead loads, shall be as specified in Section 6. The requirements of Section 7 shall apply to buried structures.

3.7.2 Secondary prestress effects
Secondary prestress effects shall be as specified in Section 8.

3.8 Live loads

3.8.1 General
The live load models specified in Clauses 3.8.2 to 3.8.12 shall apply to all span ranges.

3.8.2 Design lanes
The number of design lanes for traffic shall be determined from Table 3.4. Each design lane shall have a width, $W_c$, of $W_c/n$.

<table>
<thead>
<tr>
<th>Deck width, $W_c$, m</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0 or less</td>
<td>1</td>
</tr>
<tr>
<td>Over 6.0 to 10.0</td>
<td>2</td>
</tr>
<tr>
<td>Over 10.0 to 13.5</td>
<td>2 or 3*</td>
</tr>
<tr>
<td>Over 13.5 to 17.0</td>
<td>4</td>
</tr>
<tr>
<td>Over 17.0 to 20.5</td>
<td>5</td>
</tr>
<tr>
<td>Over 20.5 to 24.0</td>
<td>6</td>
</tr>
<tr>
<td>Over 24.0 to 27.5</td>
<td>7</td>
</tr>
<tr>
<td>Over 27.5</td>
<td>8</td>
</tr>
</tbody>
</table>

*Both should be checked.

3.8.3 CL-W loading

3.8.3.1 General
CL-W loading consists of the CL-W Truck specified in Clause 3.8.3.2 or the CL-W Lane Load specified in Clause 3.8.3.3.

A loading of not less than CL-625 shall be used for the design of a national highway network that is generally used for interprovincial transportation.

A loading exceeding CL-625 may be specified by a provincial or territorial authority for the design of certain bridges within the province or territory.

Loadings lesser or greater than CL-625 shall be used only where justified by traffic conditions and shall require Approval.

Alternatively, a traffic load may be based on site-specific vehicle and traffic conditions established by vehicle count load surveys. The resulting level of safety shall be not less than that specified by this Code. Such a loading shall require Approval.
3.8.3.2 CL-W Truck

The CL-W Truck is the idealized five-axle truck shown in Figure 3.2. The $W$ number indicates the gross load of the CL-W Truck in kilonewtons. Wheel and axle loads are shown in terms of $W$ and are also shown for the CL-625 Truck.

The wheel spacings, weight distribution, and clearance envelope of the CL-W Truck shall be as shown in Figure 3.2.

In Ontario, a CL-625-ONT Truck as specified in Annex A3.4 shall be used.

Note: The total load of the CL-625-ONT Truck is 625 kN, but the axle load distribution differs from that shown in Figure 3.2.

The CL-W and the CL-625-ONT Truck shall be placed centrally in a space 3.0 m wide that represents the clearance envelope for each Truck, unless otherwise specified by the Regulatory Authority or elsewhere in this Code.

---

**Figure 3.2**

CL-W Truck

(See Clause 3.8.3.2.)
3.8.3.3 CL-W Lane Load
The CL-W Lane Load consists of a CL-W Truck with each axle reduced to 80% of the value specified in Clause 3.8.3.2, superimposed within a uniformly distributed load of 9 kN/m, and 3.0 m wide. The CL-W Lane Load is shown in Figure 3.3.
In Ontario, a CL-625-ONT Lane Load as specified in Annex A3.4 shall be used.

![CL-W Lane Load](image)

**Figure 3.3**
CL-W Lane Load
(See Clause 3.8.3.3.)

3.8.4 Application

3.8.4.1 General
The following requirements shall apply:
(a) Truck axles that reduce the load effect shall be neglected.
(b) The uniformly distributed portion of the lane load shall not be applied to those parts of a design lane where its application decreases the load effect.
(c) For the FLS and for SLS Combination 2, the traffic load shall be one truck only, placed at the centre of one travelled lane. The lane load shall not be considered.
(d) For SLS Combination 1 and for ultimate limit states, the traffic load shall be the truck load increased by the dynamic load allowance or the lane load, whichever produces the maximum load effect. This load shall be positioned longitudinally and transversely within each design lane at a location and in the direction that produces maximum load effect. The truck width shall not project beyond the design lane, except as specified in Clause 3.8.4.3(d). The lane load and the CL-W Truck clearance envelope shall not project beyond the edge of a design lane, except as specified in Clause 3.8.4.4.

3.8.4.2 Multi-lane loading
When more than one design lane is loaded, the traffic load shall be multiplied by the applicable modification factor specified in Table 3.5. Design lanes that are loaded shall be selected to maximize the load effect.
3.8.4.3 Local components
The following requirements shall apply:

(a) For components incorporated into decks other than modular expansion joints, e.g., manhole covers and drainage gratings, the axle load considered shall be axle no. 2 of the CL-W Truck.

(b) For modular expansion joints, the axle load considered shall be axle no. 4 of the CL-W Truck.

(c) For decks and other components whose design is governed by the axle loads, the tandem axle, comprising axles nos. 2 and 3 of the CL-W Truck, or axle no. 4 of the CL-W Truck, whichever produces larger effects, shall be considered.

(d) In the design lane adjacent to a curb, railing, or barrier, the minimum distance from the centres of the wheels to the curb, railing, or barrier wall shall be 0.30 m.

Note: The axle numbers are shown in Figure 3.2.

3.8.4.4 Wheels on the sidewalk
When sidewalks and other areas adjacent to a roadway are separated from it only by curbs and not by a traffic barrier, local responses shall be computed by considering a CL-W Truck with each axle load reduced to 70% and with its wheel centres not less than 0.30 m from the face of the railing or barrier on the outer edge. This requirement shall apply only at the ultimate limit states and shall not apply to longitudinal effects in slab bridges or to main girders.

3.8.4.5 Dynamic load allowance

3.8.4.5.1 General
A dynamic load allowance shall be applied to the CL-W Truck, or any part of the truck specified in Clause 3.8.3.2, unless otherwise specified by the Regulatory Authority or elsewhere in this Code. A dynamic load allowance shall not be applied to the CL-W Lane Load specified in Clause 3.8.3.3, including that part of the CL-W Lane Load represented by axle loads.

A dynamic load allowance shall be included in loads on the superstructure and loads transferred from the superstructure to the substructure, but shall not be included in loads transferred to footings that are surrounded with earth or to those parts of piles that are below ground.

A dynamic load allowance shall increase the truck loads by the proportion of the load specified in this Section unless alternative values based on tests or dynamic analysis are approved.

3.8.4.5.2 Buried structures
The dynamic load allowance for loads on arch-type buried structures with a depth of earth cover, $D_E$, between the riding surface and the highest point of the structure shall be $0.40(1 - 0.5D_E)$, but not less than 0.10.

The dynamic load allowance for box-type buried structures shall be the value obtained from Clause 3.8.4.5.3 multiplied by the factor $(1 - 0.5D_E)$, but not less than 0.10.

---

### Table 3.5
Modification factor for multi-lane loading
(See Clause 3.8.4.2.)

<table>
<thead>
<tr>
<th>Number of loaded design lanes</th>
<th>Modification factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>0.90</td>
</tr>
<tr>
<td>3</td>
<td>0.80</td>
</tr>
<tr>
<td>4</td>
<td>0.70</td>
</tr>
<tr>
<td>5</td>
<td>0.60</td>
</tr>
<tr>
<td>6 or more</td>
<td>0.55</td>
</tr>
</tbody>
</table>
3.8.4.5.3 Components other than buried structures

For components other than buried structures, the dynamic load allowance shall be
(a) 0.50 for deck joints;
(b) 0.40 where only one axle of the CL-W Truck is used (except for deck joints);
(c) 0.30 where any two axles of the CL-W Truck, or axles nos. 1 to 3, are used; or
(d) 0.25 where three axles of the CL-W Truck, except for axles nos. 1 to 3, or more than three axles, are used.

Note: The axle numbers are shown in Figure 3.2.

3.8.4.5.4 Reduction for wood components

For wood components, the dynamic load allowance specified in Clauses 3.8.4.5.2 and 3.8.4.5.3 shall be multiplied by 0.70.

3.8.5 Centrifugal force

For structures on horizontal curves, the centrifugal force shall be computed by multiplying CL-W Truck loads, without dynamic load allowance, by \( \frac{v^2}{127r} \), which shall be taken as non-dimensional.

The centrifugal force shall be applied horizontally at the centre of each design lane, at right angles to the direction of travel, and 2.0 m above the deck surface.

3.8.6 Braking force

Braking force shall be considered only at the ultimate limit states.

For continuously supported curbs, the design load shall be a uniformly distributed lateral load of 20 kN/m.

For curbs supported at discrete points, the design load shall be a concentrated lateral load of 32 kN.

Curb loads shall be applied at the top of the curb or 250 mm above the deck surface, whichever is lower.

3.8.8 Barrier loads

3.8.8.1 Traffic barriers

The transverse, longitudinal, and vertical loads shall be as specified in Table 3.6 and shall be applied simultaneously, as specified in Clause 12.4.3.5. These loads shall be used for the design of traffic barrier anchorages and decks only. Performance levels are defined in Clause 12.4.3.2.1. The barrier loads on railings shall not be considered to act simultaneously with the curb load or with the wheel loads positioned as specified in Clauses 3.8.4.1(a) and 3.8.4.4. A dynamic load allowance shall not be applied to these loads on barriers.
Table 3.6
Loads on traffic barriers
(See Clause 3.8.8.1.)

<table>
<thead>
<tr>
<th>Performance level</th>
<th>Transverse load, kN</th>
<th>Longitudinal load, kN</th>
<th>Vertical load, kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>PL-1</td>
<td>50</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>PL-2</td>
<td>100</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>PL-3</td>
<td>210</td>
<td>70</td>
<td>90</td>
</tr>
</tbody>
</table>

3.8.8.2 Pedestrian and bicycle barriers
The load on pedestrian and bicycle barrier railings shall be a uniform load of 1.20 kN/m applied laterally and vertically simultaneously.

3.8.9 Pedestrian load
For pedestrian bridges and sidewalks on highway bridges, the pedestrian load applied to the walkway area, $p$, shall be

$$p = 5.0 - \frac{s}{30}$$

but not less than 1.6 kPa and not greater than 4.0 kPa.

For highway bridges with sidewalks, traffic loads in design lanes shall be considered together with the pedestrian load only at the ultimate limit states, with the pedestrian load reduced by 20%.

The traffic load specified in Clause 3.8.4.4 and the pedestrian load shall not be considered to act simultaneously on a sidewalk.

3.8.10 Maintenance access loads
Maintenance access loads shall be considered only at the ultimate limit states.

Service walkways and safety gratings, and the members supporting them, shall be designed for a live load of 1.6 kN uniformly distributed over a rectangular area $1.00 \times 0.50$ m placed anywhere on the walkway or grating.

Manhole steps and ladder rungs shall be designed for a load of 1.0 kN distributed over a length of 100 mm anywhere on the tread area. The tread area shall include all of the horizontal part of the step or rung except parts beyond bends or other features that effectively limit the usable tread length.

3.8.11 Maintenance vehicle load
If the width of a sidewalk on a highway bridge, or of a pedestrian bridge, is greater than 3.0 m and access is provided for maintenance vehicles, the maintenance vehicle load shown in Figure 3.4 shall be considered on the walkway area.

For sidewalks on a highway bridge, the maintenance vehicle load shall be considered only at the ultimate limit states.

The maintenance vehicle load shall not be considered to act simultaneously with the pedestrian live load or with the loading from wheels on the sidewalk specified in Clause 3.8.4.4.

3.8.12 Multiple-use structures
Where a highway bridge is used for other purposes, e.g., railway, rail transit, or other utility purposes, the loads and load factors shall be specified by the appropriate Regulatory Authority.
3.9 Superimposed deformations

3.9.1 General
Clause 3.9 specifies requirements related to the effects of temperature changes, shrinkage, creep, thermal gradients through the depth of superstructure, and foundation deformations. These requirements apply to concrete structures, steel structures, and composite structures built from concrete and steel. Analysis of temperature, shrinkage, and creep effects shall not be required for conventional wood structures, but shrinkage and swelling that are perpendicular to the grain and are due to moisture changes shall be considered.

3.9.2 Movements and load effects
Provision shall be made for all expansion and contraction that can occur as a result of variations in effective temperature, shrinkage, and creep. All load effects induced by restraint of these dimensional changes, including temporary restraints required during construction, shall be included in the analysis.

The effects of concrete creep and shrinkage shall be as specified in Section 8. Temperature differentials and foundation deformation shall be considered when the resulting distortions and displacements or the restraint thereof can cause significant load effects, or where the serviceability of the structure could be affected.

When it can be shown that inelastic behaviour reduces the load effects at the ULS and that the structure can sustain such inelastic behaviour, the reduced load effects may be considered.

3.9.3 Superstructure types
Temperature effects shall be considered for the following superstructure types:
(a) Type A: steel beam, box, or deck truss systems with steel decks, and truss systems that are above the deck;
(b) Type B: steel beam, box, or deck truss systems with concrete decks; and
(c) Type C: concrete systems with concrete decks.

3.9.4 Temperature effects

3.9.4.1 Temperature range
The temperature range shall be the difference between the maximum and minimum effective temperatures as specified in Table 3.7 for the type of superstructure. The temperature range shall be modified in accordance with the depth of the superstructure as indicated in Figure 3.5. The maximum and minimum mean daily temperature shall be taken from Figures A3.1.1 and A3.1.2.

<table>
<thead>
<tr>
<th>Superstructure type (see Clause 3.9.3.)</th>
<th>Maximum effective temperature</th>
<th>Minimum effective temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>25 °C above maximum mean daily temperature</td>
<td>15 °C below minimum mean daily temperature</td>
</tr>
<tr>
<td>B</td>
<td>20 °C above maximum mean daily temperature</td>
<td>5 °C below minimum mean daily temperature</td>
</tr>
<tr>
<td>C</td>
<td>10 °C above maximum mean daily temperature</td>
<td>5 °C below minimum mean daily temperature</td>
</tr>
</tbody>
</table>

Figure 3.5
Modifications to maximum and minimum effective temperatures
(See Clause 3.9.4.1.)
3.9.4.2 Effective construction temperature
In the absence of site-specific data, an effective construction temperature of 15 °C shall be assumed for design. This temperature shall be used to determine the effective temperature ranges for the calculation of expansion and contraction.

With Type C structures that are cast in place, the heat of hydration can cause the concrete temperature to be higher than the effective construction temperature at the time of initial set. If more precise data are not available, it shall be assumed that concrete cools by 25 °C from its initial set to the effective construction temperature.

3.9.4.3 Positioning of bearings and expansion joints
The Plans shall indicate the positioning of bearings and expansion joints in accordance with Section 11.

3.9.4.4 Thermal gradient effects
The effects of thermal gradients through the depth shall be considered in the design of Type A, B, and C structures.

A thermal gradient is positive when the top surface of the superstructure is warmer than the bottom surface.

The values of temperature differentials are given for Type A and C structures in Figure 3.6. For winter conditions, positive and negative differentials shall be considered. For summer conditions, only positive differentials shall be considered.

For composite and non-composite Type B structures, a positive temperature differential decreasing linearly by 30 °C from the top to the bottom of the deck slab shall be considered. The temperature shall be assumed to remain constant throughout the beam or truss below the slab. It shall not be necessary to consider negative differentials.

Allowances shall be made for the stresses and deformations induced when the coefficients of thermal expansion of the materials used in a composite structure differ.

3.9.4.5 Thermal coefficient of linear expansion
The thermal coefficient of linear expansion shall be as specified in Section 8 for concrete and Section 10 for steel.

![Figure 3.6](image-url)

**Figure 3.6**
Temperature differentials for Type A and C superstructures
(See Clause 3.9.4.4.)
3.10 Wind loads

3.10.1 General

Clause 3.10 specifies design wind loads for all highway structures and provides specific requirements for bridge substructures and superstructures. Guidance for determining the tendency toward aeroelastic instability in wind-sensitive bridges is also provided. All wind loads based on the reference wind pressure, \( q \), shall be treated as equivalent static loads.

Special requirements for wind tunnel testing are specified in Clause 3.10.5, with reference to the determination of specific load factors to replace those specified in Table 3.1 for wind load effects.

Annex A3.2 specifies requirements for wind loads on highway accessory supports, barriers, and slender structural elements, including the effects of vortex shedding.

3.10.1.2 Reference wind pressure

The hourly mean reference wind pressure, \( q \), shall be as specified in Table A3.1.1 for a return period of

(a) 100 years for bridge structures with any span 125 m long or longer;
(b) 50 years for bridge structures with a maximum span shorter than 125 m, luminaire support structures higher than 16 m, and overhead sign structures;
(c) 25 years for luminaire and traffic signal support structures 16 m high or shorter, and for barriers; and
(d) 10 years for roadside sign structures where a long life expectancy is not required, or for any of the structures specified in Items (a) to (c) during construction.

If the topography at the structure site can cause funnelling of the wind, the reference wind pressure shall be increased by 20%.

3.10.1.3 Gust effect coefficient

For highway bridges that are not sensitive to wind action (which includes most bridges of spans less than 125 m except those that are cable supported), the gust effect coefficient, \( C_g \), shall be taken as 2.0.

For slender, lighter structures, e.g., pedestrian bridges, luminaire, sign, and traffic signal supports, barriers, and slender structural elements, \( C_g \) shall be taken as 2.5.

For structures that are sensitive to wind action, the gust factor approach shall not be used and the wind loads shall be determined on the basis of a detailed analysis of dynamic wind action, using an Approved method that includes the effects of buffeting.

3.10.1.4 Wind exposure coefficient

The wind exposure coefficient, \( C_e \), shall not be less than 1.0 and shall be taken from Table 3.8 or calculated as \( (0.10H)^{0.2} \), where \( H \) is the height above ground of the top of the superstructure. For luminaire, sign, and traffic signal supports, and for barriers, \( H \) shall be taken to the top of the standard, support, or structure considered. The height above ground shall be measured from the foot of cliffs, hills, or escarpments when the structure is located in uneven terrain, or from the low water level for structures over water.
3.10.1.5 Non-uniform loading
Wind loads shall be applied uniformly or non-uniformly over the entire structure, whichever produces the more critical effects. Unless an analysis of non-uniform wind loads specific to the structure is undertaken, the non-uniform loading shall be 0.75 times the effective uniformly distributed load over any portion of the structure and the full effective uniformly distributed load applied over the remaining portion.

3.10.1.6 Overturning and overall stability
When the prescribed loads in the design of members are being applied, overturning, uplift, and lateral displacement shall be considered.

3.10.1.7 Alternative methods
When Approved, representative wind tunnel tests or more detailed methods of analysis may be used to establish load coefficients or design criteria different from those specified in this Section.

Wind loads derived from the results of wind tunnel tests shall be used with wind load factors determined in accordance with Clause 3.10.5.2.

3.10.2 Design of the superstructure

3.10.2.1 General
The superstructure shall be designed for wind-induced vertical and horizontal drag loads acting simultaneously. The assumed wind direction shall be perpendicular to the longitudinal axis for a straight structure or to an axis chosen to maximize wind-induced effects for a structure curved in plan.

3.10.2.2 Horizontal drag load
The following wind load per unit exposed frontal area of the superstructure shall be applied horizontally:

\[ F_h = q C_e C_g C_h \]

where \(q, C_e, \) and \(C_g\) are as specified in Clauses 3.10.1.2, 3.10.1.4, and 3.10.1.3, respectively, and \(C_h = 2.0\).

In the case of truss spans, this load shall be taken to act on the windward truss and an identical force shall be simultaneously applied to the leeward truss unless a recognized method is used to calculate the shielding effect of the windward truss.

3.10.2.3 Vertical load
The following wind load per unit exposed plan area of the superstructure shall be applied vertically:

\[ F_v = q C_e C_g C_v \]

### Table 3.8
**Wind exposure coefficient, \(C_e\)**

(See Clause 3.10.1.4.)

<table>
<thead>
<tr>
<th>Height above ground of the top of the superstructure, (H, \text{ m})</th>
<th>Wind exposure coefficient, (C_e)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 10</td>
<td>1.0</td>
</tr>
<tr>
<td>Over 10 to 16</td>
<td>1.1</td>
</tr>
<tr>
<td>Over 16 to 25</td>
<td>1.2</td>
</tr>
<tr>
<td>Over 25 to 37</td>
<td>1.3</td>
</tr>
<tr>
<td>Over 37 to 54</td>
<td>1.4</td>
</tr>
<tr>
<td>Over 54 to 76</td>
<td>1.5</td>
</tr>
<tr>
<td>Over 76 to 105</td>
<td>1.6</td>
</tr>
</tbody>
</table>
where \( q \), \( C_{e} \), and \( C_{g} \) are as specified in Clauses 3.10.1.2, 3.10.1.4, and 3.10.1.3, respectively, and \( C_{v} = 1.0 \). The vertical load shall be taken to act either upwards or downwards.

In addition to the application of \( F_{v} \) as a uniformly distributed load over the whole plan area, the effect of possible eccentricity in the application of the load shall be considered. For this purpose, the same total load shall be applied as an equivalent vertical line load at the windward quarter point of the transverse superstructure width.

### 3.10.2.4 Wind load on live load

The horizontal wind load per unit exposed frontal area of the live load shall be calculated in accordance with Clause 3.10.2.2, except that \( C_{h} \) shall be taken as 1.2. The exposed frontal area of the live load shall be the entire length of the superstructure, as seen in elevation in the direction of the wind as specified in Clause 3.10.2.1, or any part or parts of that length producing critical response, multiplied by a height of 3.0 m above the roadway surface for vehicular bridges and 1.5 m for pedestrian bridges. Areas below the top of a solid barrier wall shall be neglected.

### 3.10.3 Design of the substructure

#### 3.10.3.1 General

The substructure shall be designed for wind-induced loads transmitted to it from the superstructure and for wind loads acting directly on the substructure. Loads for wind directions both normal to and skewed to the longitudinal centreline of the superstructure shall be considered.

#### 3.10.3.2 Wind loads transmitted from the superstructure

The horizontal drag load specified in Clause 3.10.2.2 shall be resolved into transverse and longitudinal components using the skew angle modification coefficients specified in Table 3.9. These loads shall be applied as equivalent horizontal line loads at the elevation of the centroid of the exposed frontal area of the superstructure.

The vertical load specified in Clause 3.10.2.3, modified for skew angle using appropriate coefficients from Table 3.9, shall be applied as an upward or downward line load along the centreline of the superstructure or along the windward quarter point, whichever produces the more critical effect.

The vertical load and the longitudinal and transverse horizontal loads shall be applied simultaneously and the combination leading to maximum load effects in the substructure shall be used.

The requirements of Clause 3.10.2.4 shall apply in determining the wind load on the live load that is to be transferred to the substructure. The modifications specified for “Other spans” in Table 3.9 shall apply to skewed wind loads on the live load on any type of span.

Longitudinal loads shall be determined for winds parallel to the longitudinal axis of the bridge (i.e., at a skew angle of 90°) using the projected area to the wind of the bridge superstructure in the longitudinal direction.
3.10.3.3 Loads applied directly to substructure
The substructure shall be designed for directly applied horizontal drag loads. The wind load on a unit frontal exposed area of the substructure shall be calculated in accordance with Clause 3.10.2.2. The horizontal drag coefficient, \( C_h \), shall be taken as 0.7 for circular piers, 1.4 for octagonal piers, and 2.0 for rectangular and square piers.

For wind directions skewed to the substructure, the loads shall be resolved into components taken to act perpendicularly to the end and side elevations of the substructure. These load components shall be assumed to act horizontally at the centroids of the exposed areas of the end and side elevations and shall be applied simultaneously with the loads transmitted from the superstructure.

### Table 3.9
Modification of wind loads on superstructure with skew angle
(See Clause 3.10.3.2.)

<table>
<thead>
<tr>
<th>Skew angle (measured from a line normal to the longitudinal axis), degrees</th>
<th>Modification coefficients</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.00</td>
<td>0.00</td>
<td>1.00</td>
</tr>
<tr>
<td>15</td>
<td>0.93</td>
<td>0.16</td>
<td>0.88</td>
</tr>
<tr>
<td>30</td>
<td>0.87</td>
<td>0.37</td>
<td>0.82</td>
</tr>
<tr>
<td>45</td>
<td>0.63</td>
<td>0.55</td>
<td>0.66</td>
</tr>
<tr>
<td>60</td>
<td>0.33</td>
<td>0.67</td>
<td>0.34</td>
</tr>
</tbody>
</table>

### 3.10.4 Aeroelastic instability

#### 3.10.4.1 General
Aeroelastic instability, in which the motion of the structure in wind produces aerodynamic forces augmenting such motion, shall be taken into account in the design of bridges and structural components apt to be wind sensitive. The aeroelastic phenomena of vortex shedding, galloping, flutter, and divergence shall be considered where applicable.

#### 3.10.4.2 Criterion for aeroelastic instability
For a wind-sensitive structure affected by the wind actions specified in Clause 3.10.4.1, it shall be shown that the performance of the structure without further application of load factors is acceptable up to a wind speed higher than the reference wind speed, \( V_{ref} \). Unless alternative rational procedures are available, the reference wind speed shall be taken as

\[
V_{ref} = 1.24 \sqrt{\alpha_w q C_e}
\]

where
\[
\alpha_w = \text{the load factor for wind specified in Clause 3.5.1}
\]

The reference wind velocity shall be taken at deck height. Bridges and their structural components, including cables, shall be designed to be free of fatigue damage due to vortex-induced or galloping oscillations.
3.10.5 Wind tunnel tests

3.10.5.1 General
Structures that are sensitive to wind include those that are flexible, slender, lightweight, long span, or of unusual geometry. For such structures, supplementary studies by an expert in the field should be conducted, and it is possible that wind tunnel tests will be required. Representative wind tunnel tests may be used to satisfy the requirements of Clauses 3.10.4.1 and 3.10.4.2. These tests may also be used to establish the components of the overall structural loads specified in Clauses 3.10.2 and 3.10.3.

3.10.5.2 Load factors
If the overall structural loads due to wind are determined using wind tunnel tests, the load factor for wind, $\alpha_w$, in ULS Combination 4 shall be calculated as

$$\alpha_w = 0.80 \delta_w \exp(3.5kV_w)$$

where

$$k = \sqrt{\frac{V_w^2}{0.15^2 + V_w^2}}$$

The bias coefficient, $\delta_w$, and coefficient of variation, $V_w$, of the wind load effect shall be determined by the persons responsible for the wind tunnel tests and shall account for the bias and uncertainty of the reference wind pressure, the gust, the pressure and exposure coefficients, and the uncertainty of the modelling.

Wind load factors for design in the ULS Combination 3 and ULS Combination 7 shall be the product of the factor specified in Table 3.1 and the ratio ($\alpha_w / 1.65$).

3.11 Water loads

3.11.1 General
Local conditions at the site shall be considered in all cases.

3.11.2 Static pressure
Static water pressure shall be assumed to act perpendicular to the surface that is retaining the water. The pressure of water at a specific point shall be calculated as the product of the height of water above that point and the density of water.

3.11.3 Buoyancy
The effects of immersion in water or exposure to water pressure shall be considered. The beneficial effects of buoyancy shall be included, provided that they are always in existence. The non-beneficial effects of buoyancy shall be included unless the possibility of their occurrence can be excluded with certainty.

Buoyancy shall be taken as the vertical components of the static forces as calculated in accordance with Clause 3.11.2. Buoyancy shall be considered as an uplift force equivalent to the volume of water displaced.

3.11.4 Stream pressure

3.11.4.1 Longitudinal effects
The load due to flowing water acting longitudinally on a substructure element, $P$, shall be taken as $C_D\rho A v^2/2$, where the longitudinal drag coefficient, $C_D$, is as specified in Table 3.10.
Table 3.10
Longitudinal drag coefficient, $C_D$
(See Clause 3.11.4.1.)

<table>
<thead>
<tr>
<th>Upstream shape of pier</th>
<th>Longitudinal drag coefficient, $C_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Semi-circular nosed</td>
<td>0.7</td>
</tr>
<tr>
<td>Square ended</td>
<td>1.4</td>
</tr>
<tr>
<td>Wedge nosed at $\leq 90^\circ$</td>
<td>0.8</td>
</tr>
<tr>
<td>Pier with debris lodged</td>
<td>1.4</td>
</tr>
</tbody>
</table>

3.11.4.2 Lateral effects

The lateral load due to water flowing at angle, $\theta$, against a substructure element, $P_p$, shall be taken as $C_L \rho HLv^2/2$, where the lateral load coefficient, $C_L$, is as specified in Table 3.11.

Table 3.11
Lateral load coefficient, $C_L$
(See Clause 3.11.4.2.)

<table>
<thead>
<tr>
<th>Angle, $\theta$, between direction of flow and longitudinal pier axis, degrees</th>
<th>Lateral load coefficient, $C_L$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>5</td>
<td>0.5</td>
</tr>
<tr>
<td>10</td>
<td>0.7</td>
</tr>
<tr>
<td>20</td>
<td>0.9</td>
</tr>
<tr>
<td>$\geq 30$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

3.11.5 Wave action

Force effects due to wave action on bridge substructure elements exposed to environments where significant wave action can occur shall be evaluated in accordance with site-specific conditions. In the absence of such evaluations, the force against a flat surface substructure element, $F_w$, due to wave action, as a function of the wave height, $H_w$, shall be taken as $10H_w^2$.

$F_w$ shall be considered to act at mid-height of the wave, $H_w/2$, above the still water elevation. For aerodynamically curved frontal surfaces, a value of $F_w/2$ shall be used.

3.11.6 Scour action

Local conditions and past records of floods shall be consulted in designing foundation elements when scour is expected to occur. The requirements of Sections 1 and 6 shall be applied. Changes in foundation conditions resulting from the design flood shall be considered at serviceability and ultimate limit states.

3.11.7 Debris torrents

Debris torrent loads shall be considered on exposed superstructures and substructures in accordance with site-specific conditions. Sites subject to heavy rainfall of short duration, earthquakes, landslides, and rockfalls shall be investigated for debris torrents when the following conditions exist:

(a) the creek channel gradient is greater than 25° for an extended length along the channel profile;
(b) boulders and debris exist in the channel;
(c) there is a history of such events.

An expert in the field shall be consulted to determine debris torrent loads.
3.12 Ice loads

3.12.1 General
Clause 3.12 refers only to freshwater ice in rivers and lakes. For ice loads in sea water, specialist advice shall be sought.

Ice forces on bridge substructure elements shall be determined by considering the prevailing site conditions and expected form of ice action. The following interaction modes between ice and structure shall be considered:
(a) dynamic forces due to collision of moving ice sheets or floes carried by the stream current or driven by wind action (both horizontal and vertical components shall be considered);
(b) static forces due to thermal movements of continuous stationary ice sheets;
(c) lateral thrust due to arching action resulting from ice dams and ice jams; and
(d) static or dynamic vertical forces along the substructure element due to the effects of fluctuating water levels or the dynamic effects of colliding ice floes.

Data related to the anticipated thickness of ice, its direction of movement, its speed of impact, and the height of its action on the substructure element shall be obtained or derived from field surveys and records of measurements made at or near the site.

3.12.2 Dynamic ice forces

3.12.2.1 Effective ice strength
Unless more precise data is available, the following values for the effective crushing strength of ice, $p$, shall be used:
(a) the ice breaks up at melting temperature and is substantially disintegrated: 400 kPa;
(b) the ice breaks up at melting temperature and is somewhat disintegrated: 700 kPa;
(c) the ice breaks up or ice movement occurs at melting temperature and is internally sound and moving in large pieces: 1100 kPa; and
(d) the ice breaks up or ice movement occurs at temperatures considerably below the melting point or the ice: 1500 kPa.

3.12.2.2 Crushing and flexural strength
The horizontal force, $F$, due to the pressure of moving ice shall be taken as

\[ F = \begin{cases} F_b \text{ for } w/t < 6.0 \\ F_c \text{ for } w/t > 6.0 \end{cases} \]

where

\[ F_b = C_n p t^2 \]

where

\[ C_n = 0.5 \tan (\alpha + 15^\circ) \]

with $\alpha$ as shown in Figure 3.7

\[ F_c = C_a p t w \]

where

\[ C_a = \sqrt{\frac{s}{w} + 1} \]

In small streams where it is unlikely that large-size ice floes will form, the force, $F$, may be reduced by up to 50% of the value in accordance with this Clause.
3.12.2.3 Ice impact forces

3.12.2.3.1 Piers parallel to flow

Where the longitudinal axis of the pier is reasonably parallel to the direction of the movement of ice, the force, \( F \), as derived from Clause 3.12.2.2, shall be considered to act along the longitudinal axis of the pier. The following design cases shall be investigated:

(a) case 1: a longitudinal force, \( F \), plus a transverse force, \( 0.15F \); and  
(b) case 2: a longitudinal force, \( 0.5F \), plus a transverse force, \( F_t \), where

\[
F_t = \frac{F}{2\tan(0.5\beta + \theta_f)}
\]

In the absence of more precise data, \( \theta_f \) shall be taken as \( 6^\circ \). For a round-nosed edge, \( \beta \) shall be taken as \( 100^\circ \), where \( \beta \) is as shown in Figure 3.7.

**Figure 3.7**  
Pier nose angle and subtended nose angle for calculating forces due to moving ice  
(See Clauses 3.12.2.2 and 3.12.2.3.1.)
3.12.2.3.2 Piers skewed to flow
For piers with their longitudinal axis at an angle to the direction of flow, the total collision forces shall be considered to act on the projected pier width and resolved into components parallel and perpendicular to the pier shaft. The component that acts transversely on the pier shaft shall not be taken as less than 20% of the total force.

3.12.2.4 Slender piers
Where ice forces are significant, slender and flexible piers and their components, e.g., piles exposed to ice action, shall be used only when a specialist on the mechanics of ice and structure interaction is consulted.

3.12.3 Static ice forces
Where ice sheets are exposed to non-uniform thermal stresses and strains relative to the pier due to unbalanced freezing, the resulting forces on the piers shall be calculated using a compressive crushing strength of ice of not less than 1500 kPa when the ice temperature is significantly below the freezing point.

3.12.4 Ice jams
For clear openings of less than 30 m between piers or between a shoreline and a pier located in bodies of water where floating ice can occur, a pressure of 10 kPa shall be considered to act against the exposed substructure element. This force shall be applied above the level of still water for the expected thickness of the ice jam, both laterally and in the direction of the ice flow. For clear openings of more than 30 m, this force may be reduced to 5 kPa against the exposed faces.

3.12.5 Ice adhesion forces
The vertical force due to water level fluctuations, $F_v$, on a pier frozen to an ice formation shall be calculated as follows:
(a) for circular piers:
\[ F_v = 1250t^2(1.05 + 0.13R/t^{0.75}); \]
and
(b) for oblong piers:
\[ F_v = 15Lp t^{1.25} + 1250t^2(1.05 + 0.13R/t^{0.75}). \]

3.12.6 Ice accretion

3.12.6.1 General
Ice accretion loads shall be taken to occur on all exposed surfaces of superstructure members, structural supports, traffic signals, luminaires, and railings. In the case of sign panels, bridge girders, and solid barriers, ice accretion shall be considered to occur on one side only.

3.12.6.2 Load effect
The design ice thickness for ice accretion shall be the value specified in Figure A3.1.4. A unit weight of 9.8 kN/m$^3$ shall be used in calculating ice accretion loads.

3.13 Earthquake effects
Requirements for calculating effects due to earthquake forces are specified in Section 4. Relevant data pertaining to seismic zones, velocities, and accelerations are provided in Table A3.1.1 and Figures A3.1.6 and A3.1.7. Load combinations and load factors are specified in Table 3.1.
3.14 Vessel collisions

3.14.1 General
In a navigable waterway crossing where there is a risk of vessel collision, all bridge elements that could be hit shall be designed for vessel impact or adequately protected from vessel collision. The design procedure for vessel collision shall be as specified in Annex A3.3.

The following general requirements shall apply:

(a) In navigable waterways where vessel collision is possible, structures shall be
   (i) designed to resist the design vessel collision forces;
   (ii) evaluated to meet a minimum level of safety; or
   (iii) adequately protected by fenders, dolphins, berms, islands, or other devices, as appropriate.

(b) Consideration shall be given to the relationship of the bridge (including its structural dynamic response) to the following:
   (i) waterway geometry;
   (ii) size, type, loading condition, and frequency of vessels using the waterway;
   (iii) navigable water depth; and
   (iv) vessel speed and direction.

3.14.2 Bridge classification
Bridges shall be classified as follows:
(a) Class I: bridges that are of critical importance, including those that have to remain open to all traffic after a vessel collision.
(b) Class II: bridges that are of regular importance, including those that have to remain open to emergency and security vehicles after a vessel collision.

3.14.3 Assessment
Two methods, specified in Annex A3.3, may be used for assessing the classification criteria, the selection of the design vessel, and the calculation of the vessel collision forces. Method I is a simplified approach. Method II is a probabilistic approach based on $A_{F_{\text{max}}}$, the maximum annual frequency of collapse for the whole bridge.

3.14.4 Annual frequency of collapse
The annual frequency of collapse, $A_F$, for each pier and span component susceptible to ship collision shall be determined by distributing the total bridge acceptance criterion, $A_{F_{\text{max}}}$, over the number of piers and span components located in the navigable waterway.

3.14.5 Design vessel

3.14.5.1 Frequency distribution
The number of vessels, $N$, passing under the bridge shall be developed for each pier and span component being evaluated in accordance with the size, type, and loading condition of the vessels and the depth of navigable water.

3.14.5.2 Selection
For Method I, the selection of the design vessel shall be based solely on the frequency distribution of vessel traffic. For Method II, a design vessel for each pier or span component shall be selected, such that the estimated annual frequency of collapse due to vessels equal to and larger than the design vessel is less than the maximum permitted annual frequency, $A_{F_{\text{max}}}$.

3.14.6 Application of collision forces
Forces shall be applied as equivalent static forces for superstructure and pier design (see Annex A3.3).
3.14.7 Protection of piers
Protection may be provided to reduce or eliminate the exposure of bridge piers to vessel collision. Physical protection systems may include fenders, pile clusters, pile-supported structures, dolphins, islands, and combinations thereof. Such protection systems shall be considered sacrificial and be capable of stopping the vessel before contact with the pier or redirecting the vessel away from the pier.

3.15 Vehicle collision load
Highway bridge piers located less than 10 m from the edge of the road pavement shall be designed for a collision load equivalent to a horizontal static force of 1400 kN. The collision load shall be applied horizontally 1.20 m above ground level at the pier, and at 10° to the direction of travel.

3.16 Construction loads and loads on temporary structures

3.16.1 General
The weights of materials, workers, and equipment supported during construction shall be considered dead loads or live loads in accordance with Clauses 3.16.2 and 3.16.3.

The possibility of occurrence of loads due to wind, ice, and stream flow shall be determined in accordance with the expected life of the structure or the duration of the construction stage considered. A ten-year return period shall be used for these loads when they are applied.

3.16.2 Dead loads
Dead loads shall include the weights of formwork, falsework, fixed appendages, stored material, and lifting and launching devices, or parts thereof, that are not subject to movement during the construction stage considered.

3.16.3 Live loads
Live loads shall include the weights of workers, vehicles, hoists, cranes, other equipment, and structural components that are subject to movement during the construction stage considered. The live load factor to be used for construction live loads shall be 85% of the value specified for $L$ in Table 3.1.

3.16.4 Segmental construction

3.16.4.1 Erection loads
Erection loads assumed in design shall be shown on the Plans. Erection loads shall include all induced forces due to the anticipated system of temporary works, erection equipment, construction sequence, and closure forces due to misalignment corrections.

Consideration shall be given to the effects of any changes to the statics of the structural system occurring during construction and the imposition, change, and removal of any temporary supports, erection equipment, or assumed loads, including residual built-in forces, deformations, post-tensioning effects, creep, shrinkage, and thermal and any other strain-induced effects.

3.16.4.2 Construction live loads
Except for bridges constructed by incremental launching, a uniformly distributed load of not less than 500 Pa over the constructed deck area of the bridge shall be considered, to allow for the weight of miscellaneous equipment and machinery. For balanced cantilever construction, the load shall be not less than 500 Pa on one cantilever and shall be 250 Pa on the other.
Consideration shall be given to all loads from special construction equipment such as a form traveller, launching gantry or truss, lifting winch or crane, or segment delivery truck and to the static and dynamic force effects produced during segment lifting. The Plans shall require that the actual loads be obtained from the manufacturers of the equipment and Approved before construction.

Forces due to acceleration and slippage during lifting shall be considered. An equivalent static load increment equal to at least 10% of the weight of the segment and attachments shall be assumed. When accelerations are not accurately predictable and controllable, an equivalent static load increment of 100% shall be assumed.

Horizontal forces due to braking or acceleration of mobile construction equipment shall be considered in the design. Such forces shall be at least 2% of the total weight of the equipment.

3.16.4.3 Incremental launching
Incrementally launched bridges shall be designed to resist the effects of bearing construction tolerances and friction on launching bearings. When inclined launching bearings are used (as opposed to permanent horizontal bearings) the additional forces at the launching jacks and the piers shall be considered.

The coefficient of friction on launching bearings made of polished stainless steel sliding on lubricated polytetrafluoroethylene (PTFE) in compliance with Section 11 shall be assumed to vary between zero and 0.04, whichever governs holdback or pushing forces.

3.16.5 Falsework
Falsework shall be designed and detailed in accordance with CSA S269.1.
Annex A3.1 (normative)
Climatic and environmental data

Notes:
(1) This Annex is a mandatory part of this Code.
(2) See Annex CA3.1 of CSA S6.1 for the sources and derivation of the data presented in this Annex.

### Table A3.1.1
Reference wind pressure and seismic zoning
(See Clauses 3.10.1.2, 3.13, and 4.4.3.)

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Barrhead  | 315 | 380 | 430 | 490 | 1 | 0.05 | 0 | 0.00|

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<td>545</td>
<td>600</td>
<td>0</td>
</tr>
<tr>
<td>Chesterfield Inlet</td>
<td>440</td>
<td>510</td>
<td>565</td>
<td>620</td>
<td>0</td>
</tr>
<tr>
<td>Clyde River</td>
<td>615</td>
<td>765</td>
<td>890</td>
<td>1020</td>
<td>3</td>
</tr>
<tr>
<td>Coral Harbour</td>
<td>750</td>
<td>885</td>
<td>995</td>
<td>1100</td>
<td>0</td>
</tr>
<tr>
<td>Eskimo Point</td>
<td>490</td>
<td>570</td>
<td>640</td>
<td>710</td>
<td>0</td>
</tr>
<tr>
<td>Eureka</td>
<td>475</td>
<td>580</td>
<td>670</td>
<td>760</td>
<td>0</td>
</tr>
<tr>
<td>Iqaluit</td>
<td>565</td>
<td>670</td>
<td>750</td>
<td>840</td>
<td>0</td>
</tr>
<tr>
<td>Isachsen</td>
<td>680</td>
<td>800</td>
<td>900</td>
<td>1000</td>
<td>1</td>
</tr>
<tr>
<td>Nottingham Island</td>
<td>460</td>
<td>560</td>
<td>640</td>
<td>720</td>
<td>0</td>
</tr>
<tr>
<td>Rankin Inlet</td>
<td>460</td>
<td>535</td>
<td>595</td>
<td>660</td>
<td>0</td>
</tr>
<tr>
<td>Resolute</td>
<td>520</td>
<td>610</td>
<td>690</td>
<td>770</td>
<td>1</td>
</tr>
<tr>
<td>Resolution Island</td>
<td>845</td>
<td>1065</td>
<td>1235</td>
<td>1410</td>
<td>0</td>
</tr>
</tbody>
</table>
Figure A3.1.1
Maximum mean daily temperature
(See Clause 3.9.4.1.)

Analysis based on available records up to 1970. Maximum mean daily temperature, °C.
Analysis based on available records up to 1970.
Minimum mean daily temperature, °C.

Figure A3.1.2
Minimum mean daily temperature
(See Clause 3.9.4.1.)
Figure A3.1.3
Annual mean relative humidity
(See Clauses 8.4.1.5.2, 8.4.1.6.3, and 8.7.4.3.2.)
Figure A3.1.4
Ice accretion
(See Clause 3.12.6.2.)
Figure A3.1.5
Permafrost region
(See Clause 6.1.)
Figure A3.1.6
Contours of peak horizontal ground accelerations (in units of $g$) having a probability of exceedance of 10% in 50 years
(See Clauses 3.13 and 4.4.3.)
Figure A3.1.7

Contours of peak horizontal ground velocities (in units of m/s) having a probability of exceedance of 10% in 50 years

(See Clause 3.13.)
Annex A3.2 (normative)

Wind loads on highway accessory supports and slender structural elements

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

A3.2.1 General

Highway accessory supports and slender structural elements shall be designed for horizontal drag loads at the serviceability and ultimate limit states and, where appropriate, shall be designed for across-wind loads induced by vortex shedding excitation at the FLS.

The loading combinations to be considered and the load factors to be used shall be as specified in Table A3.2.1. For each loading combination, every load that is to be included shall be multiplied by the load factor specified and the resulting load effects shall be calculated. The factored load effects shall then be added together to obtain the total factored load effect.
Table A3.2.1
Load combinations and load factors for highway accessory supports and slender structural elements
(See Clauses A3.2.1, 12.5.5.3, and 12.5.5.4.1.)

<table>
<thead>
<tr>
<th>Loads</th>
<th>Permanent loads</th>
<th>Transitory loads</th>
<th>Exceptional loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>E</td>
<td>P</td>
</tr>
<tr>
<td><strong>Fatigue limit state</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FLS Combination A1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Serviceability limit state</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLS Combination A1</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td><strong>Ultimate limit states</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ULS Combination A1</td>
<td>α_D</td>
<td>α_E</td>
<td>α_P</td>
</tr>
<tr>
<td>ULS Combination A2</td>
<td>α_D</td>
<td>α_E</td>
<td>α_P</td>
</tr>
<tr>
<td>ULS Combination A3</td>
<td>α_D</td>
<td>α_E</td>
<td>α_P</td>
</tr>
</tbody>
</table>

**Note:** For ultimate limit states, the maximum or minimum value of α_D, α_E, and α_P as specified in Clause 3.5.2 shall be used.

**Legend:**

A = ice accretion load
D = dead load
E = loads due to earth pressure and hydrostatic pressure, including surcharges but excluding dead load
EQ = earthquake load
K = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains
P = secondary prestress effects
S = load due to differential settlement and/or movement of the foundation
W = wind load on structure

### A3.2.2 Horizontal drag load

The wind-induced horizontal drag load acting on the exposed frontal area of slender structural members shall be as specified in Clause 3.10.2.2, but using the values of C_n specified in Table A3.2.2.
Table A3.2.2
Horizontal drag coefficient, $C_h$
(See Clause A3.2.2.)

<table>
<thead>
<tr>
<th>Type and shape of members</th>
<th>Horizontal drag coefficient, $C_h$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two members or trusses, one in front of the other</td>
<td></td>
</tr>
<tr>
<td>Cylindrical</td>
<td>1.9</td>
</tr>
<tr>
<td>Flat</td>
<td>2.9</td>
</tr>
<tr>
<td>Three trusses forming a triangular cross-section</td>
<td></td>
</tr>
<tr>
<td>Cylindrical</td>
<td>1.7</td>
</tr>
<tr>
<td>Flat</td>
<td>2.6</td>
</tr>
<tr>
<td>Traffic signals</td>
<td>1.2</td>
</tr>
<tr>
<td>Luminaires with rounded surfaces</td>
<td>0.5</td>
</tr>
<tr>
<td>Luminaires with rectangular flat surfaces</td>
<td>1.2</td>
</tr>
<tr>
<td>Sign panels and noise barriers</td>
<td></td>
</tr>
<tr>
<td>Ratio of sides = 1</td>
<td>1.1</td>
</tr>
<tr>
<td>1 &lt; ratio of sides ≤ 10</td>
<td>1.2</td>
</tr>
<tr>
<td>Ratio of sides &gt; 10</td>
<td>1.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Single member or truss</th>
<th>With $D(qC_e)^{0.5} \leq 3.6$</th>
<th>With $3.6 &lt; D(qC_e)^{0.5} &lt; 7.2$</th>
<th>With $D(qC_e)^{0.5} \geq 7.2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylindrical</td>
<td>1.2</td>
<td>6.42 (D^{1.1}(qC_e)^{0.65})</td>
<td>0.5</td>
</tr>
<tr>
<td>Dodecagonal*</td>
<td>1.2</td>
<td>2.60 (D^{0.6}(qC_e)^{0.3})</td>
<td>0.8</td>
</tr>
<tr>
<td>Octagonal†</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>Hexdecagonal (0 \leq r &lt; 0.26)</td>
<td>1.1</td>
<td>1.37 + 1.08r (\frac{D(qC_e)^{0.5}}{13.3} - \frac{Dr(qC_e)^{0.5}}{3.3})</td>
<td>0.83 – 1.08r</td>
</tr>
<tr>
<td>(r \geq 0.26)</td>
<td>1.1</td>
<td>0.55 + (\frac{[7.2 - D(qC_e)^{0.5}]}{6.5})</td>
<td>0.55</td>
</tr>
<tr>
<td>Flat‡</td>
<td>1.7</td>
<td>1.7</td>
<td>1.7</td>
</tr>
<tr>
<td>Elliptical</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Broad side facing wind</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Narrow side facing wind</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(C_D = \frac{4 - D_1 / D_2}{3})</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Valid for member with a ratio of corner radius to distance between parallel faces equal to or greater than 0.125.
†The corners are assumed to be slightly rounded for octagonal sections. With sharp corners, a coefficient of 1.4 shall be used.
‡Flat members are shapes that are essentially flat in elevation, including plates, angles, and squares with slightly rounded corners and panels with variable message signs. A coefficient of 2.0 shall be used for single flat members, including plates, angles, and squares, with sharp corners.

Legend:

\(C_D = \) drag coefficient of cylindrical shape with a diameter of \(D_1\)

\(C_D = \) drag coefficient of cylindrical shape with a diameter of \(D_2\)

\(D = \) width or diameter of member, m

\(D_1/D_2 = \) ratio of major to minor diameter of ellipse (maximum value of 2)

\(r = \) ratio of corner radius to radius of inscribed circle
A3.2.3 Horizontal drag load on highway accessory supports

The horizontal drag load on members of highway accessory supports shall be in accordance with Clause A3.2.2. When the effects of wind loads are to be combined with those of ice accretion loads, the increase in exposed frontal area caused by ice accretion shall be considered.

For highway accessories that are less than 1 m² in area, the exposed frontal area for all wind directions shall be taken as constant and equal to the maximum exposed frontal area in any direction.

The action of wind loads on highway accessory supports shall be as shown in Figure A3.2.1 and in accordance with the following:

(a) Horizontal support members shall be designed for wind loads $W_a$ and $W_h$ applied normal to the accessories and horizontal support members, respectively, and acting at the centroids of their respective areas.

(b) Vertical support members shall be designed for wind loads imposed from any direction by applying normal and transverse wind loads simultaneously to the member in the combinations specified in Table A3.2.3. The basic load ($BL$) specified in Table A3.2.3 shall be the wind loads $W_a$, $W_h$, and $W_v$ applied normal to the accessories and support members, respectively, and acting at the centroids of their respective areas. The transverse wind load may be assumed to be equally distributed to all vertical support members.

(c) The maximum torque on single vertical support members supporting two or more horizontal support members shall be calculated by assuming that the wind load acts on a horizontal support member only if the wind load acting on that member increases the torque. The resulting torsional effects shall be combined with other effects due to full wind load. The maximum torque on single vertical support members supporting highway accessories shall be not less than the full wind load multiplied by 0.15 times the overall width of the highway accessory.

<table>
<thead>
<tr>
<th>Normal component</th>
<th>Transverse component, $t_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1.0BL$</td>
<td>$0.2BL$</td>
</tr>
<tr>
<td>$0.6BL$</td>
<td>$0.3BL$</td>
</tr>
</tbody>
</table>
Figure A3.2.1
Loads on sign, luminaire, and traffic signal support structures
(See Clause A3.2.3.)

(Continued)
Note: Resultant wind forces are applied at the centroid of each component.

Figure A3.2.1 (Concluded)

A3.2.4 Across-wind loads

A3.2.4.1 General
The dynamic effects of across-wind loads induced by vortex shedding excitation on slender structural members shall be considered at the FLS.

The stress range shall be taken as twice the maximum stress calculated in accordance with Clause A3.2.4.3.2. The stress range limit shall be taken as that corresponding to a fatigue life of over 2 000 000 cycles for the appropriate material and detail unless a detailed fatigue damage analysis shows that a different limit is appropriate.
A3.2.4.2 Vortex shedding excitation

The significance of vortex shedding excitation for a slender structural member shall be examined in accordance with Clause A3.2.4.3 as follows:

(a) For members with a constant diameter or frontal width:

\[ n_i < n_e = \frac{SV}{D} \]

where

- \( n_i \) = natural frequency of member for mode of vibration, \( i \), Hz
- \( n_e \) = frequency at which vortex shedding occurs for a member with a constant diameter or frontal width, Hz
- \( S \) = Strouhal number for the cross-sectional geometry, as specified in Table A3.2.4
- \( V \) = hourly mean wind speed at the location of the member being considered, m/sec
  \[ = 1.24 \sqrt{qC_e} \]

where

- \( q \) = hourly mean reference wind pressure for the design return period, Pa
- \( C_e \) = wind exposure coefficient specified in Clause 3.10.1.4
- \( D \) = constant diameter or frontal width of member, m

The height above ground used to calculate \( C_e \) shall correspond to the height above ground of the location of coordinate \( x \). The location at which \( n_e \) is calculated shall be taken as the top of the member.

(b) For members with a tapered diameter or frontal width:

\[ n_i < n_e(x) = \frac{SV}{D(x)} \]

where

- \( n_i \) = natural frequency of member for mode of vibration, \( i \), Hz
- \( n_e(x) \) = frequency at which vortex shedding excitation occurs at location \( x \) for a member with a tapered diameter or frontal width, Hz
- \( S \) = Strouhal number for the cross-sectional geometry, as specified in Table A3.2.4
- \( V \) = hourly mean wind speed at the location of the member being considered, m/sec
  \[ = 1.24 \sqrt{qC_e} \]

where

- \( q \) = hourly mean reference wind pressure for the design return period, Pa
- \( C_e \) = wind exposure coefficient specified in Clause 3.10.1.4
- \( D(x) \) = diameter or frontal width of a tapered member at location \( x \), m

where

- \( x \) = coordinate describing location along the member

\( n_e(x) \) shall be calculated at sufficient locations along the member to determine at which locations vortex shedding excitation can occur.
### Table A3.2.4
Vortex shedding data
(See Clause A3.2.4.2.)

<table>
<thead>
<tr>
<th></th>
<th>$S$</th>
<th>$C_L$</th>
<th>$B$</th>
<th>$L$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular cross-section</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subcritical $R_e &lt; 3 \times 10^5$</td>
<td>0.18</td>
<td>0.50</td>
<td>0.10</td>
<td>2.5</td>
</tr>
<tr>
<td>Supercritical and transcritical $R_e \geq 3 \times 10^5$</td>
<td>0.25</td>
<td>0.20</td>
<td>0.30</td>
<td>1.0</td>
</tr>
<tr>
<td>Square cross-section</td>
<td>0.11</td>
<td>0.60</td>
<td>0.25</td>
<td>3.0</td>
</tr>
<tr>
<td>Multi-sided members and rolled structural shapes</td>
<td>0.15</td>
<td>0.60</td>
<td>0.25</td>
<td>2.75</td>
</tr>
</tbody>
</table>

**Note:** The Reynolds number, $R_e$, shall be calculated as $(VD/1.5) \times 10^5$.

**Legend:**
- $B$ = band width, i.e., a measure of the variability of the vortex shedding frequency
- $C_L$ = root-mean-square (RMS) lift coefficient for the cross-sectional geometry
- $L$ = correlation length, i.e., the length (as a ratio of the diameter) over which the vortices act in phase
- $S$ = Strouhal number

### A3.2.4.3 Structural response to vortex shedding excitation

#### A3.2.4.3.1 Displacements

The magnitude of the peak member displacement, $y_i(x)$, due to vortex shedding excitation at any location, $x$, along the member for mode of vibration, $i$, shall be taken as follows:

(a) For a member with a constant diameter or frontal width:

$$y_i(x) = a_i \mu_i(x)$$

where

$$a_i = \text{modal coefficient of magnitude of the oscillatory displacement for mode of vibration, } i,$$  

for a member with a constant diameter or frontal width, m

$$= \frac{3.5 \tilde{C}_i \rho D^4 \alpha^{0.25} C}{\sqrt{B \xi_i (4\pi S)^2 \gamma M_i}} \quad \text{if } y_i(x) \leq 0.025D$$

$$= \frac{\sqrt{2} (p) \tilde{C}_i D^3 \left[ \int_0^H |\mu_i(x)| \, dx \right]}{(4\pi S)^2 \xi_i G M_i} \quad \text{if } y_i(x) > 0.025D$$

where

$$C = \frac{(H/D)^2}{1 + H/2LD} \int_0^H \frac{x^{3\alpha} \mu_i^2(x)}{H^{1 + 3\alpha}} \, dx$$

where

$\alpha$ = wind velocity profile exponent, taken as 0.36 for city centres and industrial areas, 0.25 for suburban and well-wooded areas, and 0.15 for open country with scattered trees

$\xi_i$ = structural damping for the $i$th mode, expressed as a ratio of critical damping
GMᵢ = generalized mass for mode of vibration, \( i \), kg

\[
GMᵢ = \int \frac{m(x)\muᵢ²(x)}{H} \, dx
\]

where
\( m(x) = \) mass per unit length of member at location \( x \), kg/m

\( H = \) length of member, m

\( \muᵢ(x) = \) amplitude of the member mode shape at location \( x \) for mode of vibration, \( i \)

\( \muᵢ(x) = \) air mass density, taken as 1.29 kg/m³

(b) For a member with a tapered diameter or frontal width:

\[ yᵢ(x) = aᵢ(x₁)μᵢ(x) \]

where

\( aᵢ(x₁) = \) modal coefficient of magnitude of the oscillatory displacement due to vortex shedding excitation at location \( x₁ \) for mode of vibration, \( i \), for a member with a tapered diameter or frontal width

\[
aᵢ(x₁) = \frac{2\pi L}{\sqrt{\zeta_i \psi(x₁)}} \left[ \int \frac{\muᵢ(x)}{(4\pi S)² \cdot GMᵢ} \right]^{x₁+b} \]

\[
yᵢ(x) = \frac{\sqrt{2\pi L} \cdot \hat{C} \cdot \bar{D}²(x₁) \cdot D(x) \cdot \muᵢ(x)}{(4\pi S)² \cdot \zeta_i \cdot GMᵢ} \quad \text{if} \quad yᵢ(x) > 0.025 D(x) \text{ at any location } x
\]

where

\( x₁ = \) location along a tapered member at which vortex shedding excitation is being considered

\( \psi(x₁) = \frac{dD(x₁)}{dx} + \alpha D(x₁) \quad \text{for } i \)

\( b = \) length of the member above or below location \( x₁ \) for which \( D(x) \) is within a certain percentage of \( D(x₁) \) (the percentage shall be taken as 10% unless a smaller value can be justified)

For a tapered member, \( aᵢ(x₁) \) shall be calculated for all locations, \( x₁ \), along the member at which vortex shedding excitation can occur for mode of vibration, \( i \), as determined in accordance with Clause A3.2.4.2. The largest value of \( aᵢ(x₁) \) calculated shall be used for determining \( yᵢ(x) \) and the peak inertia loads specified in Clause A3.2.4.3.2.

A3.2.4.3.2 Stresses

The maximum stresses in a member due to vortex shedding excitation shall be calculated by loading the member with the peak inertia loads acting statically. The magnitude of the peak inertia load per unit length at any location \( x \) along the member for mode of vibration, \( i \), shall be taken as

\[ Fᵢ(x) = (2\pi nᵢ)^2 yᵢ(x) m(x) \]

where

\( Fᵢ(x) = \) peak inertia load at location \( x \) for mode of vibration, \( i \), N/m

The calculation of the peak inertia loads shall take into account the mass of all components attached to the member.
A3.2.4.3.3 Damping ratios

Unless experimentally determined values are available, the value of $\zeta_i$ for members in all modes of vibration shall be taken as 0.0075 for steel and aluminum members and 0.015 for concrete members.
Annex A3.3 (normative)

Vessel collision

Notes:
(1) This Annex is a mandatory part of this Code.
(2) Taken together, this Annex and Clause 3.14 constitute a condensed version of AASHTO GVCB-1 (but written, unlike AASHTO GVCB-1, in SI units). AASHTO GVCB-1 may be used as an alternative to Clause 3.14 and this Annex for detailed design for vessel collision. It may also be used as a reference when fuller information than is provided by Clause 3.14 and this Annex is needed.

A3.3.1 Vessel frequency

A vessel frequency distribution shall be determined for the bridge site. The number of vessels, \( N \), passing under the bridge, based on size, type, loading condition, and navigable water depth, shall be developed for each pier and span element to be evaluated.

The vessel frequency distribution for vessels should be developed and modelled using dead weight tonnage (DWT) classification intervals appropriate for the waterway vessel traffic. See Annex CA3.3 of CSA S6.1.

A3.3.2 Design vessel selection

A3.3.2.1 General

Design vessel selection shall be based on Method I or Method II (Clauses A3.3.2.2 and A3.3.2.3, respectively). Once the design vessel is identified, the ship collision force can be evaluated using Clause A3.3.5.

A3.3.2.2 Method I

Note: See Clause 3.14.2 for bridge classifications.

The following requirements shall apply:
(a) Class I bridges: the design vessel size shall be such that the annual number of passages of vessels larger than the design vessel amounts to a maximum of 5% of the total annual number of merchant vessels that could impact the bridge element, but not more than 50.

(b) Class II bridges: the design vessel size shall be such that the annual number of passages of vessels larger than the design vessel amounts to a maximum of 10% of the total annual number of merchant vessels that could impact the bridge element, but not more than 200.

A3.3.2.3 Method II

Note: See Clause 3.14.2 for bridge classifications.

The following requirements shall apply:
(a) Class I bridges: the maximum annual frequency of collapse, \( AF_{\text{max}} \), for the whole bridge shall be 0.0001, i.e., a probability of 1 in 10 000.

(b) Class II bridges: the maximum annual frequency of collapse, \( AF_{\text{max}} \), for the whole bridge shall be 0.001, i.e., a probability of 1 in 1000.

The maximum annual frequency of bridge collapse for the total bridge, \( AF_{\text{max}} \), as determined in accordance with Item (a) or (b), shall be distributed over the number of pier and span elements located within the waterway, or within the distance of three times the overall length of the design vessel, on each side of the inbound and outbound vessel transit paths if the waterway is wide. This results in an acceptable risk criterion for each pier and span element of the total bridge.
The design vessel for each pier or span element shall be chosen so that the annual frequency of collapse due to vessels that are equal in size to or larger than the design vessel is less than the acceptable risk criterion specified in Item (a) or (b), as applicable.

A3.3.3 Annual frequency of collapse

A3.3.3.1 General
The annual frequency of bridge component collapse due to vessel collision, \( AF \), shall be taken as

\[
AF = (N)(PA)(PG)(PC)
\]

where
- \( N \) = the annual number of vessels, classified by type, size, and loading condition, that use the channel and can hit the bridge component
- \( PA \) = the probability of vessel aberrancy
- \( PG \) = the geometric probability of a collision between an aberrant vessel and a bridge pier or span
- \( PC \) = the probability of bridge collapse due to a collision with an aberrant vessel

\( AF \) shall be calculated for each bridge element and vessel classification. The summation of all element \( AFs \) equals the annual frequency of collapse for the entire bridge structure.

A3.3.3.2 Probability of aberrancy
The probability of vessel aberrancy, \( PA \) (the probability that a vessel will stray off course and threaten a bridge) may be determined either by statistical analysis of historical data on vessels transiting the waterway or by the following approximate method:

\[
PA = (BR)(RB)(RC)(RXC)(RD)
\]

where
- \( BR \) = aberrancy base rate (usually taken as \( 0.6 \times 10^{-4} \) for ships)
- \( RB \) = correction factor for bridge location
- \( RC \) = correction factor for current acting parallel to vessel transit path
- \( RXC \) = correction factor for cross-currents acting perpendicular to vessel transit path
- \( RD \) = correction factor for vessel traffic density

A3.3.3.3 Correction factors

A3.3.3.3.1 Factor for bridge location
Based on the relative location of the bridge in one of three waterway regions, as shown in Figure A3.3.1, the correction factor, \( RB \), shall be as follows:

- (a) for straight regions: 1.0
- (b) for transition regions:
  \[
  1.0 + \frac{\theta}{90^\circ}
  \]
  \( \theta \) is the angle from a straight path to an intersection or other bend
- (c) for turn/bend regions:
  \[
  1.0 + \frac{\theta}{45^\circ}
  \]
where
\[ \theta = \text{angle of the turn of bend specified in Figure A3.3.1, degrees} \]

(a) Turn in channel

(b) Bend in channel

Figure A3.3.1
Waterway regions for bridge location
(See Clause A3.3.3.1.)
A3.3.3.3.2 Factor for parallel currents
For currents acting parallel to the vessel transit path in the waterway, the correction factor shall be

\[ R_C = 1.0 + \frac{V_C}{5} \]

where

\[ V_C \] = current velocity component parallel to vessel transit path, m/sec

A3.3.3.3.3 Factor for cross-currents
For currents acting perpendicular to the vessel transit path in the waterway, the correction factor shall be

\[ R_{XC} = 1.0 \frac{V_{XC}}{2} \]

where

\[ V_{XC} \] = current velocity component perpendicular to vessel transit path, m/sec

A3.3.3.3.4 Factor for vessel traffic density
The correction factor, \( R_D \), selected on the basis of the ship traffic density in the waterway in the immediate vicinity of the bridge, shall be

(a) 1.0 for low-density traffic, in which vessels rarely meet, pass, or overtake each other in the immediate vicinity of the bridge;

(b) 1.3 for average-density traffic, in which vessels occasionally meet, pass, or overtake each other in the immediate vicinity of the bridge;

(c) 1.6 for high-density traffic, in which vessels routinely meet, pass, or overtake each other in the immediate vicinity of the bridge.

A3.3.3.3.5 Geometric probability
The geometric probability, \( PG \), being the conditional probability that an aberrant vessel in the zone will hit a pier, may be determined by a normal distribution to model the aberrant vessel sailing path near the bridge or by the following approximate method:

\[ PG = 1 - \frac{L_C - B}{Z_W} \]

where

\[ L_C \] = width of horizontal clearance from the pier(s), m

\[ B \] = width of ship, m

\[ Z_W \] = width of zone, m

For the same zone and pier widths, \( PG \) shall be calculated for each ship category. A similar equation based on vertical clearance may be used to calculate \( PG \) values for an impact between bridge spans and vessel superstructures.

A3.3.3.3.6 Probability of collapse
The probability of bridge collapse, \( PC \), once a bridge component has been hit by an aberrant vessel, being based on the ratio of the ultimate lateral resistance of the pier, \( H_p \), and span, \( H_s \), to the vessel impact force, \( P \), as shown in Figure A3.3.2, shall be calculated as follows:

\[ PC = 0.1 + 9(0.1 - H/P) \text{ if } 0.0 \leq H/P < 0.1 \]
\[ PC = \frac{(1 - H/P)}{9} \] if \( 0.1 \leq H/P \leq 1.0 \)

\[ PC = 0.0 \] if \( H/P > 1.0 \)

where

- \( H \) = ultimate bridge element strength, \( H_p \) or \( H_s \), MN
- \( P \) = vessel impact forces, \( P_s \), \( P_{BH} \), \( P_{DH} \), or \( P_{MT} \), MN, as specified in Clauses A3.3.5 and A3.3.7

**Figure A3.3.2**

Probability of collapse distribution

(See Clause A3.3.3.3.6.)

### A3.3.4 Design collision velocity

#### A3.3.4.1 Transit velocity in channel

The vessel transit velocity, \( V_T \), shall represent the velocity at which the design vessel is transiting the channel or waterway under normal environmental conditions.

#### A3.3.4.2 Minimum collision velocity

The minimum collision velocity, \( V_{min} \), shall not be less than the yearly mean current velocity at the bridge location. In waterways subject to seasonal flooding, flood flow velocity shall be considered in determining the collision velocity.

#### A3.3.4.3 Distribution

When an aberrant vessel wanders away from the navigation channel, its velocity shall be considered to reduce linearly to the minimum velocity over a distance equal to three times the overall length of the design vessel, as shown in Figure A3.3.3.
A3.3.5 Ship collision force on pier
The ship collision impact force shall be taken as

\[ P_S = (DWT)^{0.5} \left( \frac{V}{8.4} \right) \]

where

- \( P_S \) = equivalent static vessel collision force, MN
- \( DWT \) = dead weight tonnage of vessel, t
- \( V \) = design collision velocity, m/sec

A3.3.6 Vessel collision energy
The kinetic energy of a moving vessel to be absorbed during a non-eccentric collision with a bridge pier shall be taken as

\[ KE = \frac{(C_H)(W)(V)^2}{2 \times 10^3} \]

where

- \( KE \) = vessel collision energy, MN·m
- \( C_H \) = hydrodynamic mass coefficient
The vessel displacement tonnage, \( W \), shall be based on the loading condition of the vessel and shall include the empty weight of the vessel, plus consideration of the weight of cargo, \( DWT \), for loaded vessels, or the weight of water ballast for vessels transiting in an empty or light condition.

The following values of \( C_H \) shall be used:

- (a) for large under-keel clearances (\( \geq 0.5 \times \text{draft} \)): 1.05; and
- (b) for small under-keel clearances (\( \leq 0.1 \times \text{draft} \)): 1.25.

### A3.3.7 Ship collision force on superstructure

#### A3.3.7.1 Collision with bow

The bow collision force on a superstructure shall be taken as

\[
P_{BH} = (R_{BH})(P_S)
\]

where

- \( P_{BH} \) = ship bow collision force on an exposed superstructure, MN
- \( R_{BH} \) = ratio of exposed superstructure depth to the total bow depth
- \( P_S \) = ship collision force as specified in Clause A3.3.5, MN

For the purposes of this Clause, exposure shall be the vertical overlap between the vessel and the bridge superstructure within the depth of the collision zone.

#### A3.3.7.2 Collision with deck house

The deck house collision force on a superstructure shall be taken as

\[
P_{DH} = (R_{DH})(P_S)
\]

where

- \( P_{DH} \) = ship deck house collision force, MN
- \( R_{DH} \) = reduction factor of 0.10 for ships exceeding 100 000 \( DWT \)

\[
= \text{reduction factor of } 0.20 - \left( \frac{DWT}{100 000} \right) (0.10) \quad \text{for ships of } 100 000 \text{ } DWT \text{ or less}
\]

- \( P_S \) = ship collision force as specified in Clause A3.3.5, MN

#### A3.3.7.3 Collision with mast

The mast collision force on a superstructure shall be taken as

\[
P_{MT} = 0.10P_{DH}
\]

where

- \( P_{MT} \) = ship mast collision force, MN
- \( P_{DH} \) = ship deck house collision force as specified in Clause A3.3.7.2, MN
A3.3.8 Application of impact forces

A3.3.8.1 Pier design

A3.3.8.1.1 Design cases
Forces, both parallel and normal to the centreline of the navigable channel, shall be investigated.

For pier design, the impact force shall be applied as an equivalent static force. Two design cases shall be considered:
(a) 100% of the ship collision force, $P_s$, applied in a direction parallel to the alignment of the centreline of the navigable channel; and
(b) 50% of the ship collision force, $P_s$, applied normal to the direction of the centreline of the channel.

These impact forces shall not be taken to act simultaneously.

All portions of the bridge pier exposed to physical contact by any portion of the design vessel’s hull or bow shall be proportioned to resist the applied loads. The bow overhang, rake, or flair distance shall be considered in determining the portions of the pier exposed to contact by the vessel. Crushing of the vessel’s bow causing contact with any setback portion of the pier shall also be considered.

A3.3.8.1.2 Distribution of impact force
The impact force from both design cases shall be applied to the pier in accordance with the following requirements:
(a) To design the pier for overall stability, the design impact force shall be applied as a concentrated force on the pier at the mean high water level of the waterway, as shown in Figure A3.3.4.
(b) To design the pier for local collision forces, the design impact force shall be applied as a vertical line load equally distributed along the ship’s bow depth, as shown in Figure A3.3.5. The ship’s bow shall be considered to be raked forward when the potential contact area of the impact force on the pier is being determined.

A3.3.8.2 Superstructure design
For superstructure design, the design impact force shall be applied as an equivalent static force perpendicular to the superstructure member.

Figure A3.3.4
Ship impact concentrated force on pier
(See Clause A3.3.8.1.)
Figure A3.3.5
Ship impact line load for local collision force on pier
(See Clause A3.3.8.1.)
Annex A3.4 (normative)
CL-625-ONT live loading

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

A3.4.1 General
In Ontario, the CL-625-ONT Truck shown in Figure A3.4.1 and the CL-625-ONT Lane Load shown in Figure A3.4.2 shall be used instead of the CL-625 Truck and CL-W Lane Load, respectively.

<table>
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<tr>
<th>Axle no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
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<td>140</td>
<td>140</td>
<td>175</td>
<td>120</td>
</tr>
</tbody>
</table>

![Figure A3.4.1](Figure A3.4.1 CL-625-ONT Truck)

(See Clause A3.4.1.)

![Uniformly distributed load 9 kN/m](Uniformly distributed load 9 kN/m)

![Figure A3.4.2](Figure A3.4.2 CL-625-ONT Lane load)

(See Clause A3.4.1.)
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Section 4
Seismic design

4.1 Scope
This Section specifies minimum requirements for
(a) the seismic analysis and design of new bridge structures; and
(b) the seismic evaluation and rehabilitation of existing bridge structures.

4.2 Definitions
The following definitions apply in this Section:

Capacity-protected element — a substructure or superstructure element that has a force demand limited by the capacity of the ductile substructure element.

Concentrically braced frame with nominal ductility — a braced frame with concentric bracing designed and detailed to absorb limited amounts of energy through inelastic bending or extension of bracing members.

Connectors — mechanical devices, including bearing components and shear keys, that provide transverse or longitudinal restraint of movement of the superstructure relative to the substructure. Connectors do not include moment connections, monolithic joints, or longitudinal restrainers at expansion bearings (see Clause 4.4.10.4.2).

Damping — the dissipation of energy of a structure oscillating in one of its natural modes of vibration. It is normally expressed as a ratio of the actual value of damping to the critical value of damping. The critical value of damping is the minimum damping at which an initial motion decays without oscillation.

Design displacement — the minimum lateral seismic displacement at the centre of rigidity required for design of the isolation system (see Clause 4.10.7).

Ductile concentrically braced frame — a braced frame with concentric bracing designed and detailed to absorb energy through yielding of the braces.

Ductile substructure element — an element of a substructure that is expected to undergo reversed-cyclic inelastic deformations without significant loss of strength and is detailed to develop the appropriate level of ductility while remaining stable.

Ductility — the ability of a structural member to deform without significant loss of load-carrying capacity after yielding.

Effective damping — the value of equivalent viscous damping corresponding to the energy dissipated during cyclic response at the design displacement of the isolation system.

Effective stiffness — the value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

Effective weight — the total unfactored dead load of a superstructure and the portion of substructure elements that contribute to the inertial mass.
Elastic restraint system — the collection of structural elements that provides restraint of a seismically isolated structure for non-seismic lateral loads. The elastic restraint system may be either an integral part of isolator units or a separate device.

Emergency-route bridge — a bridge as specified in Clause 4.4.2.

Factored load — the product of a load specified in this Code and the corresponding load factor.

Factored load effect — the load effect caused by a factored load.

Factored resistance — the resistance of a member calculated using the appropriate material resistance factors.

Flexural frequency — a natural frequency of vibration of an unloaded bridge based on the flexural stiffness and mass distribution of the superstructure.

Hoop — a closed tie or continuously wound tie with seismic hooks at each end.

Isolation system — the collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the elastic restraint system if such a system is used to meet the design requirements of Clause 4.10.

Isolator unit — a device used for seismic base isolation (see Clause 4.10).

Lateral restoring force — a lateral force that tends to restore the isolator unit to its original position.

Lifeline bridge — a bridge as specified in Clause 4.4.2.

Natural frequency — the frequency of vibration of one of the natural modes of a bridge, expressed in cycles per second. The natural frequency is the inverse of the natural period.

Natural period — the duration of one complete cycle of free vibration of one of the normal modes of vibration.

Nominal resistance — the resistance of a member, connection, or structure based on the specified material properties and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Normal mode shape — the geometric configuration of a structure vibrating at one of the associated natural frequencies.

Panel zone — the area of beam-to-column connection delineated by beam and column flanges.

Probable resistance — the resistance of a member, calculated by taking into account the expected development of large strains and associated stresses larger than the minimum specified yield values taken as the nominal resistance times a factor greater than 1.0 (see Clause 4.4.10.4.3)

Regular bridge — a bridge as specified in Clause 4.4.5.3.2.

Response modification factor — a factor specified in Clauses 4.4.8.1, 4.11.9, and 4.12.2.

Response spectrum — the envelope of maximum response of a single degree-of-freedom oscillator subjected to a particular disturbance, plotted as a function of the natural period or frequency of the oscillator.

Restrainer — a tie, cable, or other device designed for limiting displacements at expansion bearings.
Return period — the average time in years between the equalling or exceeding of an event. The inverse of the return period is approximately the probability of equalling or exceeding the event in one year.

Seismic cross-tie — a single bar with a seismic hook at one end and, at the other end, a hook with a bend of at least 90° and at least a 12-bar diameter extension.

Seismic hook — a hook with a bend of at least 135° and an extension of not less than the larger of ten bar diameters or 150 mm.

Static pushover analysis — a static analysis involving a step-by-step force-deformation analysis procedure accounting for inelastic effects.

Time-history analysis — a dynamic analysis obtained by determining the response using a step-by-step integration of an acceleration-time seismic ground response.

Total design displacement — the maximum lateral seismic displacement of an isolator for the testing requirements of an isolation system (see Clause 4.10.7).

4.3 Abbreviation and symbols

4.3.1 Abbreviation
The following abbreviation applies in this Section:
PHA — Peak horizontal ground acceleration, \( g \)

4.3.2 Symbols
The following symbols apply in this Section:
\[ A = \text{zonal acceleration ratio (dimensionless)} \]
\[ A_b = \text{bonded area of rubber} \]
\[ A_c = \text{area of core of a spirally reinforced compression member measured out-to-out of spirals, mm}^2 \]
\[ A_g = \text{gross cross-sectional area, mm}^2 \]
\[ A_r = \text{reduced net bonded area of rubber, } A_b (1 - \Delta/B) \]
\[ A_{sh} = \text{total cross-sectional area of tie reinforcement, including supplementary cross-ties with a vertical spacing of } s \text{ and crossing a section with a core dimension of } h_c, \text{ mm}^2 \]
\[ b = \text{width of the compression face of the member, mm} \]
\[ b_{bf} = \text{width of the beam flange} \]
\[ C = \text{member reserve capacity calculated in accordance with Clause 4.11.9} \]
\[ C_f = \text{factored compressive force in column} \]
\[ C_r = \text{factored compressive resistance of column (see Clause 10.9.3)} \]
\[ C_{sm} = \text{elastic seismic response coefficient for the } m \text{th mode of vibration (dimensionless)} \]
\[ D = \text{dead load, as defined in Clause 3.2} \]
\[ d = \text{effective depth, being the distance from the extreme compression fibre to the centroid of the tensile force, mm; depth of column} \]
\[ d_b = \text{nominal bar diameter} \]
\[ d_i = \text{design displacement at the centre of rigidity of the isolation system in the direction under consideration; lateral displacement under earthquake loads} \]
\( E \) = modulus of elasticity of elastomer

\( EQ \) = earthquake load, expressed as a modified design force (see Clauses 4.4.10.3.2 and 4.4.10.4.2)

\( F \) = statically equivalent seismic force

\( F_d \) = design force for connections for bridges in Seismic Performance Zone 1

\( F_n \) = maximum negative force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of \( \Delta_n \)

\( F_{n,\text{min}} \) = minimum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of \( \Delta_n \)

\( F_p \) = maximum positive force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of \( \Delta_p \)

\( F_{p,\text{min}} \) = minimum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of \( \Delta_p \)

\( f_y \) = specified minimum yield stress, MPa

\( f_{c'} \) = specified compressive strength of concrete, MPa

\( f_{cr} \) = cracking strength of concrete, MPa

\( f_{y} \) = minimum specified yield strength of reinforcing bars, MPa

\( g \) = acceleration due to gravity

\( H \) = for abutments, the average height of the columns supporting the bridge deck to the next expansion joint, mm; for columns or piers, the column or pier height, mm; for hinges within a span, the average height of the two adjacent columns or piers, mm

Note: The value of \( H \) for single-span bridges is 0.0.

\( h_c \) = core dimension of a tied column in the direction under consideration, mm

\( I \) = importance factor (dimensionless)

\( k \) = bridge lateral stiffness; modification factor specified in Table 4.6

\( k = \) material constant

\( k_{\text{eff}} \) = effective stiffness of an isolator unit, determined by prototype testing

\( k_{\max} \) = maximum effective stiffness of an isolation system at the design displacement in the horizontal direction under consideration

\( k_{\text{min}} \) = minimum effective stiffness of an isolation system at the design displacement in the horizontal direction under consideration

\( L \) = length of the bridge deck to the adjacent expansion joint or to the end of the bridge deck, mm

Note: For hinges within a span, \( L \) is the sum of the distances to either side of the hinge. For single-span bridges, \( L \) is the length of the bridge deck (see Figure 4.1).

\( L/r \) = slenderness ratio of brace

\( M_{pr} \) = probable flexural resistance of column

\( M_{px} \) = plastic moment resistance in strong direction of bending

\( m \) = mass per unit length of a structure, kg/m

\( N \) = minimum support length measured normal to the face of the abutment or pier, mm

\( n \) = natural frequency of vibration, Hz

\( P \) = maximum vertical load resulting from the combination of dead load plus live load (including seismic live load, if applicable) using a \( \gamma \) factor of 1.0

\( P_e \) = equivalent uniformly distributed static seismic loading in uniform-load method (see Clause 4.5.3.1)

\( p_e(x) \) = equivalent static earthquake loading applied to represent the primary mode of vibration, kN/m

\( P_f \) = factored axial load at a section at the ultimate limit state, N

\( p_o \) = arbitrary uniform lateral load
\[ R = \text{response modification factor (dimensionless)} \]
\[ R_{\text{req}} = \text{required response modification factor (see Clause 4.11.9)} \]
\[ R_{\text{prov}} = \text{provided response modification factor (see Clause 4.11.10)} \]
\[ r_y = \text{radius of gyration of a member about its weak axis} \]
\[ S = \text{dimensionless site coefficient (see Clause 4.4.6); shape factor of an elastomeric bearing (see Clauses 4.10.12.2 and 11.2)} \]
\[ S_e = \text{seismic force effect (see Clause 4.11.9)} \]
\[ S_i = \text{dimensionless site coefficient for isolation design for the given soil profile, as specified in Clause 4.10.4} \]

\[ s = \text{vertical spacing of transverse reinforcement} \]
\[ T = \text{natural period of a structure, } s \]
\[ T_e = \text{period of seismically isolated structure, } s, \text{ in the direction under consideration} \]
\[ T_m = \text{period of vibration of the } m \text{th mode, } s \]
\[ t = \text{thickness of flange} \]
\[ V_c = \text{factored shear resistance provided by tensile stresses in the concrete, } N \]
\[ V_f = \text{factored shear force at a section, } N \]
\[ V_r = \text{factored shear resistance; factored shear resistance of column web} \]
\[ V_{s,max} = \text{maximum value of } V_s(x) \]
\[ V_e(x) = \text{deformation corresponding to } p_o \]
\[ W = \text{effective weight of a bridge} \]
\[ W(x) = \text{dead load of the bridge superstructure and tributary substructure, expressed in weight per unit length of the bridge} \]
\[ w = \text{web thickness} \]
\[ \alpha = \text{generalized participation factor used in the single-mode spectral method} \]
\[ \beta = \text{equivalent viscous damping ratio for the isolation system; generalized participation factor used in the single-mode spectral method, } kN-m \]
\[ \gamma = \text{generalized mass factor used in the single-mode spectral method, } kN-m^2 \]
\[ \Delta = \text{shear deflection in the bearing} \]
\[ \Delta_c = \text{instantaneous compressive deflection} \]
\[ \Delta_n = \text{maximum negative displacement of an isolator unit during each cycle of prototype testing} \]
\[ \Delta_p = \text{maximum positive displacement of an isolator unit during each cycle of prototype testing} \]
\[ \Delta_s = \text{imposed lateral displacement} \]
\[ \varepsilon_c = \text{compression strain in bearing due to vertical loads} \]
\[ \varepsilon_{eq} = \text{shear strain due to } d_i \]
\[ \varepsilon_{sc} = \text{shear strain due to vertical loads} \]
\[ \varepsilon_{sh} = \text{shear strain due to maximum horizontal displacement resulting from creep, post-tensioning, shrinkage, and thermal effects calculated between the installation temperature and the least favourable extreme temperature} \]
\[ \varepsilon_{sr} = \text{shear strain due to imposed rotation} \]
\[ \varepsilon_u = \text{minimum elongation-at-break of rubber} \]
\[ \theta = \text{rotation imposed on bearing} \]
\[ \lambda = \text{slenderness parameter (see Clause 10.9.3)} \]
\[ \rho_h = \text{ratio of area of horizontal shear reinforcement to gross concrete area of a vertical section} \]

Note: For design of the isolation system, \( W \) is the total seismic dead load weight of the structure above the isolation interface.
\[ \rho_s = \text{ratio of volume of spiral reinforcement to total volume of core, out-to-out of spirals of spirally reinforced compression members} \]

\[ \rho_v = \text{ratio of area of vertical shear reinforcement to gross concrete area of a horizontal section} \]

\[ \Sigma k_{\text{eff}} = \text{sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure segment, calculated at displacement } d_i \]

\[ \phi_c = \text{resistance factor for concrete, as specified in Clause 8.4.6} \]

\[ \phi_s = \text{resistance factor for structural steel (see Clause 10.5.7) or reinforcing steel (see Clause 8.4.6)} \]

\[ \psi = \text{skew of support measured from a line normal to the span direction, degrees} \]

### 4.4 Earthquake effects

#### 4.4.1 General

Force effects arising from horizontal earthquake motions shall be determined in accordance with Clause 4.4.5 on the basis of the elastic seismic response coefficient, \( C_{sm} \), specified in Clause 4.4.7 and the effective weight of the bridge. Seismic design force effects for ductile substructure elements shall be adjusted by the response modification factors specified in Clause 4.4.8.1 and designed and detailed in accordance with Clauses 4.7 and 4.8. Displacements shall be determined using a response modification factor of 1.0 and an importance factor of 1.0.

Force effects arising from vertical earthquake motions shall be considered accounted for by using the load factors on dead loads specified in Table 3.2.

Earthquake load effects for capacity-protected members shall be determined from elastic design forces or in accordance with capacity design principles for forces resulting from inelastic action of members with which they connect.

Special bridges such as arches, cable-supported structures, and large trusses require special studies and shall be designed using seismic-resistant design principles providing a minimum level of safety comparable to that intended in this Code.

Soil liquefaction, liquefaction-induced ground movements, slope instability, increases in lateral earth pressure, soil-structure interaction, and approach fill settlements shall be considered in accordance with Clause 4.6.

#### 4.4.2 Importance categories

For the purpose of Clause 4.4, the Regulatory Authority shall place bridges into one of the following three importance categories:

(a) lifeline bridges;

(b) emergency-route bridges; and

(c) other bridges.

The basis of classification shall include social/survival and security/defence requirements. In classifying a bridge, consideration shall be given to possible future changes in conditions and requirements.

Lifeline bridges are generally those that carry or cross over routes that need to remain open to all traffic after the design earthquake, which is an event with a 10% probability of exceedance in 50 years (equivalent to a 15% probability of exceedance in 75 years and a return period of 475 years). Lifeline bridges also need to be usable by emergency vehicles and for security/defence purposes immediately after a large earthquake, e.g., a 1000-year return period event (7.5% probability of exceedance in 75 years).

Emergency-route bridges are generally those that carry or cross over routes that should, at a minimum, be open to emergency vehicles and for security/defence purposes immediately after the design earthquake.
4.4.3 Zonal acceleration ratio
The zonal acceleration ratio, $A$, to be used in the application of Clause 4.4 shall be determined in accordance with one of the following Tables, except that a qualified specialist shall be consulted to determine the zonal acceleration ratio for sites located close to active faults or with PHA values greater than 0.40:
(a) Table A3.1.1 (for locations not listed in Table A3.1.1, the highest value for adjacent locations listed in the Table shall be used unless site-specific data is obtained); or
(b) Table 4.1, using the PHA specified in Figure A3.1.6 or as provided by the Geological Survey of Canada using the seismic hazard methodology used to generate Figure A3.1.6.

4.4.4 Seismic performance zones
Bridges shall be assigned to one of the four seismic performance zones specified in Table 4.1 using the zonal acceleration ratio, $A$, obtained from Clause 4.4.3.

Table 4.1
Seismic performance zones
(See Clauses 4.4.3, 4.4.4, 4.6.6, 4.10.2, 4.10.3, and 4.10.6.2.1.)

<table>
<thead>
<tr>
<th>Range of PHA, $g$, for 10% probability of exceedance in 50 years</th>
<th>Zonal acceleration ratio, $A$</th>
<th>Lifeline bridges (see Clause 4.4.2(a))</th>
<th>Emergency-route and other bridges (see Items (b) and (c) of Clause 4.4.2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 ≤ PHA &lt; 0.04</td>
<td>0</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>0.04 ≤ PHA &lt; 0.08</td>
<td>0.05</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>0.08 ≤ PHA &lt; 0.11</td>
<td>0.1</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>0.11 ≤ PHA &lt; 0.16</td>
<td>0.15</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>0.16 ≤ PHA &lt; 0.23</td>
<td>0.2</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>0.23 ≤ PHA &lt; 0.32</td>
<td>0.3</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>0.32 or greater</td>
<td>0.4</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>

4.4.5 Analysis for earthquake loads

4.4.5.1 General
The minimum analysis requirements for seismic effects shall be as specified in Clauses 4.4.5.2 and 4.4.5.3.

For the modal methods of analysis specified in Clause 4.4.5.3, the elastic design spectrum shall be that given by the equation in Clause 4.4.7.1.

Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, the minimum requirements specified in Clauses 4.4.10.2 and 4.4.10.5 shall apply.

Details for analysis of dynamic effects are specified in Clause 4.5.

4.4.5.2 Single-span bridges

4.4.5.2.1 Analysis requirements
Seismic analysis shall not be required for single-span bridges regardless of seismic performance zone, except that single-span truss bridges in Seismic Performance Zones 2, 3, and 4 shall be analyzed as specified for regular multi-span bridges in Clause 4.4.5.3.
4.4.5.2.2 Other requirements

Connectors and restrainers between the superstructure and the abutments shall be designed for the minimum force requirements specified in Clause 4.4.10.1.

Single-span truss bridges in Seismic Performance Zones 2, 3, and 4 shall be designed for the elastic forces ($R = 1.0$).

For all seismic zones, end diaphragms in girder bridges shall be designed to remain elastic while transmitting forces equal to the connection forces specified in Clause 4.4.10.

Minimum support length requirements shall be satisfied at each abutment as specified in Clause 4.4.10.5.

For Seismic Performance Zones 2, 3, and 4, the applicable requirements of Clause 4.6 shall be satisfied.

4.4.5.3 Multi-span bridges

4.4.5.3.1 Analysis requirements

For multi-span structures, the minimum analysis requirements shall be as specified in Table 4.2.

Bridges may also be analyzed using an inelastic analysis such as a time-history analysis (see Clause 4.5.3.4) or a static pushover analysis (see Clause 4.5.3.5). Care needs to be taken with the modelling of the structure, selection of the input time histories, and interpretation of the results.

The seismic analysis of earthquake load effects for multi-span trusses and the diaphragms of girder bridges in Seismic Performance Zones 2, 3, and 4 shall also include an assessment of earthquake load effects on the superstructure components. The analysis requirements shall be as specified in Table 4.2 and shall be based on an appropriate global model of the superstructure and substructure. The response modification factors and design requirements specified in Clauses 4.4.8 and 4.4.10 shall also apply.

### Table 4.2

Minimum analysis requirements for multi-span bridges

(See Clauses 4.4.5.3.1 and 4.10.6.1.)

<table>
<thead>
<tr>
<th>Seismic performance zone</th>
<th>Lifeline bridges</th>
<th>Emergency-route bridges</th>
<th>Other bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Regular</td>
<td>Irregular</td>
<td>Regular</td>
</tr>
<tr>
<td>1</td>
<td>—</td>
<td>—</td>
<td>No seismic analysis required (see Clause 4.4.5.1)</td>
</tr>
<tr>
<td>2</td>
<td>MM</td>
<td>MM</td>
<td>UL</td>
</tr>
<tr>
<td>3</td>
<td>MM</td>
<td>TH*</td>
<td>MM</td>
</tr>
<tr>
<td>4</td>
<td>MM</td>
<td>TH*</td>
<td>MM</td>
</tr>
</tbody>
</table>

*Requires Approval. The multi-mode method may be used if appropriate.

Legend:
- **MM** = multi-mode spectral method (see Clause 4.5.3.3)
- **SM** = single-mode spectral method (see Clause 4.5.3.2)
- **TH** = time-history method (see Clause 4.5.3.4)
- **UL** = uniform-load method (see Clause 4.5.3.1)

4.4.5.3.2 Regular and irregular bridges

A bridge shall be considered regular if it has fewer than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry, and no large changes in these parameters from span to span or support to support (abutments excluded) as specified in Table 4.3. All other bridges shall be considered irregular.
4.4.6 Site effects

4.4.6.1 General
The effects of site conditions on bridge response shall be included in the determination of seismic loads for bridges.

The site coefficient, $S$, specified in Table 4.4, shall be based on the soil profile types specified in Clauses 4.4.6.2 to 4.4.6.5.

Subject to the requirements of Clause 4.4.6.6, a site coefficient need not be explicitly identified if a site-specific seismic response coefficient is developed by a qualified specialist.

### Table 4.4
**Site coefficient, $S$**
(See Clauses 4.4.6.1 and 4.4.6.6.)

<table>
<thead>
<tr>
<th>Soil profile type</th>
<th>Site coefficient, $S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.0</td>
</tr>
<tr>
<td>II</td>
<td>1.2</td>
</tr>
<tr>
<td>III</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>2.0</td>
</tr>
</tbody>
</table>

### 4.4.6.2 Soil Profile Type I
Soil Profile Type I is a profile with
(a) rock of any characteristic, shale-like or crystalline in nature (such material can be characterized by a shear wave velocity greater than 750 m/s or by another appropriate means of classification); or
(b) stiff soil conditions where the soil depth is less than 60 m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

---

Table 4.3
**Regular bridge requirements**
(See Clause 4.4.5.3.2.)

<table>
<thead>
<tr>
<th>Number of spans</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum subtended angle (curved bridge)</td>
<td>90°</td>
<td>90°</td>
<td>90°</td>
<td>90°</td>
<td>90°</td>
</tr>
<tr>
<td>Maximum span length ratio from span to span</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Maximum bent or pier stiffness ratio from span to span (excluding abutments)</td>
<td>—</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>2</td>
</tr>
</tbody>
</table>

- Continuous superstructure or multiple simple spans with longitudinal restrainers and transverse restraint at each support or a continuous deck slab
- Multiple simple spans without restrainers or a continuous deck slab

**Note:** All ratios are expressed in terms of the smaller value.

---

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4.4.6.3 Soil Profile Type II
Soil Profile Type II is a profile with stiff clay or deep cohesionless soils where the soil depth exceeds 60 m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

4.4.6.4 Soil Profile Type III
Soil Profile Type III is a profile with soft to medium-stiff clays and sands, characterized by 9 m or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

4.4.6.5 Soil Profile Type IV
Soil Profile Type IV is a profile with soft clays or silts greater than 12 m in depth. These materials can be characterized by a shear wave velocity less than 150 m/s and can include loose natural deposits or non-engineered fill.

4.4.6.6 Other soil profile types
For other soil profile types, or where the soil properties are not known in sufficient detail to determine the soil profile type with confidence, the Engineer shall use his or her judgment to select a site coefficient from Table 4.4 that conservatively represents the amplification effects at the site. The soil profile coefficients shall apply to all foundation types, including pile-supported and spread footings.

4.4.7 Elastic seismic response coefficient

4.4.7.1 General
Unless otherwise specified in Clause 4.4.7.2, the elastic seismic response coefficient, $C_{sm}$, for the $m$th mode of vibration shall be

$$C_{sm} = \frac{1.2A/S}{T_m^{2/3}} \leq 2.5AI$$

where

$A$ = zonal acceleration ratio specified in Clause 4.4.3  
$S$ = site coefficient specified in Clause 4.4.6  
$T_m$ = period of vibration of the $m$th mode, s  
$I$ = importance factor based on the importance category specified in Clause 4.4.2  
= 3.0 for lifeline bridges, but need not be taken greater than the value of $R$ for the ductile substructure elements specified in Table 4.5  
= 1.5 for emergency-route bridges  
= 1.0 for other bridges

4.4.7.2 Exceptions
The following exceptions shall apply:

(a) For Soil Profile Type III or Type IV soils in areas where the zonal acceleration ratio is equal to or greater than 0.30, $C_{sm}$ need not exceed $2.0AI$.

(b) For Soil Profile Type III or Type IV soils, $C_{sm}$ for modes other than the fundamental mode that have periods less than 0.3 s shall be taken as

$$C_{sm} = AI(0.8 + 4.0T_m)$$

(c) For structures in which the period of vibration of any mode exceeds 4.0 s, the value of $C_{sm}$ for that mode shall be taken as

$$C_{sm} = \frac{3AI}{T_m^{1/3}}$$
4.4.7.3 Site-specific elastic response coefficient

Site-specific response spectra may be used with Approval, except that the response spectra ordinates shall not be less than $0.8C_{sm}$.

4.4.8 Response modification factors

4.4.8.1 General

The seismic design force effects for the ductile substructure elements shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, $R$, specified in Table 4.5. Response modification factors depend on the ability of the ductile substructure element to develop an appropriate level of ductility and energy absorption and shall be used only when all of the design and detailing requirements specified in Table 4.5 are satisfied for the element.

The lateral load-resisting substructure elements shall be designed and detailed to be ductile, i.e., have a minimum $R$ of 2.0.

For bridges of slab, beam-girder, or box girder construction with a structurally continuous reinforced concrete deck from pier to pier (or abutment to abutment), a detailed analysis of earthquake effects on superstructure components shall not be required. However, an analysis of cross-frames or diaphragms between girders at the abutments and piers shall be required.

<table>
<thead>
<tr>
<th>Ductile substructure elements</th>
<th>Response modification factor, $R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall-type piers in direction of larger dimension</td>
<td>2.0</td>
</tr>
<tr>
<td>Reinforced concrete pile bents</td>
<td></td>
</tr>
<tr>
<td>Vertical piles only</td>
<td>3.0</td>
</tr>
<tr>
<td>With batter piles</td>
<td>2.0</td>
</tr>
<tr>
<td>Single columns</td>
<td></td>
</tr>
<tr>
<td>Ductile reinforced concrete</td>
<td>3.0</td>
</tr>
<tr>
<td>Ductile steel</td>
<td>3.0</td>
</tr>
<tr>
<td>Steel or composite steel and concrete pile bents</td>
<td></td>
</tr>
<tr>
<td>Vertical piles only</td>
<td>5.0</td>
</tr>
<tr>
<td>With batter piles</td>
<td>3.0</td>
</tr>
<tr>
<td>Multiple-column bents</td>
<td></td>
</tr>
<tr>
<td>Ductile reinforced concrete</td>
<td>5.0</td>
</tr>
<tr>
<td>Ductile steel columns or frames</td>
<td>5.0</td>
</tr>
<tr>
<td>Braced frames</td>
<td></td>
</tr>
<tr>
<td>Ductile steel braces</td>
<td>4.0</td>
</tr>
<tr>
<td>Nominally ductile steel braces</td>
<td>2.5</td>
</tr>
</tbody>
</table>

**Note:** See Clauses 4.7 and 4.8 for design and detailing requirements.

4.4.8.2 Application

Seismic forces shall be assumed to act in any horizontal direction.

The appropriate $R$-factor shall be used for each orthogonal axis of the substructure.

A wall-type concrete pier may be analyzed as a single column in the weak direction of the pier if all of the requirements for columns specified in Clause 4.7.4.2 are satisfied.
4.4.9 Load factors and load combinations

4.4.9.1 General
The load factors and load combinations shall be in accordance with Clause 3.5. The earthquake load, EQ,
shall be determined in accordance with Clause 4.4.9.2.

4.4.9.2 Earthquake load cases
Unless otherwise specified in Clause 4.4.9.3, the elastic seismic effects on each of the principal axes of a
component resulting from analyses in the two perpendicular horizontal directions shall be combined to
form two load cases as follows:
(a) 100% of the absolute value of the effects resulting from an analysis in one of the perpendicular
directions combined with 30% of the absolute value of the force effects from the analysis in the
second perpendicular direction; and
(b) 100% of the absolute value of the effects from the analysis in the second perpendicular direction
combined with 30% of the absolute value of the force effects resulting from the analysis in the first
perpendicular direction.

The effects of vertical ground motion shall be accounted for by using the load factors on dead load
specified in Table 3.2.

4.4.9.3 Time-history analysis
For spectrum-compatible time-histories in accordance with Clause 4.5.3.4, separate analyses shall be
carried out in the two perpendicular directions and the resulting forces shall be combined in accordance
with Clause 4.4.9.2.

For a time-history analysis using site-specific acceleration time-histories in which the two horizontal
ground motion components are considered simultaneously, the resulting forces need not be combined as
specified in Clause 4.4.9.2. The effects of vertical ground motion shall be accounted for by using the load
factors on dead loads specified in Table 3.2 or by using a site-specific vertical acceleration time-history.

4.4.10 Design forces and support lengths

4.4.10.1 General
For single-span bridges in any seismic performance zone, the minimum design connection force effect in
the restrained directions between the superstructure and the substructure shall be the tributary dead load
at the abutment multiplied by the zonal acceleration ratio and the site coefficient for the site (with a
minimum value of $A = 0.05$). This force shall be considered to act in each horizontally restrained direction.

For multi-span bridges, the applicable requirements of Clauses 4.4.10.2 to 4.4.10.7 shall be satisfied.

For all bridges, the minimum support lengths at expansion bearings shall be in accordance with
Clause 4.4.10.5. Alternatively, longitudinal restrainers shall be provided in accordance with
Clause 4.4.10.6.

4.4.10.2 Seismic Performance Zone 1
Where transverse or longitudinal restraint of the superstructure is provided relative to the substructure, the
restraining element shall be designed to resist a horizontal seismic force in each restrained direction equal to
(a) 0.10 times the tributary dead load, for $A = 0.0$; or
(b) 0.20 times the tributary dead load, for $A = 0.05$.
For each uninterrupted segment of a superstructure, the tributary dead load at the line of fixed bearings
used to determine the longitudinal connection force shall be the total dead load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the
transverse direction, the tributary dead load used to determine the transverse connection design force
shall be the dead load reaction at that bearing.
4.4.10.3 Seismic Performance Zone 2

4.4.10.3.1 General
Structures in Seismic Performance Zone 2 shall be analyzed in accordance with the minimum requirements specified in Clause 4.4.5.

4.4.10.3.2 Modified seismic design forces
The modified seismic design forces for Zone 2 shall be determined in accordance with the requirements of Clause 4.4.10.4, except that the nominal resistance of the ductile substructure elements shall be used instead of their probable resistance.

4.4.10.4 Seismic Performance Zones 3 and 4

4.4.10.4.1 General
Structures in Seismic Performance Zones 3 and 4 shall be analyzed in accordance with the minimum requirements specified in Clause 4.4.5.

4.4.10.4.2 Modified seismic design forces
For load effects in ductile substructure elements, seismic design forces, e.g., moments in columns, piers, and pile bents or axial forces in braces, shall be determined by dividing the elastic seismic forces obtained in accordance with Clause 4.4.9.2 by the appropriate response modification factor, \( R \), specified in Table 4.5. The seismic design forces so determined shall be termed modified seismic design forces.

Seismic design forces for capacity-protected elements, e.g., superstructures, cap-beams, beam-column joints, and foundations (including footings, pile caps, and piles, but not including pile bents and retaining walls), shall be determined using elastic design forces obtained in accordance with Clause 4.4.9, with \( R = 1.0 \) and \( I = 1.0 \). Alternatively, capacity-protected elements may be designed to have factored resistances equal to or greater than the maximum force effects that can be developed by the ductile substructure element(s) attaining their probable resistance.

Connectors shall be designed to transmit, in their restrained directions, the maximum force effects determined from 1.25 times the elastic seismic forces \((R = 1.0 \text{ and } I = 1.0)\), but these forces need not exceed the force that can be developed by the ductile substructure element attaining 1.25 times its probable resistance.

4.4.10.4.3 Yielding mechanisms and design forces in ductile substructures
The yielding mechanism shall be considered to form prior to any other failure mode due to overstress or instability in the structure and/or in the foundation. Except for pile bents, yielding shall be permitted in ductile substructure elements such as columns, piers, or braces only at locations where the elements can be readily inspected and/or repaired. The nominal and probable resistances shall be determined for the final details of the member chosen and hence may be somewhat larger than the resistance required from the design procedure.

For yielding mechanisms involving flexural hinging in ductile substructure elements such as columns, piers, and bents, inelastic hinging moments shall be taken as their probable resistance determined by multiplying the flexural nominal resistance of concrete sections by 1.30 and of steel sections by 1.25.

The shear and axial design forces for columns, piers, and pile bents due to earthquake effects shall be as follows:

(a) Shear forces: either the unreduced elastic design shear determined in accordance with Clause 4.4.10.4.2, with \( R = 1.0 \), or the shear corresponding to inelastic hinging of the column determined from statics that take into consideration the probable flexural resistance of the member and its effective height. For flared columns and columns adjacent to partial-height walls, the top and
bottom flares and the height of the walls shall be considered in determining the effective column height. If the column foundation is significantly below ground level, the possibility of the hinge forming above the foundation shall be considered.

(b) Axial forces: either the unreduced elastic design axial force determined in accordance with Clause 4.4.10.4.2 (using an R-factor of 1.0) or the axial force corresponding to inelastic hinging of the column in a bent.

4.4.10.4.4 Undesirable failure modes and design forces in ductile substructure elements
Ductile substructure elements shall be designed so that undesirable failure modes such as shear failures in concrete columns and local buckling of steel columns or braces are avoided.

4.4.10.5 Minimum support length requirements for displacements
Unless longitudinal restrainers in accordance with Clause 4.4.10.6 are provided, bridge support lengths at expansion bearings shall accommodate the greater of the maximum displacement calculated in accordance with Clause 4.4.5 (with R = 1.0) and the empirical support length, $N_s$, calculated in accordance with this Clause. Bearings restrained for longitudinal movement shall be designed in accordance with Clause 4.4.10.1, 4.4.10.2, 4.4.10.3.2, or 4.4.10.4.2.

The empirical support length, $N_s$, shall be the minimum support length in millimetres measured normal to the face of the abutment or pier (excluding the concrete cover) and shall be calculated as follows:

$$N_s = K \left[ 200 + \frac{L}{600} + \frac{H}{150} \right] \left[ 1 + \frac{\psi^2}{8000} \right]$$

where

$K$ = modification factor specified in Table 4.6
$L$ = the length of the bridge deck to the adjacent expansion joint or to the end of the bridge deck, mm (see Figure 4.1)
$H$ = for abutments, the average height of the columns supporting the bridge deck to the next expansion joint, mm
$H$ = for columns or piers, the column or pier height, mm
$H$ = for hinges within a span, the average height of the two adjacent columns or piers, mm
$H$ = for single-span bridges, 0.0 mm
$\psi$ = skew of support measured from a line normal to the span direction, degrees
Figure 4.1
Dimensions for minimum support lengths
(See Clause 4.4.10.5.)

Table 4.6
Modification factor, $K$
(See Clause 4.4.10.5.)

<table>
<thead>
<tr>
<th>Seismic performance zone</th>
<th>Modification factor, $K$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
</tr>
<tr>
<td>$A = 0$</td>
<td></td>
</tr>
<tr>
<td>Soil Profile Type I or II</td>
<td>0.5</td>
</tr>
<tr>
<td>Soil Profile Type III or IV</td>
<td>1.0</td>
</tr>
<tr>
<td>$A = 0.05$ (all soil profile types)</td>
<td>1.0</td>
</tr>
<tr>
<td>2 (all $A$ and all soil profile types)</td>
<td>1.0</td>
</tr>
<tr>
<td>3 (all $A$ and all soil profile types)</td>
<td>1.5</td>
</tr>
<tr>
<td>4 (all $A$ and all soil profile types)</td>
<td>1.5</td>
</tr>
</tbody>
</table>
4.4.10.6 Longitudinal restrainers
Restrainers shall be designed to ensure integrity under excessive forces or movements without experiencing brittle failures. Friction shall not be considered to be an effective restrainer.

Restrainers shall be designed for a force calculated as three times the zonal acceleration ratio, $A$, multiplied by the dead load of the lighter of the two adjoining spans or parts of the structure, but the factor $3A$ shall not be less than 0.2.

If the restrainer is at a point where relative displacement of sections of the superstructure will occur due to effects such as temperature change and shrinkage, sufficient slack shall be provided in the restrainer so that the restrainer does not restrict such movements.

Where restrainers are to be provided at columns or piers, the restrainer of each span may be attached to the column or pier rather than interconnecting adjacent spans. Where a column or pier could be subject to instability due to ground liquefaction or excessive ground movements, the restrainer shall be attached to the column or pier.

The connections of a restrainer to the superstructure or substructure shall be designed to resist 125% of the ultimate restrainer capacity. Restrainers shall be designed to remain elastic under the design seismic forces specified in this Section.

4.4.10.7 Hold-down devices
For bridges in Seismic Performance Zones 2, 3, and 4, hold-down devices shall be provided at supports and at hinges where the vertical force effect resulting from the seismic load cases specified in Clause 4.4.9.2 or 4.4.9.3 opposes and exceeds 50% but is less than 100% of the reaction due to dead loads. In this case, the net upward force for the design of the hold-down device shall be taken as 10% of the reaction due to dead loads that would be exerted if the span were simply supported.

If the vertical seismic forces result in net uplift, the hold-down device shall be designed to resist the larger of
(a) 120% of the difference between the vertical seismic force and the reaction due to dead loads; and
(b) 10% of the reaction due to dead loads that would be exerted if the span were simply supported.

4.5 Analysis

4.5.1 General
The minimum analysis requirements for seismic effects are specified in Clauses 4.4.5.2 and 4.4.5.3. The four types of analysis are described in Clauses 4.5.3.1 to 4.5.3.4.

In the analysis methods specified in Clause 4.5.3, the actual weight shall be taken as the effective weight.

In the modelling of reinforced concrete sections, either uncracked or cracked cross-sectional properties shall be used when the periods and force effects are calculated. The effects of cracking shall be taken into account in calculating deflections.

For the modal methods specified in analysis specified in Clauses 4.5.3.2 and 4.5.3.3, the elastic design spectrum shall be in accordance with Clause 4.4.7.

Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, the minimum requirements specified in Clauses 4.4.10.2 and 4.4.10.5 shall apply.

4.5.2 Single-span bridges
Seismic analysis of single-span bridges shall not be required, regardless of seismic zone, except as required by Clause 4.4.5.2.1.
4.5.3 Multi-span bridges

4.5.3.1 Uniform-load method
The uniform load method is applicable to both transverse and longitudinal earthquake motions. The equivalent uniformly distributed static seismic loading, \(P_e\), shall be taken as

\[
P_e = \frac{C_{sm} W}{L}
\]

where

\(W\) = effective weight of the bridge
\(L\) = total length of the bridge

In determining \(C_{sm}\) from Clause 4.4.7.1, the period of vibration of the bridge, \(T\), shall be taken as

\[
T = 2\pi \sqrt{\frac{W}{gK}}
\]

where

\(g\) = acceleration due to gravity, m/s\(^2\)
\(K\) = lateral stiffness of the bridge

\[p_o L\]

\[V_{s,\text{max}}\]

\(V_{s,\text{max}}\) = maximum static displacement of the bridge due to an arbitrary uniform lateral load, \(p_o\)

4.5.3.2 Single-mode spectral method
The single-mode spectral method of analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction, as appropriate. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape or by a more rigorous dynamic analysis. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, \(C_{sm}\), specified in Clause 4.4.7.1 and the corresponding spectral displacement. This amplitude shall be used to determine force effects.

The intensity of the equivalent static seismic loading, \(p_e(x)\), shall be taken as

\[
p_e(x) = \frac{\beta C_{sm} W(x) V_1(x)}{\gamma}
\]

where

\(\beta\) = \(\int W(x) V_1(x) \, dx\)
\(C_{sm}\) = elastic seismic response coefficient in Clause 4.4.7.1
\(\gamma\) = deformation corresponding to \(p_o\)

\(p_o\) = an arbitrary uniform lateral load
\(W(x)\) = effective weight of the bridge
In determining $C_{vm}$, the period of vibration of the bridge, $T$, shall be taken as

$$T = 2\pi \sqrt{\frac{\gamma}{p_{o} g \alpha}}$$

where

$$\alpha = \int V_{x}(x) \, dx$$

### 4.5.3.3 Multi-mode spectral method

The multi-mode spectral method of analysis shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. A three-dimensional model shall be used to represent the structure.

The number of modes used in the analysis shall be such that 90% mass participation of the superstructure in the direction under consideration is accounted for. The elastic seismic response spectrum specified in Clause 4.4.7 shall be used for each mode.

The member forces and displacements shall be estimated using an accepted modal combination procedure. For bridges with closely spaced modes (within 10% of each other in terms of natural frequency), the complete quadratic combination (CQC) method or the absolute sum of the modal quantities shall be used.

### 4.5.3.4 Time-history method

The time-histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the Regulatory Authority. Unless the Regulatory Authority otherwise directs, five spectrum-compatible time histories shall be used when site-specific time-histories are not available. The spectrum used to generate these five time-histories shall be the seismic response spectrum specified in Clause 4.4.7. If site-specific time-histories are used, they shall include the site soil profile effects and be modified by the importance factor, $I$.

Every step-by-step time-history method of analysis used for elastic or inelastic analysis shall satisfy the requirements of Clause 5.11.

If an inelastic time-history method of analysis is used, the $R$-factors shall be taken as 1.0.

The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material hysteretic properties.

### 4.5.3.5 Static pushover analysis

The static pushover analysis shall be a step-by-step force deformation response analysis that takes account of inelastic response and the structural detailing specified in the design, e.g., anchorage of reinforcement for reinforced concrete members and connection details for steel members. Possible local and global instability and brittle failure modes shall be considered. The analysis results may be used to determine the deformation capacity of the structure.

### 4.6 Foundations

#### 4.6.1 General

In addition to satisfying the requirements of Section 6, the requirements specified in Clauses 4.6.2 to 4.6.6 shall also be satisfied in Seismic Performance Zones 2, 3, and 4.

#### 4.6.2 Liquefaction of foundation soils

An evaluation shall be made of the potential for liquefaction of foundation soils and the impact of liquefaction on bridge foundations.
If liquefiable soils are identified and pose a hazard to the bridge, one or more of the following measures shall be taken:

(a) use of an appropriate foundation type, e.g., deep piles or piers that extend below the zone of liquefiable soil. These foundation elements shall be designed to withstand ground-movement-induced soil loads;

(b) soil improvement methods such as densification, removal and replacement, grouting, and dewatering or providing drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled; and

(c) design of bridge structures to withstand the predicted ground movements.

4.6.3 Stability of slopes
An analysis shall be carried out to assess the effect of seismic forces on the stability of soil and rock slopes adjacent to the bridge.

If the analysis shows that slope instability is likely during or following an earthquake, the effect of this instability on the bridge foundations, particularly regarding slope movement, shall be evaluated. If the movements are unacceptable, slope-stabilizing measures shall be carried out to reduce such movements.

4.6.4 Seismic forces on abutments and retaining walls
Seismically induced lateral soil pressures on the back of abutment and retaining walls shall be included in the design where applicable. These pressures may be calculated using the Mononobe-Okabe method.

4.6.5 Soil-structure interaction
When deemed appropriate by the Regulatory Authority, the interaction of soil-structure foundation systems due to earthquake loading shall be evaluated.

4.6.6 Fill settlement and approach slabs
Unless exempted by the Regulatory Authority, settlement or approach slabs providing structural support between approach fills and abutments shall be provided. These approach slabs shall be adequately tied to the abutments.

Positive ties to the abutment shall be capable of resisting a design force, \( F_s \), calculated as follows:

\[
F_s = (\mu + A)W_s
\]

where

\( \mu \) = coefficient of friction between the slab and the underlying soil

\( A \) = zonal acceleration ratio (from Table 4.1)

\( W_s \) = permanent vertical reaction between slab and soil

This connection shall be free to rotate so that moment will not be transferred to the abutment backwall when the approach fill settles.

4.7 Concrete structures

4.7.1 General
Concrete structures shall satisfy the requirements of Clauses to 4.7.2 to 4.7.5 and the applicable requirements of Section 8.

4.7.2 Seismic Performance Zone 1
Bridges in Seismic Performance Zone 1 shall satisfy the requirements of Clause 4.4.10.2.
4.7.3 Seismic Performance Zone 2
Bridges in Seismic Performance Zone 2 shall satisfy the requirements of Clause 4.4.10.3. The transverse reinforcement at the top and bottom of a column shall be as specified in Clauses 4.7.4.2.5 and 4.7.4.2.6.

4.7.4 Seismic Performance Zones 3 and 4

4.7.4.1 General
Bridges in Seismic Performance Zones 3 and 4 shall satisfy the requirements of Clauses 4.4.10.4 and 4.7.4.2 to 4.7.4.4.

4.7.4.2 Column requirements

4.7.4.2.1 General
For the purposes of Clauses 4.7.4.2.2 to 4.7.4.2.7, a vertical support shall be considered to be a column if the ratio of the clear height to the maximum plan dimension of the support is equal to or greater than 2.5. For a flared column, the maximum plan dimension shall be taken at the minimum section of the flare. For supports with a ratio of clear height to maximum plan dimension of less than 2.5, the requirements of Clause 4.7.4.3 shall apply.

4.7.4.2.2 Longitudinal reinforcement
The area of longitudinal reinforcement shall not be less than 0.008 or more than 0.06 times the gross cross-sectional area, \( A_g \), of the column. The centre-to-centre spacing of longitudinal bars shall not exceed 200 mm.

4.7.4.2.3 Flexural resistance
The biaxial resistance of columns shall not be less than that required to resist the forces specified in Clause 4.4.10.4.

4.7.4.2.4 Column shear and transverse reinforcement
The factored shear force, \( V_{f} \), on each principal axis of each column and concrete pile bent shall be as specified in Clause 4.4.10.4.3.

The amount of transverse reinforcement shall not be less than that determined in accordance with Clause 8.9.3.

The following requirements shall apply to the plastic hinge regions at the top and bottom of the column and pile bents:

(a) In the plastic hinge regions, when the minimum factored axial compression force exceeds \( 0.10 f_c A_g \), \( V_c \) shall be as specified in Clause 8.9.3. \( V_c \) shall be taken as zero when the minimum factored axial compression force is zero. For values of minimum factored axial compression force between zero and \( 0.1 f_c A_g \), linear interpolation may be used to determine the value of \( V_c \).

(b) The plastic hinge region shall be assumed to extend from the soffit of girders or cap beams at the top of columns to the top of foundations at the bottom of columns. This distance shall be taken as the greatest of

(i) the maximum cross-sectional dimension of the column;
(ii) one-sixth of the clear height of the column; or
(iii) 450 mm.

The plastic hinge region at the top of the concrete pile bent shall be taken as that specified for columns. At the bottom of the pile bent, the plastic hinge region shall be considered to extend from three times the maximum cross-section dimension below the calculated point of maximum moment, taking into account soil-pile interaction, to a distance of not less than the maximum cross-section dimension, but not less than 500 mm above the ground line.
4.7.4.2.5 Transverse reinforcement for confinement at plastic hinge regions

The cores of columns and concrete pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The transverse reinforcement for confinement shall have a yield strength not more than that of the longitudinal reinforcement and the spacing shall be in accordance with Clause 4.7.4.2.6.

For a circular column, the ratio of spiral reinforcement, $\rho_s$, shall not be less than the greater of that determined in accordance with Clause 8.14.4.2 or

$$\rho_s = \frac{0.12 f'c}{f_y} \left[ 0.5 + \frac{1.25 \rho_l}{\phi_c f'c A_g} \right]$$

where

$$0.5 + \frac{1.25 \rho_l}{\phi_c f'c A_g} \geq 1.0$$

Within plastic hinge regions, splices in spiral reinforcement shall be in accordance with Clause 8.14.4.2. Lap splices in longitudinal reinforcement shall be used only as specified in Clause 4.7.4.2.7.

For rectangular columns, the total cross-sectional area, $A_{sh}$, of transverse reinforcement shall not be less than the greater of

$$A_{sh} = 0.30 sh_c \frac{f'c}{f_y} \left[ \frac{A_g}{A_t} - 1 \right]$$

$$A_{sh} = 0.12 sh_c \frac{f'c}{f_y} \left[ 0.5 + \frac{1.25 \rho_l}{\phi_c f'c A_g} \right]$$

where

$$0.5 + \frac{1.25 \rho_l}{\phi_c f'c A_g} \geq 1.0$$

and $s$ is the vertical spacing of transverse reinforcement.

$A_{sh}$ shall be calculated for both principal axes of a rectangular column and the larger value shall be used.

Transverse reinforcement in plastic hinge regions shall be provided by single or overlapping hoops or spirals. Seismic cross-ties having the same bar size as the tie may be used. Each end of the seismic cross-tie shall engage a peripheral longitudinal reinforcing bar. Seismic cross-ties shall be alternated so that hooks that do not qualify as seismic hooks are not adjacent to each other in the horizontal and vertical directions.

4.7.4.2.6 Spacing of transverse reinforcement for confinement

Transverse reinforcement for confinement shall be provided in the plastic hinge regions specified in Clause 4.7.4.2.4 and shall extend into the top and bottom connections in accordance with Clause 4.7.4.4.

The centre-to-centre spacing shall not exceed the smallest of 0.25 times the minimum component dimension, six times the diameter of the longitudinal reinforcement, or 150 mm.

The centre-to-centre spacing of interlocking spirals or hoop cages in oblong columns shall not be greater than 0.75 times the diameter of the spiral or hoop cage. A minimum of four vertical bars shall be located within each overlapping region of the spirals or hoops.

4.7.4.2.7 Splices

Splices shall satisfy the requirements of this Clause and Clause 8.15.9.
Lap splices in longitudinal reinforcement shall not be located in plastic hinge regions and shall be permitted only within the centre half of column height unless the splices are located in a region where it is demonstrated that plastic hinging will not occur. The splice length shall not be less than the greater of 60 bar diameters or 400 mm. The centre-to-centre spacing of the transverse reinforcement over the length of the splice shall not exceed the smaller of 0.25 times the minimum cross-section dimensions of the component or 100 mm.

Welded splices in accordance with Clause 8.15.9.2 or mechanical connection splices in accordance with Clause 8.4.4.4 may be used if not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the distance between splices of adjacent bars is greater than the larger of 600 mm or 40\(d_b\), measured along the longitudinal axis of the column.

4.7.4.3 Wall-type piers
The requirements of this Clause shall apply to the design for the strong direction of a wall-type pier. The weak direction may be designed as a column in accordance with Clause 4.7.4.2. If the wall-type pier is not designed as a column in its weak direction, the limitations for shear resistance specified in this Clause shall apply.

The reinforcement ratio, both horizontally, \(\rho_{h}\), and vertically, \(\rho_{v}\), in any wall-type pier shall not be less than 0.0025, and \(\rho_{v}\) shall not be less than \(\rho_{h}\).

Reinforcement spacing, both horizontally and vertically, shall not exceed 450 mm. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The shear resistance, \(V_r\), of the pier shall be taken as the lesser of \(2.25\phi_c f_{cr} b d\) and \((0.41\phi_c f_{cr} + \rho_{h} f_{y} b d)\). Horizontal and vertical reinforcement shall be provided at each face of a pier. Splices in horizontal reinforcement shall be staggered and splices in horizontal and vertical layers shall not occur at the same location.

Ties for end wall reinforcement need not extend across the strong direction.

4.7.4.4 Column connections
The design forces for column connections shall be those of capacity-protected elements in accordance with Clause 4.4.10.4.2. The development length for all longitudinal steel shall be 1.25 times that specified in Clause 8.15.2.

Column transverse reinforcement, as specified in Clause 4.7.4.2.5, shall be continued for a distance not less than the greater of 0.5 times the maximum column dimension or 400 mm from the face of the column connection into the adjoining component.

The shear resistance provided by the concrete in the joint of a frame or bent, in the direction under consideration, shall not exceed \(2.5\phi_c f_{cr} b d\).

4.7.5 Piles

4.7.5.1 General
Pile reinforcing details shall meet the requirements of Clauses 4.7.5.2 to 4.7.5.4 and 8.23.

4.7.5.2 Seismic Performance Zone 1
No additional design provisions need to be considered for Seismic Performance Zone 1.

4.7.5.3 Seismic Performance Zone 2

4.7.5.3.1 General
Piles for structures in Seismic Performance Zone 2 shall meet the requirements of Clause 4.4.10.3.

Concrete piles shall be anchored to the pile footing or cap by embedment of pile reinforcement or by anchorages to develop uplift forces. The embedment length shall not be less than 1.25 times the development length required for the reinforcement specified in Clause 8.15.

Concrete-filled pipe piles shall be anchored by at least four dowels with a minimum steel ratio of 0.01. Dowels shall be embedded in the manner normally used for concrete piles.
4.7.5.3.2 Cast-in-place concrete piles
Longitudinal reinforcement shall be provided for cast-in-place concrete piles in the upper end of the pile for a length not less than the greater of one-third of the pile length or 2500 mm, with a minimum steel ratio of 0.005 provided by at least four bars. Spiral reinforcement or equivalent ties of not less than 10M bars shall be provided at a pitch not exceeding 225 mm, except that the pitch shall not exceed 75 mm within a length not less than the greater of 600 mm or 1.5 times the maximum cross-section dimension below the pile footing reinforcement or cap reinforcement.

4.7.5.3.3 Precast concrete piles
Longitudinal reinforcement in precast concrete piles shall not be less than 0.01 times the cross-sectional area and shall consist of at least four bars or tendons.

Spiral reinforcement or equivalent ties in precast concrete piles shall not be less than 10M bars at a pitch not exceeding 225 mm, except that the pitch shall not exceed 75 mm within a length not less than the greater of 600 mm or 1.5 times the maximum cross-section dimension below the pile footing reinforcement or cap reinforcement.

4.7.5.4 Seismic Performance Zones 3 and 4

4.7.5.4.1 General
In addition to meeting the requirements specified in Clause 4.7.5.3, piles in Seismic Performance Zones 3 and 4 shall meet the requirements of Clauses 4.7.5.4.2 to 4.7.5.4.5.

4.7.5.4.2 Confinement length
The upper end of every pile shall be reinforced as a potential plastic hinge region except when it can be established that there is no possibility of significant lateral deflection of the pile. The potential plastic hinge region shall extend from the underside of the pile footing or cap over a length that is the greater of twice the maximum cross-section dimension or 600 mm. Where a plastic hinge can form at a lower level, the confinement transverse reinforcement shall be provided to the lower level.

4.7.5.4.3 Confinement reinforcement
The transverse reinforcement within the confinement length, as specified in Clause 4.7.5.4.2, shall be in accordance with the requirements for columns in Clause 4.7.4.2.5.

4.7.5.4.4 Cast-in-place concrete piles
Longitudinal reinforcement shall be provided for cast-in-place concrete piles for the full length of the piles. In the upper two-thirds of the pile, the longitudinal steel ratio, provided by not fewer than four bars, shall not be less than 0.0075. Spiral reinforcement or equivalent ties of not less than 10M bars shall be provided at a pitch not exceeding 225 mm, except that the pitch shall not exceed 75 mm for the top portion of the pile over a distance not less than the greater of 1200 mm or twice the maximum cross-section dimension, and confinement reinforcement shall be in accordance with Clause 4.7.5.4.3.

4.7.5.4.5 Precast concrete piles
The longitudinal reinforcement in precast concrete piles shall not be less than 1% of the cross-sectional area of the pile and shall consist of at least four bars or tendons.

Spiral reinforcement or equivalent ties in precast concrete piles shall not be less than 10M bars at a pitch not exceeding 225 mm, except for the top 1200 mm, where the pitch shall not exceed 75 mm and the confinement reinforcement shall be in accordance with Clause 4.7.5.4.3.
4.8 Steel structures

4.8.1 General
Steel structures shall meet the requirements of Clause 4.8 as well as the applicable requirements of Section 10. Clause 4.8 shall not apply to structures that are designed to resist full elastic seismic loads.

There shall be a continuous and clear load path or paths. Proper load transfer shall be considered in designing foundations, substructures, superstructures, and connections.

Welds located in regions of expected inelastic deformations shall be complete penetration welds. Partial penetration groove welds shall not be permitted in these regions.

Abrupt changes in cross-sections of members shall not be permitted in regions of expected inelastic deformation unless demonstrated to be acceptable by analysis and such acceptability is supported by research results.

4.8.2 Materials
Ductile substructure elements shall be made of steels conforming to CSA G40.21, Grade 350A, 350AT, 300W, 350W, 300WT, or 350WT. Materials other than these may be used if Approved and the probable and nominal strengths are correctly established.

Other elements shall be made of steels in accordance with Clause 10.4.1.

4.8.3 Sway stability effects
The sway effects produced by the vertical loads acting on the structure in its displaced configuration shall be determined from a second-order analysis. Alternatively, the requirements of CAN/CSA-S16 may be applied.

4.8.4 Steel substructures

4.8.4.1 General
Clauses 4.8.4.2 to 4.8.4.4 shall apply to single-tier steel substructures of single-level bridges. For other structures, the requirements of Clause 4.8.5 shall apply.

4.8.4.2 Seismic Performance Zone 1
Steel substructures in Seismic Performance Zone 1 shall meet the requirements of Clause 4.4.10.2.

4.8.4.3 Seismic Performance Zone 2

4.8.4.3.1 General
Steel substructures in Seismic Performance Zone 2 shall meet the requirements of Clauses 4.8 and 4.4.10.3.

4.8.4.3.2 Ductile moment-resisting frames and bents

4.8.4.3.2.1 General
Ductile moment-resisting frames and bents shall meet the requirements of Clause 4.8.4.4.2, as modified by Clauses 4.8.4.3.2.2 and 4.8.4.3.2.3.

4.8.4.3.2.2 Columns
Columns shall be designed as ductile substructure elements. The maximum axial compressive load limit of Clause 4.8.4.4.2.2 shall be replaced by 0.60AgFy.
4.8.4.3.2.3 Beams, panel zones, and connections
Beams, panel zones, moment-resisting connections, and column base connections shall be designed as capacity-protected elements as specified in Clause 4.4.10.3.2. The nominal flexural resistance of the column shall be determined from Clause 4.8.4.4.2.3, with $\phi_s$ taken as unity.

4.8.4.3.3 Ductile single-column structures
Ductile single-column structures shall meet the requirements of Clause 4.8.4.4.2, except that the maximum axial compressive load limit of Clause 4.8.4.4.2.2 shall be replaced by $0.60A_gF_y$.

4.8.4.3.4 Ductile concentrically braced frames
Ductile concentrically braced frames and bents shall meet the requirements of Clause 4.8.4.4.3.

4.8.4.3.5 Concentrically braced frames and bents with nominal ductility
Concentrically braced frames and bents with nominal ductility shall meet the requirements of Clause 4.8.4.4.4, except that braces in chevron-braced frames need not comply with Clause 4.8.4.4.3 but shall meet the requirements of Clause 4.8.4.4.3.

4.8.4.3.6 Other framing systems
Other framing systems shall meet the requirements of Clause 4.8.4.4.5 or 4.8.5.

4.8.4.4 Seismic Performance Zones 3 and 4

4.8.4.4.1 General
Steel substructures in Seismic Performance Zones 3 and 4 shall meet the requirements of Clause 4.8 and the applicable requirements of Clause 4.4.10.4.

4.8.4.4.2 Ductile moment-resisting frames and single-column structures

4.8.4.4.2.1 General
Clause 4.8.4.4.2 shall apply to ductile moment-resisting frames and bents constructed with I-shape beams and columns connected with their webs in a common plane. Columns shall be designed as ductile structural elements. The beams, the panel zone at column-beam intersections, and the connections shall be designed as capacity-protected elements in accordance with Clause 4.4.10.4.2.

Moment-resisting frames that do not meet these requirements shall be designed in accordance with the requirements for ductile moment-resisting frames of Clause 27 of CAN/CSA-S16.1 using $R = 5$, or in accordance with Clause 4.8.5.

4.8.4.4.2.2 Columns
Columns shall be Class 1 sections in accordance with Section 10. Welded sections shall have web-to-flange welds proportioned to develop the full tensile capacity of the web.

The resistance of columns to combined axial load and bending shall be determined in accordance with Clauses 10.8.3 and 10.9.4. The factored axial compression due to the combination of seismic load and permanent loads shall not exceed $0.30A_gF_y$.

The factored shear resistance, $V_r$, developed by the column web shall be taken as $0.55\phi_s w_d F_y$.

The potential plastic hinge locations near the top and base of each column shall be laterally supported and the unsupported distance from these locations shall not exceed $980r_y/\sqrt{F_y}$.

These lateral supports shall be provided either directly to the flanges or indirectly through a column web stiffener or a continuity plate. Each column flange lateral support shall resist a force of not less than 2% of the nominal column flange strength ($b_{bf}tF_y$) at the support location. The possibility of complete load reversal shall be considered.
Splices that incorporate partial joint penetration groove welds shall be located away from the potential plastic hinge locations at a minimum distance equal to the greatest of
(a) one-fourth the clear height of column;
(b) twice the column depth; and
(c) 1 m.

Fasteners connecting the separate elements of built-up columns shall have resistances able to support full yielding at potential plastic hinge locations.

4.8.4.4.2.3 Beams
The factored resistances of the beams shall be determined in accordance with Clause 10.10 or 10.11. At a joint between beams and columns, the sum of the factored resistances of the beams shall not be less than the sum of the probable resistances of the column(s) framing into the joint. The probable resistance of column(s) shall be taken as 1.25 times their nominal flexural capacity, determined as follows unless demonstrated otherwise by rational analysis:

\[ 1.18M_{px} \left[ 1 - \frac{C_f}{A_y F_y} \right] \leq M_{px} \]

4.8.4.4.2.4 Seismic design forces for panel zones and connections
Column-beam intersection panel zones, moment-resisting connections, and column base connections shall be designed as capacity-protected elements in accordance with Clause 4.4.10.4.2.

4.8.4.4.2.5 Additional requirements for panel zones and connections
Panel zones shall be designed in such a manner that the vertical shearing resistance is determined in accordance with Clause 10.10.5, using \( F_t = 0 \). Diagonal stiffeners may be used.

Beam-to-column connections shall have resistances not less than the resistances of the beam specified in Clause 4.8.4.4.2.3. Beam flange continuity plates shall be proportioned to meet the stiffener requirements of Clause 10.18.5.3 and shall be connected to both column flanges and the web. They shall be provided on both sides of the panel zone web, finish with a total width at least 0.8 times the flange width of the opposing flanges, and meet the b/t limit of a Class 3 projecting element specified in Clause 10.9.2.

Flanges and connection plates in bolted connections shall have a factored net section ultimate resistance in accordance with Item (b) or (c) of Clause 10.8.2 at least equal to the factored gross area yield resistance specified in Clause 10.8.2(a).

4.8.4.4.3 Ductile concentrically braced frames

4.8.4.4.3.1 General
Braces are the ductile substructure elements in ductile concentrically braced frames. The modified design forces for these members shall be determined in accordance with Clause 4.4.10.4.2.

4.8.4.4.3.2 Bracing systems
Diagonal braces shall be oriented in such a manner that in any planar frame at least 30% of the horizontal shear carried by the bracing system is carried by tension braces and at least 30% is carried by compression braces. Frames in which seismic load resistance is provided by any of the following shall not be considered ductile concentrically braced frames:
(a) chevron bracing or V-bracing, in which pairs of braces are located either above or below a beam and meet the beam at a single point within the middle half of the span;
(b) K-bracing, in which pairs of braces meet a column on one side near its mid-height; and
(c) knee bracing.

4.8.4.4.3.3 Bracing members
Bracing members shall have a slenderness ratio, \( L/r \), less than 1900/\( \sqrt{F_y} \).
In built-up bracing members, the slenderness ratio of the individual parts shall be not greater than 0.5 times the slenderness ratio of the member as a whole. Symmetrical open sections shall be Class 1 in accordance with Section 10.

Width-thickness ratios shall not exceed \(145/\sqrt{F_y}\) (\(330/\sqrt{F_y}\) for rectangular and square hollow structural sections and \(13,000/F_y\) for circular hollow structural sections).

The factored compressive resistance of a brace shall be determined as the product of \(C_r\) specified in Clause 10.9.3 and a reduction factor equal to \((1 + 0.35\lambda)\). This factor need not be applied if the tension braces acting in the same plane as the compression brace have sufficient reserve capacity to compensate for the reduction.

4.8.4.4.3.4 Brace connections
Brace connections shall be designed as capacity-protected elements in accordance with Clause 4.4.10.4.2. The controlling probable resistance shall be taken as the axial tensile yield strength of the brace, \(A_g F_y\). Eccentricities in brace connections shall be minimized.

Brace connections, including gusset plates, shall be detailed to avoid brittle failures due to rotation of the brace when it buckles. This ductile rotational behaviour shall be allowed for, either in the plane of the frame or out of it, depending on the slenderness ratios.

Fasteners that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the fastener, have resistances able to support one-half of the yield load of the smaller component being joined, with this force assumed to act at the centroid of the smaller member.

4.8.4.4.3.5 Columns, beams, and other connections
Columns, beams, beam-to-column connections, and column splices that participate in the lateral-load-resisting system shall be designed as capacity-protected elements in accordance with Clauses 4.4.10.4.2 and 4.8.4.4.3.4 and shall also meet the following requirements:

(a) Columns, beams, and connections shall resist forces arising from load redistribution following brace buckling or yielding. The brace compressive resistance shall include the reduction factor specified in Clause 4.8.4.4.3.3 if this creates a more critical condition.

(b) Column splices made with partial penetration groove welds and subject to net tension forces due to overturning effects shall have factored resistances not less than 50% of the flange yield load of the smaller segment.

4.8.4.4.4 Concentrically braced frames with nominal ductility

4.8.4.4.4.1 General
Braces are the ductile substructure elements in nominally ductile concentrically braced frames. The modified design forces for these members shall be determined in accordance with Clause 4.4.10.4.2.

4.8.4.4.4.2 Bracing systems
Bracing systems considered to have nominal ductility include tension diagonal bracing, chevron or V-bracing, and direct tension-compression diagonal bracing. K-braced frames, in which pairs of braces meet a column near its mid-height, and knee-braced frames shall not be considered concentrically braced frames with nominal ductility.

4.8.4.4.4.3 Bracing members
Inclined compression bracing members shall be Class 2 sections as specified in Section 10 or shall have cross-section elements that can undergo limited straining while sustaining the yield stress.

4.8.4.4.4.4 Brace connections
Brace connections shall be designed as capacity-protected elements in accordance with Clause 4.4.10.4.2. The controlling probable resistance shall be taken as the axial tensile yield strength of the brace, \(A_g F_y\).

For tension-only bracing, the load selected shall be multiplied by an additional factor of 1.10.
4.8.4.4.4.5 Columns, beams, and connections
Columns, beams, and connections shall be designed as capacity-protected elements in accordance with Clauses 4.4.10.4.2 and 4.8.4.4.3.5.

4.8.4.4.4.6 Chevron-braced and V-braced systems
Braces in chevron-braced frames shall meet the requirements of Clause 4.8.4.4.3.3.

The beam attached to chevron braces or V-braces shall be continuous between columns and its top and bottom flanges shall be designed to resist a lateral load of 1.5% of the flange yield force at the point of intersection with the braces. The beam shall
(a) resist the combined effect corresponding to one brace attaining its reduced compressive resistance, as specified in Clause 4.8.4.4.3.3, with the other brace attaining its tensile capacity, \( A_g F_y \), and the permanent loads acting on the beam; or
(b) be a Class 1 section, as specified in Section 10, and the beam connections at the columns shall resist the load effects corresponding to plastic hinging at the brace intersection point. When such a beam is supported from below by chevron braces, it shall have adequate nominal resistance to support its permanent loads without the support provided by the braces.

4.8.4.4.5 Ductile eccentrically braced frames
Ductile eccentrically braced frames may be proportioned in accordance with Clause 27.7 of CAN/CSA-S16, using \( R = 5 \).

4.8.5 Other systems
Other framing systems and frames that incorporate special bracing, base isolation, or other energy-absorbing devices, or special ductile superstructure elements, shall be designed on the basis of published research results, observed performance in past earthquakes, or special investigation, and shall require Approval.

4.9 Joints and bearings

4.9.1 General
The requirements of Section 11 shall apply to joints and bearings.

4.9.2 Seismic design forces
The seismic design forces shall be in accordance with Clause 4.4.10.

4.10 Seismic base isolation

4.10.1 General
Clause 4.10 specifies requirements for isolator units and for the seismic isolation design of highway bridges.

Design requirements for isolation bearings are specified in Clauses 4.10.2 to 4.10.10. These requirements provide a revised design procedure for isolation bearings that allows for the possibility of large displacements resulting from the seismic response. General test requirements are specified in Clause 4.10.11. Requirements for elastomeric isolators are specified in Clauses 4.10.12 and 4.10.13. Additional requirements for sliding isolators are specified in Clauses 4.10.14 and 4.10.15. The requirements of Section 11 shall also apply.

Isolation systems without self-centring capabilities shall not be used.
4.10.2 Zonal acceleration ratio
The zonal acceleration ratio, $A_z$, shall be as specified in Table 4.1 but not less than 0.1.

**Note:** The zonal acceleration ratio specified in Table 4.1 for seismic isolation design is the same as that for conventional design.

4.10.3 Seismic performance zones
The seismic performance zones, which delineate the method of analysis and the minimum design requirement, are the same as those for conventional design and are specified in Table 4.1.

4.10.4 Site effects and site coefficient
The site coefficient for seismic isolation design, $S_i$, which accounts for the site condition effects on the elastic response coefficient, shall be as specified in Table 4.7.

<table>
<thead>
<tr>
<th>Soil profile type (see Clauses 4.4.6.2 to 4.4.6.5)</th>
<th>Site coefficient for seismic isolation design, $S_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.0</td>
</tr>
<tr>
<td>II</td>
<td>1.5</td>
</tr>
<tr>
<td>III</td>
<td>2.0</td>
</tr>
<tr>
<td>IV</td>
<td>2.7*</td>
</tr>
</tbody>
</table>

*Site-specific studies should be used for isolated bridges on Type IV soils.

4.10.5 Response modification factors and design requirements for substructure
Response modification factors, $R$, for all substructures shall be limited to 1.5, whereas substructures for lifeline and emergency-route bridges shall be designed to remain elastic ($R = 1.0$). For all isolated bridges, the design and detailing requirements for substructures in Seismic Performance Zones 2, 3, and 4 shall, at a minimum, be equivalent to the requirements for structures in Seismic Performance Zone 2.

4.10.6 Analysis procedures

4.10.6.1 General
Table 4.2 shall be used to determine the applicable analysis procedure. The application of the applicable analysis procedure to isolated bridges shall be as specified in Clause 4.10.6.2 or 4.10.6.3. However, for isolation systems where the effective damping (expressed as a percentage of critical damping) exceeds 30%, a three-dimensional non-linear time-history analysis shall be performed using the hysteresis curves of the isolation system unless the value of $B$ in Clause 4.10.6.2.1 is limited to 1.7.
4.10.6.2 Uniform-load/single-mode spectral analysis

Note: See Clauses 4.5.3.1 and 4.5.3.2 for the uniform-load and single-mode spectral methods.

4.10.6.2.1 Statically equivalent seismic force and coefficient

Except for the case where a soil profile for the bridge site is Type IV, the statically equivalent seismic force, \( F \), shall be

\[ F = C'_{sm} W \]

where

\[ C'_{sm} = \text{elastic seismic response coefficient for isolated structures} \]

\[ = \frac{A S_i}{B T_e} \leq 2.5 \frac{A}{B} \]

\( W \) = dead load of the superstructure segment supported by isolation bearings

The displacement, \( d_i \), across the isolation bearings (in millimetres) shall be

\[ d_i = \frac{250 A S_i T_e}{B} \]

where

\( A \) = zonal acceleration ratio from Table 4.1

\( B \) = damping coefficient from Table 4.8 for the direction under consideration

\( S_i \) = dimensionless site coefficient for isolation design for the given soil profile, as specified in Table 4.7

\( T_e \) = period of vibration, s

\[ = 2\pi \sqrt{\frac{W}{\sum k_{eff} g}} \]

where

\( \sum k_{eff} \) = sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure segment, calculated at displacement \( d_i \)

\( g \) = acceleration due to gravity
4.10.6.2.2 Application of uniform-load/single-mode method of analysis

The statically equivalent force determined in accordance with Clause 4.10.6.2.1, which is associated with the displacement across the isolation bearings, shall be applied using either the uniform-load method or the single-mode spectral method of analysis independently along two perpendicular axes and combined as specified in Clause 4.4.9.2. The effective linear stiffness of the isolators used in the analysis shall be calculated at the design displacement.

4.10.6.3 Multi-mode spectral analysis

Note: See Clause 4.5.3.3 for the multi-mode spectral method.

Where the appropriate ground motion response spectrum for the isolated modes is specified by Clause 4.10.6.2.1, an equivalent linear response spectrum analysis shall be performed in accordance with Clause 4.5.3. The ground motion response spectrum specified in Clause 4.4.7 shall be used for all other modes of vibration. The effective linear stiffness of the isolators shall be calculated at the design displacements.

The combination of orthogonal seismic forces shall be in accordance with Clause 4.4.9.2.

---

Table 4.8
Damping coefficient, $B$
(See Clauses 4.10.6.2.1 and 4.10.11.2.)

<table>
<thead>
<tr>
<th>Equivalent viscous damping, $\beta$ (% of critical)</th>
<th>Damping coefficient, $B$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 2$</td>
<td>0.8</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>1.2</td>
</tr>
<tr>
<td>20</td>
<td>1.5</td>
</tr>
<tr>
<td>30</td>
<td>1.7</td>
</tr>
<tr>
<td>40</td>
<td>1.9</td>
</tr>
<tr>
<td>50</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: The percentage of critical damping shall be verified by a test of the isolation system’s characteristics as specified in Clause 4.10.11.3.3. The damping coefficient shall be based on linear interpolation for damping levels other than those specified in this Table. For isolation systems where the effective damping exceeds 30% of critical, a three-dimensional non-linear time-history analysis shall be performed using the hysteresis curves of the system, unless $B$ is limited to 1.7.
4.10.6.4 Time-history analysis

**Note:** See Clause 4.5.3.4 for the time-history method.

For isolation systems requiring a time-history analysis, the following requirements shall apply:

(a) The isolation system shall be modelled using the non-linear deformational characteristics of the isolators determined and verified by test in accordance with Clause 4.10.11.

(b) Pairs of horizontal ground motion time-history components shall be selected from different recorded events and modified to be compatible with the design spectra of Clause 4.4.7. The following methods may be used to achieve this modification:

(i) time histories may be scaled so that their 5%-damped response spectra do not fall below the design spectra of Clause 4.4.7 by more than 10% in the period range of 1 to 5 s or by more than 20% in the range below 1 s; or

(ii) time histories may be scaled so that the square root of the sum of the squares (SRSS) of the 5%-damped spectrum of the scaled components does not fall below 1.3 times the design spectra of Clause 4.4.7 for the period range of 1 to 5 s.

(c) At least three appropriate pairs of time histories shall be developed and each pair shall be applied simultaneously to the model. The maximum response of the parameter of interest shall be used for the design.

4.10.7 Clearance and design displacements for seismic and other loads

The design displacements in the two orthogonal directions for clearance purposes shall be the maximum displacement determined in each direction from the analysis. The required clearance for lifeline and emergency-route bridges shall be 1.25 times the maximum displacements calculated.

The total design displacement for the testing requirements of Clause 4.10.11 shall be the maximum of 50% of the elastomer shear strain in an elastomeric-based system or the maximum displacement that results from the combination of loads specified in Clause 4.4.9.2.

Horizontal deflections in the isolators resulting from load combinations involving wind loads on structure and traffic, braking forces, and centrifugal forces, as specified in Table 3.1, as well as thermal movements, shall be calculated and adequate clearance shall be provided.

4.10.8 Design forces for Seismic Performance Zone 1

The seismic design force of the connection between superstructure and substructure at each isolator for bridges in Seismic Performance Zone 1, \( F_A \), shall be

\[
F_A = k_{\text{eff}} d_i
\]

where

- \( k_{\text{eff}} \) = effective linear stiffness of the isolation bearing calculated at displacement \( d_i \)
- \( d_i \) = displacement of the isolated superstructure as specified in Clause 4.10.6.2.1, using a minimum zonal acceleration ratio, \( A \), of 0.10

4.10.9 Design forces for Seismic Performance Zones 2, 3, and 4

The requirements of Clauses 4.4.10.3 and 4.4.10.4 and the response modification factor and design requirements of Clause 4.10.5 shall apply in Seismic Performance Zones 2, 3, and 4.

The seismic design forces for columns and piers shall not be less than the forces resulting from the yield level of a softening system, the friction level of a sliding system, or the ultimate capacity of a sacrificial seismic-restraint system. In all cases, the larger of the static or dynamic conditions shall apply.

4.10.10 Other requirements

4.10.10.1 Non-seismic lateral forces

Isolated structures shall resist all non-seismic lateral load combinations applied above the isolation system, including load combinations involving wind loads on the structure and the traffic, braking forces, and centrifugal forces specified in Table 3.1.
An elastic restraint system shall be provided to limit lateral displacements of the isolation system caused by non-seismic forces, to a value satisfactory to the design Engineer.

4.10.10.2 Lateral restoring force
The isolation system shall be configured to produce a lateral restoring force such that the lateral force at the design displacement is at least 0.025W greater than the lateral force at 50% of the design displacement.

4.10.10.3 Vertical load stability
The isolation system shall provide a factor of safety of at least 3.0 for vertical loads (dead load plus live load) in its laterally undeformed state. It shall also be designed to be stable under the dead load plus or minus any vertical load resulting from seismic effects at a horizontal displacement of 1.5 times the total design displacement for isolation systems with a lateral restoring force. If the design is based on maximum credible response spectra, the 1.5 and 3.0 coefficients shall be reduced to 1.1 and 2.2, respectively.

4.10.10.4 Cold weather requirements
Cold weather performance shall be considered in the design of all types of isolation systems in sustained low-temperature zones.

4.10.11 Required tests of isolation system

4.10.11.1 General
The deformation characteristics and damping values of the isolation system used in the design and analysis shall be based on the tests specified in Clause 4.10.11. Tests on similarly sized isolators may be used to satisfy the requirements of Clause 4.10.11. Such tests shall validate design properties that can be extrapolated to the actual sizes used in the design.

The design shall also be based on manufacturers’ pre-Approved or certified test data.

4.10.11.2 Prototype tests
The following requirements shall apply to prototype tests:

(a) Prototype tests shall be performed on two full-size specimens of each type and size similar to that used in the design. The tests specimens shall include the elastic restraint system, if such a system is used in the design. The specimens tested shall not be used for construction.

(b) For each cycle of tests, the force-deflection and hysteretic behaviour of the test specimens shall be recorded.

(c) The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load similar to the typical or average dead load on the isolators of a common type and size. The total design displacement of these tests shall be in accordance with Clause 4.10.7:

(i) 20 fully reversed cycles of loading at a lateral force corresponding to the maximum non-seismic design force;

(ii) three fully reversed cycles of loading at each of the following increments of the total design displacement: 0.25, 0.50, 0.75, 1.0, and 1.25; and

(iii) 15 \(S_i/B\) cycles, but not fewer than 10 fully reversed cycles of loading at 1.0 times the total design displacement and a vertical load similar to dead load. \(B\) shall be determined from Table 4.8.

(d) The vertical load-carrying elements of the isolation system shall be statically tested at the displacements resulting from the requirements of Clause 4.10.10.3. In these tests, the maximum downward force shall be taken as the load of 1.25D plus the increased vertical load due to earthquake effects, and the minimum downward force shall be taken as 0.8D minus the vertical load due to earthquake effects, where \(EQ\) is any vertical load resulting from horizontal seismic loads.

(e) If a sacrificial elastic restraint system is used, the ultimate capacity shall be established by test.
4.10.11.3 Determination of force-deflection characteristics

4.10.11.3.1 General

The following requirements shall apply:

(a) The force-deflection characteristics of the isolation system shall be based on the cyclic load test results for each fully reversed cycle of loading.

(b) The effective stiffness of an isolator unit, \( k_{\text{eff}} \), shall be calculated for each cycle of loading as follows:

\[
\begin{align*}
    k_{\text{eff}} &= \frac{F_p - F_n}{\Delta_p - \Delta_n},
\end{align*}
\]

where \( F_p \) and \( F_n \) are the maximum positive and maximum negative forces, respectively, and \( \Delta_p \) and \( \Delta_n \) are the maximum positive and maximum negative test displacements, respectively. If the minimum effective stiffness is to be determined, \( F_{p,\text{min}} \) and \( F_{n,\text{min}} \) shall be used in the equation.

4.10.11.3.2 System adequacy

The performance of the test specimens shall be deemed to be adequate if the following conditions are satisfied:

(a) The force-deflection plots of all tests specified in Clause 4.10.11.2 have a positive incremental force-carrying capacity.

(b) For each increment of test displacement specified in Clause 4.10.11.2(c)(ii), the following conditions are met:

- (i) there is less than a ±10% change from the average effective stiffness of a given test specimen over the required three cycles of test; and
- (ii) there is not more than a 10% difference in the average values of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test.

(c) There is not more than a 20% increase or 20% decrease in the effective stiffness between the first cycle and any subsequent cycle of each test specimen for the cyclic tests specified in Clause 4.10.11.2(c)(iii).

(d) There is not more than a 20% decrease in the effective damping of each test specimen for the cyclic tests specified in Clause 4.10.11.2(c)(iii).

(e) All specimens of vertical load-carrying elements of the isolation system remain stable at the displacements specified in Clause 4.10.10.3 for the static loads specified in Clause 4.10.11.2(d).

4.10.11.3.3 Design properties of the isolation system

The following requirements shall apply to the design properties of the isolation system:

(a) The minimum and maximum effective stiffness of the isolation system shall be determined as follows:

- (i) The value of \( k_{\text{min}} \) shall be based on the minimum effective stiffnesses of individual isolator units as determined by the cyclic tests of Clause 4.10.11.2(c)(ii) at a displacement amplitude equal to the design displacement.

- (ii) The value of \( k_{\text{max}} \) shall be based on the maximum effective stiffnesses of individual isolator units as determined by the cyclic tests of Clause 4.10.11.2(c)(ii) at a displacement amplitude equal to the design displacement.

(b) The equivalent viscous damping ratio, \( \beta \), of the isolation system shall be calculated as follows:

\[
\beta = \frac{1}{2\pi} \cdot \frac{\text{Total area}}{\sum k_{\text{max}} d_i^2}
\]

where the total area represents the energy absorbed by the isolation system in one cycle and shall be taken as the sum of the areas inside the hysteresis loops of all isolators. The hysteresis loop area of each isolator shall be taken as the minimum area of one cycle obtained from the three hysteresis loops established by the cyclic tests of Clause 4.10.11.2(c)(ii) at a displacement amplitude equal to the design displacement.
4.10.12 Elastomeric bearings — Design

4.10.12.1 General
Isolator units that use elastomeric bearings shall be designed in accordance with Clause 4.10.12. Additional test requirements are specified in Clause 4.10.13.

The requirements of Clause 4.10.12 shall be considered supplemental to those of Section 11. The requirements of Clause 4.10.12 shall govern in the event of a conflict with those of Section 11.

The design procedures specified in Clause 4.10.12 are based on service loads excluding impact. Elastomeric bearings used in isolation systems shall be reinforced using integrally bonded steel reinforcement. Fabric reinforcement shall not be permitted.

4.10.12.2 Shear strain components for isolation design
The four components of shear strain in the bearing shall be calculated as follows:

(a) Shear strain due to compression by vertical loads, \( \varepsilon_{sc} \), shall be calculated as follows:

\[
\varepsilon_{sc} = 6S \varepsilon_c
\]

where

\[
S = \text{shape factor of the bearing, as defined in Clause 11.2}
\]

\[
\varepsilon_c = \frac{\Delta_c}{\Sigma t_i}
\]

where

\[
\Sigma t_i = \text{sum of the thicknesses of the deformable rubber layers}
\]

\[
= \frac{\Delta_c}{T}
\]

\[
= \frac{P}{A_c E (1 + 2kS^2)}
\]

The effects of creep of the elastomer shall be added to the instantaneous compressive deflection, \( \Delta_c \), when long-term deflections are considered. They shall not be included when the requirements of Clause 4.10.12.4 are applied. Long-term deflections shall be calculated from information relevant to the elastomer compound used, if it is available. If it is not, the values specified in Clause 11.6.6 shall be used as a guide.

(b) Shear strain due to imposed lateral displacement, \( \varepsilon_{sh} \), shall be calculated as follows:

\[
\varepsilon_{sh} = \frac{\Delta s}{T}
\]

where

\[
T = \Sigma t_i
\]

(c) Shear strain due to earthquake-imposed displacement, \( \varepsilon_{eq} \), shall be calculated as follows:

\[
\varepsilon_{eq} = \frac{d_i}{T}
\]

where

\[
T = \Sigma t_i
\]

(d) Shear strain due to rotation, \( \varepsilon_{sr} \), shall be calculated as follows:

\[
\varepsilon_{sr} = \frac{B^2 \theta}{2t_i T}
\]

where

\[
T = \Sigma t_i
\]
4.10.12.3 Limiting criteria for allowable vertical loads
The allowable vertical load on an elastomeric isolation bearing shall be governed by limitations on the equivalent shear strain in the rubber due to different load combinations and to the stability requirements specified in Clauses 4.10.12.4 to 4.10.12.6.

4.10.12.4 Combinations of shear strains due to service loads
The following two criteria shall be satisfied for service loads that include dead load plus live load, thermal effects, creep, shrinkage, and rotation:

(a) \( \varepsilon_{sc} + \varepsilon_{sh} + \varepsilon_{sr} \leq 0.5 \varepsilon_u \); and

(b) \( \varepsilon_{sc} \leq 0.33 \varepsilon_u \).

0.5 \( \varepsilon_u \) shall not exceed 5.0.

4.10.12.5 Combinations of shear strains due to dead and seismic loads
The following criterion shall be satisfied for seismic loads that include dead load and seismic load, seismic design displacements, and rotation:

\[
\varepsilon_{sc} + \varepsilon_{eq} + \varepsilon_{sr} \leq 0.75 \varepsilon_u
\]

4.10.12.6 Stability against overturning
Elastomeric isolation bearings shall be shown either by test or analysis to be capable of resisting the vertical loads specified in Clause 4.10.11.2(d) at the seismic design displacements specified in Clause 4.10.7.

4.10.13 Elastomeric bearings — Construction

4.10.13.1 General
Isolator units that use elastomeric bearings shall be constructed in accordance with Clause 4.10.13. The requirements of Clause 4.10.13 shall be considered supplemental to Section 11. The requirements of Clause 4.10.13 shall govern in the event of a conflict with those of Section 11.

Elastomeric bearings used in isolation systems shall be reinforced using integrally bonded steel reinforcement. Fabric reinforcement shall not be permitted.

Seismic isolation bearings shall meet the requirements of Clause 4.10.13.2 and Section 11.

4.10.13.2 Additional requirements for elastomeric isolation bearings

4.10.13.2.1 General
In addition to the material and bearing tests required by Clauses 4.10.10 and 4.10.11, the tests specified in Clauses 4.10.13.2.2 and 4.10.13.2.3 shall be performed on elastomeric isolation bearings.

4.10.13.2.2 Compression
A 12 h sustained proof load test on each bearing shall be performed. The compressive load for the test shall be 1.5 times the sum of the maximum dead load plus live load. If bulging suggests poor laminate bond, the bearing shall be rejected.

4.10.13.2.3 Combined compression and shear
A minimum of 20% of the bearings shall be selected at random for testing in combined compression and shear. The bearings may be tested in pairs. The compressive load shall be the dead load and the bearings shall be subjected to five complete reversed cycles of loading to shear strains of \( \pm 0.5 \).

Test results shall be within \( \pm 10\% \) of those values assumed in design. Bearings that test outside this range may be accepted only on Approval.
4.10.14 Sliding bearings — Design
Sliding bearings may be used for isolation systems if Approved. The Regulatory Authority shall specify appropriate materials and design parameters. The requirements of Clause 11.6 for PTFE bearing surfaces shall be satisfied.

4.10.15 Sliding bearings — Construction
Isolator units that use sliding bearings shall be constructed in accordance with Section 11.

4.11 Seismic evaluation of existing bridges

4.11.1 General
The seismic evaluation of existing bridges shall be conducted by Engineers knowledgeable in the field of earthquake engineering. The Owner or those having jurisdiction shall be responsible for the identification and prioritization of existing bridges that require seismic evaluation.

4.11.2 Bridge classification
The Owner or those having jurisdiction shall classify bridges in accordance with Clause 4.4.2. For lifeline bridges, special studies shall be performed to evaluate their seismic performance. The earthquake level and procedure used for evaluating lifeline bridges shall be specified by the Owner or those having jurisdiction and shall, at a minimum, comply with the requirements of this Code. For emergency-route and other bridges, the requirements of Clause 4.11 shall apply.

4.11.3 Damage levels

4.11.3.1 Moderate damage
Moderate damage is damage that does not cause collapse of a bridge and following which the bridge can be repaired to full strength without full closure; access to emergency vehicles is available almost immediately after the earthquake and limited access to normal traffic is available within a few days.

4.11.3.2 Significant damage
Significant damage is damage that does not cause collapse of a bridge but can take several weeks or months to repair; limited access to emergency and light traffic is sometimes available after a few days, but full service is not available until repairs are completed.

4.11.4 Performance criteria
Emergency-route bridges predicted to withstand the major earthquake with moderate or less damage shall be considered satisfactory. Other bridges predicted to withstand the major earthquake with significant or less damage shall be considered satisfactory.

4.11.5 Evaluation methods

4.11.5.1 Minimum analysis requirements for evaluation
The minimum analysis requirements for evaluation shall be as specified in Table 4.9.
Table 4.9  
Minimum analysis requirements for evaluation  
(See Clause 4.11.5.1.)

<table>
<thead>
<tr>
<th>Seismic performance zone</th>
<th>Single-span bridges</th>
<th>Multi-span bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Emergency-route</td>
<td>Other</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>2</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>3</td>
<td>LE</td>
<td>None</td>
</tr>
<tr>
<td>4</td>
<td>LE</td>
<td>LE</td>
</tr>
</tbody>
</table>

Legend:  
LE = limited seismic evaluation required (see Clause 4.11.5.2)  
MM = multi-mode spectral method (see Clause 4.5.3.3)  
None = no seismic evaluation required  
SM = single-mode spectral method (see Clause 4.5.3.2)

4.11.5.2 Limited evaluation  
Limited evaluation shall require the following:  
(a) Available seat width shall be checked for the minimum requirements specified in Clause 4.4.10.5 or longitudinal restrainers complying with Clause 4.4.10.6 shall be provided.  
(b) Bearings shall be checked for a force demand not less than 20% of the tributary dead load in the restrained directions.  
(c) The potential for soil-liquefaction-induced ground movements, slope instability, approach fill settlements, and increases in lateral earth pressure shall be considered.

4.11.6 Load factors and load combinations for seismic evaluation  
In lieu of specific provisions provided by the Regulatory Authority, seismic evaluation of existing bridges shall be based on the following load factors and load combination:

\[ 1.0D + 1.0EQ \]

Seismic forces and displacements resulting from orthogonal loading shall be combined in accordance with Clause 4.4.9.2.

4.11.7 Minimum support length  
For bridges requiring detailed evaluation, available support lengths at expansion bearings shall be checked for the greater of the maximum displacement calculated in accordance with Clause 4.11.5 or the empirical seat width requirements specified in Clause 4.4.10.5. Alternatively, longitudinal restrainers complying with Clause 4.4.10.6 shall be provided.

4.11.8 Member capacities  
4.11.8.1 General  
For the purposes of Clause 4.11 only, member capacities, \( C \), shall be the unfactored nominal resistance of the member.

4.11.8.2 Material strengths  
Material strengths shall be evaluated in accordance with Clause 14.7.
4.11.8.3 Nominal resistance
For existing structural members meeting all of the design and detailing requirements of this Code, the
nominal resistance shall be calculated in accordance with Clauses 4.7 and 4.8.
For members not meeting all of the design and detailing requirements of this Code, account shall be
taken of the effects of any differences. Differences to be accounted for shall include, but not be limited to,
the following:
(a) For steel members, the influence of the width/thickness (b/t) ratios for local buckling shall be
considered in evaluating their nominal resistance.
(b) Steel members whose slenderness ratios exceed those allowed by this Code shall be considered to act
in tension only unless their behaviour under compression is evaluated based on verified research
results.
(c) For reinforced concrete members with inadequately anchored or spliced steel bars, premature bond
failure shall be considered in evaluating their nominal flexural resistance based on verified research
results.
(d) For reinforced concrete members, the concrete contribution to the shear resistance shall be reduced
as the ductility demand increases in the structural member being evaluated.
(e) Inadequately detailed beam-column joints and column-footing joints shall be checked for shear
capacity.

4.11.8.4 Effects of deterioration
The nominal resistances of existing members shall be reduced to account for any member defects or
deterioration in accordance with Clause 14.14.3.

4.11.9 Required response modification factor
For each structural element and connector, the required response modification factor, \( R_{\text{req}} \), shall be found
such that the following equation is satisfied:

\[
R_{\text{req}} = \frac{S_e}{C}
\]

where

\( S_e \) = seismic force effect assuming all members remain elastic, calculated in accordance with
Clause 4.11.5, except as limited by capacities of other members
\( C \) = member reserve capacity after the effects of dead load have been considered, calculated in
accordance with Clause 4.11.8

4.11.10 Response modification factor of existing substructure elements
The response modification factor provided, \( R_{\text{prov}} \), shall be the \( R \)-factor specified in Clause 4.4.8.1 when all
members and joints satisfy the design and detailing requirements of Clauses 4.7 and 4.8.
For members and joints not so detailed, \( R_{\text{prov}} \) shall be determined from

(a) an assessment of the consequences of specific detailing, and with due consideration for all possible
failure modes and the expected length of inelastic deformations on the overall performance of the
bridge. The selected levels for acceptable response modification factors shall meet the performance
requirements specified in Clause 4.11.4; and

(b) results from reversed-cyclic loading tests of structural components constructed to simulate as-built
details, which provide a means for evaluating the influence of important details and for determining a
suitable \( R_{\text{prov}} \).

4.11.11 Evaluation acceptance criteria
The structural elements and connectors of an existing bridge shall be deemed acceptable for seismic
evaluation if \( R_{\text{prov}} \) is greater than or equal to \( R_{\text{req}} \).
If any element or connector does not meet this requirement, rehabilitation in accordance with
Clause 4.12 shall be carried out unless it can be demonstrated by non-linear analysis that the
consequences would not be detrimental to the performance of the bridge.
4.11.12 Other evaluation procedures
The lateral strength (pushover analysis) and time-history analysis methods may be used for seismic evaluation of existing bridges. Care shall be taken with the modelling of the structure, selection of the input time histories, and interpretation of the analysis results.

4.11.13 Bridge access
The loss of access resulting from an abutment structural failure, adjacent slope failure, or approach fill settlement shall be evaluated for emergency route bridges located in Seismic Performance Zones 3 and 4.

4.11.14 Liquefaction of foundation soils
The potential for liquefaction of the foundations soils shall be evaluated for the following bridges:
(a) bridges located in Seismic Performance Zone 4; and
(b) multi-span bridges classified as emergency-route bridges and located in Seismic Performance Zone 3.
If subsoil liquefaction is likely and the foundation movements are unacceptable, mitigation measures shall be taken (see Clause 4.6).

4.11.15 Soil-structure interaction
When deemed appropriate by the Regulatory Authority, the interaction of soil-structure foundation systems to earthquake loadings shall be evaluated.

4.12 Seismic rehabilitation

4.12.1 Performance criteria
Seismic rehabilitations shall be designed so that a minimum level of safety is provided. This level shall be
(a) comparable to that intended for new bridges; or
(b) as prescribed by the Regulatory Authority.

4.12.2 Response modification factor for rehabilitation
The response modification factor, $R$, for the rehabilitated ductile substructure element shall be determined in accordance with Clause 4.11.10 but shall not exceed the smaller of
(a) the value of $R$ from Table 4.5 corresponding to the type of substructure element; or
(b) 5.0.

4.12.3 Seismic rehabilitation

4.12.3.1 In the design of the rehabilitation measures, the following design aspects shall be accounted for:
(a) the fact that increased stiffness due to strengthening can attract higher seismic loads;
(b) the influence of the rehabilitation measures on fatigue life;
(c) the influence of the rehabilitation measures on alteration of load paths;
(d) the fact that strengthening some members can result in larger force demands on other members (including superstructure members), connections, and foundations;
(e) the need to design rehabilitation measures to prevent damage to inaccessible underground foundations;
(f) restraint on thermal movement due to added restrainers;
(g) the fact that improvement of foundation soils can induce movements or tilting of the substructure;
(h) the need for proper planning of the sequence if rehabilitation is applied in stages;
(i) the need (if applicable) for adequate maintenance and inspection of the rehabilitated structure at regular intervals; and
(j) such other design aspects as are applicable to the rehabilitation measures.
4.12.3.2
In the design of the rehabilitation measures, the following requirements shall apply:
(a) column rehabilitation jackets shall terminate 100 mm from the top of the footing and the bottom of
the cap beam;
(b) if uplift occurs near the base of a structure, care shall be taken to ensure adequate guiding for this
movement. Due consideration shall be given to other effects, e.g., loss of support and impact;
(c) if base isolators are employed, care shall be taken in assessing the structural stability at other limit
state combinations (e.g., wind);
(d) the durability of the rehabilitation measures shall be assessed; and
(e) a complete re-analysis of the rehabilitated structure in both the longitudinal and transverse directions
shall be conducted to assess the performance of the rehabilitated structure.

4.12.4 Seismic rehabilitation techniques
Seismic rehabilitation techniques that have been analytically and experimentally verified shall be used,
subject to Approval.

Note: Seismic rehabilitation techniques include the following:
(a) isolation of ground motion from the structure by “base isolation” bearings or other means;
(b) increasing the ductility of the system with or without strengthening;
(c) introduction of energy-dissipating devices;
(d) installation of restrainers, bumpers, or both between spans;
(e) alteration of load paths;
(f) increasing available support lengths both longitudinally and transversely;
(g) making provision for inelastic hinging to occur;
(h) strengthening;
(i) improvement of liquefaction-prone foundation soils; and
(j) stabilization of approach fills and adjacent slopes.
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Section 5
Methods of analysis

5.1 Scope
This Section specifies analysis requirements for bridges, including but not limited to bridges of the following superstructure types:

(a) slab;
(b) voided slab;
(c) deck-on-girder, including slab-on-girder, steel-grid-deck-on-girder, and wood-deck-on-girder;
(d) shear-connected beam;
(e) truss;
(f) arch;
(g) rigid frame and integral abutment;
(h) bridges incorporating wood beams;
(i) box girder — single cell;
(j) box girder — multi-cell;
(k) box girder — multi-spine;
(l) cable stayed; and
(m) suspension.

Note: In this Section, these are referred to as Type A, Type B, etc.

5.2 Definitions
The following definitions apply in this Section:

Cantilever slab — that portion of a deck slab that lies outside the outermost girder or web or lies outside the outermost lines of support.

Deck — an element of a bridge superstructure that carries and distributes wheel loads to the substructure.

Degree-of-freedom — one of a number of translations or rotations that are required to define the movement of a node.

Distortional stresses — stresses developed in a cross-section because of distortion in its own plane.

Divergence — an aerodynamic instability in torsion that is analogous to column buckling and usually occurs at wind speeds beyond the range normally considered in the design.

Effective width — a reduced width of a flange or deck that enables a member to be proportioned on the basis of uniform stress.

External portion of a bridge — a part of the transverse cross-section of a bridge, as follows:

(a) for a solid slab bridge, the outermost 2.00 m of the transverse cross-section on either side of the bridge;
(b) for a voided slab bridge with rectangular or circular voids, that part of the transverse cross-section on either side of the bridge that is on the outer side of the vertical plane bisecting the outermost void;
(c) for a slab-on-girder bridge, that part of the transverse cross-section on either side of the bridge that is on the outer side of the vertical plane midway between the outermost girder and the girder next to it; and
(d) for a multi-cell or multi-spine box girder bridge, that part of the transverse cross-section on either side of the bridge that is on the outer side of the vertical plane that bisects the area of the outermost cell.
Flexural rigidity — the bending stiffness of a beam, $EI$, or the bending stiffness per unit width or per unit length of an orthotropic plate, $D_x$ or $D_y$, respectively.

Floor beam — a transverse structural member that supports a deck or longitudinal stringers and spans between longitudinal girders, trusses, or arches.

Floor system — that portion of a bridge superstructure that directly supports traffic, including, where present, the deck, floor beams, and stringers.

Grid — a plane assembly of intersecting beams subject to loading perpendicular to the plane formed by the assembly and characterized by the fact that interaction between beams takes place only at their intersections.

Idealization — representation of a structure or load for purposes of analysis or testing.

In-plane forces — forces acting in the plane of an element, member, or system.

Integral abutment bridge — a bridge in which there is no expansion joint between the bridge superstructure and its abutment(s) and structural continuity with the abutment(s) is preserved.

Interface shear — shear between a deck and a supporting beam.

Internal portion of a bridge — that part of the transverse cross-section of a bridge contained between the two external portions.

Large-deflection theory — a theory that assumes that deflections caused by the application of loads alter the behaviour of a structure to the extent that they need to be considered in the analysis of the structure.

Longitudinal direction — the direction of traffic flow.

Longitudinal moment — moment in the longitudinal vertical plane about a transverse axis.

Longitudinal torsion — torsion about a longitudinal axis.

Longitudinal vertical shear — vertical shear in the longitudinal vertical plane associated with change of longitudinal moment.

Mathematical model — a conceptual approximation to a structure for purposes of analysis.

Modification factor — a factor applicable to highway live loads (see Clause 3.8.4.2).

Moment of inertia — the second moment of area of the cross-section of a component with respect to a centroidal axis in the plane of the area, unless otherwise specified in this Section.

Multi-cell bridge — a box girder bridge with three or more webs per box and in which the bottom flange is continuous in the transverse direction.

Multi-spine bridge — a box girder bridge in which the bottom flange is discontinuous in the transverse direction, thus forming spines mutually connected only by the deck slab and transverse diaphragms (if present).

Orthotropic deck — a deck made of steel plates stiffened with open or closed steel ribs welded to the undersides of the steel plates.

Orthotropic plate — a plate with different flexural and torsional rigidities in orthogonal directions.

Radius of curvature — the radius at any point on a longitudinal line joining the centroids of transverse cross-sections, when viewed in plan.
Rigid frame bridge — a bridge in which the piers, abutments, or both are structurally continuous with the longitudinal components of the superstructure.

Shallow superstructure bridge type — one of the bridge types in which in-plane distortions of the transverse cross-section are negligible, i.e., slab, slab-on-girder, voided slab, and shear-connected beam bridge types (but only, in the case of shear-connected beams, if the interconnections of adjacent beams provide continuity of transverse flexural rigidity across the cross-section).

Shear-connected beams — longitudinal beams placed side by side, with connections between adjacent beams, e.g., shear keys, for transferring transverse vertical shear.

Skew angle — the angle between the longitudinal centreline of a bridge and a line normal to the centreline of bearings.

Skew span (for a bridge in which the plan-form is a parallelogram) — the length of the unsupported edge.

Skew width (for a bridge in which the plan-form is a parallelogram) — the width of the deck parallel to the supported edge.

Small-deflection theory — analysis based on the assumption that deflections caused by the application of loads do not have a significant effect on the accuracy of the analysis and can therefore be ignored in the calculation of force effects.

Span — the length between the supports of a bridge, taken along the line of the main supporting members.

Spine — a portion of a bridge cross-section that comprises a portion of the deck, two webs, and a lower flange and thereby constitutes a closed box.

Stringers — longitudinal structural members spanning between floor beams.

Transverse direction — the direction perpendicular to the longitudinal direction at any point on a bridge.

Transverse moment — moment in the transverse vertical plane about a longitudinal axis.

Transverse torsion — torsion about a transverse axis.

Voided slab — a concrete slab with circular or rectangular voids described in Clause 5.5.2 and geometrical and structural configurations that make the effects of cell distortion negligible.

Warping stresses — in-plane stresses that are developed in a cross-section and are due to restraint of the in-plane displacements associated with warping of the cross-section’s components.

Width — the following distances:
(a) for a bridge in which the unsupported edges are parallel, the perpendicular distance between the unsupported edges of the bridge; and
(b) for a bridge in which the unsupported edges are not parallel, the distance between the unsupported edges along a line perpendicular to the centreline of the bridge at the point of consideration.
5.3 Abbreviations and symbols

5.3.1 Abbreviations
The following abbreviations apply in this Section:
FLS — fatigue limit state
SLS — serviceability limit state
ULS — ultimate limit state

5.3.2 Symbols
The following symbols apply in this Section:
- \( A \) = factor for calculating cantilever moments in deck slabs (see Figure 5.2)
- \( A_s \) = total area of stiffeners in width \( B \), as shown in Figure 5.6 for orthotropic steel decks
- \( A_x \) = equivalent area per unit width of the transverse section of a voided slab
- \( A_y \) = equivalent area per unit length of the longitudinal section of a voided slab
- \( a \) = length of the longer side of a rectangular section; distance between adjacent transverse bracing members; for orthotropic steel decks, the distance centre-to-centre of longitudinal ribs (see Table 5.11)
- \( B \) = width of a bridge; for orthotropic steel decks, the spacing of transverse or longitudinal beams, as shown in Figure 5.6
- \( B_e \) = a reduced value of \( B \)
- \( B_l \) = for orthotropic steel decks, the length of the cantilever
- \( b \) = length of the shorter side of a rectangular section; one-half of the transverse span of a deck slab, as shown in Figure 5.5
- \( b_e \) = a reduced value of \( b \)
- \( C \) = transverse distance of the wheel load from the supported edge of a cantilever slab, m; modification factor for simplified analysis of skewed slab-on-girder bridges (see Table CA5.1.3 of CSA S6.1)
- \( C_e \) = correction factor used to adjust the \( F \) value for longitudinal moment to account for the vehicle edge distance
- \( C_l \) = correction factor used to adjust the \( F \) value for longitudinal moment and longitudinal vertical shear
- \( D_{VE} \) = vehicle edge distance for slab-on-girder bridges, m, as shown in Figure 5.1
- \( D_t \) = distribution width for transverse moment in timber decks
- \( D_x \) = longitudinal flexural rigidity per unit width
- \( D_{xy} \) = longitudinal torsional rigidity per unit width
- \( D_y \) = transverse flexural rigidity per unit length
- \( D_{yx} \) = transverse torsional rigidity per unit length
- \( D_1 \) = coupling rigidity per unit width
- \( D_2 \) = coupling rigidity per unit length
- \( DLA \) = dynamic load allowance, as defined in Clause 3.2
- \( ds \) = for box girder cross-sections, the length of each side of thickness \( t \)
- \( E \) = modulus of elasticity; a distribution width for one line of wheels for steel grid decks spanning longitudinally
- \( E_L \) = modulus of elasticity of wood in the direction \( L \) shown in Figure A5.2.2
- \( E_T \) = modulus of elasticity of wood in the direction \( T \) shown in Figure A5.2.2
- \( E_c \) = modulus of elasticity of concrete
\( E_s \) = modulus of elasticity of steel
\( e \) = for orthotropic steel decks, the distance centre-to-centre between adjacent closed ribs
\( F \) = when a bridge superstructure is analyzed in accordance with simplified methods, a width dimension that characterizes load distribution for a bridge, as specified in Clauses 5.7.1.2 to 5.7.1.5 and 5.7.1.6.2, m
\( F_m \) = when a bridge superstructure is analyzed in accordance with simplified methods, an amplification factor to account for the transverse variation in maximum longitudinal moment intensity, as compared to the average longitudinal moment intensity
\( F_v \) = when a bridge superstructure is analyzed in accordance with simplified methods, an amplification factor to account for the transverse variation in maximum longitudinal vertical shear intensity, as compared to the average longitudinal vertical shear intensity
\( G \) = shear modulus
\( G_{LT} \) = shear modulus of wood with respect to axes \( L \) and \( T \) shown in Figure A5.2.2
\( G_c \) = shear modulus of concrete
\( G_s \) = shear modulus of steel
\( I \) = moment of inertia of the cross-section of a beam
\( i_L \) = longitudinal moment of inertia per unit width
\( i_T \) = transverse moment of inertia per unit length
\( J \) = torsional inertia of a beam
\( j_L \) = longitudinal torsional inertia per unit width
\( j_T \) = transverse torsional inertia per unit length
\( K \) = torsional constant for a rectangular section
\( k \) = constant used in calculating maximum transverse vertical shear intensity due to live load in shear-connected beam bridges (see Clause 5.7.1.8.1)
\( L \) = for simply supported spans, the span; for continuous spans, the span specified in Clause A5.1.2
\( L_s \) = stringer span
\( \ell_1, \ell_2 \) = distances between points of inflection, as shown in Figure 5.6 for orthotropic steel decks
\( M_T \) = maximum longitudinal moment for one lane width of truck or lane loading, as applicable, including dynamic load allowance
\( M_g \) = for girder-type bridges, the maximum longitudinal moment per girder due to live load, including the effects of amplification for transverse variation in maximum longitudinal moment intensity and dynamic loading
\( M_{g,avg} \) = average moment per girder due to live load, determined by sharing equally the total live load moment on the bridge cross-section among all girders in the cross-section
\( M_x \) = longitudinal bending moment per unit width
\( M_{xy} \) = longitudinal torsional moment per unit width
\( M_y \) = transverse bending moment per unit length
\( m \) = for bridges of the solid cross-section type, e.g., slabs, voided slabs, and wood deck bridges that span longitudinally, the maximum longitudinal moment per metre of width due to live load
\( m_{avg} \) = for bridges of the solid cross-section type, e.g., slabs, voided slabs, and wood deck bridges that span longitudinally, the average longitudinal moment per metre of width due to live load
\( N \) = number of girders or longitudinal wood beams in the bridge deck width, \( B \)
\( n \) = modular ratio, \( E_s/E_c \), for steel and concrete; modular ratio of girder or beam material to slab material; number of design lanes on a bridge
\( P \) = the 87.5 kN wheel load of the CL-625 Truck
\( R \) = mean radius of curvature of the curved portion of a bridge that is curved in plan
\( R_l \) = modification factor for multi-lane loading in accordance with Clause 3.8.4.2
\( r_t \) = ratio of the thickness, \( t_1 \), of the deck slab at the exterior edge of a bridge deck to the thickness, \( t_2 \), at the edge of the flange of the external girder
\( S \) = centre-to-centre spacing of longitudinal girders of a deck-on-girder bridge; centre-to-centre spacing of circular voids of a voided slab bridge; centre-to-centre spacing of spines of a multi-spine bridge
\( S_c \) = transverse distance from the longitudinal external edge of a bridge deck to the supported edge of cantilevered slabs located at the outside face of the web of the external girder
\( S_e \) = equivalent span of concrete deck in (see Clause 5.7.1.7.1)
\( S_y \) = transverse shear rigidity per unit length
\( s_y \) = shear area per unit length, as specified in Clause A5.2.1
\( t \) = overall thickness of a slab
\( t_v \) = depth of circular or rectangular void in voided slabs
\( t_t \) = slab thickness at the external edge of the deck slab
\( t_2 \) = slab thickness at the edge of the flange of the external girder
\( V_T \) = maximum longitudinal vertical shear for one lane width of truck or lane loading, as applicable, including dynamic load allowance
\( V_g \) = for girder-type bridges, the maximum longitudinal vertical shear per girder due to live load, including the effects of amplification for transverse variation in maximum longitudinal vertical shear intensity and dynamic loading
\( V_g,_{avg} \) = average shear per girder due to live load, determined by sharing equally the total live load shear on the bridge cross-section among all girders in the cross-section
\( V_y \) = maximum intensity of transverse vertical shear in shear-connected beam bridges
\( \nu \) = for slab, voided slab, and wood deck bridges that span longitudinally, the maximum longitudinal vertical shear per metre of width due to live load, including the effects of amplification for transverse variation in maximum longitudinal vertical shear intensity and dynamic loading
\( \nu_{avg} \) = average shear per metre of width due to live load for slab, voided slab, and wood deck bridges that span longitudinally, determined by sharing uniformly the total live load shear on the bridge cross-section over the width of the bridge cross-section
\( W_e \) = width of a design lane, m
\( \psi \) = skew angle; for orthotropic steel decks, the effective plate width factor for interior portions of the deck, as shown in Figure 5.6
\( \psi_p \) = for orthotropic steel decks, the effective plate width factor for exterior portions of the deck, as shown in Figure 5.6

\( x, y \) = coordinates of a reference point on a cantilever slab, as shown in Figure 5.2
\( \beta \) = parameter specified in Clause 5.7.1.3
\( \epsilon \) = skew parameter specified in Clause 5.6.1.1
\( \mu \) = lane width modification factor
\( \nu \) = Poisson's ratio
5.4 General requirements

5.4.1 Application
The requirements of Clauses 5.4.2 to 5.4.13 shall apply to bridges of all types, subject to the requirements of Clause 5.5 for certain types of short- and medium-span bridges and Clause 5.10 for long-span bridges.

5.4.2 Analysis for limit states
All bridges and bridge elements shall be modelled in such a manner that their analysis accurately predicts their behaviour at each relevant limit state.

For the purpose of analysis, materials shall be treated as elastic unless otherwise permitted in this Section or Approved. The elastic properties and characteristics of the materials shall be determined in accordance with Sections 8 to 10 and 16.

Inelastic properties of materials may be used for analysis of structures designed to resist
(a) ship impact;
(b) earthquake at ultimate limit states; and
(c) accidental collision forces.

For inelastic behaviour, materials shall be deemed to have residual strength when strained past their elastic limit only when their behaviour is known to be ductile or, in the case of concrete, when adequate confinement reinforcement is provided in accordance with Section 4.

5.4.3 Modelling

5.4.3.1 General
The geometry and structural characteristics of a bridge shall be modelled in such a manner that an analysis based on the model accurately reflects the behaviour of the bridge.

Small-deflection theory or large-deflection theory shall be used for the analysis as applicable and as required by Clauses 5.4.3.2 and 5.4.3.3, respectively.

The plan geometry requirements of Clause A5.1.3 shall also apply to bridge geometry modelling.

5.4.3.2 Small-deflection theory
Beams, girders, trusses, braced frames, grillages, slabs, and connections designed in accordance with this Code shall be considered structures to which small-deflection theory applies. Accordingly, the effects of deflection on analysis of the structural system may be ignored.

5.4.3.3 Large-deflection theory
Arch bridges, suspension bridges, cable-stayed bridges, catenaries, and frames where sidesway is permitted by this Code shall be analyzed using large-deflection theory unless analysis or past experience with similar structures indicates that small-deflection theory is adequate.

5.4.4 Structural responses
Type A to K bridges (see Clause 5.1) shall be analyzed for the relevant structural responses specified in Table 5.1. Bridges of other types shall be analyzed for all of the responses specified in Table 5.1. Deformations shall be calculated in accordance with Clause 5.4.6.
### Table 5.1 Structural responses

(See Clauses 5.4.4, 5.4.7, 5.5.2, 5.5.4, 5.5.5.1, 5.5.5.2, 5.5.7, and A5.1.5.)

<table>
<thead>
<tr>
<th>Bridge type</th>
<th>Longitudinal moment</th>
<th>Transverse moment</th>
<th>Longitudinal torsion</th>
<th>Longitudinal vertical shear</th>
<th>Transverse vertical shear</th>
<th>In-plane forces</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Slab</td>
<td>X</td>
<td>X</td>
<td></td>
<td>X</td>
<td>X†</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>B. Voided slab</td>
<td>X</td>
<td>X</td>
<td>X†</td>
<td>X</td>
<td>X†</td>
<td>X‡</td>
<td>Voids shall meet the requirements specified in Clause 5.5.2</td>
</tr>
<tr>
<td>C. Deck-on-girder</td>
<td>X</td>
<td>X</td>
<td>X†</td>
<td>X</td>
<td>X†</td>
<td>X‡</td>
<td>Longitudinal moment shall be taken by girder only or girder plus slab, depending on whether construction is non-composite or composite, respectively. Transverse moment taken by deck only (need not be considered if the deck is a slab designed in accordance with the empirical method specified in Clause 8.18.4). The bridge shall be treated as A, B, or C, as applicable.</td>
</tr>
<tr>
<td>D. Shear-connected beam</td>
<td>X</td>
<td>X</td>
<td>X†</td>
<td>X</td>
<td>X§</td>
<td>X‡</td>
<td>Longitudinal moment and longitudinal vertical shear shall be calculated as for multi-spine bridges of concrete construction</td>
</tr>
<tr>
<td>E. Truss</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>The requirements of Clause 5.5.4 shall apply</td>
</tr>
<tr>
<td>F. Arch</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>The requirements of Clause 5.5.4 shall apply</td>
</tr>
</tbody>
</table>

(Continued)
<table>
<thead>
<tr>
<th>Bridge type</th>
<th>Longitudinal moment</th>
<th>Transverse moment</th>
<th>Longitudinal torsion</th>
<th>Longitudinal vertical shear</th>
<th>Transverse vertical shear</th>
<th>In-plane forces</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>G. Rigid frame and integral abutment</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>The requirements of Clause 5.5.4 shall apply</td>
</tr>
<tr>
<td>Members supporting floor system</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor system</td>
<td>—</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>XX</td>
<td></td>
</tr>
<tr>
<td>H. Incorporating wood beams</td>
<td>X</td>
<td>X††</td>
<td>X††</td>
<td>X</td>
<td>X§§</td>
<td>XX</td>
<td>Live load deflections shall be considered and the requirements of Section 9 shall be followed</td>
</tr>
<tr>
<td>I. Box girder — Single cell***</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>J. Box girder — Multi-cell***</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>K. Box girder — Multi-spine***</td>
<td>X</td>
<td>X†</td>
<td>X††</td>
<td>X</td>
<td>X‡‡</td>
<td>X§§</td>
<td></td>
</tr>
</tbody>
</table>

*Not required for dead load if the requirements of Clause 5.6.1.1 are met.
†Not required for live load if the requirements of Clause 5.7.1.1 are met.
‡ Applies only to prestressing.
§ Applies only to live load.

**Using appropriate orientation, treat similarly to Type A, B, C, D, or H.
†† Applies only to deck-on-stringer type.
‡‡ Applies only to wood plank decks under live load.
§§ Applies only to prestressed wood decks.

The requirements of Clause 5.5.7 shall apply. Torsional warping need be considered only in the case of steel and steel-composite construction. For concrete box girders, torsional warping need not be considered.

Distortional warping need not be considered in the case of concrete or steel-composite construction if
(a) adequate diaphragms or cross-frames are present over the supports;
(b) in addition to those over the supports, a minimum of two intermediate diaphragms per span are present for concrete construction and three per span for steel-composite construction; and
(c) adjacent diaphragms are spaced not more than 18 m apart in the case of concrete construction and not more than 12 m apart in the case of steel-composite construction.

**Note:** An “X” indicates that the response shall be considered.
5.4.5 Factors affecting structural responses
In deriving the appropriate structural responses, all relevant factors in the design, both geometric and non-geometric, shall be taken into account, including, but not limited to, the following:

(a) continuity of spans;
(b) plan geometry, including skewness and curvature;
(c) edge stiffening;
(d) longitudinal variation of transverse section;
(e) transverse variation of longitudinal section;
(f) diaphragms and cross-frames;
(g) wind bracing;
(h) interaction of floor system and its support system;
(i) barrier and parapet walls;
(j) support boundary conditions;
(k) movement of supports;
(l) temperature;
(m) creep, shrinkage, and relaxation;
(n) elastic shortening; and
(o) construction sequence.

The factors specified in this Clause shall be taken account of in accordance with Annex A5.1, as applicable.

5.4.6 Deformations

5.4.6.1 General
Deformations shall be calculated using material properties specified in Sections 8 to 10 and 16, as applicable. Deformations due to prestress shall be taken into account in accordance with the requirements of Clause 5.4.11.

5.4.6.2 Dead load deflections
In the calculation of dead load deflection, the effects of construction sequence, prestressing, creep, and shrinkage shall be considered. The simplified method specified in Clause 5.6.1.2 may be used for bridges meeting the requirements of Clause 5.6.1.1.

5.4.6.3 Live load deflections
In the absence of a more rigorous analysis, live load deflections of a bridge that satisfy the requirements of Clause 5.7.1.1 may be determined as follows:
(a) Regardless of the transverse position of the live load, the average deflection of the transverse cross-section of the bridge may be determined by treating the bridge as a beam and calculating the deflection of the beam due to flexure.
(b) For compliance with the deflection limitation requirements of Clause 3.4.4, the maximum deflection of the transverse cross-section of a bridge of shallow superstructure may be estimated using the method specified in Clause 5.7.1.2.2.

The maximum deflection of any point on the transverse cross-section of a multi-spine box girder bridge may be estimated using the same method specified in Clause 5.7.1.2.2, in accordance with the additional requirements of Clause 5.7.1.3.

5.4.7 Diaphragms and bracing systems
Diaphragms and bracing systems shall comply with the applicable requirements of Sections 8 to 10, Table 5.1, and Clause A5.1.5.
5.4.8 Analysis of deck slabs
Concrete deck slabs that are proportioned in accordance with Clause 8.18.4 need not be analyzed, except that the cantilever portions of a deck slab that are outside the outermost girders shall be analyzed in accordance with Clause 5.7.1.6.1. Concrete deck slabs not designed in accordance with Clause 8.18.4 shall be analyzed by elastic methods, except that at the ULS a yield line method may be used in lieu of elastic analysis.

Deck components, other than monolithic concrete slabs designed in accordance with Clause 8.18.4, may be analyzed using elastic methods or the simplified methods specified in Clause 5.7.1.7.

5.4.9 Analysis for redistribution of force effects
The effect of creep and shrinkage on redistribution of force effects shall be considered.

5.4.10 Analysis for accumulation of force effects due to construction sequence
The accumulation of force effects due to the construction sequence shall be considered. For calculation of force effects at a particular stage in the construction sequence, elastic methods shall be used and the material properties shall be those appropriate to that stage of construction.

5.4.11 Analysis for effects of prestress
Force effects arising from prestress, including secondary force effects in statically indeterminate structures, shall be taken into account.

5.4.12 Analysis for thermal effects
Thermal effects shall be included in the analysis, as specified in Clause 3.9.

5.4.13 Secondary stability effects
Secondary effects related to overall structural stability and member stability shall be included in the analysis, in accordance with Clause 5.12 and, as applicable, Sections 8 to 10 and 16.

5.5 Requirements for specific bridge types

5.5.1 General
For Type A to K bridges (see Clause 5.1), the general requirements specified in Clause 5.4 shall be subject to the requirements specified in Clauses 5.5.2 to 5.5.8. For other types of bridges, engineering judgment shall be used to determine what further requirements apply.

5.5.2 Voided slab — Limitation on size of voids
If a bridge is to be treated as a voided slab type, the limiting size of the voids shall be as follows:
(a) For circular voids, the diameter of the void shall not exceed 80% of the total depth of the slab, and the spacing of the voids, centre-to-centre, shall not be less than the total depth of the slab.
(b) For rectangular voids, the thickness of the web defined by adjacent voids shall be not less than 20% of the total depth of the section. The depth of the void shall not exceed 80% of the total depth of the section and the transverse width of the void shall not exceed 1.5 times its depth.

Where the voids do not comply with the requirements of Item (a) or (b), as applicable, the bridge shall be treated as being of the multi-cell box girder type unless adequate diaphragms are provided to prevent cell distortion in accordance with Table 5.1, in which case the bridge may be treated as a voided slab.
5.5.3 Deck-on-girder
For the structural responses of deck-on-girder bridges, the following shall apply:
(a) The longitudinal moment shall be assumed to be resisted only by the girders if the structural action is non-composite, and by the girders plus an appropriate portion of the deck if the structural action is composite.
(b) The transverse moment shall be assumed to be taken by the deck; the action of cross-bracing or diaphragms, if present, may be taken into account.

5.5.4 Truss and arch
For the structural responses of truss and arch bridges, the following shall apply:
(a) The floor system shall be analyzed for the structural responses specified in Table 5.1, with the exception of longitudinal torsion, for which analysis shall be optional.
(b) Axial forces for each member of the truss or arch shall be considered, including the effects of axial offset or eccentricity at panel points.
(c) The overall stability of the truss shall be considered.
(d) The stability of the compression chord of a pony truss shall be considered.
(e) In-plane and out-of-plane buckling of components shall be considered.
(f) In the analysis of long-span arches, the deflected shape of the structure shall be used in the formulation of equilibrium. Short- to medium-span arches may be analyzed using magnification correction methods.

5.5.5 Rigid frame and integral abutment types
5.5.5.1 Rigid frame
For rigid frame bridges, the analysis for structural responses shall meet the requirements for slab, voided slab, deck-on-girder, or truss and arch bridges (as applicable) specified in Clauses 5.5.2 to 5.5.4 and Table 5.1, and shall also include in-plane forces and bending moments induced by frame action.

5.5.5.2 Integral abutment
For integral abutment bridges, the analysis for structural responses shall meet the requirements for slab and deck-on-girder bridges (as applicable) specified in Clauses 5.5.3 and Table 5.1. The negative moment region in proximity to an integral abutment shall be established by the Engineer using appropriate methods of elastic analysis. In-plane forces and bending moments induced by frame action occurring after establishment of continuity between the superstructure and the substructure shall be included in the analysis.

5.5.6 Transverse wood deck
For bridges incorporating laminated wood decks spanning transversely between longitudinal girders or stringers, only the transverse moment need be analyzed.

5.5.7 Box girder
For the structural responses of box girder bridges, the following shall apply:
(a) For dead load, all of the structural responses specified in Table 5.1 shall be considered. The warping and distortional effects introduced during steel and steel-composite construction shall also be considered, as shall the distortional effects that occur during construction in concrete box girders that are temporarily of open section.
(b) For live load, all of the structural responses specified in Table 5.1 shall be considered. Warping and distortional effects shall be included in the analysis unless the bridge complies with the requirements of Clause 5.7 for the purposes of simplified analysis and
(i) in the case of concrete and steel-composite construction, the requirements of Table 5.1 regarding diaphragms or cross-frames are met; and
(ii) in the case of composite steel box girders of the multi-spine type, the requirements of Clauses 10.12.5.1 and 10.12.7 are met.
5.5.8 Single-spine bridges
A torsionally stiff, closed section single-girder superstructure may be idealized for global force effects as a single-spine beam if the ratio of the span length to the width exceeds 2.5. The width to be used in applying this criterion shall be the average distance between the outside faces of the exterior webs and the span length shall be taken as follows:
(a) for simply supported bridges, the length between bearing lines; and
(b) for continuous and/or skewed bridges, the length of the longest side of the rectangle that can be drawn within the plan view of the width of the smallest span.
A horizontally curved, torsionally stiff single-spine superstructure may be analyzed for global force effects as a curved spine beam. The location of the centreline of such a beam shall be taken at the centre of the cross-section and the eccentricity of dead loads shall be taken into account.

5.6 Dead load
5.6.1 Simplified methods of analysis (beam analogy method)

5.6.1.1 Conditions for use
For dead load analysis, the beam analogy method specified in Clause 5.6.1.2 may be used for bridges satisfying the following conditions:
(a) the width is constant;
(b) the support conditions are closely equivalent to line support at the ends of the bridge and, in the case of multi-span bridges, at intermediate supports;
(c) for slab and voided slab bridges, the skew parameter, $\varepsilon = B \tan \frac{\psi}{L}$, does not exceed 1/6, and for slab-on-girder bridges built with shored construction, the skew parameter, $\varepsilon = S \tan \frac{\psi}{L}$, does not exceed 1/18, where $S$ is the centre-to-centre spacing of longitudinal web lines or girders and $B$ is the width of the bridge;
Note: For slab-on-girder bridges built with unshored construction, no limitation on the value of the skew parameter, $\varepsilon$, applies.
(d) for bridges that are curved in plan and built with shored construction, the radius of curvature, span, and width satisfy the requirements of Clause A5.1.3.2;
(e) slab and voided slab bridges are
   (i) of substantially uniform depth across a transverse section; or
   (ii) tapered in the vicinity of a free edge, provided that the length of the taper in the transverse direction does not exceed 2.5 m; and
(f) for a bridge with longitudinal girders and an overhanging deck slab, the overhang is not more than 1.80 m and does not exceed 60% of the mean spacing between the longitudinal girders or, for box girder bridges, 60% of the spacing of the two outermost adjacent webs.
When these conditions are not fully met, engineering judgment shall be used to determine whether the bridge satisfies these conditions to an extent sufficient for the beam analogy method to apply.

5.6.1.2 Description of method
A bridge satisfying the requirements of Clause 5.6.1.1 shall be treated as a beam for the purpose of determining longitudinal moments, longitudinal vertical shears, and deflections due to dead load. The whole of the bridge superstructure, or that part of the bridge superstructure contained between two parallel vertical planes running in the longitudinal direction, may be considered. The dead load of the cast deck and superimposed dead load shall be distributed in accordance with engineering judgment and in a manner that satisfies overall equilibrium.

Any element of superstructure in which the load is carried mainly by flexure in one direction, including but not limited to the following, may be analyzed as a beam:
(a) a rectangular slab supported along two opposite edges only;
(b) a rectangular slab supported along three or four edges and with a width more than twice the span;
(c) floor beams and associated portions of a deck; and
(d) stringers and associated portions of a deck.

5.6.2 Refined methods of analysis
For short- and medium-span bridges, a refined method of analysis in accordance with Clause 5.9 shall be used when the requirements of Clause 5.6.1 are not met. In cases where the requirements of Clause 5.6.1 are met, a refined method in accordance with Clause 5.9 may nevertheless be used.

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

5.7 Live load

5.7.1 Simplified methods of analysis

5.7.1.1 Conditions for use
For live load analysis, the simplified methods of Clauses 5.7.1.2 to 5.7.1.5 may be used for bridges satisfying the following conditions:

(a) the width is constant;
(b) the support conditions are closely equivalent to line support at the ends of the bridge and, in the case of multi-span bridges, at intermediate supports;
(c) for slab bridges and slab-on-girder bridges with skew, the skew parameter requirements of Clause A5.1.3.1 are met;
(d) for bridges that are curved in plan, the radius of curvature, span, and width satisfy the applicable requirements of Clause A5.1.3.2;
(e) slab and voided slab bridges are
   (i) of substantially uniform depth across a transverse section; or
   (ii) tapered in the vicinity of a free edge, provided that the length of the taper in the transverse direction does not exceed 2.5 m;
(f) for slab-on-girder bridges, there are at least three longitudinal girders of equal flexural rigidity and equal spacing, or with variations from the mean in rigidity and spacing of not more than 10% in each case;
(g) for multi-spine bridges, there are at least two longitudinal girders of nearly equal flexural rigidity and equal spacing;
(h) for a bridge with longitudinal girders and an overhanging deck slab, the overhang is not more than 1.80 m and does not exceed 60% of the mean spacing between the longitudinal girders or, for box girder bridges, the spacing of the two outermost adjacent webs;
(i) for a continuous span bridge, the requirements of Clause A5.1.2 are met;
(j) for multi-spine bridges, each spine has only two webs, and, for steel and steel-composite multi-spine bridges, the requirements of Clause 10.12.5.1 are met; and
(k) for multi-spine bridges, the number of lanes is larger than or equal to the number of spines and $B/L$ is less than or equal to 1.0.

When these conditions are not fully met, engineering judgment shall be used to determine whether the bridge satisfies these conditions to an extent sufficient for the appropriate simplified method to apply.

For the purpose of using simplified methods of analysis for live load, superstructure types shall be categorized as specified in Table 5.2.
# Table 5.2
Superstructure categories for simplified methods of analysis for live load
(See Clause 5.7.1.1.)

<table>
<thead>
<tr>
<th>Category</th>
<th>Type of superstructure</th>
<th>Applicable Clauses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow superstructure</td>
<td>Slab</td>
<td>5.7.1.2 (longitudinal moment)</td>
</tr>
<tr>
<td></td>
<td>Voided slab that complies with Clause 5.5.2*</td>
<td>5.7.1.4 (longitudinal vertical shear)</td>
</tr>
<tr>
<td></td>
<td>Slab-on-girder</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel-grid-deck-on-girder</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Wood-deck-on-girder</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Wood deck on longitudinal wood beam</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stress-laminated wood deck bridge spanning longitudinally</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Longitudinal nail-laminated wood deck bridge</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Longitudinal laminates of wood-concrete composite decks</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shear-connected-beam bridge in which the interconnection of adjacent beams is such as to provide continuity of transverse flexural rigidity across the cross-section.</td>
<td>5.7.1.2 (longitudinal moment)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7.1.4 (longitudinal vertical shear)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7.1.8 (transverse vertical shear)</td>
</tr>
<tr>
<td>Multi-spine bridge</td>
<td>Box girder bridge in which the boxes are connected only by the deck slab and transverse diaphragms, if present. The bottom flanges of the boxes are discontinuous.</td>
<td>5.7.1.3 (longitudinal moment)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7.1.5 (longitudinal vertical shear)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7.1.8 (transverse vertical shear)</td>
</tr>
<tr>
<td></td>
<td>Shear-connected beam bridge in which the interconnection of adjacent beams is such as not to provide continuity of transverse flexural rigidity across the cross-section.</td>
<td>5.7.1.3 (longitudinal moment)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7.1.5 (longitudinal vertical shear)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7.1.8 (transverse vertical shear)</td>
</tr>
</tbody>
</table>

*Multi-cell box girders with diaphragms in accordance with Clause 5.4.4 may be treated as voided slabs for the purposes of simplified methods of analysis; otherwise, a refined method of analysis in accordance with Clause 5.9 shall be used.

## 5.7.1.2 Longitudinal bending moments in shallow superstructures

### 5.7.1.2.1 Longitudinal bending moments for ultimate and serviceability limit states

For a bridge classified as a shallow superstructure and satisfying all of the applicable conditions listed in Clause 5.7.1.1, the method specified in Clause 5.7.1.2.1.2 may be used for obtaining governing live load moments in the internal and external portions of the bridge in the absence of a more refined method.
5.7.1.2.1.2

Longitudinal bending moment diagrams shall be obtained by treating the bridge as a beam for two load cases. The first load case shall comprise one truck consisting of two lines of wheels as specified in Clause 3.8.3.2, multiplied by the appropriate factor \(1 + DLA\) specified in Clause 3.8.4.5. The second load case shall comprise the lane load specified in Clause 3.8.3.3. The governing moments per design lane thus obtained shall be designated \(M_T\).

The following requirements shall also apply:

(a) For girder-type bridges and bridges with longitudinal wood beams, the longitudinal moment per girder, \(M_g\), shall be calculated as follows:

\[
M_g = F_m M_{g,\text{avg}}
\]

where

\[
F_m = \text{amplification factor to account for the transverse variation in maximum longitudinal moment intensity, as compared to the average longitudinal moment intensity}
\]

\[
= \frac{SN}{F \left[1 + \frac{\mu CF}{100}\right]} \geq 1.05
\]

where

\[
S = \text{centre-to-centre girder spacing, m}
\]

\[
F = \text{width dimension that characterizes load distribution for a bridge, m}
\]

\[
\left[1 + \frac{\mu CF}{100}\right] = \text{lane width correction factor}
\]

\[
\mu = \frac{W_e - 3.3}{0.6} \leq 1.0
\]

where

\[
W_e = \text{width of design lane, m, calculated in accordance with Clause 3.8.2}
\]

\[
C_f = \text{percentage correction factor obtained from Table 5.3}
\]

For bridges with \(S_c\) greater than 0.5, \(F_m\) for external girders shall be multiplied by 1.05.

\[
M_{g,\text{avg}} = \text{average moment per girder due to live load determined by sharing equally the total moment on the bridge cross-section among all girders in the cross-section}
\]

\[
= \frac{nM_T R_L}{N}
\]

where

\[
n = \text{number of design lanes in accordance with Clause 3.8.2}
\]

\[
M_T = \text{maximum moment per design lane at the point of the span under consideration, as specified in this Clause}
\]

\[
R_L = \text{modification factor for multi-lane loading in accordance with Clause 3.8.4.2 or 14.9.4.2}
\]

\[
N = \text{number of girders or longitudinal wood beams in the bridge deck width, B}
\]

\[
F = \frac{F_4 n R_L}{2.80}
\]
where

\[ F_4 = \text{value of } F \text{ for four design lanes obtained from Table 5.3} \]

and the value for \( C_f \) obtained from Table 5.3 for \( n = 4 \) shall be used without modification.

(b) For slab bridges, voided slab bridges, and wood deck bridges that span longitudinally, the moment per metre of width, \( m \), shall be calculated as follows:

\[ m = F_m m_{\text{avg}} \]

where

\[ F_m = \frac{B}{F \left[ 1 + \frac{\mu C_f}{100} \right]} \geq 1.05 \]

where

\[ B = \text{total width of bridge, regardless of whether tapered edges are present} \]

\[ m_{\text{avg}} = \frac{nM_r R_l}{B_e} \]

where

\[ B_e = \text{effective width of the bridge, calculated by reducing the total width, } B, \text{ for the effects} \]

\[ \text{of tapered edges, if present, as specified in Clause A5.1.4} \]

\( F \) and \( C_f \) shall be obtained from Table 5.3 and \( F \) shall be modified for bridges with more than four design lanes in accordance with Clause 5.7.1.2.1.2(a).

**Table 5.3**

\textit{F and C\textsubscript{f} for longitudinal bending moments in shallow superstructures corresponding to ultimate and serviceability limit states}

(See Clauses 5.7.1.2.1.2 and 5.7.1.2.2.2.)

<table>
<thead>
<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>Highway class (see Clause 1.4.2.2) and applicability</th>
<th>No. of design lanes</th>
<th>Portion</th>
<th>( 3 , m &lt; L \leq 10 , m )</th>
<th>( L &gt; 10 , m )</th>
<th>( C_f, % )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type \textit{A or B} \( \text{or for design of new bridges or evaluation} )</td>
<td>\textit{A or B} \( \text{for design of new bridges or evaluation} )</td>
<td>1</td>
<td>External</td>
<td>3.80 + 0.04L</td>
<td>4.20</td>
<td>16 – (36/L)</td>
</tr>
<tr>
<td>2</td>
<td>Internal</td>
<td>4.00 + 0.04L</td>
<td>4.40</td>
<td>16 – (36/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>External</td>
<td>7.10</td>
<td>7.10</td>
<td>20 – (40/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Internal</td>
<td>7.60 – (6/L)</td>
<td>7.30 – (3/L)</td>
<td>20 – (40/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>\textit{C or D} \( \text{for evaluation only} )</td>
<td>\textit{A or B} \( \text{for design of new bridges or evaluation} )</td>
<td>1</td>
<td>External</td>
<td>7.90 + 0.21L</td>
<td>10.80 – (8/L)</td>
<td>16 – (30/L)</td>
</tr>
<tr>
<td>2</td>
<td>Internal</td>
<td>5.90 + 0.41L</td>
<td>10.80 – (8/L)</td>
<td>16 – (30/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>External</td>
<td>10.10 + 0.26L</td>
<td>14.30 – (16/L)</td>
<td>16 – (30/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Internal</td>
<td>7.40 + 0.56L</td>
<td>14.00 – (10/L)</td>
<td>16 – (30/L)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(Continued)
<table>
<thead>
<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>No. of design lanes</th>
<th>Portion</th>
<th>Highway class (see Clause 1.4.2.2) and applicability</th>
<th>$F$, m</th>
<th>$3 \ m \leq L \leq 10 \ m$</th>
<th>$L &gt; 10 \ m$</th>
<th>$C_f$, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type C slab-on-girder</td>
<td>1</td>
<td>External</td>
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Table 5.3 (Continued)

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<th>Highway class (see Clause 1.4.2.2) and applicability</th>
<th>No. of design lanes</th>
<th>Portion</th>
<th>3 m &lt; L ≤ 10 m</th>
<th>L &gt; 10 m</th>
<th>Cf, %</th>
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<td>Type H</td>
<td>A or B (for design of new bridges or evaluation)</td>
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<td>C or D (for evaluation only)</td>
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<td>External or Internal</td>
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<tr>
<td>C or D (for evaluation only)</td>
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<td>External or Internal</td>
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<td></td>
<td>2</td>
<td>External or Internal</td>
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<td>External or Internal</td>
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<td>External or Internal</td>
<td>—</td>
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</tr>
</tbody>
</table>

*For girder bridges with laminated wood decks not 140 or 290 mm thick, linear interpolation based on the values for 140 and 290 mm decks shall be used.

**Note:** Only Class A highways shall be used for determining F values for the design of new bridges. In the evaluation of existing bridges, Class A, B, C, or D highways may be used for determining F values, in accordance with the requirements of the Regulatory Authority.
5.7.1.2.2 Longitudinal bending moments and associated deflections for fatigue limit state and superstructure vibration

5.7.1.2.2.1 For a bridge classified as a shallow superstructure and satisfying all of the applicable conditions listed in Clause 5.7.1.1, the method specified in Clause 5.7.1.2.2.2 may be used for obtaining governing longitudinal moments in the absence of a more refined method.

5.7.1.2.2.2 The following requirements shall apply:

(a) The values of \( F \), \( C_f \), and \( C_e \) shall be obtained using Tables 5.4 and 5.5 for the internal and external portions of the cross-section, in accordance with the type of bridge, the number of design lanes, the span, \( L \), and, in the case of slab-on-girder bridges, the vehicle edge distance, \( D_{VE} \), as shown in Figure 5.1. For bridges other than the slab-on-girder type, there shall be no need to consider \( D_{VE} \). When the value of \( D_{VE} \) is greater than 3.00 m, it shall be taken as 3.00 m for the purposes of this Clause. The span, \( L \), for continuous spans shall be as specified in Clause A5.1.2.

(b) The longitudinal bending moments and deflections shall be calculated by treating the bridge as a beam loaded by two lines of wheels that comprise one truck, as specified in Clause 3.8.3.2. The bending moments and deflections shall be multiplied by \( 1 + DLA \), where \( DLA \) is the relevant dynamic load allowance for a single vehicle or portion of a vehicle, as applicable, to obtain the live load longitudinal bending moments and deflections for the entire cross-section of the bridge. The governing moments thus obtained shall be designated \( M_T \), which shall be distributed in the cross-section in accordance with Item (c).

(c) For girder-type bridges and bridges with longitudinal wood beams, the longitudinal moment per girder, \( M_g \), shall be calculated as follows:

\[
M_g = F_m M_{g,\text{avg}}
\]

where

\[
F_m = \frac{S N}{1 + \frac{\mu C_f}{100}} + \frac{C_e}{100} \geq 1.05
\]

where

- \( S \) = centre-to-centre girder spacing, m
- \( F \) = width dimension that characterizes load distribution for a bridge, obtained from Table 5.4. For the internal girders of slab-on-girder bridges consisting of two or more lanes, the value of \( F \) obtained from Table 5.4 shall be modified by the following factor, which accounts for the variation of \( F \) with girder spacing \( S \):

For \( 10 \text{ m} \leq L \leq 50 \text{ m} \):

\[
F = F_{\text{tab}} \left[ 1.00 + (0.29S - 0.35) \left( \frac{L - 10}{40} \right) \right]
\]

For \( L > 50 \text{ m} \):

\[
F = F_{\text{tab}} (0.29S + 0.65)
\]

where \( F_{\text{tab}} \) is the value of \( F \) for the internal girders from Table 5.4 and the girder spacing, \( S \), is limited to \( 1.2 \text{ m} \leq S \leq 3.6 \text{ m} \). A value of 3.6 m shall be used for \( S \) if \( S \) exceeds 3.6 m.
\[ \mu = \frac{W_e - 3.3}{0.6} \leq 1.0 \]

where
\[ W_e = \text{width of design lane, m, calculated in accordance with Clause 3.8.2} \]

\[ C_f = \text{percentage correction factor obtained from Table 5.4} \]

\[ C_e = \text{percentage correction factor for vehicle edge distance obtained from Table 5.5} \]

For bridges with \( S_c \) greater than 0.5S, \( F_m \) for external girders shall be multiplied by 1.05.

\[ M_{g,\text{avg}} = \frac{M_{g,\text{avg}}}{N} \]

where
\[ M_g = \text{average moment per girder determined by sharing equally the total moment on the bridge cross-section among all girders in the cross-section} \]
\[ N = \text{number of girders or longitudinal wood beams in the bridge deck width, } B \]

The amplification factor, \( F_m \), shall apply to the calculation of maximum deflection. The maximum deflection of a girder for satisfying superstructure vibration requirements shall be determined by applying \( (F_m/N) \) trucks per girder and using the appropriate stiffness characteristics of the girder.

For slab bridges, voided slab bridges, and wood deck bridges that span longitudinally, the moment per metre of width, \( m \), shall be calculated as follows:

\[ m = F_m m_{avg} \]

where
\[ F_m = \frac{B}{F \left[ 1 + \frac{\mu C_f}{100} \right]} \geq 1.05 \]

where
\[ B = \text{total width of the bridge, regardless of whether tapered edges are present} \]
\[ m_{avg} = \frac{M_{g,\text{avg}}}{B_c} \]

where
\[ B_c = \text{effective width of the bridge, calculated by reducing the total width, } B, \text{ for the effects of tapered edges, if present, as specified in Clause A5.1.4} \]

\( F \) and \( C_f \) shall be obtained from Table 5.4 and the amplification factor, \( F_m \), shall apply to the calculation of maximum deflection. The maximum deflection of a point on the transverse section of a slab or voided slab bridge for satisfying superstructure vibration requirements shall be determined by applying \( (F_m/B) \) trucks per metre of width and using the appropriate stiffness characteristics of a 1 m wide section of slab.
### Table 5.4
F and $C_f$ for longitudinal bending moments in shallow superstructures corresponding to the fatigue limit state
(See Clause 5.7.1.2.2.2.)

<table>
<thead>
<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>No. of design lanes</th>
<th>Portion</th>
<th>$F$, m</th>
<th>$L &gt; 10$ m</th>
<th>$C_f$, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$3 \text{ m} &lt; L \leq 10 \text{ m}$</td>
<td>$L &gt; 10 \text{ m}$</td>
<td></td>
</tr>
<tr>
<td>Type A or B</td>
<td>1</td>
<td>External</td>
<td>$3.80 + 0.04L$</td>
<td>4.20</td>
<td>16 – (36/L)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>$4.00 + 0.04L$</td>
<td>4.40</td>
<td>16 – (36/L)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>External</td>
<td>$3.60 + 0.26L$</td>
<td>7.00 – (8/L)</td>
<td>16 – (36/L)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>$3.20 + 0.30L$</td>
<td>6.40 – (2/L)</td>
<td>16 – (36/L)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>External</td>
<td>$3.30 + 0.30L$</td>
<td>9.60 – (33/L)</td>
<td>16 – (36/L)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>$3.00 + 0.40L$</td>
<td>9.80 – (29/L)</td>
<td>12 – (36/L)</td>
</tr>
<tr>
<td></td>
<td>4 or more</td>
<td>External</td>
<td>$3.40 + 0.30L$</td>
<td>12.00 – (56/L)</td>
<td>10 – (30/L)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>$3.00 + 0.44L$</td>
<td>12.00 – (46/L)</td>
<td>10 – (30/L)</td>
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<tr>
<td>Type C slab-on-girder</td>
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<td>External</td>
<td>$3.30^*$</td>
<td>3.50 – (2/L)</td>
<td>5 – (12/L)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>$3.30 + 0.05L$</td>
<td>4.40 – (6/L)</td>
<td>5 – (12/L)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>External</td>
<td>$3.60^*$</td>
<td>3.80 – (2/L)</td>
<td>5 – (15/L)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>$2.80 + 0.12L$</td>
<td>4.60 – (6/L)</td>
<td>5 – (15/L)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>External</td>
<td>$3.60 + 0.01L$</td>
<td>3.70 + (L – 10)</td>
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</tr>
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<td></td>
<td>Internal</td>
<td>$2.80 + 0.12L$</td>
<td>4.80 – (8/L)</td>
<td>0</td>
</tr>
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<td></td>
<td>4 or more</td>
<td>External</td>
<td>$3.80^*$</td>
<td>3.80 + (L – 10)</td>
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<td></td>
<td></td>
<td>Internal</td>
<td>$2.80 + 0.12L$</td>
<td>5.00 – (10/L)</td>
<td>0</td>
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<td>3.40</td>
<td>3.40</td>
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<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>2.80 + 0.06L</td>
<td>4.20 – (8/L)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>External</td>
<td>3.60</td>
<td>3.80 – (2/L)</td>
<td>0</td>
</tr>
<tr>
<td></td>
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<td>Internal</td>
<td>3.00 + 0.06L</td>
<td>4.40 – (8/L)</td>
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<tr>
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<td>External</td>
<td>3.60</td>
<td>3.80 – (2/L)</td>
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<td>Internal</td>
<td>3.00 + 0.06L</td>
<td>4.40 – (8/L)</td>
<td>0</td>
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<tr>
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<td>4 or more</td>
<td>External</td>
<td>3.60</td>
<td>3.80 – (2/L)</td>
<td>0</td>
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<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>3.00 + 0.06L</td>
<td>4.40 – (8/L)</td>
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<td></td>
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<td>3.30 + 0.05L</td>
<td>4.80 – 20/(L + 10)</td>
<td>5 – (12/L)</td>
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<tr>
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<td>3.90 – (2/L)</td>
<td>6 – (15/L)</td>
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<td></td>
<td>Internal</td>
<td>3.30 + 0.05L</td>
<td>5.40 – 3.20 √L – 6</td>
<td>6 – (15/L)</td>
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<td>3</td>
<td>External</td>
<td>3.50 + 0.02L</td>
<td>3.90 – (2/L)</td>
<td>10 – (21/L)</td>
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<tr>
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<td></td>
<td>Internal</td>
<td>3.40 + 0.06L</td>
<td>5.50 – 1350/(L + 20)^2</td>
<td>10 – (21/L)</td>
</tr>
<tr>
<td></td>
<td>4 or more</td>
<td>External</td>
<td>3.50 + 0.02L</td>
<td>3.90 – (2/L)</td>
<td>10 – (21/L)</td>
</tr>
<tr>
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<td></td>
<td>Internal</td>
<td>3.60 + (L^2/200)</td>
<td>6.00 – (19/L)</td>
<td>10 – (21/L)</td>
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</table>

(Continued)
<table>
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<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>No. of design lanes</th>
<th>Portion</th>
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<th>$3 \text{ m} &lt; L \leq 10 \text{ m}$</th>
<th>$L &gt; 10 \text{ m}$</th>
<th>$C_f %$</th>
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<td>External or Internal</td>
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<td></td>
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<tr>
<td></td>
<td>2</td>
<td>External or Internal</td>
<td>2.80</td>
<td>2.80</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>External or Internal</td>
<td>2.80</td>
<td>2.80</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 or more</td>
<td>External or Internal</td>
<td>2.80</td>
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<td></td>
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<tr>
<td>Type C steel-grid-deck-on-girder with deck 100 mm thick or thicker</td>
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<td>External or Internal</td>
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<tr>
<td></td>
<td>2</td>
<td>External or Internal</td>
<td>3.70</td>
<td>3.70</td>
<td>0</td>
<td></td>
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<tr>
<td></td>
<td>3</td>
<td>External or Internal</td>
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<td>3.70</td>
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<td></td>
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<tr>
<td></td>
<td>4 or more</td>
<td>External or Internal</td>
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<td>3.70</td>
<td>0</td>
<td></td>
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<tr>
<td>Type C with wood plank deck</td>
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<td>External or Internal</td>
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<td>2.40</td>
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<td></td>
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<tr>
<td></td>
<td>2</td>
<td>External or Internal</td>
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<td>2.40</td>
<td>0</td>
<td></td>
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<tr>
<td></td>
<td>3</td>
<td>External or Internal</td>
<td>2.40</td>
<td>2.40</td>
<td>0</td>
<td></td>
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<tr>
<td></td>
<td>4 or more</td>
<td>External or Internal</td>
<td>2.40</td>
<td>2.40</td>
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<td></td>
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<td>External or Internal</td>
<td>3.70</td>
<td>—</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3 or more</td>
<td>External or Internal</td>
<td>3.70</td>
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(Continued)
Table 5.4 (Concluded)

<table>
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<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>No. of design lanes</th>
<th>Portion</th>
<th>$F$, m</th>
<th>$3 \text{ m} &lt; L \leq 10 \text{ m}$</th>
<th>$L &gt; 10 \text{ m}$</th>
<th>$C_f$, %</th>
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<td>Type H longitudinal laminates of wood-concrete composite deck bridge</td>
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<td>External or Internal</td>
<td>3.20</td>
<td>3.20</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>External or Internal</td>
<td>3.30</td>
<td>3.30</td>
<td>0</td>
<td></td>
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<tr>
<td></td>
<td>3</td>
<td>External or Internal</td>
<td>3.30</td>
<td>3.30</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 or more</td>
<td>External or Internal</td>
<td>3.30</td>
<td>3.30</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

*For girder bridges with laminated wood decks not 140 or 290 mm thick, linear interpolation based on the values for 140 and 290 mm decks shall be used.

Table 5.5

$C_e$ for longitudinal bending moments in shallow superstructures corresponding to the fatigue and vibration limit state

(See Clause 5.7.1.2.2.2.)

<table>
<thead>
<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>No. of design lanes</th>
<th>Portion</th>
<th>$C_e$, %</th>
<th>$3 \text{ m} &lt; L \leq 20 \text{ m}$</th>
<th>$L &gt; 20 \text{ m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type C slab-on-girder</td>
<td>1</td>
<td>External</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>External</td>
<td>$30(D_{VE} - 1)[1 + 0.4(D_{VE} - 1)^2]$</td>
<td>$30(D_{VE} - 1)\left[1 + \frac{160(D_{VE} - 1)^2}{L^2}\right]$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>External</td>
<td>$26(D_{VE} - 1)[1 + 0.4(D_{VE} - 1)^2]$</td>
<td>$26(D_{VE} - 1)\left[1 + \frac{160(D_{VE} - 1)^2}{L^2}\right]$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 or more</td>
<td>External</td>
<td>$26(D_{VE} - 1)[1 + 0.4(D_{VE} - 1)^2]$</td>
<td>$26(D_{VE} - 1)\left[1 + \frac{160(D_{VE} - 1)^2}{L^2}\right]$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Other types</td>
<td>1 or more</td>
<td>External or Internal</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>
5.7.1.3 Longitudinal bending moments in multi-spine bridges

5.7.1.3.1 If all of the applicable conditions listed in Clause 5.7.1.1 are satisfied, the simplified method specified in Clause 5.7.1.3.2 may be used for multi-spine bridges.

5.7.1.3.2 The value of \( \beta \) shall be calculated as follows:

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

where

- \( B \) = for ultimate and serviceability limits states, the width of the bridge
- \( B \) = for the fatigue limit state, the width of the bridge, but not greater than three times the spine spacing, \( S \)
- \( D_x \) = total bending stiffness, \( EI \), of the bridge cross-section divided by the width of the bridge
- \( D_{xy} \) = total torsional stiffness, \( GJ \), of the bridge cross-section divided by the width of the bridge

The longitudinal bending moment per spine shall be calculated using the methods specified in Clause 5.7.1.2.1 for the ultimate and serviceability limit states and in Clause 5.7.1.2.2 for the fatigue limit state, except that \( S \) shall be taken as the centreline-to-centreline spacing of the spines and the applicable values of \( F \) and \( C_f \) shall be obtained from Table 5.6. No distinction shall be made between internal and external portions of the cross-section, and the value of \( C_e \) shall be taken as zero. At ultimate and serviceability limit states for bridges with more than four design lanes, the value of \( F \) shall be calculated as follows:
5.7.1.4 Longitudinal vertical shear in shallow superstructures

5.7.1.4.1 Longitudinal vertical shear for ultimate and serviceability limit states

5.7.1.4.1.1 For a bridge classified as a shallow superstructure and satisfying all of the applicable conditions listed in Clause 5.7.1.1, the method specified in Clause 5.7.1.4.1.2 may be used for obtaining governing live load shears in the internal and external portions of the bridge.

5.7.1.4.1.2 Longitudinal vertical shear diagrams shall be obtained by treating the bridge as a beam for two load cases. The first load case shall comprise one truck consisting of two lines of wheels, as specified in Clause 3.8.3.2, multiplied by the appropriate factor \((1 + DLA)\) specified in Clause 3.8.4.5. The second load case shall comprise the lane load specified in Clause 3.8.3.3. The governing shears per design lane thus obtained shall be designated \(V_T\).

The following requirements shall also apply:

(a) For girder-type bridges and bridges with longitudinal wood beams, the longitudinal vertical shear per girder, \(V_g\), shall be calculated as follows:

\[
V_g = F_v V_{g,\text{avg}}
\]

where

\[
F_v = \frac{S}{N} \frac{F}{F_{\text{avg}}}
\]

where

\[
S = \text{centre-to-centre girder spacing, m}
\]

\[
F = \text{width dimension that characterizes load distribution for a bridge}
\]

\[
V_{g,\text{avg}} = \text{average shear per girder determined by sharing equally the total shear on the bridge cross-section among all girders in the cross-section}
\]

\[
= \frac{nV_R R_t}{N}
\]

Table 5.6

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Number of design lanes</th>
<th>(F), m</th>
<th>(C_f), %</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS or SLS</td>
<td>2</td>
<td>8.5 – 0.3(\beta)</td>
<td>16 – 2(\beta)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>11.5 – 0.5(\beta)</td>
<td>16 – 2(\beta)</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>14.5 – 0.7(\beta)</td>
<td>16 – 2(\beta)</td>
</tr>
<tr>
<td>FLS</td>
<td>2 or more</td>
<td>8.5 – 0.9(\beta)</td>
<td>16 – 2(\beta)</td>
</tr>
</tbody>
</table>
where

\( n \) = number of design lanes in accordance with Clause 3.8.2

\( V_T \) = maximum shear per design lane at the point of the span under consideration, as specified in this Clause

\( R_L \) = modification factor for multi-lane loading in accordance with Clause 3.8.4.2 or 14.9.4.2

\( N \) = number of girders or longitudinal wood beams in the bridge deck width, \( B \)

For bridges with up to four design lanes, \( F \) shall be obtained from Table 5.7. For girder-type bridges, where the spacing, \( S \), of longitudinal girders is less than 2.00 m, the value of \( F \) obtained from Table 5.7 shall be multiplied by \((S/2)^{0.25}\). This reduction factor shall not apply to solid slabs, transversely prestressed laminated-wood bridges, or longitudinal laminates of wood-concrete composite decks.

For bridges with more than four design lanes, the value of \( F \) shall be calculated as follows:

\[
F = F_4 \cdot \frac{nR_L}{2.80}
\]

where

\( F_4 \) = value of \( F \) for four design lanes obtained from Table 5.7 and multiplied by \((S/2)^{0.25}\) if \( S \) is less than 2.00 m

(b) For slab bridges, voided slab bridges, and wood deck bridges that span longitudinally, the longitudinal vertical shear per metre of width, \( v \), shall be calculated as follows:

\[
v = F_v \cdot v_{avg}
\]

where

\( F_v \) = \( \frac{B}{F} \geq 1.05 \)

where

\( B \) = total width of the bridge, regardless of whether tapered edges are present

\( F \) = width dimension that characterizes load distribution for a bridge, obtained from Table 5.7

\( v_{avg} \) = \( \frac{nV_T R_L}{B_e} \)

where

\( B_e \) = effective width of the bridge, calculated by reducing the total width, \( B \), for the effects of tapered edges, if present, as specified in Clause A5.1.4

For voided slab bridges where the centre-to-centre spacing, \( S \), of longitudinal web lines is less than 2.00 m, the value of \( F \) obtained from Table 5.7 shall be multiplied by \((S/2)^{0.25}\). For bridges with more than four design lanes, the value of \( F \) shall be calculated as follows:

\[
F = F_4 \cdot \frac{nR_L}{2.80}
\]

where

\( F_4 \) = value of \( F \) for four design lanes obtained from Table 5.7 and multiplied by the factor \((S/2)^{0.25}\) if \( S \) is less than 2.00 m
Table 5.7

<table>
<thead>
<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>Number of design lanes and class of highway</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>All classes</td>
</tr>
<tr>
<td>Type A</td>
<td>2.60 + 0.45√L</td>
</tr>
<tr>
<td>Type B</td>
<td>3.60</td>
</tr>
<tr>
<td>Type C</td>
<td></td>
</tr>
<tr>
<td>Slab-on-girder</td>
<td>3.50</td>
</tr>
<tr>
<td>With laminated wood deck</td>
<td>3.00*</td>
</tr>
<tr>
<td>With plank wood deck</td>
<td>2.40*</td>
</tr>
<tr>
<td>Steel-grid-deck-on-girder</td>
<td></td>
</tr>
<tr>
<td>Deck less than 100 mm thick</td>
<td>2.30*</td>
</tr>
<tr>
<td>Deck 100 mm thick or thicker</td>
<td>3.00*</td>
</tr>
<tr>
<td>Type H</td>
<td></td>
</tr>
<tr>
<td>Longitudinal wood beams</td>
<td>3.30*</td>
</tr>
<tr>
<td>with transverse laminated wood deck</td>
<td></td>
</tr>
<tr>
<td>Stress-laminated wood deck bridge spanning longitudinally</td>
<td>3.30</td>
</tr>
<tr>
<td>Longitudinal nail-laminated wood deck bridge</td>
<td>1.70</td>
</tr>
<tr>
<td>Longitudinal laminates of wood-concrete composite deck bridge</td>
<td>2.60</td>
</tr>
</tbody>
</table>

*These values of F are the maximums to be used for internal and external girders. In addition, for external girders, the load shall be the reaction of the wheel loads, assuming that the flooring between the stringers acts as a simple beam, but F shall not be greater than the value specified for internal girders.
5.7.1.4.2 Longitudinal vertical shear for fatigue limit state

For a bridge satisfying all of the applicable conditions listed in Clause 5.7.1.1, live load longitudinal vertical shear may be calculated using the method specified in Clause 5.7.1.4.1, except that the values of \( F \) shall be obtained from Table 5.8 and \( V_T \) shall be calculated using a single truck on the bridge, in one lane only, such that \( n = 1 \) and \( R_L = 1.00 \).

Table 5.8

F for longitudinal vertical shear corresponding to fatigue limit state, m
(See Clause 5.7.1.4.2.)

<table>
<thead>
<tr>
<th>Bridge type (see Clause 5.1)</th>
<th>Number of design lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Type A</td>
<td>2.60 + 0.45(\sqrt{L})</td>
</tr>
<tr>
<td>Type B</td>
<td>3.60</td>
</tr>
<tr>
<td>Type C</td>
<td></td>
</tr>
<tr>
<td>Slab-on-girder</td>
<td>3.50</td>
</tr>
<tr>
<td>With laminated wood deck</td>
<td>3.00*</td>
</tr>
<tr>
<td>With plank wood deck</td>
<td>2.40*</td>
</tr>
<tr>
<td>Steel-grid-deck-on-girder</td>
<td></td>
</tr>
<tr>
<td>Deck less than 100 mm thick</td>
<td>2.30*</td>
</tr>
<tr>
<td>Deck 100 mm thick or thicker</td>
<td>3.00*</td>
</tr>
<tr>
<td>Type H</td>
<td></td>
</tr>
<tr>
<td>Longitudinal wood beams with transverse laminated wood deck</td>
<td>3.30*</td>
</tr>
<tr>
<td>Stress-laminated wood deck bridge spanning longitudinally</td>
<td>3.30</td>
</tr>
<tr>
<td>Longitudinal nail-laminated wood deck bridge</td>
<td>1.70</td>
</tr>
<tr>
<td>Longitudinal laminates of wood-concrete composite deck bridge</td>
<td>2.60</td>
</tr>
</tbody>
</table>

*These values of \( F \) are the maximums to be used for internal and external girders. In addition, for external girders, the load shall be the reaction of the wheel loads, assuming that the flooring between the stringers acts as a simple beam, but \( F \) shall not be greater than the value specified for internal girders.

5.7.1.5 Longitudinal vertical shear in multi-spine bridges

5.7.1.5.1

If all of the applicable conditions listed in Clause 5.7.1.1 are satisfied, the simplified method specified in Clause 5.7.1.5.2 may be used.
5.7.1.5.2

For bridges with up to four design lanes, the value of \( F \) for longitudinal vertical shear shall be obtained from Table 5.9. The method specified in Clause 5.7.1.4 for the applicable limit state shall be used, with \( S \) being the centreline-to-centreline spacing of spines and \( N \) the number of spines; the shear thus obtained shall be for one spine. The shear force determined for one spine shall be equally distributed to the two webs of the spine. The value of \( F \) for the purposes of this Clause shall be such that \( F \) is greater than or equal to \( F_{m} \).

At ultimate and serviceability limit states for bridges with more than four design lanes, the value of \( F \) shall be calculated as follows:

\[
F = F_{4} \frac{nR_{t}}{2.80}
\]

where

\( F_{4} \) = value of \( F \) for four design lanes obtained from Table 5.9

Table 5.9

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Number of design lanes</th>
<th>( F, \text{m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS or SLS</td>
<td>2</td>
<td>7.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>9.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>11.2</td>
</tr>
<tr>
<td>FLS</td>
<td>2 or more</td>
<td>4.25</td>
</tr>
</tbody>
</table>

5.7.1.6 Deck slab moments due to loads on the cantilever overhang

5.7.1.6.1 Transverse moments due to wheel loads on the cantilever overhang

For a cantilever slab of constant or linearly varying thickness, the intensity of transverse moment \( M_{y} \) due to a concentrated load \( P \) shall be calculated as follows:

\[
M_{y} = \frac{2PA}{\pi} \left[ \frac{1}{\left( 1 + \left[ \frac{A \cdot x}{C - y} \right]^{2} \right)} \right]^{2}
\]

where

\( A \) = coefficient obtained from Figure 5.2
\( C \) = transverse distance of the load from the supported edge of the cantilever slab
\( x \) and \( y \) are the coordinates shown in Figure 5.2, with \( y \) less than \( C \).

The relevant design moment intensity shall be obtained by multiplying \( M_{y} \) by (1 + DLA).

For the design moment intensity due to the vertical axle loads of the CL-625 Truck, the effects of individual loads shall be obtained and superimposed or, alternatively, the design moment intensity due to the CL-625 Truck may be obtained directly, without calculation, from Table 5.10 for stiffened and unstiffened overhangs, as applicable (Table 5.10 includes the factor [1 + DLA]).
For those portions of the cantilever slab that are within a distance $S_c$ of an unsupported transverse edge of the slab, the transverse moment intensity shall be assumed to be $2M_y$ unless a more rigorous analysis is used.

**5.7.1.6.1.2 Transverse moments in the interior panel next to the cantilever overhang**

In the absence of a more refined method of analysis, the transverse moments in the interior panel next to the cantilever overhang may be assumed to vary linearly from the values calculated in accordance with Clause 5.7.1.6.1.1 at the root of the cantilever overhang to zero at the girder next to the exterior girder.
Figure 5.2
Calculation of $A$
(See Clause 5.7.1.6.1.1.)

(a) Slab without edge stiffening

(b) Slab with edge stiffening
Table 5.10
Maximum cantilever moments, $M_y$, due to unfactored CL-625 Truck wheel loads (DLA included), kN•m/m
(See Clause 5.7.1.6.1.1.)

<table>
<thead>
<tr>
<th>$S_c$, m</th>
<th>$r_t = 1.00$</th>
<th>$r_t = 0.833$</th>
<th>$r_t = 0.667$</th>
<th>$r_t = 1.00$</th>
<th>$r_t = 0.833$</th>
<th>$r_t = 0.667$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>35</td>
<td>36</td>
<td>37</td>
<td>33</td>
<td>34</td>
<td>36</td>
</tr>
<tr>
<td>1.50</td>
<td>42</td>
<td>44</td>
<td>47</td>
<td>34</td>
<td>35</td>
<td>37</td>
</tr>
<tr>
<td>2.00</td>
<td>53</td>
<td>55</td>
<td>58</td>
<td>36</td>
<td>38</td>
<td>41</td>
</tr>
<tr>
<td>2.50</td>
<td>71</td>
<td>74</td>
<td>78</td>
<td>50</td>
<td>53</td>
<td>55</td>
</tr>
<tr>
<td>3.00</td>
<td>90</td>
<td>94</td>
<td>98</td>
<td>65</td>
<td>67</td>
<td>70</td>
</tr>
<tr>
<td>3.75</td>
<td>117</td>
<td>122</td>
<td>128</td>
<td>86</td>
<td>89</td>
<td>92</td>
</tr>
</tbody>
</table>

Note: For the wheel load position relative to the free edge, see Figure 5.3.

5.7.1.6.2 Local longitudinal moment in cantilever slabs (main reinforcement parallel to traffic)
For longitudinal cantilever spans not longer than 3 m, the maximum intensity of local longitudinal moment, $M_x$, in kN•m/m, shall be calculated as follows:

$$M_x = PC/F$$

where

- $P = 87.5$ kN wheel load of the CL-W Truck
- $C = $ longitudinal distance of $P$ from the line of transverse support, m
- $F = 0.35C + 1.00$ (but shall not exceed 2.10 m)

The relevant design moment shall be obtained by multiplying $M_x$ by $(1 + DLA)$. 
For longitudinal cantilever spans longer than 3 m, the methods specified in Clause 5.7.1.2.1 shall be used, with the span length, \( L \), being taken as twice the cantilever span. The live loading to be used shall be in accordance with Clause 3.8.4.3(c).

**5.7.1.6.3 Transverse moments in cantilever slabs due to railing loads**

In determining transverse moments in cantilever slabs resulting from the barrier or railing loads specified in Clause 3.8.8.1 and applied in accordance with Clause 12.4.3.5, the method of analysis shall be

(a) a refined method in accordance with Clause 5.9; or

(b) yield line theory.

**5.7.1.7 Transverse bending moments in decks**

**5.7.1.7.1 Concrete decks slabs supported on longitudinal girders**

Concrete deck slabs shall be analyzed for positive and negative bending moments resulting from loads applied on the slabs. The analysis shall consider the bending moments induced in the longitudinal direction that agree with the assumptions used in the analysis of the transverse bending moments. The cantilever portions of concrete deck slabs shall be analyzed for transverse negative bending moments resulting from loads on the cantilever portions of the slabs or horizontal loads on barriers and railings. The cantilever portions of concrete deck slabs may be analyzed using Clause 5.7.1.6.1.

Concrete deck slabs (other than their cantilever portions that are proportioned in accordance with the empirical design method of Clause 8.18.4 for the CL-625 Truck) need not be analyzed for transverse bending moments due to live load.

Concrete deck slabs that are supported on longitudinal girders may be analyzed for transverse bending using the simplified elastic method in which the maximum unfactored transverse moment intensity in the portion of the deck slab between the outer girders due to the CL-625 Truck shall be determined as follows:

(a) Except for portions of the deck slab within 1 m of a free edge, the deck slab shall be designed for an unfactored transverse live load moment intensity as follows:

(i) for simple span deck slabs: \((S_e + 0.6)P/10 \text{ kN·m/m, where } S_e \text{ is the effective transverse span in } \text{metres (equal to the smaller of the centre-to-centre spacing of the girder webs and the clear span between girder webs plus the deck thickness)} \text{ and } P \text{ is 87.5 kN, the maximum wheel load of the CL-625 Truck}; \text{ and}

(ii) for deck slabs continuous over three or more supports, the maximum bending moment, either positive or negative, shall be assumed to be 80% of that determined for a simple span. These moments shall be increased by the dynamic load allowance for a single axle, as specified in Clause 3.8.4.5.3.

(b) The portion of a deck slab within 1 m of a transverse free edge shall be reinforced to twice the level of the transverse reinforcement in the other portions of the deck slab unless equivalent local stiffening by diaphragms is provided in accordance with a requirement in another Section of this Code.

(c) The longitudinal moment intensity for distribution of wheel loads to be used with the transverse moment intensity specified in Item (a) shall be taken as \(120/(S_e^{0.5})\%\) (but not to exceed 67% of the maximum transverse moment intensity) and shall be applied as a positive moment that produces tension in the bottom portion of the deck slab. The longitudinal reinforcement necessary to resist the longitudinal moment shall be used in the centre half of the span. The percentage may be reduced by 50% in the end quarters of the span.

**5.7.1.7.2 Steel grid decks**

**5.7.1.7.2.1 General**

Transverse bending moments due to live load in steel grid decks shall be determined as specified in this Clause and Clauses 5.7.1.7.2.2 and 5.7.1.7.2.3.

The grid floor shall be designed as continuous. For concrete-filled floors, moments may be determined in accordance with Clause 5.7.1.7.1 using the simplified elastic method for concrete decks.
The requirements for load distribution specified in Clauses 5.7.1.7.2.2 and 5.7.1.7.2.3 assume that the floor is composed of main elements that span between girders, stringers, or cross-beams, and of secondary elements that are capable of transferring load between the main elements. Reinforcement for secondary elements shall consist of bars or shapes welded to the main steel.

5.7.1.7.2.2 Decks filled with concrete
Floors filled with concrete that span perpendicular to the direction of traffic may be analyzed using the elastic method for concrete deck design for load distribution and moment calculation.

Floors that span longitudinally shall be designed for longitudinal moments determined by distributing one line of truck wheel loads over a width \( E = 1.22 + 0.06S \leq 2.1 \) m, where \( S \) is the span in metres.

Longitudinal edge beams shall be provided for all cantilevered slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb or slab and parapet.

The unfactored live load moment for the longitudinal edge beam shall equal 0.1\( P S \) kN-m for simple spans and 0.08\( P S \) kN-m for continuous spans, where \( S \) is the span, in metres, between points of support. Transverse cantilevered beams, diaphragms, or substructure locations are considered points of support. \( P \) is the maximum wheel load of the CL-625 Truck (87.5 kN), which shall be increased by (1 + \( DLA \)).

The strength of the composite steel and concrete slab shall be determined using the “transformed area” method.

5.7.1.7.2.3 Open decks
A wheel load of one-tenth of the total weight of the CL-625 Truck (62.5 kN), further increased by (1 + \( DLA \)), shall be distributed over a length and width equal to the wheel dimensions specified in Clause 3.8.3.2.

The strength of the section shall be determined using the moment of inertia method.

Edges of open grid steel decks shall be supported by suitable means as required. These supports may be longitudinal, transverse, or both, as required to support all edges properly.

5.7.1.7.3 Transverse laminated wood decking on sawn timber stringers
For bridges with sawn timber stringers, the maximum transverse moment intensity, \( M_y \), due to the CL-625 Truck or to Level 2 or 3 evaluation trucks shall be calculated as follows:

\[
M_y = 2.40 + 0.47L_s \quad \text{for bridges with one design lane}
\]

\[
= 2.19 + 0.56L_s \quad \text{for bridges with more than one design lane}
\]

In these equations, \( L_s \), the stringer span, is in metres and \( M_y \) is in kN-m/m.

5.7.1.7.4 Transverse stress-laminated wood deck-on-girders
For bridges with stress-laminated wood decks, the transverse moment in the decking shall be calculated by assuming that a transverse line of wheels is sustained uniformly by a transverse strip of the decking of a width, \( D_t \), measured in the longitudinal direction of the bridge span, with \( D_t = 0.30 + 0.4S \) for decks with edge stiffening at the transverse free edges, and in which the flexural rigidity of the stiffening beam is greater than or equal to that of a transverse strip of the decking with a width, measured in the longitudinal direction of the bridge span, of 0.25 m. In this equation \( D_t \) and \( S \) are in metres and \( S \) is the girder spacing. If the stiffening beam is absent or has a flexural rigidity less than specified in this Clause, \( D_t = 0.30 + 0.14S \) shall be used.

5.7.1.7.5 Transverse nail-laminated wood deck-on-girders
Transverse bending moments due to live load on transverse nail-laminated wood deck-on-girders shall be determined by distributing a wheel load over a width of 0.4 m plus the thickness of the wearing surface.